



Addis Ababa University

Addis Ababa Institute of Technology

School of Civil and Environmental Engineering

**INVESTIGATING THE EFFECT OF SCOURING ON HYDRAULIC PERFORMANCE
OF RIVER CROSS DRAINAGE STRUCTURE:**

(Case study of Ginchi Awash Bridge)

A thesis Submitted to Addis Ababa Institute of Technology, School of Graduate
Studies, Addis Ababa University

In partial Fulfillment of the Requirement for the Degree of Masters of Science in
Civil and Environmental Engineering
Major in Hydraulic Engineering

By

Hawi Adicho

Advisor: Dr. Yilma Sileshi

Addis Ababa

Ethiopia

June 2018

**Addis Ababa University
Addis Ababa Institute of Technology
School of Graduate Studies**

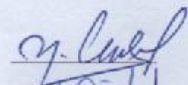
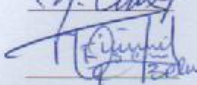
**INVESTIGATING THE EFFECT OF SCOURING ON THE HYDRAULIC
PERFORMANCE OF CROSS DRAINAGE STRUCTURE:**

(A case Study of Ginchi Awash Bridge)

A thesis submitted and presented to the school of graduate studies of Addis Ababa University in partial fulfillment of the degree of Masters of Science in Civil Engineering (Hydraulic Engineering)

By
Hawi Adicho

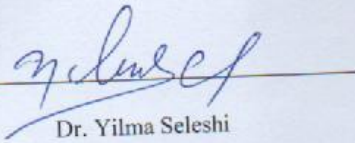
Approval by Board of Examiners

	<u>Name</u>	<u>Signature</u>	<u>Date</u>
1. Advisor	<u>Dr. Yilma Seleshi</u>		<u>03 Dec 2018</u>
2. External Examiner	<u>Dr. Fiseha Behulu</u>		<u>15/12/2018</u>
3. Internal Examiner	<u>Dr. Daneal Fekersillassie</u>	<u>Daneal FBS</u>	<u>15/12/2018</u>
4. Chairman	<u>Dr. Agizew Niguse</u>	_____	<u>15/12/2018</u>

Dr. Agizew Nigussie
Dean, School of Civil &
Environmental Engineering

Certification

The undersigned certify that he has read the Thesis entitled Investigating the Effect of Scouring on the Hydraulic Performance of Cross Drainage Structure: *(A case Study of Ginchi Awash Bridge)* and here by recommend for acceptance by the Addis Ababa University in partial fulfillment of the requirements for the degree of Master of Science.



Dr. Yilma Seleshi

(Advisor)


03 Dec 2018

Date

Declaration

I the undersigned, declare that the thesis is my own work, and has not been presented for a degree at any university and that all sources of materials used for the thesis have been fully acknowledged.

Name: Hawi Adicho

Signature: 

Addis Ababa Institute of Technology

Addis Ababa University

Acknowledgment

I would like to acknowledge everyone who played a role in my academic accomplishments. First of all I would like to praise almighty GOD for his incomparable provision until the end of my work. Secondly my parents, who supported me with love and understanding, without you, I could never have reached this current level of success. Next I would like to express my sincere gratitude to my advisor Dr.Yilma Sileshi for giving me valuable guidance and continuous support from the beginning of this work to the end. My sincere appreciation also goes to all staff members of Civil and Environmental Engineering specially Hydraulic Engineering stream and others who helped me to carry out my thesis work through remarkable encouragement, advice, material support and collaboration in every aspect. Thank you all for your unwavering support.

ABSTRACT

Bridges that cross the rivers and valleys are vital components of the road network that contributes significantly to the national development and public daily life. Any damage or collapse of bridge can risk the lives of road users as well as create serious influence to the entire country. Ginchi Awash Bridge is found at 79.5km west of Addis Ababa Ethiopia, which is bounded in geographical coordinate of $38^{\circ}35'E$ to $38^{\circ}7'E$ longitude and $9^{\circ}8'N$ to $9^{\circ}1'N$ latitude. Ginchi Awash Bridge is the one which this thesis work is conducted on, in case the bridge is in risk due to the scour at the abutment and the river bed of the bridge structure. Hence this research provides the investigation of the effect of scouring on the hydraulic performance of the Bridge from the hydrological and hydraulic analysis. From the hydrological aspect the design flood at different return periods is estimated by Flood Frequency Analysis method and the peak discharge for 100 year return period is $291.04m^3/s$. Based on the result from HEC-RAS model the significant bed scour was modeled and abutment local scour depth for the estimated design flood of 100 year return period is 4.12m at left abutment and 4.84m at right abutment and the contraction scour or bed scour is 1.66m. The total scour depth is the sum of local abutment scour depth and contraction scour depth which is 5.78m at left abutment and 6.50m at right abutment. This is mainly caused by constriction of the main channel and about 70.5 percent of river bed and bank material is sand and gravel which are rapidly eroded under water action. In addition to this the steady flow water surface profile is checked and the result shows that the water surface elevation of Ginchi Awash River at the Bridge is lower than the low chord elevation of the Bridge, so the flood could not over top even for design flood of 100 year return period. Finally the Scour mitigation methods are studied in HEC-RAS software. Taking into account the possible geotechnical failure repair and construction costs, constructing riprap at the bridge abutment is recommended and its detail design calculation is discussed.

Key words: - Bridge, Flood frequency Analysis, HEC-RAS, Local Abutment Scour, Contraction Scour, Degradation

Table of Contents

Certification	iii
Declaration.....	Error! Bookmark not defined.
Acknowledgment	iv
ABSTRACT.....	v
List of Figure.....	ix
List of Table.....	xi
List of Abbreviation.....	xii
1.0. INTRODUCTION	1
1.1 General.....	1
1.2. Statement of Problem.....	2
1.3. Objective.....	3
1.3.1. General Objective	3
1.3.2. Specific Objectives	3
1.4. Research Question	4
1.5. Organization of Thesis	5
2.0. LITERATURE REVIEW	6
2.1. Major Causes of Bridge Failure	6
2.1.1. Hydrological Failure of Bridges	6
2.1.2. Hydraulic Failure of Bridges.....	6
2.2. Major Hydraulic Causes of Bridges Failures	7
2.2.1. Bridge Scouring	7
2.2.2. Sediment Aggradation and Debris Accumulation.....	9
2.2.3. Inadequate Opening Size	10
2.3. Bridge Failure in Ethiopia.....	10
2.4. Scour in different Bed Materials	11
2.4.1. Scour in Gravel and Boulder.....	11
2.4.2. Scour in Cohesive Sediments.....	12
2.4.3. Scour in Rock Materials.....	12
2.5. Hydrological Analysis.....	13
2.5.1. Flood Frequency Analysis	13

2.5.2.	Runoff Routing	14
2.5.3.	Rational Method.....	14
2.5.4.	SCS Unit Hydrograph	15
2.6.	Hydraulic Analysis.....	16
2.6.1.	Hydraulic model (HEC-RAS).....	16
2.6.2.	Steady Flow Water Surface Profile computation in HEC-RAS.....	16
2.6.3.	Determination of Manning’s Roughness Coefficient.....	17
2.6.4.	Bridge Scour Analysis using HEC-RAS model.....	17
2.6.5.	Sediment Transport.....	18
3.0.	STUDY AREA AND DATA.....	19
3.1.	Location of the Study Area	19
3.2.	Topography	20
3.3.	Climate.....	21
3.4.	Hydrology	22
3.5.	Soil.....	23
3.6.	Land use Land Cover	24
4.0.	MATERIAL AND METHOD	26
4.1.	Introduction.....	26
4.2.	Data analysis	27
4.2.1.	Quality Control	27
4.2.2.	Best-Fit Distribution	27
4.3.	Hydrological Method	29
4.3.1.	Flood Frequency Analysis	29
4.4.	Hydraulic Method	32
4.4.1.	Hydraulic Model (HEC-RAS)	32
4.4.2.	Input Data for HEC-RAS Model.....	32
4.4.3.	Steady Flow Water Surface Profile computation in HEC-RAS.....	36
4.4.4.	Bridge Scour Analysis Using HEC-RAS Model	38
4.4.5.	Sediment Transport.....	43
4.4.6.	Procedures in HEC-RAS Model	45
5.0.	RESULT AND DISCUSSION	49

5.1.	Data Analysis	49
5.1.1.	Test for Outlier	49
5.1.2.	Best-Fit Distribution Result	50
5.2.	Hydrological Analysis.....	50
5.2.1.	Delineation of the Catchment	50
5.2.2.	Computation of Peak Discharge	52
5.3.	Grain Size Analysis (Sieve Analysis)	52
5.4.	Hydraulic Analysis (Using HEC-RAS model)	54
5.4.1.	Steady Flow Water Surface Profile including the Bridge at different return period.....	54
5.4.2.	Sediment Transport Analysis Results	56
5.4.3.	Bridge Scour Depth Result	63
5.5.	Mitigation Measurements	67
5.5.1.	Design of Riprap at Abutment	71
6.0.	CONCLUSION AND RECOMMENDATION.....	75
6.1.	Conclusion	75
6.2.	Recommendation	77
	REFERANCE	78
	APPENDICES	80
	Appendix A: Data outlier Test.....	81
	Appendix B: Flood frequency Analysis of Peak Discharge Estimation Result.	82
	Appendix B.1: General Extreme Value 1 Gumbell’s Distribution method	82
	Appendix B.2: Log-Pearson Type III Distribution Method.....	84
	Appendix B.3: Log-Normal Distribution Method.	87
	Appendix C: River cross section data at Downstream and Upstream of Ginchi Awash Bridge (Surveying Data).....	88
	Appendix D.1: Grain Size Analysis of River Bed Soil Sample Data.	94
	Appendix D.2: Grain Size Analysis of left and right Bank Soil Sample Data.....	96
	Appendix H: Sediment data.	98

List of Figure

Figure 1-1: Bridge scour problem at the River bed and Abutment of Ginchi Awash Bridge (April 02, 2018).	3
Figure 3-1: Ginchi Awash River Catchment location.....	19
Figure 3-2: Map of Topography for Ginchi Awash Catchment.	20
Figure 3-3: Climate Map of Ginchi Awash Catchment.	21
Figure 3-4: Annual maximum mean daily stream flow data of Ginchi Awash River.	22
Figure 3-5: Annual Instantaneous daily stream flow data of Ginchi Awash River.	23
Figure 3-6: Map of soil type for Ginchi Awash Catchment.	24
Figure 3-7: Map Land use Land cover for Ginchi Awash Catchment.....	25
Figure 4-1: Conceptual Frame Work for the methodology.	26
Figure 4-2: Surveying of Ginchi Awash River cross section data (April 02, 2018).....	33
Figure 4-3: Sieve Analysis of Grain size distribution of River bed and River Bank channel at Addis Ababa institute of Technology soil lab (April 02, 2018).	35
Figure 4-4: Representation of terms in energy equation (Source: Gray, 2010).....	37
Figure 4-5: Correction for Abutment Skew, K_2	43
Figure 4-6: Skew angle and Bridge abutment alignment with respect to the flow direction.....	43
Figure 4-7: HEC-RAS River reach and cross section representation.	46
Figure 4-8: Bridge Data Editor on HEC-RAS Model.....	47
Figure 4-9: Hydraulic Design Editor Window for Scour at Bridge.....	48
Figure 5-1: Data checking for Best-Fit Distribution.....	50
Figure 5-2: Ginchi Awash Catchment.	51
Figure 5-3: Grain size distribution curve and the grain size of 50 percent finer of Ginchi Awash River bed material.	53
Figure 5-4: Grain size distribution curve and the grain of 50 percent finer of Ginchi Awash left and right Bank material.	54
Figure 5-5: Water surface and Energy grade line profile under Bridge opening and over river channel for 2 year return period design flood.	55
Figure 5-6: Water surface and Energy grade line profile under Bridge opening and over the river channel for 100 year return period design flood.	55
Figure 5-7: HEC-RAS River Bed profile plot along the river for 100 year return period of design flood (Q100).....	56
Figure 5-8: HEC-RAS River bed profile plot along the river for 2 year return period of design discharge (Q2).....	58
Figure 5-9: HEC-RAS Bed change profile at cross section 6 (upstream) and 5 (downstream) of the Bridge crossing structure for Q2.....	61

Figure 5-10: HEC-RAS Bed change profile at cross section 6 (upstream) and 5 (downstream) of the Bridge crossing structure for Q100.	62
Figure 5-11: Total scour depicted for design flood of 2 year return period.	63
Figure 5-12: Total scour depicted for design flood of 100 year return period.	65
Figure 5-13: Total scour depth depicted for design flood of 100 year return period.....	70
Figure 5-14: Total scour depth depicted for design of 2 year return period.	70
Figure 5-15: Rough Sketch of Riprap on Abutment of Ginchi Awash Bridge.....	74
Figure C0-1: River cross section profile Downstream of the Bridge crossing.....	92
Figure C0-2: River cross section profiles Upstream of the Bridge crossing.	93

List of Table

Table 3-1: Rainfall Station, Coordinate and Length of record period.	21
Table 4-1: Ginchi Awash Bridge Dimensions.	36
Table 4-2: Exponent for mode of bed material transport, K1.	41
Table 4-3: Correction factor for abutment shape, K1.	42
Table 5-1: Data Analysis for Outlier test.	49
Table 5-2: Estimation of Peak discharge Using General Extreme Value, Log-Pearson type III and Log-Normal distribution methods.	52
Table 5-3: Summary of Peak discharge using General Extreme Value (Gumbell's) distribution method.	52
Table 5-4: The Bed elevation change result of sediment transport analysis for design flood of 100 year return period.	57
Table 5-5: The Bed elevation change result of sediment transport analysis for design flood of 2 year return period.	59
Table 5-6: Combined local and contraction Bridge scour depth result at main channel and flood plain of Bridge crossing for design flood of 2 year return period.	64
Table 5-7: Combined local and contraction Bridge scour depth result at main channel and flood plain of Bridge crossing for design flood of 100 year return period.	66
Table 5-8: Scour depth of Ginchi Awash Bridge at Original case and including different mitigation methods.	68
Table 5-9: Improvement of Scour mitigation methods.	68

List of Abbreviation

APE	Annual Exceedence Probability
a.m.sl	above mean see level
CN	Curve Number
CRCL	Cereal land Cover system Light
CRCM	Cultivated land Rainfed land Cover system Moderate
DEM	Digital Elevation Model
ERA	Ethiopian Roads Authority
FCD	Forest montane Coniferous, Dense
FHWA	Federal Highway Authority
GIS	Geographical Information System
HEC-HMS	Hydraulic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydraulic Engineering Center River Analysis System
MoWIE	Ministry of Water Irrigation and Electric
NMA	National Metrological Agency
SCS	Soil Conservation Service
SRTM	Satellite Resolution Topographic Map

1.0. INTRODUCTION

1.1 General

Bridges 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 bridge can risk the lives of road users as well as create serious influence to the entire country. Bridges are hydraulic structures which are built on water bodies to transport traffic flow from one side of the river or flood plain to next side. Undermining the bridge opening capacity, sediment and debris accumulation and scouring effects are the major problems in bridge hydraulic design and these factors play a great roll behind failures of bridge structure all around the world (ERA Drainage Design Manual, 2013).

Scour is a natural phenomenon caused by erosion on alluvial or gravel beds by a flowing stream. Local scour refers to the scour caused by an obstruction in the channel. Local scour around bridge foundation elements is one of the most common reasons for bridge collapse and has caused huge economic loss around the world (Chang, 2002). According to a nation-wide study conducted by the US Federal Highway Administration, 75% of 383 bridge failures in 1973 involved abutment damage and 25% involved pier damage (Chang, 1973). During the flooding in Georgia in 1994 over 500 bridges were damaged due to the scour (Richardson and Davis, 2001). Bridge scour has been identified as the most common cause of highway bridge failures and it accounts for about 60% of all bridge collapse in the United States (Deng and Cai, 2009).

Hydraulic problem on bridge structures is a great problem also in Ethiopia. Many bridge and culvert structures failed due to flooding and scouring. In the northern part of the country most bridge failures are due to scouring effects whereas in the southern and eastern part of the country flooding and overtopping are the major failure types. Ginchi Awash Bridge is an example to show bridge hydraulic performance reduction due to scouring failure at the abutment and at the foundation of the bridge. This bridge is arch concrete bridge with single span. From historical event document local abutment scouring is started to transpire on this Bridge during high flood event which is occurred in 1996 GC.

The bed material of Bridge location has a great role in live bed contraction scouring existence. It means that the bed material of Bridge location fall velocity is less than sheer force and velocity of the water which results live bed local abutment scouring. In addition to this, the constriction of the river channel has a great contribution in increment of flow velocities under the bridge structure which directly results live bed contraction scouring.

The significance of this thesis work is investigating the main causes of Bridge scouring at Ginchi Awash Bridge from hydrological and hydraulic perspective. It also determines an appropriate method for scour mitigation which allows the authorities to prepare and evaluate the flood resistance of a critical Bridge structures. By increasing the scour resilience of Bridge, it will decrease the effect of floods and its financial and social consequences.

1.2. Statement of Problem

Now a day, Ethiopia is investing billions of birr for the construction of roads and highway including bridge structures in order to enhance the economic activity of the country. Eventhough, a huge amount of money is invested on the sector, perhaps due to the gaps in detail study and design problems of the bridge structures the country loses huge amount of money every year. Ginchi Awash Bridge is the one which is affected by scouring, eventhough the maintenance is taken more than two times there is still scouring at the abutment and foundation (River bed) of the bridge.

The Ginchi Awash Bridge abutment scouring is happened from the local abutment scouring which involves the removal of material from around the abutments and it is caused by an acceleration of flow and resulting vortices induced by the flow obstructions. The Bridge foundation or River bed scouring is happened from contraction scouring which involves the removal of material from the bed and banks across all the channel width. This component of scour can result from a contraction of the flow area and the bed and banks material of Ginchi Awash River is sand and gravel which are classified as loose granular soil type and highly eroded under water action.

During the high flood season the channel bank is eroded highly and this is mainly caused from the deforestation of the channel bank which results the reducing of the stability of soil.



Figure 1-1: Bridge scour problem at the River bed and Abutment of Ginchi Awash Bridge (April 02, 2018).

Figure 1-1 clearly shows that the exact problem of Ginchi Awash bridge at the River bed of the bridge which is the by reason of constriction of the bridge and at the abutment of the bridge due to the local scour at abutment. So generally, this scour problem remain a great distressing to the society those are leaving around and to vehicles which are using this bridge to cross Ginchi Awash River.

1.3. Objective

1.3.1. General Objective

The general objective of this research is to investigate the effect of scouring on hydraulic performance of the Ginchi Awash Bridge hydrological and hydraulic analysis.

1.3.2. Specific Objectives

- ✓ To evaluate the design floods of the Bridge at different return period.
- ✓ To evaluate the scour depth of Ginchi Awash Bridge for different design flood.
- ✓ To identify the dominant causes of scouring on the Bridge.
- ✓ To propose mitigation methods that can improve the Bridge scouring and hydraulic performance of Ginchi Awash Bridge.

1.4. Research Question

To achieve the aim of this thesis work objective the following questions are raised and answered under the result and discussion part.

- ✓ Is the current Bridge opening size sufficient enough to pass the design flood of 100 year return period without causing any Bridge failure?
- ✓ Does the Bridge construction have the effect on the computed scour depth and River bed change at the Bridge cross section?
- ✓ What are the main causes of scouring both at abutment and River bed of the Bridge?
- ✓ What are the possible mitigation methods that can be used to prevent the problem and to increase the operational life of the Bridge?

1.5. Organization of Thesis

This Thesis paper generally organized into six chapters.

- ✓ Chapter One includes the introduction of the Bridge scour failure from the general of a worldwide to that of Ginchi Awash Bridge. Addition to this the objective of the study, problem statement, research questions is included under this chapter.
- ✓ Chapter Two is all about literature review under this chapter the MSc. thesis of others researcher, journal papers, books and different manuals are reviewed.
- ✓ Chapter Three attempts to discuss the general characteristic of the study area which includes the description of the location of the Bridge, the topography, climate, hydrology, soil and land use land cover of the study area.
- ✓ Chapter Four is the section where the methods that used in data analysis, hydrological analysis and hydraulic analysis of the study is presented in.
- ✓ Chapter Five is the result and discussion section in which the objective of the study is achieved and this section of the paper includes the result of data analysis, hydrological analysis (peak discharge estimation), the hydraulic analysis (abutment and river bed scour depth of the Bridge) and finally the scour mitigation methods are discussed under this chapter.
- ✓ Chapter Six is the last section of this paper which consist of the conclusion and recommendation of the thesis work which is based on the result and discussion part.

2.0. LITERATURE REVIEW

2.1. Major Causes of Bridge Failure

2.1.1. Hydrological Failure of Bridges

Hydrological failures mainly associated with the determination of the peak discharge that the Bridge is capable to pass without having significant problems. Less input data, error in data analysis and selection of inappropriate modeling methods will lead to undermining or over estimation of peak discharges which will result reduction or over design of Bridge opening size. Over design results increase in total cost of the structure whereas reduction in opening size will result in total failure of the Bridge during peak flow seasons of the year. In order to avoid this, a detail Hydrological study needs to be carried out in the catchment where the bridge is going to be constructed. The stream channels are occasionally straightened or modified in some way to accommodate the required structure. In streams flowing over bedrock this approach is adequate but in alluvial streams it may prove inadequate (Brown, 1981).

2.1.2. Hydraulic Failure of Bridges

Hydraulic failure of the bridge structure includes inadequate flow capacity to over-topping of the bridge deck or the approach embankments, increased loading on the structure from water, sediment or debris and failure of foundations or supports as a result of bridge scouring (Rohani A., 2002).

The major damage on Bridges at river crossing occurs during high flood seasons. Many researchers studied the Hydraulic design of Bridges in order to investigate major causes of failure of different river Bridges around the world. Smith (1976, 1977) studied 143 Bridge failures that occurred throughout the world between 1847 and 1975. From the study it is possible to understand that almost half of the failures were due to floods.

Flood events have a great potential to wash the foundation and the superstructure of a large number of Bridges structures. When there is excess flow in the stream and when this excess flows exceed the design flood, the flow will bypass the bridge and the most severe consequences will be scour and foundation failure (Les Hamill, 2004).

2.2. Major Hydraulic Causes of Bridges Failures

2.2.1. Bridge Scouring

Scour is a phenomenon in which the level of the River bed becomes lower under the effect of water erosion, leading to the exposure of bridge foundations. This happens either because of the increase of flow speed around the river piers or because of the long time erosion of the riverbed. Different materials scour at different rates. Loose granular soils are rapidly eroded by flowing water, while cohesive or cemented soils are more scour resistant. However, ultimate scour in cohesive or cemented soils can be as deep as scour in sand-bed streams. Under constant flow conditions, scour will reach maximum depth in sand and gravel bed materials in hours; cohesive bed materials in days; glacial tills, sand stones and shales in months; limestones in years and dense granites in centuries. Under flow conditions more typical of actual bridge crossings, several floods will be needed to attain maximum scour (FHWA, 2012).

A factor in scour at highway crossings and encroachments is whether the scour is clear-water or live-bed scour. Clear-water scour occurs where there is no transport of bed material upstream of the crossing or encroachment and live-bed scour occurs where there is transport of bed material from the upstream reach into the crossing or encroachment (HEC 18 - Evaluating Scour at Bridges).

A study on the issues of bridge abutment and foundation scouring or highway crossing (Bridge and Culvert) outlet erosion from Mekelle to Adigrat Highway - Tigray, Ethiopia, which is done by Dawit Hadera and Bizuneh Asfaw (2016) shows that the crossing structures in the study area are highly affected by scouring, due to runoff are flowing to concentration from the whole catchment to one direction and give speed to erode the soil. In these study the authors try to analyse both hydrological and hydraulic aspect of two culvert one from Kehen and the other from Betehaweriat near Adigrat and one bridge Agulae river bridge have been selected. For the bridge the hydrologic and hydraulic analysis was carried out using SCS method and HEC-RAS 4.1.0 respectively and using SCS method and HY-8 for Kehen and rational method and HY-8 for Betehaweriat culverts respectively. From the result of study the remedial measures were recommended from hydraulic analysis results of the bridge and culvert. The recommendations

are constructing riprap at the abutment of Agulae Bridge and constructing of energy dissipaters for the Kehen and Betehaweriat culverts.

Total scour at a highway crossing is comprised of three components, these are:

1. Long-term aggradation and degradation,
2. Contraction scour and
3. Local scour.

2.2.1.1. Long-term Aggradation and Degradation

These are long-term streambed elevation changes due to natural or man-induced causes which can affect the reach of the river on which the bridge is located. Aggradation involves the deposition of material eroded from the channel or watershed upstream of the bridge, whereas degradation involves the lowering or scouring of the bed of a stream due to a deficit in sediment supply from upstream (HEC 18 - Evaluating Scour at Bridges).

2.2.1.2. Contraction Scour

Contraction scour in a natural channel involves the removal of material from the bed and banks across all or most of the channel width. This component of scour can result from a contraction of the flow area or change in downstream control of the water surface elevation. The scour is the result of increased velocities and shear stress on the bed of the channel. Contraction of the flow by bridge approach embankments encroaching onto the floodplain and/or into the main channel is the most common cause of contraction scour. Contraction scour can be either clear-water or live-bed. Live-bed contraction scour typically occurs during the rising stage of a runoff event, while refilling of the scour hole occurs during the falling stage. Also, clear-water scour at low or moderate flows can change to live-bed scour at high flows. This cyclic nature creates difficulties in measuring contraction scour after a flood event (FHWA, 2012).

2.2.1.3. Local scour

Local scour involves removal of material from around piers, abutments, spurs, and embankments. It is caused by an acceleration of flow and resulting vortices induced by the flow

obstructions, and is usually cyclic in nature. Local scour can also be either clear-water or live-bed scour (FHWA, 2012).

2.2.2. Sediment Aggradation and Debris Accumulation

Sediment aggradation at Bridge cross section results in serious problem on the overall Hydraulic performance of a Bridge. The main effect of channel bed aggradation and fill is to reduce bridge clearance. However, aggradation may continue to the extent where new hazards like meanders cut off and channel pattern change will also generate. In addition, aggradation may lead to bank erosion as bar formation changes flow paths, and decreased channel capacity will increase flooding with the potential for damage to the bridge and its approaches (Jean-Louis, Briaud et.al, 2001).

Build-up of trash and debris can significantly affect the hydraulic performance of Bridges. Difficulties are normally associated with small single span Bridges which tend to be more easily blocked than large multi-span structures. This may result in large increase in water level upstream and associated flooding. Debris may partially restrict the flow leading to significant scour around piers or abutments threatening the safety of the structure. Debris which is caught against or between piers can result in enhanced Hydraulic forces by increasing the effective pier width. The extent of these force is not easily predicted and usually be most severe when the river is in flood (The Design of Highway Bridges for Hydraulic Action, 1994).

A study on the issues of “Effect of Sediment Aggradation on the Hydraulic Performance of River Cross Drainage Structures (The case of Mekanisa Bridge on Mekanisa to Gofa Highway Road)” is done by Natnael Hailu (2016) shows that, the sediment aggradation at bridge location plays a significant role in the reduction of the hydraulic performance of the Bridge. The effect of reduction in the opening size of the Bridge is already being observed on the area including flooding and bank overtopping which puts pressure on the society living along the river bank. In this study the authors try to analyse both hydrological and hydraulic aspect of Mekanisa Bridge and the hydrological and hydraulic analysis are carried out using HEC-HMS and HEC-RAS respectively. From the result of hydraulic analysis the recommended remedial measures are retaining wall must be provided for the protection of potential sediment material movement to

the downstream of the bridge from the river cross sections which are located on bends and curves and to reduce the bank erosion and he recommend the River training work at the downstream side of the bridge, this is mainly because the river alignment bends just after it passes through the Bridge and returns back to the original alignment after it follows a half circular river alignment.

2.2.3. Inadequate Opening Size

In general, given a particular design discharge at a given crossing, the shorter a bridge the more backwaters it will create. This same smaller bridge will also have higher velocities through the bridge opening and an increased potential for scour at the bridge foundation. A longer bridge at this same crossing will generate a smaller amount of backwater and will have lower velocities and potential for scour. Policy considerations and economics require an understanding of the impacts that the bridge could have on the flow of water in the floodplain and the impacts it might have on adjacent properties (ERA Drainage manual 2013).

2.3. Bridge Failure in Ethiopia

Ethiopia has experienced many cases of bridge collapse and their serious consequence over decades. Presently more than 20,000 km of federal road administrated by ERA and the road network contains about 3,000 bridges and more than 27,000 small drainage structures. Among them about 35% of Ethiopian bridges were constructed more than 60 years ago showing signs of severe deterioration due that they have not been properly maintained since their construction (Beza N. 2010).

According to Short Study on Ethiopian Arch Bridge (2017) among 3,000 bridges in Ethiopia 602 are Arch bridges which are constructed between 1930s and 1980s. The study exposed among old bridges which suddenly failed down in rainy season is those masonry arch bridge. According to the first nationwide bridge Inventory done in 2005/06, in all road segments under ERA administration, except in Soddo District where no arch bridge exists. Most of them are constructed during Italian invasion before 1945. These bridges are serving for more than 7 decades and currently they are not compatible and suitable to the current flow of traffic and cargos as they were not designed to it.

In Ethiopia many of Masonry arch bridges still in need of total replacement. It is because those bridges are under a risk of failure caused by: -

1. Since many of them are constructed in 1930-40s and served for more than 70 years, no matter how they are maintained now they are not reliable to carry the current and future traffic load,
2. The material type, which is masonry wall and gravel fill is weathered enough in the past many years and the surface cover is wear out in many cases despite good looking asphalt pavement is now,
3. The current traffic volume and weight is very different from designed and intended service at that time and by nature of the masonry arch structure (not reinforced) once the key stone and
4. The central part starts collapsing; the whole structure can fall down in a moment and can't spare time to somehow provide temporary solution (Short Study on Ethiopian Arch Bridge, 2017).

2.4. Scour in different Bed Materials

2.4.1. Scour in Gravel and Boulder

The gravel and boulder Bed Rivers are characterized by large median size and high standard deviation of the bed material size distribution. The size distribution affects the scour depth significantly. When the standard deviation is large and the bed material contains some non-moving sizes of sediment, the coarser particles tend to accumulate in the scour hole and form an armor layer which tends to inhibit further scour. Hence, in general, greater the standard deviation less will be scour compared to scour in uniform bed material. This topic has also not been studied extensively in the past. Details on this are also available in Kothyari and Garde, (2007).

When a bridge pier is constructed in a gravel-bed river, as the scour progresses during the flood, coarser particles will accumulate in the scour hole and armoring effect will be caused. As a result, the scour depth will be much smaller than that in an alluvial river with relatively finer and less non- uniform material. In the case of uniform or nearly uniform sediments it has been found

by some investigators that clear water scour is slightly greater than scour under sediment transporting flows. Similar conclusion cannot necessarily be true for highly non uniform sediments. The two effects that would govern scour depth in sediment transporting flow would be sediment load falling into scour hole and relative coarsening. This process however needs to be further studied by carrying out extensive experiments.

2.4.2. Scour in Cohesive Sediments

In India the estimation of scour around bridge piers is done by using Lacey-Inglis method. This method is commonly used in sandy strata. However, recently there has been a debate on whether this method can be used in clayey strata exhibiting certain amount of cohesion. Examples have been quoted by Kand (1993) indicating that structures built in clayey soils with shallow foundations have stood intact for several decades or a few centuries. On the other hand, using Lacey-Inglis method for such soils would give excessive scour depth and need deep foundations.

2.4.3. Scour in Rock Materials

Rock strata are encountered on many occasions while driving the well of bridge foundations. Great difficulty is encountered by the practitioners in sinking the foundation well through rocky strata. Realistic estimation of maximum scour level is thus very important in such scenarios.

Depending upon their type, rocks can have different resistance characteristics against shearing action of flow. Based on their resistance the rocks are termed as completely weathered, highly weathered, moderately weathered or slightly weathered rock formation. Mostly the slightly weathered and the moderately weathered rocks are non-erodible even when they are subjected to large shear stresses. The erodibility behavior of highly weathered rocks varies with the magnitude of shearing action or stream power (Annanadale, 1995). For example for the new proposed railway bridge on Yamuna at Delhi the highly weathered rock was discerned to be erodible near the river general bed level but it was non-erodible at lower (Singh et al., 2006). The reduced shear stress for design flow conditions at elevations lower than river bed was worked out in this case by the mathematical model of Kothyari et al. (1992 b).

2.5. Hydrological Analysis

The first analysis process in the hydraulic design for a bridge is the hydrological analysis or the calculation of design flood and sometimes discharges for historical floods. The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental for the design of highway drainage structures. Errors in the estimates will result in a structure that is either undersized and causes more drainage problems or oversized and costs more than necessary. On the other hand, it is important to realize that any hydrological analysis is only estimation. There are several floods that need to be calculated. Firstly, design floods for a range of probabilities up to an annual exceedence probability of 1% (AEP 1%). This range should include the standard AEPs of 50, 20, 10, 5, 2 and 1% for most projects. As well the flood with an annual exceedence probability of 0.05% is needed for the structural design of the bridge. In some cases, other large floods are also needed in special circumstances. Sometimes only a few design floods may be needed. If there is sufficient data for model calibration, floods should also be calculated for the specific historical flood events that are used for calibration (Hydraulic Guidelines for Bridge Design).

There are different methods in hydrological analysis of bridge structures and those methods are depend up on the availability and length of available stream flow data and metrological data for either the bridge site itself, elsewhere on the river or on a nearby similar catchment. The general methods are as follows:

2.5.1. Flood Frequency Analysis

At site flood frequency analysis is used if there is a long term stream gauge located near the bridge site. If the gauge is located close to the bridge site, say with a difference in catchment area of less than 10%, the gauge can be used directly, with the calculated discharges modified for the small differences in catchment area. If the gauge is located further away, the flood frequency analysis can be used to help estimate parameters for a runoff routing model. Flood frequency analysis is usually the most reliable method for calculating design flood discharges, but it only provides peak discharges, not flood hydrographs. If flood hydrographs are also needed, say for calculating times of closure, these must be calculated in some other way. The two methods are by scaling recorded hydrographs or by using a runoff routing model. Even if the nearest stream

gauge is located some distance from the bridge site or even in a neighboring catchment, flood frequency analysis on this station may be of value to obtain local regional flood estimates to compare with general regional values (Hydraulic Guidelines for Bridge Design).

In this research flood frequency analysis or statistical method of estimating peak discharge will be used, since ERA drainage design manual recommend that the statistical analysis method to estimate peak flows for the gauged site of stream flow record length of greater than 10 year. The followings are different methods used in flood frequency analysis for estimation of design flood:-

- I. General Extreme Value 1 (Gumbell's) distribution method.
- II. Log-Pearson type III distribution method.
- III. Log-Normal distribution method

The detail equations and their description of those methods are found under methodology part of this paper.

2.5.2. Runoff Routing

This is a rainfall-runoff model where flood hydrographs are calculated from rainfall using one of a range of different software packages. The most common packages used for bridge design include HEC-HMS, but there are several others that can also be appropriate. Rainfall inputs are calculated using "HEC-HMS Rainfall and Runoff". Model parameters can be calibrated using when design floods have been calculated, they should be checked for consistency. Runoff routing analysis gives both flood peak discharges and hydrograph (Hydraulic Guidelines for Bridge Design).

2.5.3. Rational Method

The Rational Method provides estimates of peak runoff rates for small urban and rural watersheds of less than 50 hectares (0.5 square km) and in which natural or man-made storage is small. It is best suited to the design of urban storm drain systems, small side ditches and median ditches, and driveway pipes. It shall be used with caution if the time of concentration exceeds 30 minutes. Rainfall is a necessary input for this method of flow estimation. The rational formula estimates the peak rate of runoff at any location in a catchment area as a function of the

catchment area; runoff coefficient; and mean rainfall intensity, for duration equal to the time of concentration (ERA Drainage manual 2013).

The rational formula is expressed as:

$$Q = 0.00278CIA \quad 2.1$$

Where:

Q = maximum rate of runoff, m³/s

C = runoff coefficient representing a ratio of runoff to rainfall

I = average rainfall intensity for a duration equal to the time of concentration, for a selected return period, mm/hr.

A = catchment area tributary to the design location, ha

2.5.4. SCS Unit Hydrograph

Techniques developed by the U. S. Soil Conservation Service (12) for calculating rates of runoff require the same basic data as the Rational Method: catchment area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm.

However, the SCS method is most suited for computing flood peaks and run of volumes for catchments smaller than 65km², with slopes of less than 30% and a time of concentration (Tc) less than 10 hours (ERA Drainage manual 2013).

The SCS runoff equation is therefore a method of estimating direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$Q = \frac{(P-Ia)^2}{(P-Ia)+S} \quad 2.2$$

Where: Q = accumulated direct runoff, mm

P = accumulated rainfall (potential maximum runoff), mm

Ia = initial abstraction including surface storage, interception, and infiltration prior to runoff, mm

S = potential maximum retention, mm.

The relationship between I_a and S was developed from experimental catchment area data. It removes the necessity for estimating I_a for common usage. The empirical relationship used in the SCS runoff equation is:

$$I_a = 0.2S \quad 2.3$$

Substituting $0.2*S$ for I_a , the SCS rainfall-runoff equation becomes:

$$Q = \frac{(P-0.2*S)^2}{(P+0.8*S)} \quad 2.4$$

S is related to the soil and cover conditions of the catchment area through the CN. CN has a range of 0 to 100, and S is related to CN by:

$$S = \frac{25400}{CN-254} \quad 2.5$$

Where S = potential maximum retention, mm.

CN= curve number.

2.6. Hydraulic Analysis

2.6.1. Hydraulic model (HEC-RAS)

Hydraulic analysis was performed in order to determine the hydraulic performance of the Bridge and to calculate the flow patterns, flood levels and flow velocities for the bridge and nearby river reach. The most commonly used hydraulic analysis procedure used for routine bridge scour analysis is HEC-RAS, developed by the United States Army Corps of Engineers. This software is well suited to the hydraulic analysis of bridges and culverts. And this model is widely used around the world for such structures. The HEC-RAS model is suitable to compute the contraction scour and local scour at piers and abutments at fixed channel cross-sections including Bridge cross-section. To model a Bridge scour event, HEC-RAS uses a quasi-unsteady flow series (HEC-RAS, User's Manual, 2010).

2.6.2. Steady Flow Water Surface Profile computation in HEC-RAS

This component of the model is intended to calculate water surface profiles for steady gradually varied flow. It is also capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles. The basic computational procedure is based on the solution of the one-dimensional energy equation and momentum equations. Momentum equation is applied in situations where the water surface profile is rapidly varied flow type.

This happens where there is hydraulic jump, flow through bridges, and when flow is constricted (Hydraulic Reference Manual, 2010).

2.6.3. Determination of Manning's Roughness Coefficient

Manning' n selection of an appropriate value for Manning's n is very significant to the accuracy of the computed water surface profiles. The value of manning's n is highly variable and depends on a number of factors including: surface roughness, vegetation, channel irregularities, channel alignment, scour and deposition, obstructions, size and shape of the channel, stage and discharge, seasonal changes, temperature, and suspended material and bed load. This method is used for the design flood levels at crossing sites after the design discharges have been estimated by the hydrological methods of statistical analysis or flood frequency analysis.

$$Q = \frac{1}{n} * S^{\frac{1}{2}} * A * R^{\frac{2}{3}} \quad 2.6$$

Where:

Q	=	Discharge in [m ³ /sec]
R	=	Hydraulic mean depth [m] = A/P
A	=	Cross-sectional flow area [m ²]
P	=	Wetted perimeter [m]
S	=	Longitudinal bed slope [%]
n	=	Manning's roughness coefficient

2.6.4. Bridge Scour Analysis using HEC-RAS model

The scour computations in HEC-RAS are comprised of three components: Contraction scour, Pier scour, and Abutment scour. This computation is based upon the methods outline in Hydraulic Engineering Circular No. 18 (FHWA, 2001). In order to perform a bridge scour analysis first we need to develop hydraulic model of the river reach containing the bridge to be analyzed. The model must include the downstream and upstream cross section of the river, in order to evaluate the long term effects of the bridge on the water surface profile.

2.6.5. Sediment Transport

This component of the modeling system is intended for the simulation of one dimensional sediment transport or movable boundary calculations resulting from erosion and deposition over moderate time periods typically years, although applications to single flood events also possible (Hydraulic Reference Manual, 2010). The model is designed to simulate long-term trends of erosion and deposition in a stream channel that might result from modifying the frequency and duration of the water discharge and stage, or modifying the channel geometry. This model is suitable to determine the rate of sedimentation at fixed channel cross-sections including Bridge cross-section which is the aim of this study.

To model a sediment transport event, HEC-RAS uses a quasi-unsteady flow series. A quasi-unsteady flow series is created from a storm hydrograph by dividing the hydrograph into flow duration steps, in which the flow, stage, temperature, and sediment loading are considered constant. The flow durations are further broken down into computation increments, in which the bed elevation and hydrodynamics are updated after each increment. Finally, computational increments are further subdivided into the bed mixing time step to consider change in the composition of bed material due to removal or addition of materials. Therefore, rather than modeling a constantly changing flow hydrograph, HECRAS models a constant flow over the flow duration, and only allows bed elevation and hydrodynamics to change between each computational increment. This approach helps to increase model stability when simulating sediment transport events.

3.0. STUDY AREA AND DATA

3.1. Location of the Study Area

The study area is approximately bounded between the geographic coordinate of 38°35'52.97"E to 38°7'43.76"E longitude and 9°8'10.99"N to 9°1'41.75"N latitude and conducted in Ginchi Awash Catchment, Dandi district, West Shewa zone of Oromia Regional state, central Ethiopia. The catchment is located in Awash Basin and covers the total catchment area of 47.9km². The study area is located at 79.5Km west of Addis Ababa at an altitude of ranging between 2243-3030 m above sea level. Ginchi Awash Bridge is placed at the outlet of Ginchi Awash Catchment and it used to carry traffic from Addis Ababa city and Ginchi to Ambo town.

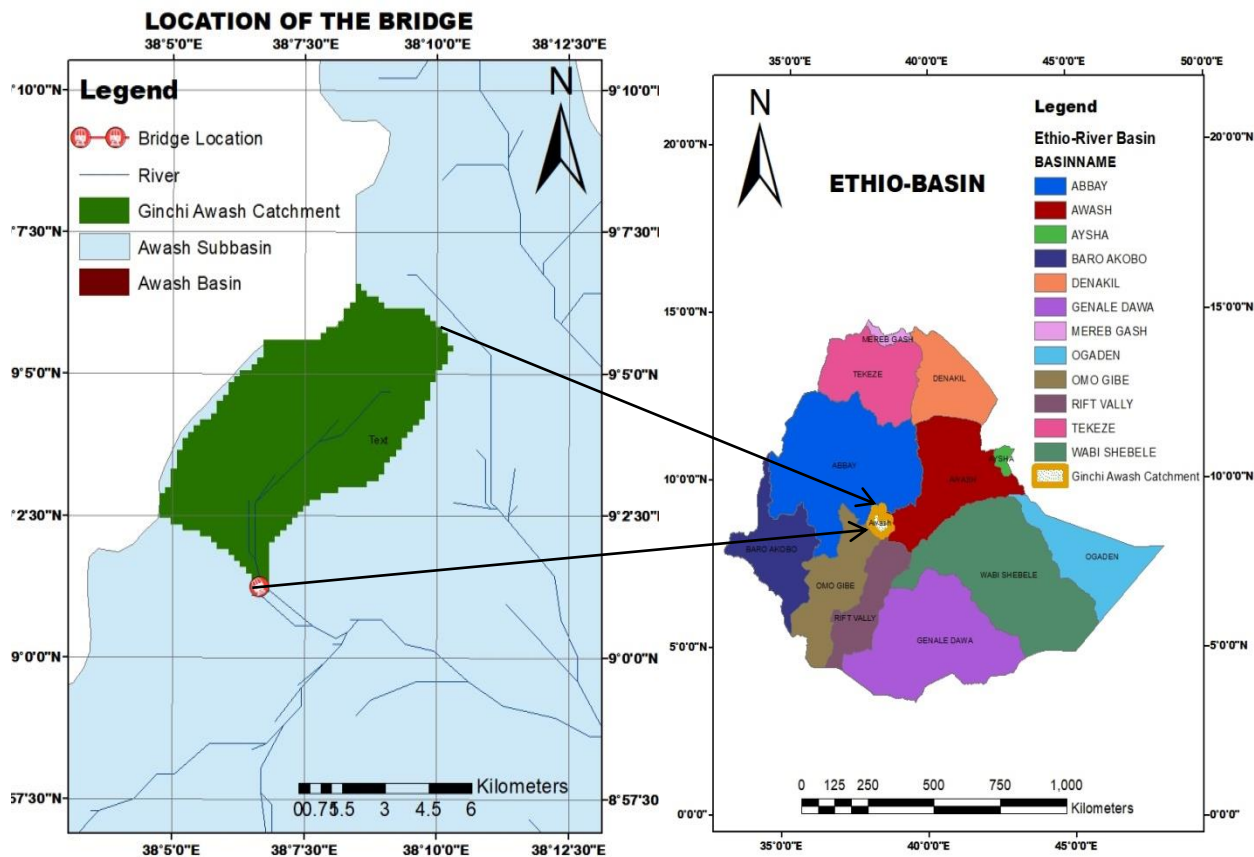


Figure 3-1: Ginchi Awash River Catchment location.

3.2. Topography

The study area was stratified into three, based on altitudinal ranges Low Land (2243-2400m.a.s.l.), Medium land (>2400-2750m.a.s.l.) and High Land (>2750-3030m.a.s.l.). The data under topography includes collection of geological map and topographical map of the area. From Ministry of Water, Irrigation and Electric the complete Digital Elevation Model (DEM) data of the project area on 90mx90m Grid together with GIS files (such as GIS data of Land use, soil types etc.) are collected. The catchment delineation has been made on the Digital Topographic Map (SRTM 90m*90m Grid digital Elevation Models). The catchments parameters such as average slope, length of the longest water course, difference in elevation between the crossing and water divide, the main channel length and its slope are determined from the above listed maps and software's.

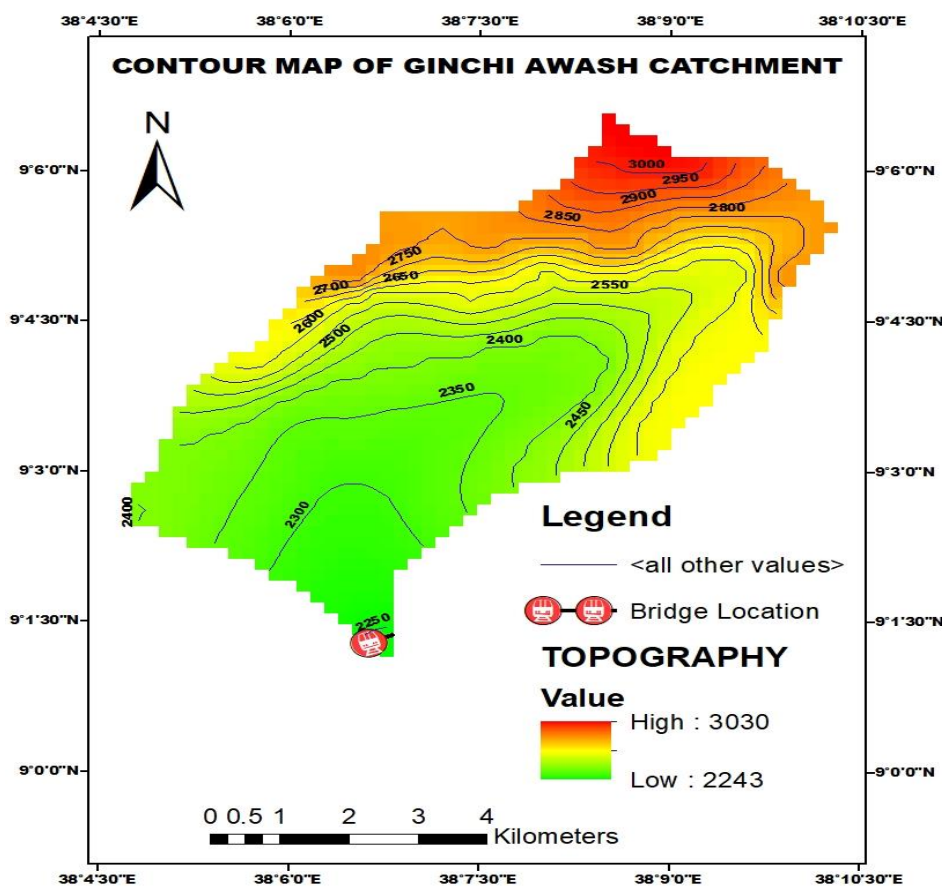


Figure 3-2: Map of Topography for Ginchi Awash Catchment.

3.3. Climate

The minimum and maximum temperature of the district is 15⁰C and 31⁰C, respectively. The district receives average annual rainfall ranging from 750–1,300 mm (lowland=750-800 mm, midland=800-950 and highland=950-1300). The main rainy season is from June to end of September. The observed daily time series rainfall data at three gauging stations which are at the bridge location and near to Bridge locations are collected from national metrological agency (NMA) and discussed in the following table.

Table 3-1: Rainfall Station, Coordinate and Length of record period.

Metrological Station	Coordinate		Elevation (m.a.s.l)	Record Period
	Latitude	Longitude		
Ginchi at Awash station	9.02	38.13	2132	(1985-2016 GC)
Asgori met station	8.79	38.33	2072	(1985-2015 GC)
Welenkomi met station	9.25	38.08	2952	(1987-2016 GC)

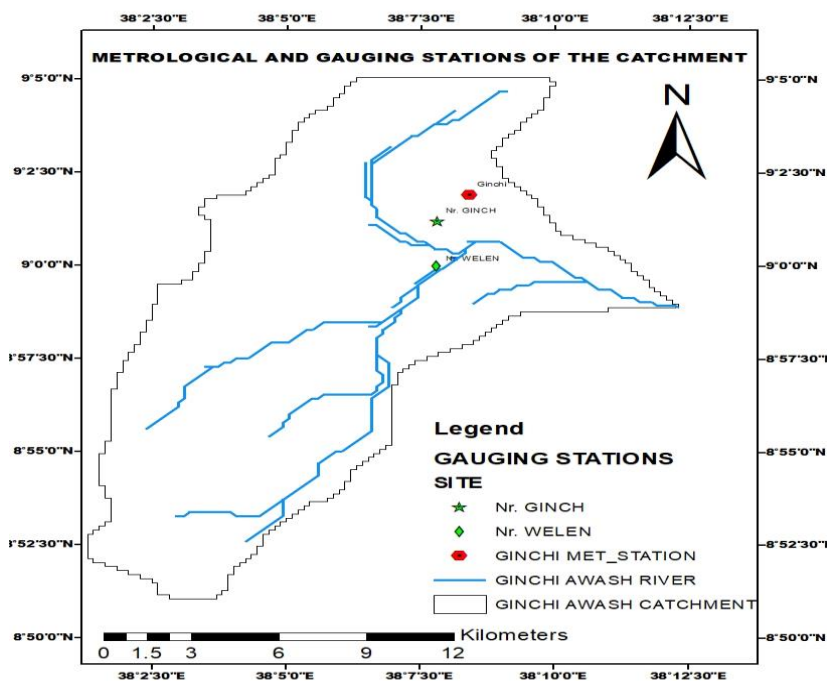


Figure 3-3: Climate Map of Ginchi Awash Catchment.

3.4. Hydrology

The Awash river originates from a high plateau near Ginchi town, of Mount Warqe, and flows along the rift valley into the Afar triangle and ends in saline Lake Abbe. The mean annual daily stream flow data of 17 years (1994-2010 near to the Ginchi Awash River Bridge location) is collected from Ministry of Water, Irrigation and Electric. Since ERA Drainage manual recommend statistical method of peak discharge estimation for stream gauged of more than 10 year record period in this research the annual Instantaneous daily flow data is directly used in flood frequency analysis to estimate the peak discharge that the bridge is capable to pass through its opening size.

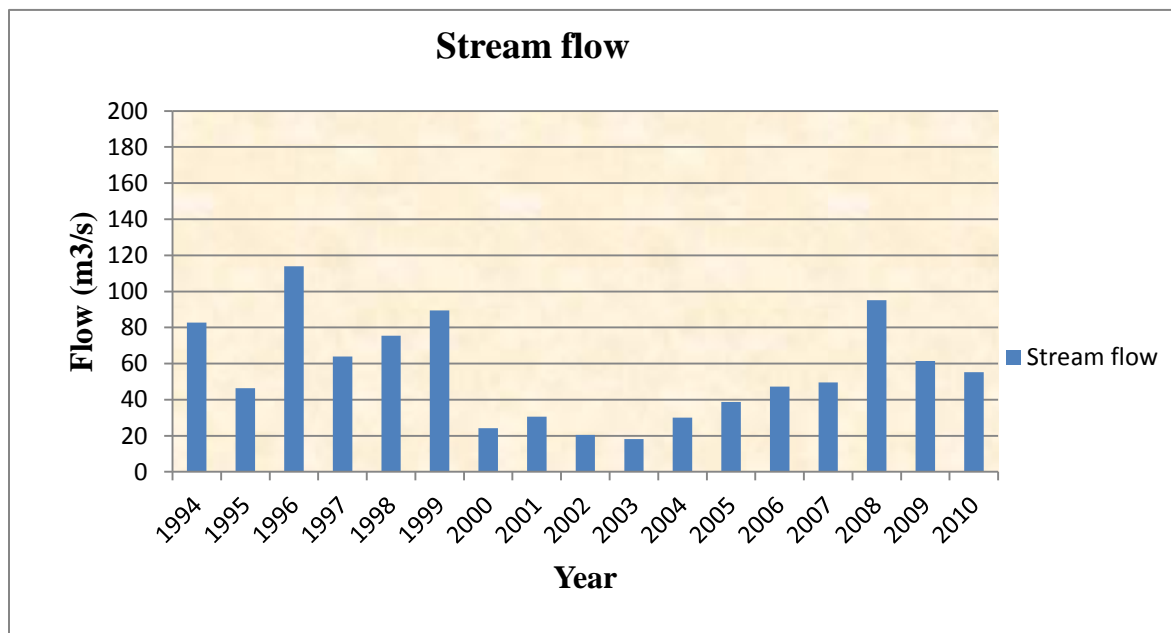


Figure 3-4: Annual maximum mean daily stream flow data of Ginchi Awash River.

For the catchment area of less than 300 square kilometers it is not recommended to use the annual average daily flow data directly. Hence the annual instantaneous daily flow is must to be calculated from the annual average daily flow by using the following Fuller’s equation.

$$Q_{inst} = Q_{avr} * 2A^{-0.3} + Q_{avr} \tag{3.1}$$

Where

Q_{inst} = instantaneous peak discharge (m^3/s).

Q_{avr} = maximum mean daily flow (m^3/s).

A = catchment area in square miles (mi^2).

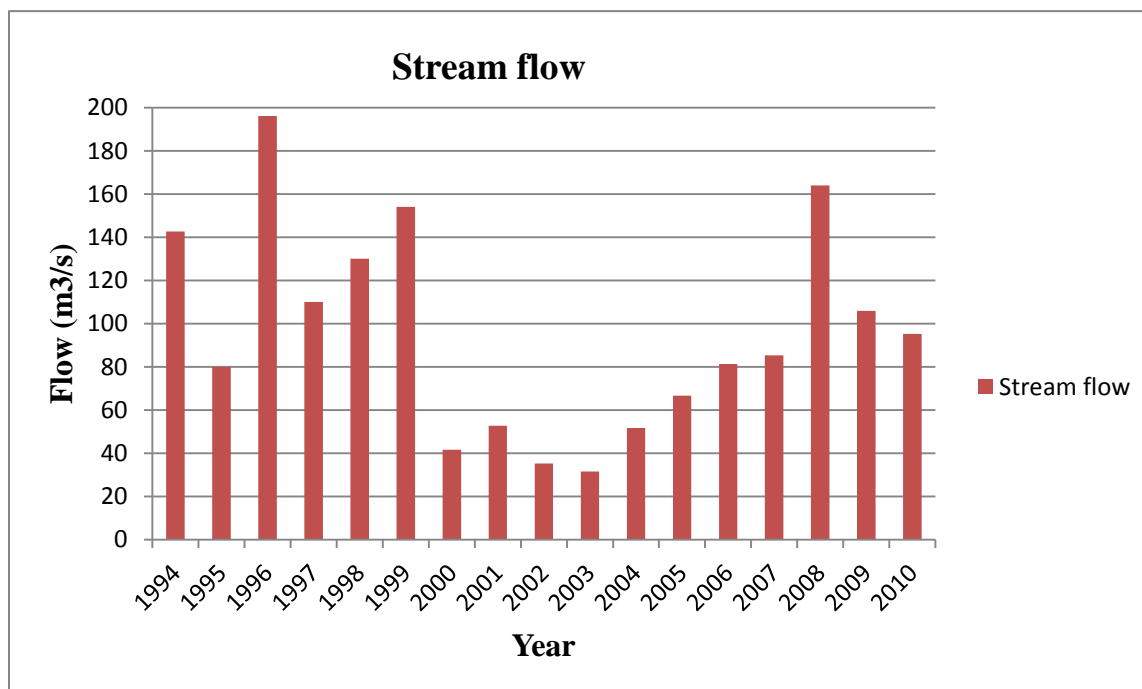


Figure 3-5: Annual Instantaneous daily stream flow data of Ginchi Awash River.

3.5. Soil

The soil of the district is classified in to three major soil types which are 58.2% of Eurtic Cambisols, 21.6% of Pellic Vertisols and 20.2% Eurtic Nitisols. Eurtic Cambisols is the soil which has a little agricultural value and it is the most erodible soil and because of the erodibility of the soil at the Bridge location the scouring is arisen at the abutment and the foundation of the bridge structure. Pellic Vertisols these soils are heavy clay soils in flat areas, having a pounced dry season during which they shrink and have deep cracks in a polygonal pattern. During the wet season the clay swells and causes pressure in the sub-soil. Pellic Vertisols are dark usually occupying areas which are water-logged during the rainy season. Another difficulty of Pellic Vertisols is that the permeability of the sub soils is very low which causes the rushing out and

having stagnant water during rainy season. And the third type of soil in the catchment is Eurtic Nitisols these soils are Nitisols with a base saturation of 50 percent or more. They are found on almost flat to sloping terrain in highlands and this type of soil are deep, clayey red soil and have very good physical properties. They have a uniform profile, are porous, have a stable structure and deep rooting volume and their moisture storage capacity is high.

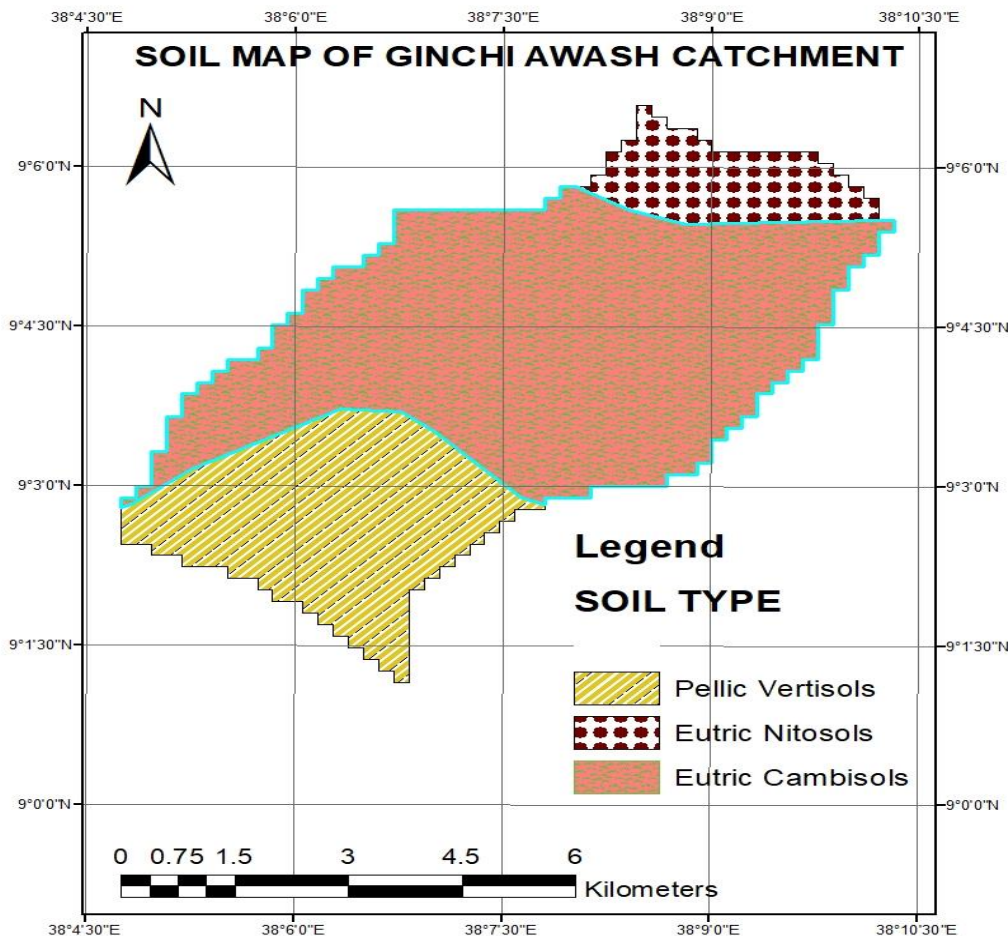


Figure 3-6: Map of soil type for Ginchi Awash Catchment.

3.6. Land use Land Cover

The descriptions of catchment using land use land cover dynamic are capable means of characterizing constantly changing catchment behavior on the basis of multiple considerations. Currently the general land use and land cover of the study area is classified

as three major land use land cover. This is characterized as Cultivated Land, Rainfed, Cereal Land cover system, Lightly stocked (CRCL) which covers 21.7% of the catchment area, Cultivated Land, Rainfed Land cover system, Moderately stocked (CRCM) which covers 37.1% of the catchment area and the third one is Forest, Montane Coniferous, Dense (FCD) which covers 41.2%. In the North Eastern part of the catchment, the land is covered by Chilimo Forest, dominantly with thin acacia trees and this place is found at the upside of the Ginchi Awash River which facilitates infiltration of precipitation into the ground rather it changes to runoff and flows to the stream.

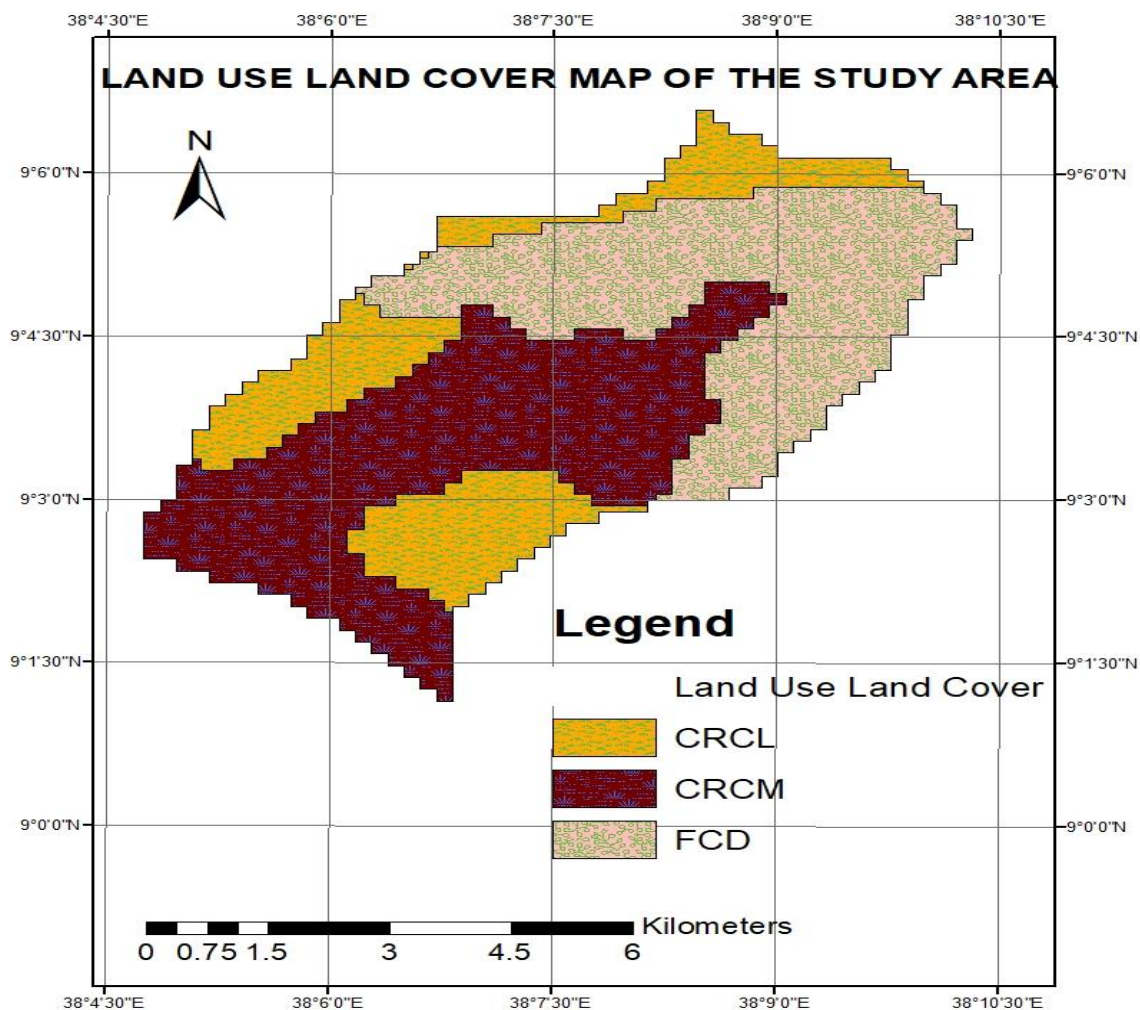


Figure 3-7: Map Land use Land cover for Ginchi Awash Catchment.

4.0. MATERIAL AND METHOD

4.1. Introduction

This chapter generally tries to discuss the methods that will be used in this thesis work, which include data analysis those emphasizes the assessment of the data collected from governmental institutions and field survey. The hydrological analysis methods of peak flood estimation is the second major part of this chapter which will be investigated by the flood frequency analysis method and after the hydrological analysis and all basic input parameters for the HEC-RAS model is completed the hydraulic analysis will take place. In the hydraulic analysis the steady flow water profile, sediment transport simulation and hydraulic bridge scour of Ginchi Awash Bridge will be studied.

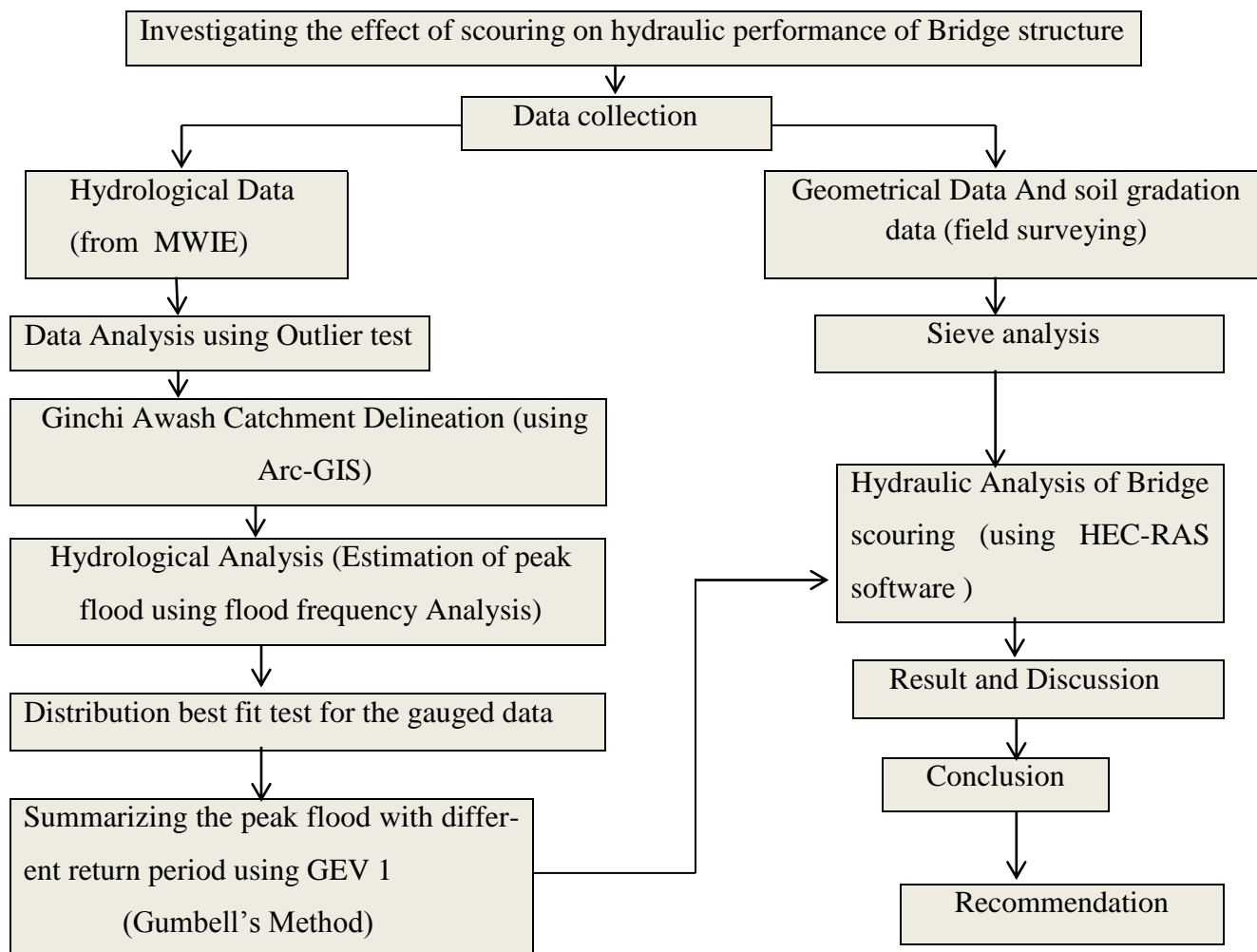


Figure 4-1: Conceptual Frame Work for the methodology.

4.2. Data analysis

Data analysis is the process of evaluating data using analytical and logical reasoning to examine each component of the data provided. This form of analysis is just one of the many steps that must be completed when conducting a research experiment. Data from various sources is gathered, reviewed, and then analyzed to form some sort of finding or conclusion. There are a variety of specific data analysis methods, some of which include test for outliers and goodness of distribution fit test by like Easy-fit software.

4.2.1. Quality Control

An outlier is an observation that deviates significantly from the bulk of the data, which may be due to errors in data collection, or recording, or due to natural causes. The presence of outliers in the data causes difficulties when fitting a distribution to the data. Low & high outliers are both possible and have different effect on the analysis. The Grubbs & Beck (1972) test (G-B) May be used to detect outliers. In this test the quantities XH & XL are calculated by using equations shown below:-

$$XH = \exp(\bar{X} + K_N \sigma) \quad 4.1$$

$$XL = \exp(\bar{X} - K_N \sigma) \quad 4.2$$

Where \bar{X} and σ are the mean and standard deviation of the natural logarithms of the sample respectively. The Grub and Beck statistic tabulated for various sample size and significance levels by Grubbs and Beck (1972). The following approximation proposed by Pilon et al. (1985) is used, where N is the sample size.

$$K_N = -3.62201 + 6.28446N^{1/4} - 2.49835N^{1/2} + 0.491436N^{3/4} - 0.037911N \quad 4.3$$

Sample values greater than XH are considered to be high outliers, while those less than XL is considered to be low outliers.

4.2.2. Best-Fit Distribution

Best fit distribution is all about the selection and evaluation of parent distribution. Era drainage manual recommended the tree distribution methods to estimate the peak discharge and in this study the selection of the best fit distribution from that tree distribution method is based on L-Moment Ratio Diagrams.

A diagram is based on $LC_s (\tau_3)$ versus $LC_k (\tau_4)$ such as that can be used similar to the conventional Moment Ratio Diagrams to identify appropriate distribution for the sample. L-moments obtain their name from their construction as linear combinations of order statistics (Hosking and Wallis).

The L-moment measure of location and L-moment ratio measures of scale, skewness and kurtosis are:

Location, mean:

$$\text{Mean} = L_1 \quad 4.4$$

Scale, L-Cv (τ_2):

$$\tau_2 = L_2/L_1 \quad 4.5$$

L- Skewness (τ_3):

$$\tau_3 = L_3/L_2 \quad 4.6$$

L-Kurtosis (τ_4):

$$\tau_4 = L_4/L_2 \quad 4.7$$

Where $L_1 = \beta_0 \quad 4.8a$

$$L_2 = 2\beta_1 - \beta_0 \quad 4.8b$$

$$L_3 = 6\beta_2 - 6\beta_1 + \beta_0 \quad 4.8c$$

$$L_4 = 20\beta_3 - 30\beta_2 + 12\beta_1 - \beta_0 \quad 4.8d$$

And, where the data (x_{1-n}) are first ranked in ascending order from 1 to n and:

$$\beta_0 = \frac{1}{n} \sum_{j=1}^n X_j \quad 4.8e$$

$$\beta_1 = \frac{1}{n} \sum_{j=2}^n X_j [(j-1)/(n-1)] \quad 4.8f$$

$$\beta_2 = \frac{1}{n} \sum_{j=3}^n X_j [(j-1)(j-2)/[(n-1)(n-2)]] \quad 4.8g$$

$$\beta_3 = \frac{1}{n} \sum_{j=4}^n X_j [(j-1)(j-2)(j-3)/[(n-1)(n-2)(n-3)]] \quad 4.8h$$

Some useful relationships for constructing the L-moment ratio diagram for some common distribution are given by:

For Gumbels Extreme Value-I (GEV I) distribution method the L-Kurtosis is:

$$\tau_4 = 0.10701 + 0.1109(\tau_3) + 0.84838(\tau_3)^2 - 0.06669(\tau_3)^3 + 0.00567(\tau_3)^4 - 0.04208(\tau_3)^5 + 0.03763(\tau_3)^6 \quad 4.9$$

For Log-Normal distribution method the L-Kurtosis is:

$$\tau_4 = 0.12282 + 0.77518(\tau_3)^2 + 0.12279(\tau_3)^4 - 0.13638(\tau_3)^6 + 0.113638(\tau_3)^8 \quad 4.10$$

For Log-Pearson Type III distribution method the L-Kurtosis is:

$$\tau_4 = 0.1224 + 0.30115(\tau_3)^2 + 0.95812(\tau_3)^4 - 0.13638(\tau_3)^6 + 0.19383(\tau_3)^8 \quad 4.11$$

4.3. Hydrological Method

The first analysis process in the hydraulic design for a bridge is the hydrological analysis or the calculation of design flood and sometimes discharges for historical floods. The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental for the design of highway drainage structures. Errors in the estimates will result in a structure that is either undersized and causes more drainage problems or oversized and costs more than necessary. There are different methods in hydrological analysis of bridge structures and those methods are depend up on the availability and length of available stream flow data and metrological data for either the bridge site itself, elsewhere on the river or on a nearby similar catchment. The general methods are as follows:

4.3.1. Flood Frequency Analysis

At site flood frequency analysis is used if there is a long term stream gauge located near the bridge site. If the gauge is located close to the bridge site, say with a difference in catchment area of less than 10%, the gauge can be used directly, with the calculated discharges modified for the small differences in catchment area. If the gauge is located further away, the flood frequency analysis can be used to help estimate parameters for a runoff routing model. Flood frequency analysis is usually the most reliable method for calculating design flood discharges (Hydraulic Guidelines for Bridge Design).

In this research since the stream at the bridge site is gauged and the gauge is located close to the Bridge site, flood frequency analysis or statistical method of estimating peak discharge will be used, because of ERA drainage design manual recommend that the statistical analysis to estimate peak flood for the gauged site of stream flow record of sufficient length. The following three

methods are the statistical distribution methods which are used for calculation of peak discharge from stream flow data. But before using the methods it is must to decide of choosing only one method depend up on the best fit test and goodness-of-fit test of the distribution method and the method of choosing distribution is discussed under data analysis part.

In this thesis work the three major flood frequency analysis methods are selected based on the ERA Drainage manual which adopted to estimate the design flood. These methods are General Extreme Value I (GEV-I), Log-Pearson type III and Log-Normal.

4.3.1.1. General Extreme Value 1 (Gumbell’s) Distribution Method

This method was introduced by Gumbel (1941) and commonly known as Gumbel’s distribution. It is one of the most widely used probability distribution function for extreme values in hydrological and metrological studies for prediction of peak discharge. Gumbel’s equation for practical use is:

$$X_T = \bar{X} + K \sigma_{n-1} \tag{4.12}$$

Where σ_{n-1} = standard deviation of the sample of size N = $\sqrt{\frac{\sum(X-\bar{X})^2}{N-1}}$

K = frequency factor expressed as $K = \frac{Y_T - \bar{Y}_n}{S_n}$

Y_T = reduced variate, a function of T and given by $Y_T = -\left[\ln. \ln \frac{T}{T-1}\right]$

Or $Y_T = -\left[0.834 + 2.303 \log. \log \frac{T}{T-1}\right] \tag{4.13}$

\bar{Y}_n = reduced mean a function of sample size N.

S_n = reduced standard deviation, a function of sample size N.

4.3.1.2. Log-Pearson type III Distribution Method

This distribution is extensively used in USA for project sponsored by the US Government. In this the first the variate is first transformed into logarithmic form (base 10) and the transformed data is then analyzed. If X is the variate of a random hydrologic series, then the series of variates

Where

$$Z = \log x \tag{4.14}$$

are first obtained. For this Z series, for any recurrence interval T.

$$Z_T = \bar{Z} + K_Z \sigma_Z \tag{4.15}$$

Where K_Z = a frequency factor which is a function of recurrence interval T and the Coefficient of skew C_s .

σ_Z = standard deviation of the Z variate sample.

$$= \sqrt{\sum (z - \bar{z})^2 / (N - 1)} \quad 4.16$$

C_s = coefficient of skew of variate Z

$$= \frac{N \sum (z - \bar{z})^3}{(N-1)(N-2)(\sigma_Z)^3} \quad 4.17$$

\bar{Z} = mean of the z values.

N = sample size = number of years of record.

4.3.1.3. Log-Normal Distribution Method

If the random variable $y = \log X$ is normally distributed, then X is said to be lognormal distributed. Chow (1954) reasoned that this distribution is applicable to hydrologic variables formed as the products of other variables since if: $X = X_1 X_2 X_3 \dots X_n$ and then $Y = \log X$, $X = \sum_{i=1}^n \log X_i = \sum_{i=1}^n Y_i$ which tends to the normal distribution for large z provided that the X_i are independent and identically distributed. The lognormal distribution has been found to describe the distribution of hydraulic conductivity in a porous medium. (Freeze, 1975)

Where

$$X_T = \text{antilog}(Y_T) \quad 4.18$$

$$Y_T = \bar{Y} + K_Y * \delta_Y \quad 4.19$$

$$K_Y = W - \left[\frac{2.516 + 0.8028W + 0.0103W^2}{1 + 1.4328W + 0.1893W^2 + 0.0013W^3} \right] \quad 4.20$$

Where K_Y is a function of w.

$$W = \left[\ln \left(\frac{1}{p^2} \right) \right]^{\frac{1}{2}} \quad 4.21$$

When $p > 0.5$, $1-p$ is substituted for p.

$$\bar{Y} = \sum Y / N \quad 4.22$$

$$\sigma_{n-1} = \sqrt{\frac{\sum (X - \bar{X})^2}{N-1}} \quad 4.23$$

Where σ_{n-1} = standard deviation of sample N
 X = maximum discharge
 \bar{X} = mean discharge

4.4. Hydraulic Method

There are different commonly worldwide adopted methods which are used to perform the hydraulic analysis of the bridge and culvert structures. These are HEC-RAS software and HY-8 respectively. In this study this software is used in the Hydraulic analysis of Ginchi Awash Bridge.

4.4.1. Hydraulic Model (HEC-RAS)

Hydraulic analysis will be performed in order to determine the steady flow water surface, scour depth of the bridge both at left and right abutment and the sediment transport analysis to know the bed elevation change. In addition to this hydraulic analysis is performed to calculate flow velocities for the bridge and nearby river reach. The most commonly used hydraulic analysis procedure used for routine bridge scour analysis is HEC-RAS, developed by the United States Army Corps of Engineers. This software is well suited to the hydraulic analysis of bridges and culverts. And this model is widely used around the world for such structures (ERA Drainage manual 2013)

4.4.2. Input Data for HEC-RAS Model

In this dissertation there are different input data for HEC-RAS model and it involves some procedures. The followings are some input data in HEC-RAS model.

4.4.2.1. Geometric Data:

Survey data collected from field at the bridge site by using survey instrument and field observation likes; ground surface profile, flow direction, reach length, elevation of the areas, Manning's n value, abutment condition, banks both right and left, and bridge alignment. Basic elements of geometric data are the following:

I. River cross section data

Boundary geometry for the analysis of flow in natural stream is specified in terms of ground surface profiles (cross section) and the measured distance between them (reach lengths). Cross sections are located at intervals along a stream to characterize the flow carrying capability of the stream and its adjacent floodplain. The required information for a cross section consists of: the river, reach and river station identifiers, station points and elevation, downstream reach lengths, Manning's roughness coefficients, main channel bank stations, and contraction and expansion coefficients. So as shown in the figure below using total station the geometry data is collected both at upstream and downstream of Ginchi Awash Bridge crossing. At upstream of the bridge crossing and downstream of the Bridge crossing, six and five cross sections are collected respectively with an interval of approximately 20m apart.



Figure 4-2: Surveying of Ginchi Awash River cross section data (April 02, 2018).

After data collection is completed the boundary of collected data is exported to Eagle Point software for further analysis. Eagle point software is used to generate the contour map of the area and also to generate cross section data with further station numbers at the desired locations. The station number and the elevation data were exported to excel and then to HEC-RAS.

II. Manning's Roughness coefficient Value n

For Main channel

An effective approach to estimate the Manning's roughness in river is to select a grain roughness value of Manning's n for the bed material and then add other components of roughness to the

grain roughness. The grain roughness is represented by the size and shape of grains of that from the stream bed. The Strickler equation is used to determine the Manning's roughness for bed material of D_{50} is less than 4mm. The following is Strickler equation which is used to calculate roughness value of Manning's n :

$$n = 0.0132(D_{50})^{\frac{1}{6}}$$

4.24

Where n = Manning's roughness value for main channel of the River

D_{50} = median diameter of the bed material, mm (which is 3mm in case of this paper).

So Manning's roughness for main channel is, $n=0.016$ for main channel is used in HEC-RAS software.

For River banks

At study sites on determination of Manning's roughness n value for stream widths are less than 30m, vegetation (brush, grass, and trees) that covers more than 25 percent of the wetted perimeter causes the computed roughness coefficient to increase by 0.002 to 0.012 during the growing season (William F. Coon, 1998). 0.012 is taken in case of this paper.

$$n = 0.0132(D_{50})^{\frac{1}{6}} + 0.012$$

Where

n = Manning's roughness value for main channel of the River.

D_{50} = median diameter of the River bank material, mm (which is 0.95mm in case of this paper).

III. Contraction or Expansion Coefficient

Contraction or expansion of flow due to changes in the cross section is common cause of energy losses within a reach (between two cross sections). Around bridge location since the flow is abrupt contraction and expansion coefficients of 0.3 and 0.5 are used respectively. While for

others cross section as the change of flow is small, coefficients of contraction and expansion are typically 0.3 and 0.1 respectively.

4.4.2.2. Soil Sample and Gradation Data

Soil sample and gradation data is used in grain size and percent of finer computation which is the basic input in HEC-RAS software during the sediment transport and Bridge scour analysis. This data was collected during field surveying and the soil samples under the bridge bed and on the upstream of right and left bank were collected. The mechanical sieve analysis is performed to determine the distribution of the coarser, large-sized particles and the finer particles. The following figure shows that the sieve analysis provision in Addis Ababa institute of Technology soil lab.



Figure 4-3: Sieve Analysis of Grain size distribution of River bed and River Bank channel at Addis Ababa institute of Technology soil lab (April 02, 2018).

4.4.2.3. Salient Features of Bridge Structures

In HEC-RAS, the bridge dimensions must be defined in order to conduct the analysis. The Ginchi Awash Bridge is built in 1939, located west of Addis Ababa, Ethiopia. It spans over the Ginchi Awash River and used to carry traffic from Ginchi to Ambo town. The Ginchi Awash Bridge is a single span arch concrete bridge and is 19.5m long and 9.5m wide (source, ERA, Bridge management). The bridge is supported by two abutments. The overall dimensions of the bridge are summarized in Table 4-1.

Table 4-1: Ginchi Awash Bridge Dimensions.

Bridge Dimensions (m)	
Bridge Overall Length	19.5
Deck Height	5.3
Deck width	9.5
Height between high chord and low cord	1.15

4.4.3. Steady Flow Water Surface Profile computation in HEC-RAS

This component of the model is intended to calculate water surface profiles for steady gradually varied flow. It is also capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles. The basic computational procedure is based on the solution of the one-dimensional energy equation and momentum equations (Hydraulic Reference Manual, 2010). In case of this thesis work because of the bridge do not have pier the Energy equation will be used in order to calculate water surface profile.

The water surface profiles are computed from one cross section to the next by solving the energy equation with an iterative procedure called the standard step method. The energy equation based on Figure 4-5 is written as follows:

$$Z_2 + Y_2 + \frac{a_2 V_2^2}{2g} = Z_1 + Y_1 + \frac{a_1 V_1^2}{2g} + h_e \quad 4.25$$

Where:

Z_1, Z_2 = elevation of main channel inverts.

Y_1, Y_2 = depth of water at cross sections

V_1, V_2 = average velocities.

a_1, a_2 = velocity weighting coefficient.

h_e = energy head loss.

This can be further simplified using the figure given below;

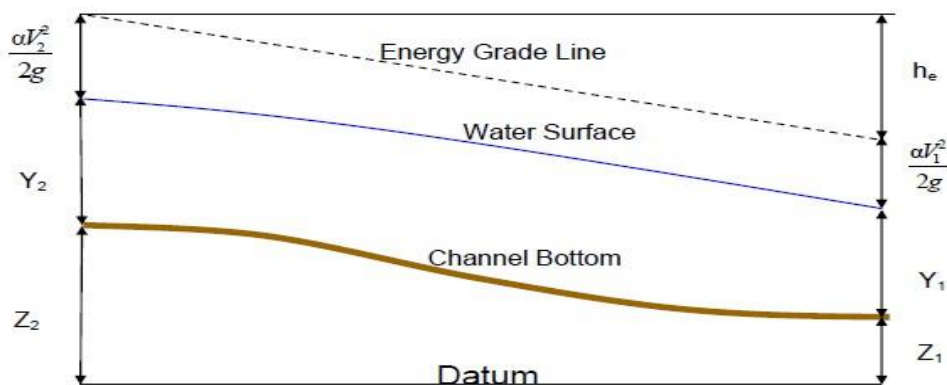


Figure 4-4: Representation of terms in energy equation (Source: Gray, 2010).

The energy head loss (h_e) can be estimated by adding friction head loss (h_f) and contraction/expansion head loss (h_{ce}). Friction head loss is a product of energy grade line slope (S_f) and length of the channel between each cross section. The energy head loss can be expressed using Equation 4.17;

$$h_e = LS_f + c \left[\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g} \right] \quad 4.26$$

Where

h_e = energy head loss

L = discharge weighted reach length.

S_f = representative friction slope between two sections.

C = expansion or contraction loss coefficient.

V_1, V_2 = average velocities (total discharge / total flow area).

g = gravitational acceleration.

α_1, α_2 = velocity weighting coefficients.

Stream cross sections may not always taken perpendicular to the flow direction. Due to this left and right side of the channel will have different length. For this reason HEC-RAS considers weighted channel length during computation process and expressed in Equation 4.18 below.

$$L = \frac{L_{lob}Q_{lob} + L_{ch}Q_{ch} + L_{rob}Q_{rob}}{Q_{lob} + Q_{ch} + Q_{rob}} \quad 4.27$$

Where:

L = discharge weighted reach length

L_{lob} , L_{ch} , L_{rob} , are left overbank, main channel and right overbank length respectively.

Q_{lob} , Q_{ch} , Q_{rob} , are discharges at left overbank, main channel and right overbank section of the channel respectively.

The energy grade line and the velocity head can also be determined from Manning's equation applying successive iterations:

$$Q = kS_f^{\frac{1}{2}} \quad 4.28$$

Where:

Q = discharge in the channel

$$K = \text{conveyance constant}, \quad k = \frac{1}{n} AR^{\frac{2}{3}} \quad 4.29$$

S_f = friction slope

During computation of contraction and expansion losses the program assumes that as contraction is occurring whenever the velocity head downstream is greater than the velocity head upstream. Likewise, when the velocity head upstream is greater than the velocity head downstream, the program assumes that a flow expansion is occurring (Gray, 2010).

4.4.4. Bridge Scour Analysis Using HEC-RAS Model

The scour computations in HEC-RAS are comprised of three components: Contraction scour, Local scour at Pier, and Local scour Abutment. But in case of this study since the bridge do not consist of pier the only consideration is contraction scour and local abutment scour. This computation is based upon the methods outline in Hydraulic Engineering Circular No. 18 (FHWA, 2012). In order to perform a bridge scour analysis first we need to develop hydraulic model of the river reach containing the bridge to be analyzed. The model must include the downstream and upstream cross section of the river, in order to evaluate the long term effects of the bridge on the water surface profile.

4.4.4.1. Contraction Scour

Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction or by a bridge. From continuity, a decrease in flow area results in an increase

in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction and more bed material is removed from the contracted reach than is transported into the reach. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases and, in the riverine situation, the velocity and shear stress decrease until relative equilibrium is reached; i.e., the quantity of bed material that is transported into the reach is equal to that removed from the reach.

Contraction scour equations are based on the principle of conservation of sediment transport. In the case of live-bed scour, this simply means that the fully developed scour in the bridge cross-section reaches equilibrium when sediment transported into the contracted section equals sediment transported out. As scour develops the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity. For live-bed scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance. For clear-water scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material.

To determine if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c and compare it with the mean velocity V of the flow in the main channel or overbank area upstream of the bridge opening. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V$), then clear-water contraction scour will exist. If the critical velocity is less than the mean velocity ($V_c < V$), then live-bed contraction scour will exist. To calculate the critical velocity the following equation will be used. Neill's equation with S_s equal to 2.65

$$V_c = 11.52 Y_1^{\frac{1}{6}} D_{50}^{\frac{1}{3}} \quad 4.30$$

Where

V_c = critical velocity which will transport bed materials of size D_{50} and smaller, m/s

S_s = specific gravity of bed material

y_1 = depth of upstream flow, m/s.

Live-bed contraction equation

A modified version of Laursen's 1960 equation below for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section. The original equation is given with the consideration of Manning's n . The modification is to eliminate the ratio of Manning's n . The HEC-RAS model assumes the following equations for bed material is being transported in the upstream section.

$$y_2 = y_1 \left(\frac{Q_2}{Q_1} \right)^{\frac{6}{7}} \left(\frac{W_1}{W_2} \right)^{K_1} \quad 4.31$$

$$y_s = y_2 - y_0 \quad 4.32$$

Where y_s = Average depth of contraction scour ,m.

y_1 = average depth in the upstream main channel, m.

y_2 = average depth in the contracted section, m.

y_0 = Average depth in the main channel or in flood plain at the contracted section before Scour, m.

W_1 = bottom width of the upstream main channel, m

W_2 = bottom width of main channel in the contracted section, m

Q_1 = flow in the upstream channel transporting sediment, m^3/s

Q_2 = flow in the contracted channel, m^3/s . Often this is equal to the total discharge unless the total flood flow is reduced by relief bridges or water overtopping the approach roadway.

n_2 = Manning's n for contracted section

n_1 = Manning's n for upstream main channel

K_1 & K_2 = exponents determined below depending on the mode of bed material transport

Table 4-2: Exponent for mode of bed material transport, K1.

V_*/w	K_1	K_2	Mode of Bed Material Transport
<0.50	0.59	0.066	Mostly contact Bed Material
0.50 to 2.0	0.64	0.21	Some suspended Bed Material discharge
>2.0	0.69	0.37	Mostly suspended Bed Material discharge

$V_* = (gy_1S_1)^{1/2}$ shear velocity in the upstream section, m/s

W = median fall velocity of the bed material based on the D_{50}

g = acceleration of gravity (32.2 m/s^2)

S_1 = slope of energy grade line of main channel, m/m

D_{50} = median diameter of the bed material, mm

4.4.4.2. Local Scour

Local scour involves removal of material from around piers, abutments, spurs, and embankments. The basic mechanism causing local scour at piers or abutments is the formation of vortices (known as the horseshoe vortex) at their base. The horseshoe vortex results from the pileup of water on the upstream surface of the obstruction and subsequent acceleration of the flow around the nose of the pier or embankment. The action of the vortex removes bed material from around the base of the obstruction. The transport rate of sediment away from the base region is greater than the transport rate into the region, and, consequently, a scour hole develops. As the depth of scour increases, the strength of the horseshoe vortex is reduced, thereby reducing the transport rate from the base region (HEC-18).

Local Scour at Abutments

Abutment scour depends on the interaction of the flow obstructed by the abutment and roadway approach and the flow in the main channel at the abutment. The discharge returned to the main channel at the abutment is not simply a function of the abutment and roadway length in the field case.

As a check on the potential depth of scour to aid in the design of the foundation and placement of rock riprap or guide banks, Froehlich’s live-bed scour equation or an equation from HIRE can be used in HEC-RAS model so in case of this paper Froehlich’s live bed scour equation is preferred on the software because of the wetted embankment length divide by approach depth both at left and right abutment for all design flood case is less than 25 and HEC-18 recommend Froehlich’s equation for such case.

$$y_s = 2.27K_1K_2(L')^{0.43}F_r^{0.61} + y_a \tag{4.33}$$

Where K_1 = correction factor for abutment shape (see Table 4-3)

K_2 = coefficient for angle of embankment to flow = $(\theta/90)^{0.13}$

L' = length of abutment (embankment) projected normal to flow, m

A_e = the flow area of the approach cross section obstructed by the embankment, m^2

F_r = Froude Number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$

V_e = Q_e/A_e , m/s

Q_e = the flow obstructed by the abutment and approach embankment, m^3/s

y_a = average depth of flow on the floodplain, m

y_s = scour depth, m

Table 4-3: Correction factor for abutment shape, K_1 .

Description	K_1
Vertical-wall Abutment	1.00
Vertical-wall Abutment with wing wall	0.82
Spill-through Abutment	0.55

Since the Ginchi Awash Bridge abutment is vertical-wall abutment with wing wall $K_1=0.82$ is used in this investigation.

The correction factor K_2 , for angle of attack can be taken from Figure 4-6.

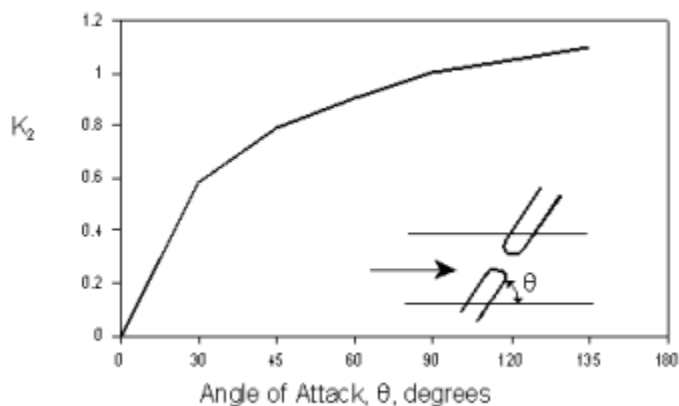


Figure 4-5: Correction for Abutment Skew, K_2 .



Figure 4-6: Skew angle and Bridge abutment alignment with respect to the flow direction.

But in case of this paper the abutment skew is 45 degree at right and left abutment. So for the right abutment $K_2 = (\theta/90)^{0.13} = (42/90)^{0.13} = 0.91$.

4.4.5. Sediment Transport

This component of the modeling system is intended for the simulation of one dimensional sediment transport or movable boundary calculations resulting from erosion and deposition over moderate time periods typically years, although applications to single flood events also possible (Hydraulic Reference Manual 2010). The model is designed to simulate long-term trends of

erosion and deposition in a stream channel that might result from modifying the frequency and duration of the water discharge and stage, or modifying the channel geometry. This model is suitable to determine the rate of sedimentation at fixed channel cross-sections including Bridge cross-section which is the aim of this study.

To model a sediment transport event, HEC-RAS uses a quasi-unsteady flow series. A quasi-unsteady flow series is created from a storm hydrograph by dividing the hydrograph into flow duration steps, in which the flow, stage, temperature, and sediment loading are considered constant. The flow durations are further broken down into computation increments, in which the bed elevation and hydrodynamics are updated after each increment. Finally, computational increments are further subdivided into the bed mixing time step to consider change in the composition of bed material due to removal or addition of materials. Therefore, rather than modeling a constantly changing flow hydrograph, HECRAS models a constant flow over the flow duration, and only allows bed elevation and hydrodynamics to change between each computational increment. This approach helps to increase model stability when simulating sediment transport events.

Between each cross section, HEC-RAS solves for sediment continuity using the Exner Equation. The Exner equation is used by calculating a sediment transport capacity within the active layer of the channel based on the current hydrodynamics of the cross section.

$$-\frac{\partial Q_s}{\partial X} = (1 - \lambda_p)\beta \frac{\partial \eta}{\partial t} \quad 4.34$$

Where: B= channel width

η = channel elevation

λ_p = active layer porosity

t= time

x= distance and,

Q_s = transported sediment load

This equation simply states that the change of sediment volume in a controlled section is equal to the difference between the inflowing and out flowing loads. The sediment continuity equation is solved by computing transport capacity through the control volume associated with each cross section. This capacity is compared to the sediment supply entering the control volume. If the capacity is greater than supply there is a sediment deficit which is satisfied by eroding bed sediments. If supply exceeds capacity there is a sediment surplus causing material to deposit (Hydraulic Reference Manual 2010).

HEC-RAS determines transport potential for a single representative soil grain. Transport potential is the hydrodynamic capacity of the water to transport a single grain size sediment material. There are a number of equations developed to determine the transport potential of flowing water based on range of grain size distribution which is available on HEC-RAS Technical Reference Manual, (2010).

After the appropriate transport potential function selected the next step in HECRAS will be determining the transport capacity in order to determine single representative sediment transport model of the system. This is done by multiplying the transport potential for each grain class and percentage of that grain class in the bed and summarized in Equation below.

$$T_c = \sum_{j=1}^n B_j T_j \quad 4.35$$

Where: T_c =total transport capacity

n =the number of grain size class

β =percentage of the material in the particular grain size class

Using the output from this Equation and solving the Exner continuity Equation HEC-RAS solves for Bed elevation change in stream channels.

4.4.6. Procedures in HEC-RAS Model

After all the necessary data used in the model are collected the followings are the procedures to run all the required hydraulic analysis of the Bridge structure.

To run the steady flow analysis first the geometric data should be inserted. In order to insert geometric data first River name (Awash) and the reach (Ginchi Awash) are defined. Once river and reach name defined simple layout of the River plotted on the HEC-RAS window. Then cross section editor window opened. Once the geometry of the cross sections were surveyed and determined using Eagle Point software then the cross section output transferred to excel software and after that cross section data exported to HEC-RAS geometric data editor. In this part of the software component the station and elevation of cross sections, the distance between the banks, the manning's roughness coefficient, expansion and contraction coefficients were all inserted.

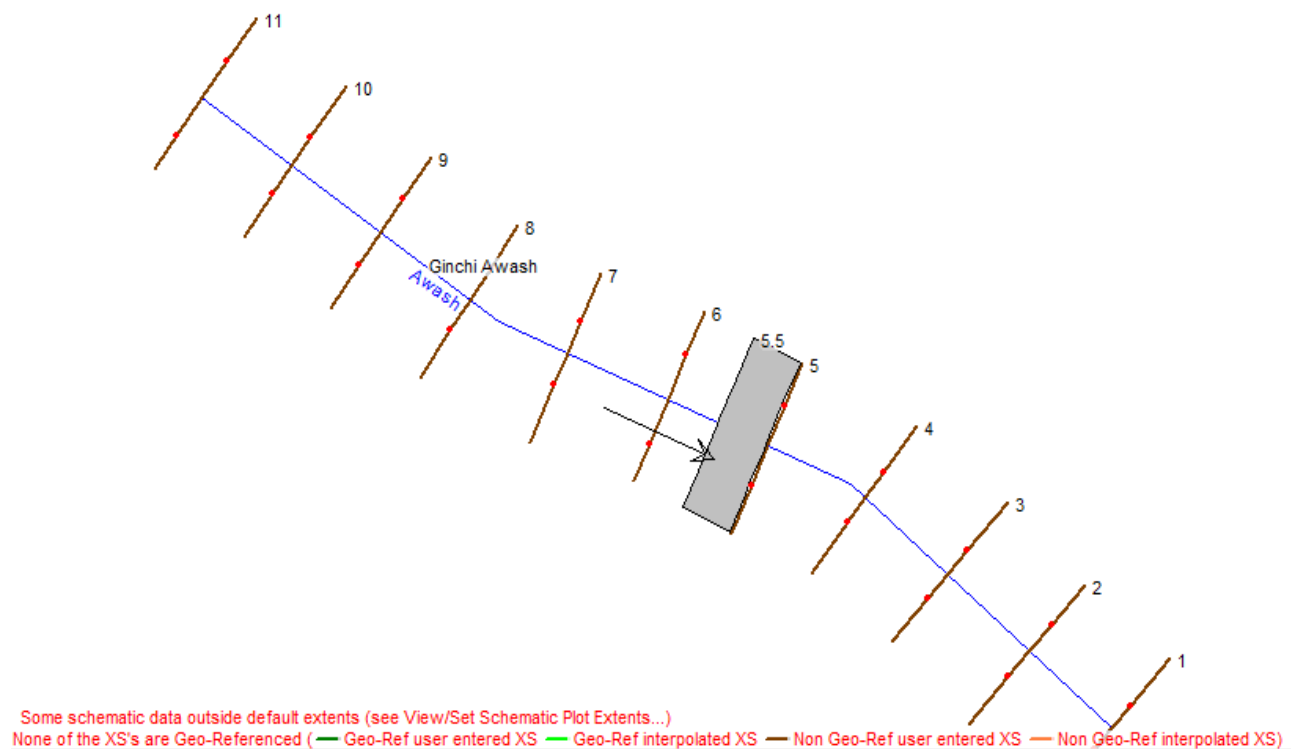


Figure 4-7: HEC-RAS River reach and cross section representation.

The bridge cross section and layout is inserted in the bridge and culvert data editor window. All necessary cross section data of the bridge which are used to represent the actual bridge shape were inserted in this part of the software component. The bridge length, the bridge height, high and low chords of the bridge, width of the bridge finally the abutments were all inserted in this editor.

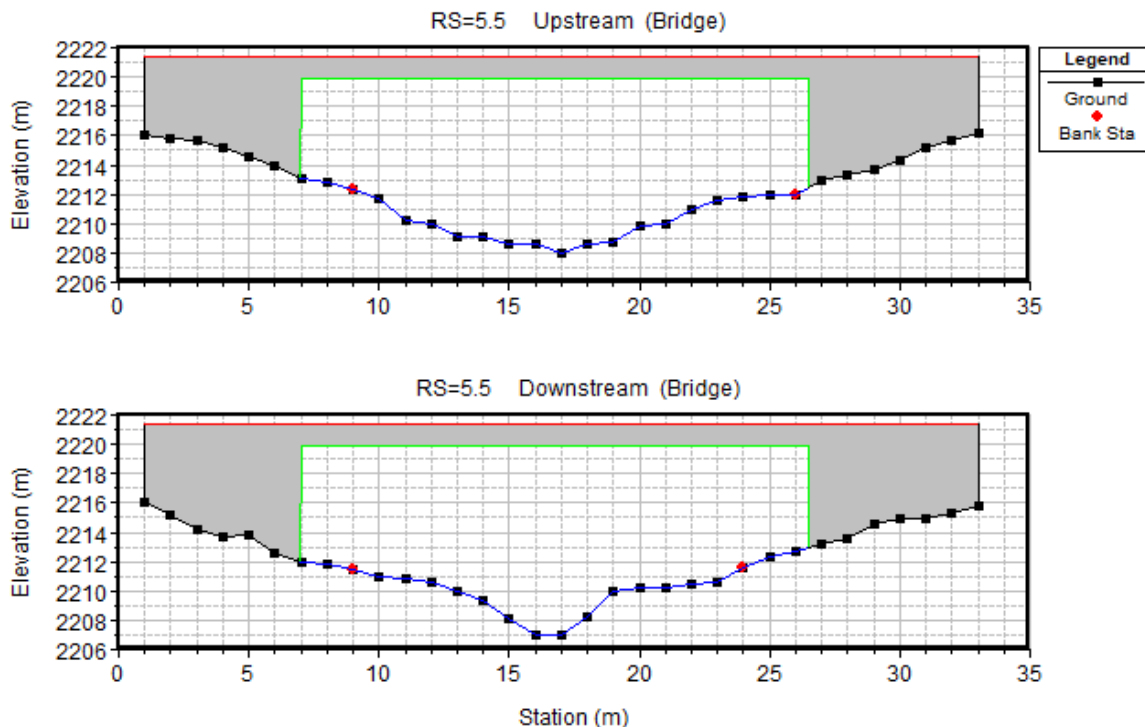


Figure 4-8: Bridge Data Editor on HEC-RAS Model.

After carrying out all the necessary geometric input data then the estimated design flood by flood frequency analysis results are inserted into the steady flow data editor section. The boundary conditions for the model also selected from the reach boundary condition editor tab. For the case of this study normal depth boundary condition was selected since no observed data was available. Normal depth assumes slope of the river bed is equivalent to the energy grade line of the flowing water. Slope of the river bed is determined by dividing the elevation difference of river bed elevation at upstream and downstream of the bridge by the respective channel length difference. Finally from Run tab plan was created and steady flow analysis is carried out and the outputs were collected in the view tab.

The second major requirement of this thesis work is determining movable bed load or sediment analysis at specific cross section locations. To simulate the sediment transport, first sediment data is inserted into the model using sediment data editor window. Sediment data includes the initial conditions, transport parameters, transport functions, soil sample gradation results and for the boundary conditions sediment series data that gathered from Ministry of Water, Irrigation

and Electric is used. For the transport parameter Laursen method is selected because of only Laursen equation of transport function in HEC-RAS consider soil gradation up to silt range.

The next essential step during sediment analysis was to determine and insert the quasi-unsteady flow data. The current version of the HEC-RAS model is only capable of modeling sediment transport by converting the unsteady flow hydrograph into a series of blocks so that every sediment calculation remains constant for the defined duration. For the case of this thesis the design peak discharges will be used in order to visualize their impact on the Bridge cross section. The final step is running the software and collecting both tabulated and graphical outputs of the model.

The third and basic of this thesis work is computing the scour depth of the Bridge both at left and right abutment. To compute the scour depth of bridge first open Hydraulic Design tab then selecting the bridge scour from type section then all data are avail by default. But the values of D50 and K_1 are inserted manually.

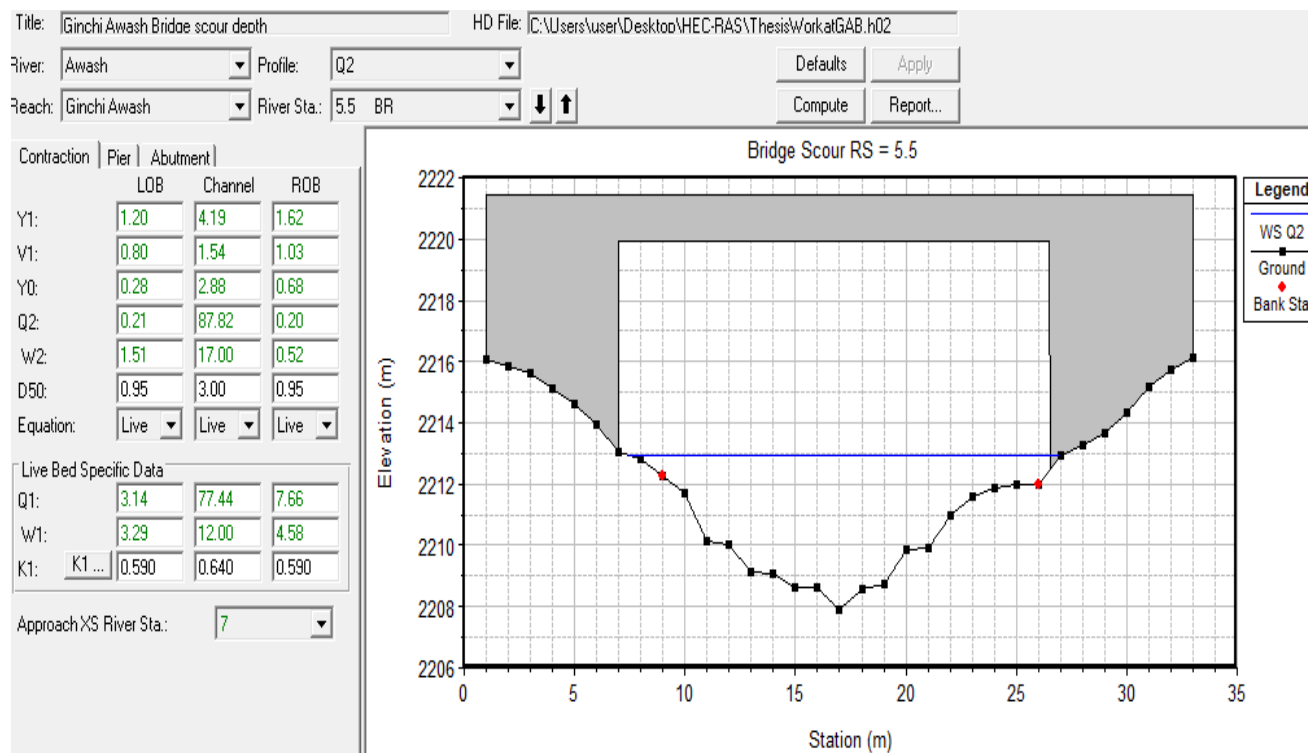


Figure 4-9: Hydraulic Design Editor Window for Scour at Bridge.

5.0. RESULT AND DISCUSSION

5.1. Data Analysis

Data analysis is the process of evaluating data using analytical and logical reasoning to examine each component of the data provided. So in this research the hydrological data which means the stream gauged of 17 years recorded data is analyzed by using the following tests.

5.1.1. Test for Outlier

As outliers affect the statistics, especially higher order statistics, is recommended to test for outliers and eliminating the higher and lower data before using it in the flood frequency analysis of estimating the design flood. So by using the equation discussed under the method section of this paper the data is analyzed by outlier test and the entire stream data gathered is checked not to be higher and lower to use in hydrological analysis.

Table 5-1: Data Analysis for Outlier test.

No Rec	Year	X(Max Q)	Y=Log X	(Y-y) ²	(Y-y) ³	Outlier	
						Higher	Lower
1	1994	82.79	1.92	0.05394	0.012529	Ok	Ok
2	1995	46.44	1.67	0.00036	-0.000007	Ok	Ok
3	1996	113.87	2.06	0.13741	0.050934	Ok	Ok
4	1997	63.88	1.81	0.01432	0.001713	Ok	Ok
5	1998	75.46	1.88	0.03687	0.007080	Ok	Ok
6	1999	89.41	1.95	0.07059	0.018753	Ok	Ok
7	2000	24.16	1.38	0.09156	-0.027707	Ok	Ok
8	2001	30.59	1.49	0.04008	-0.008023	Ok	Ok
9	2002	20.40	1.31	0.14152	-0.053236	Ok	Ok
10	2003	18.24	1.26	0.18034	-0.076582	Ok	Ok
11	2004	30.01	1.48	0.04342	-0.009049	Ok	Ok
12	2005	38.66	1.59	0.00970	-0.000956	Ok	Ok
13	2006	47.19	1.67	0.00014	-0.000002	Ok	Ok
14	2007	49.49	1.69	0.00008	0.000001	Ok	Ok
15	2008	95.178	1.98	0.08575	0.025109	Ok	Ok
16	2009	61.432	1.79	0.01054	0.001083	Ok	Ok
17	2010	55.25	1.74	0.00321	0.000182	Ok	Ok

Total	=	942.44	28.66	0.919815	-0.058178
Mean	=	55.44	1.69		
St.Dev	=	28.20	0.24		
Skew	=		-0.30		
Coff. Var.	0.491				
Stan.Error	11.90				

5.1.2. Best-Fit Distribution Result

Based on the result from the L-Moment Ratio Diagram the flood frequency analysis of distribution method is decided. This method of selecting and evaluating the best fit distribution shows that the Best-Fit distribution for the gathered stream flow data is Extreme Value 1 (Gumbell's Distribution method).

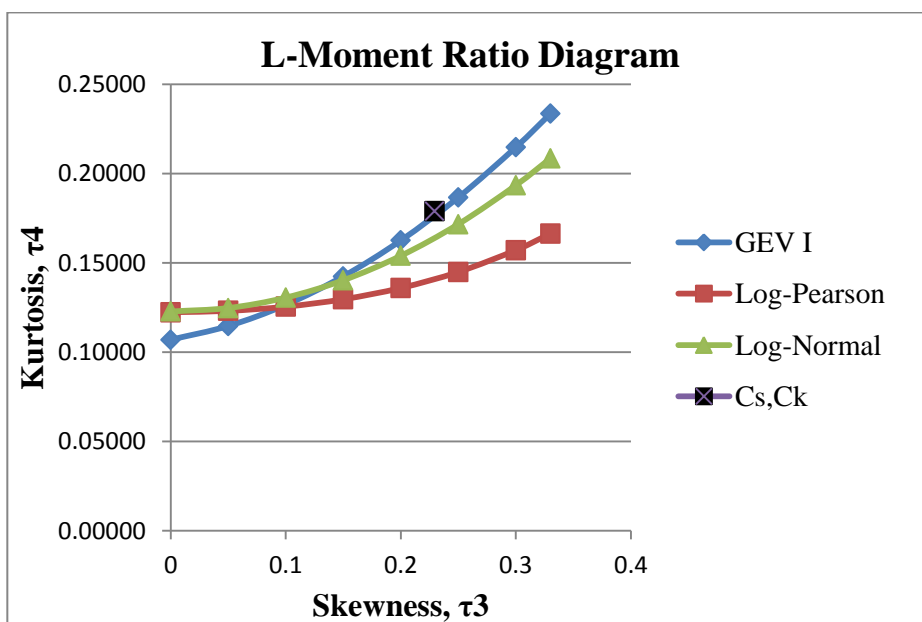


Figure 5-1: Data checking for Best-Fit Distribution

5.2. Hydrological Analysis

5.2.1. Delineation of the Catchment

The catchment delineation is performed in Arc view GIS 10.3 with GIS Hydrology extensions using the Digital Elevation Model generated by SRTM 90mx 90m Grid digital topographic map which gives usually accurate catchment delineations. The Ginchi Awash River catchment covers

47.9km² areas at the bridge crossing outlet location and its elevation varies from 3000 m at the highest point and 2250 m at the crossing location. Under the catchment delineation the Topography, Soil type and Land -Use Land-Cover is also considered and is sited under study area description part of this paper.

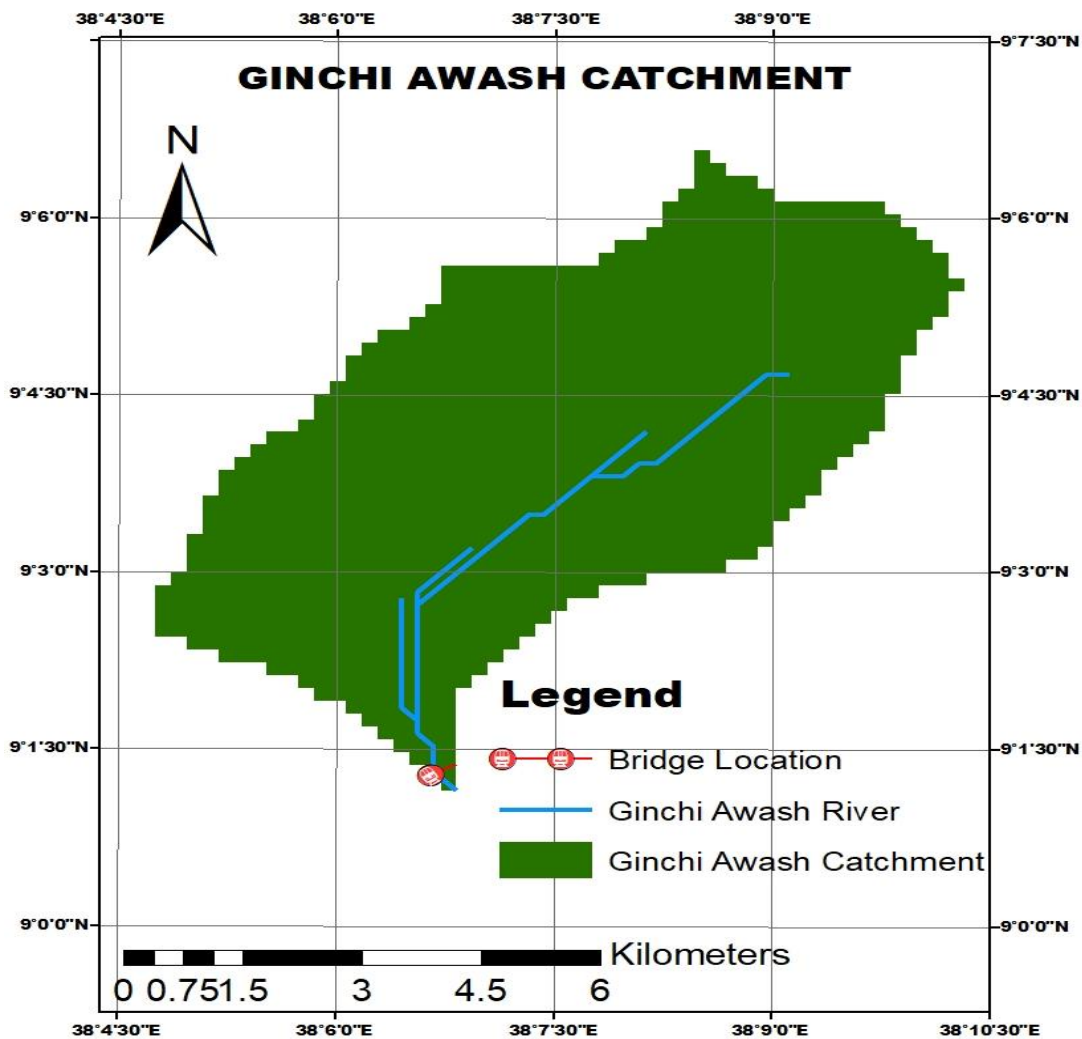


Figure 5-2: Ginchi Awash Catchment.

5.2.2. Computation of Peak Discharge

Based on 17 years of recorded annual daily instantaneous stream flow data at Ginchi Awash River the peak discharge for five different return periods were determined using flood frequency analysis methods and tabulated in Table 5-2. The detail calculation is found on Appendix B.

Table 5-2: Estimation of Peak discharge Using General Extreme Value, Log-Pearson type III and Log-Normal distribution methods.

Maximum discharge corresponding to each return period			
Return period	Log-Normal (m ³ /s)	GEV 1(m ³ /s)	Log-Person III(m ³ /s)
Q2	83.537	88.237	85.880
Q10	169.517	178.478	166.118
Q25	219.661	223.897	206.940
Q50	239.101	257.591	237.173
Q100	280.259	291.037	266.917

From Data checking of Best-fit test by using L-Moment ratio diagram (Figure 5-1) General Extreme Value (Gumbell's) is the best-fit distribution method for the available hydrological data of Ginchi Awash River. So based on this analysis the following peak discharges are summarized with respect to their return period.

Table 5-3: Summary of Peak discharge using General Extreme Value (Gumbell's) distribution method.

Return Period (Year)	2	10	25	50	100
Peak Discharge(m ³ /s)	88.237	178.478	223.897	257.591	291.037

5.3. Grain Size Analysis (Sieve Analysis)

Grain size analysis is performed to determine the percentage of different grain sizes contained within soil. HEC-RAS model needs D50 of particle size in hydraulic analysis of Bridge scour for

main channel and left and right bank of the channel and percent of finer for some standard grain diameter in Sediment transport analysis. D50 for River Bed and left and right Bank material is 3mm and 0.95mm respectively. Those values are taken from particle size distribution curve which is developed from grain size analysis. See Figure 5-3 and Figure 5-4.

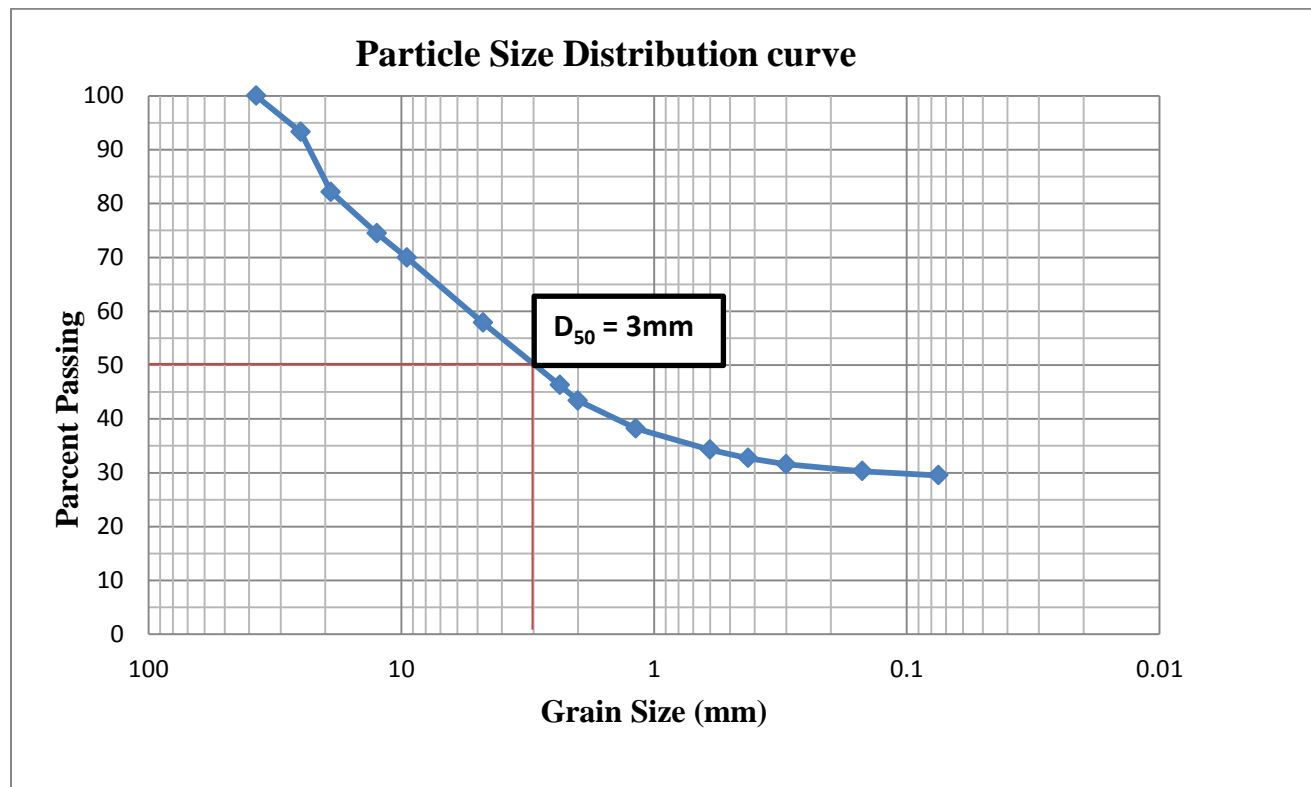


Figure 5-3: Grain size distribution curve and the grain size of 50 percent finer of Ginchi Awash River bed material.

From the above figure of Grain size distribution curve of the river bed material the percentage of Gravel, sand and fine material is 53.73, 16.77 and 29.51 respectively and shows that the more than 60 percent of bed material is Gravel and sand.

From the below figure 5-4 of Grain size distribution curve of the river bank material the percentage of Gravel, sand and fine material is 37.87, 22.51 and 39.62 respectively and shows that the more percentage of material at the river bank is the fine material.

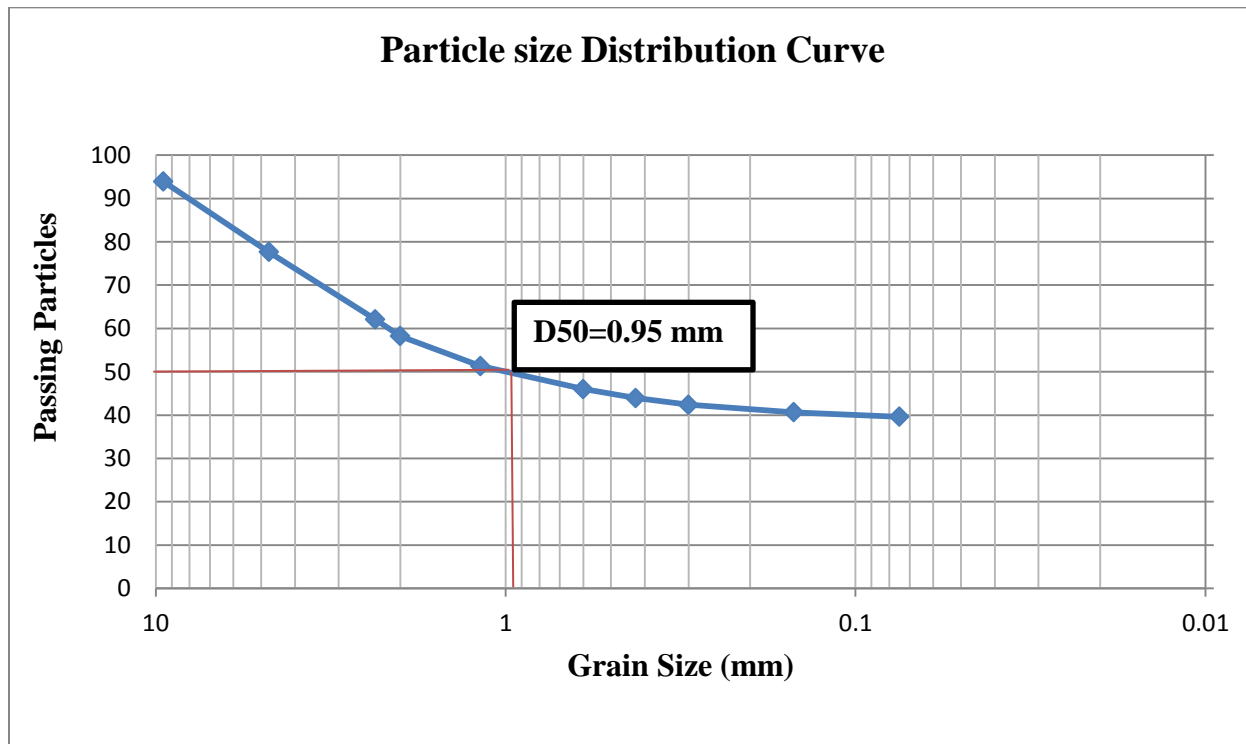


Figure 5-4: Grain size distribution curve and the grain of 50 percent finer of Ginchi Awash left and right Bank material.

5.4. Hydraulic Analysis (Using HEC-RAS model)

A HEC-RAS model was generated based on the geometry of the river channel and dimensions of Ginchi Awash Bridge. The peak flood of different return periods were also taken into account to run the model. After all of the inputs are complete, a steady flow analysis is conducted and the sediment transport simulation and the hydraulic design bridge scour depths are triggered on HEC-RAS.

5.4.1. Steady Flow Water Surface Profile including the Bridge at different return period.

The estimated design discharge from flood frequency analysis is used to determine the water surface profile. In this section we can analysis that whether the bridge opening size is adequate to pass the flood coming in any return period or not.

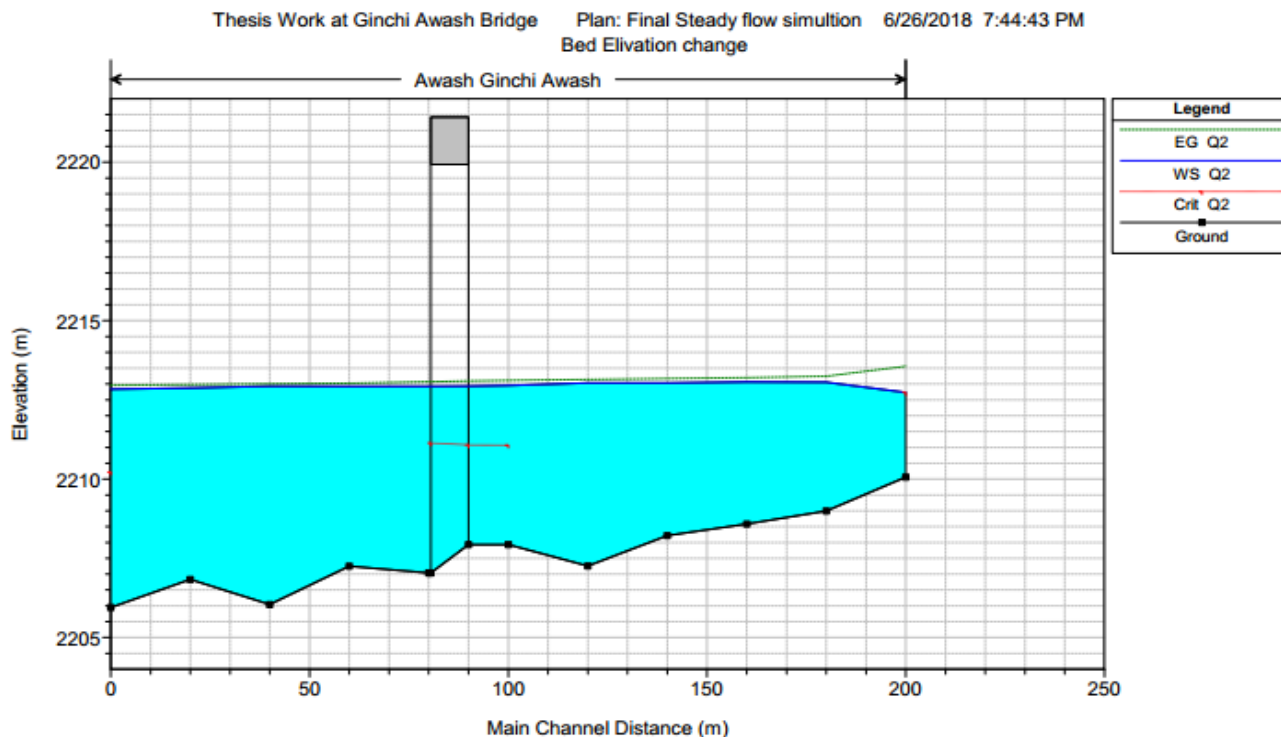


Figure 5-5: Water surface and Energy grade line profile under Bridge opening and over river channel for 2 year return period design flood.

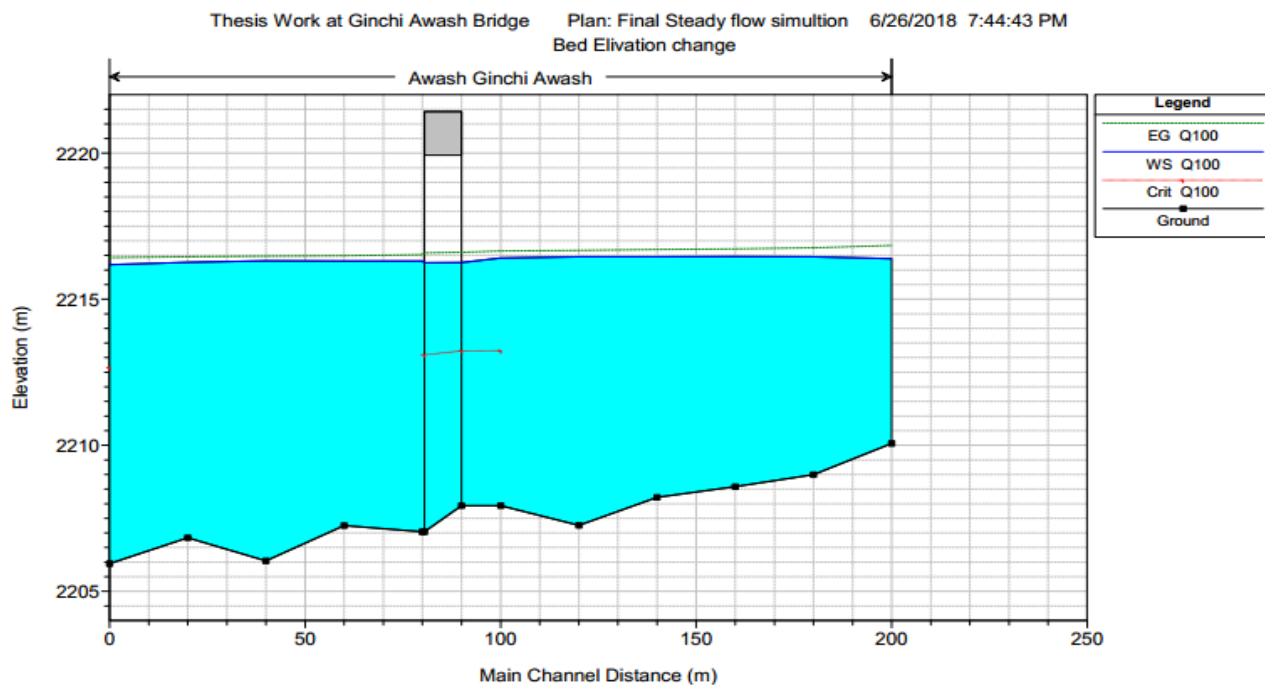


Figure 5-6: Water surface and Energy grade line profile under Bridge opening and over the river channel for 100 year return period design flood.

From the above Figure 5- and Figure 5-6 it is seen that the water surface elevation for both 100 year and 2 year return period design flood is less than the low chord of the bridge so from this it can be concluded that the bridge is not over topped in all case of design flood.

5.4.2. Sediment Transport Analysis Results

Based on the current cross section, bed material of the river and the Quasi Unsteady flow, HEC-RAS computed the bed elevation change along the river bed. For the purpose of comparison, the less frequent 100 year peak flood and the more frequent 2 year peak flood were used as an input and the following bed elevation change results were determined.

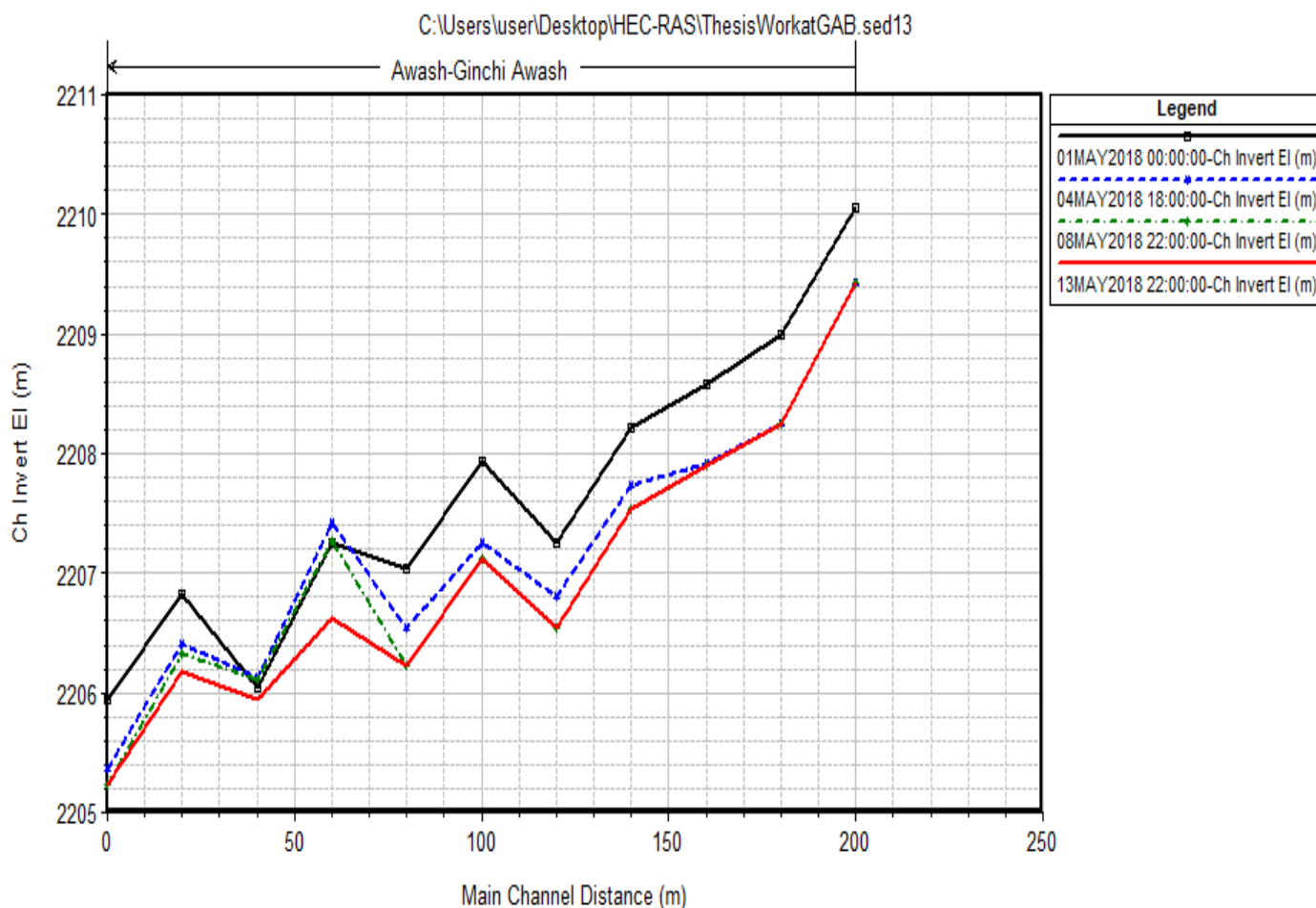


Figure 5-7: HEC-RAS River Bed profile plot along the river for 100 year return period of design flood (Q100).

From Figure above we can conclude that there exists a series of sediment scouring both at the upstream and downstream side of the Bridge which is the major problem in Ginchi Awash Bridge and the bridge is located between cross section 6 (upstream) and cross section 5 (downstream) which means of between 80m and 100m of the main channel distance, where the bed elevation change is highly occurred and results the Bridge scouring. The bed elevation change outputs are compiled in Table 5-4 below.

Table 5-4: The Bed elevation change result of sediment transport analysis for design flood of 100 year return period.

River	Reach	RS	Channel Distance (m)	(01MAY 2018 00:00:00) Channel Invert El (m)	(04MAY 2018 18:36:00) Channel Invert El (m)	(08MAY 2018 23:24:00) Channel Invert El (m)	(13MAY 2018 22:00:00) Channel Invert El (m)	Final bed elevation change (m)
Awash	Ginchi Awash	11	20.0001	2210.06	2209.42	2209.42	2209.417	-0.643
Awash	Ginchi Awash	10	20.0001	2208.99	2208.25	2208.25	2208.25	-0.741
Awash	Ginchi Awash	9	20.0001	2208.58	2208.16	2207.91	2207.907	-0.673
Awash	Ginchi Awash	8	20.0001	2208.22	2207.83	2207.54	2207.535	-0.687
Awash	Ginchi Awash	7	20.0001	2207.26	2207.25	2206.74	2206.517	-0.743
Awash	Ginchi Awash	6	20.0001	2207.93	2207.74	2207.11	2207.114	-0.816
Awash	Ginchi Awash	5	20.0001	2207.04	2206.57	2206.38	2206.225	-0.82
Awash	Ginchi Awash	4	20.0001	2207.25	2207.38	2207.45	2207.186	-0.633

Awash	Ginchi Awash	3	20.0001	2206.04	2206.08	2206.14	2206.056	0.092
Awash	Ginchi Awash	2	20.0001	2206.83	2206.82	2206.82	2206.546	-0.646
Awash	Ginchi Awash	1	0	2205.95	2205.43	2205.23	2205.228	-0.722

In order to investigate the short term effects of the river flow on the river bed profile, more frequent flood must be used in the model. For this purpose the 2 year return period river flow were used. Based on the 2 year peak flood, the following bed elevation change results were determined.

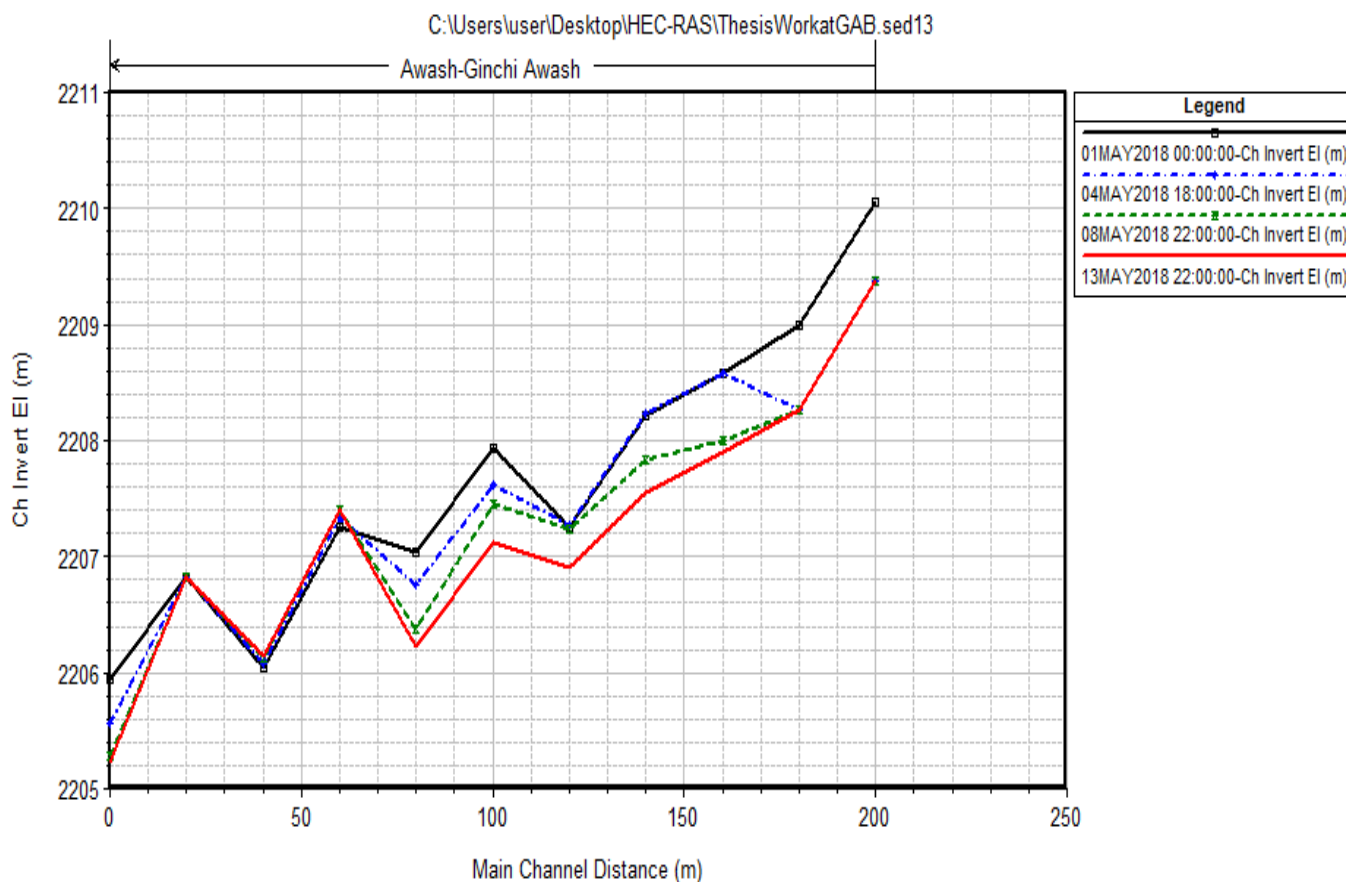


Figure 5-8: HEC-RAS River bed profile plot along the river for 2 year return period of design discharge (Q2).

Figure 5-8 also showed the same pattern of sediment degradation as in Figure 5-7 the major difference here is the magnitude of bed elevation change due to degradation is reduced, which is directly related to the reduction of input flood. The outputs of bed elevation are summarized in Table 5-5 below.

Table 5-5: The Bed elevation change result of sediment transport analysis for design flood of 2 year return period.

River	Reach	RS	Channel Distance (m)	(01MAY 2018 00:00:00) Channel Invert El (m)	(04MAY 2018 18:36:00) Channel Invert El (m)	(08MAY 2018 23:24:00) Channel Invert El (m)	(13MAY 2018 22:00:00) Channel Invert El (m)	Final bed elevation change (m)
Awash	Ginchi Awash	11	20.0001	2210.06	2209.36	2209.36	2209.382	-0.678
Awash	Ginchi Awash	10	20.0001	2208.99	2208.28	2208.27	2208.264	-0.726
Awash	Ginchi Awash	9	20.0001	2208.58	2208.66	2208.27	2207.906	-0.674
Awash	Ginchi Awash	8	20.0001	2208.22	2208.26	2208.09	2207.549	-0.671
Awash	Ginchi Awash	7	20.0001	2207.26	2207.3	2207.31	2207.904	-0.354
Awash	Ginchi Awash	6	20.0001	2207.93	2207.65	2207.66	2207.116	-0.814
Awash	Ginchi Awash	5	20.0001	2207.04	2206.84	2206.51	2206.226	-0.814
Awash	Ginchi Awash	4	20.0001	2207.25	2207.31	2207.37	2207.405	0.155
Awash	Ginchi Awash	3	20.0001	2206.04	2206.06	2206.09	2206.405	0.104

Awash	Ginchi Awash	2	20.0001	2206.83	2206.75	2206.75	2206.144	-0.004
Awash	Ginchi Awash	1	0	2205.95	2205.7	2205.41	2205.229	-0.721

For both the least frequent (100 year return period of design flood) and the most frequent (2 year return period of design flood) cases the HEC-RAS model results showed that river bank and river bed scouring exists both at upstream and downstream of the Bridge which consequences the Bridge abutment and foundation to be scoured. Figure 5-9 and Figure 5-10 shows that the bed elevation change occurred at the exact upstream and downstream cross section of the bridge for design flood of 2 year return period and design flood of 100 year return period respectively.

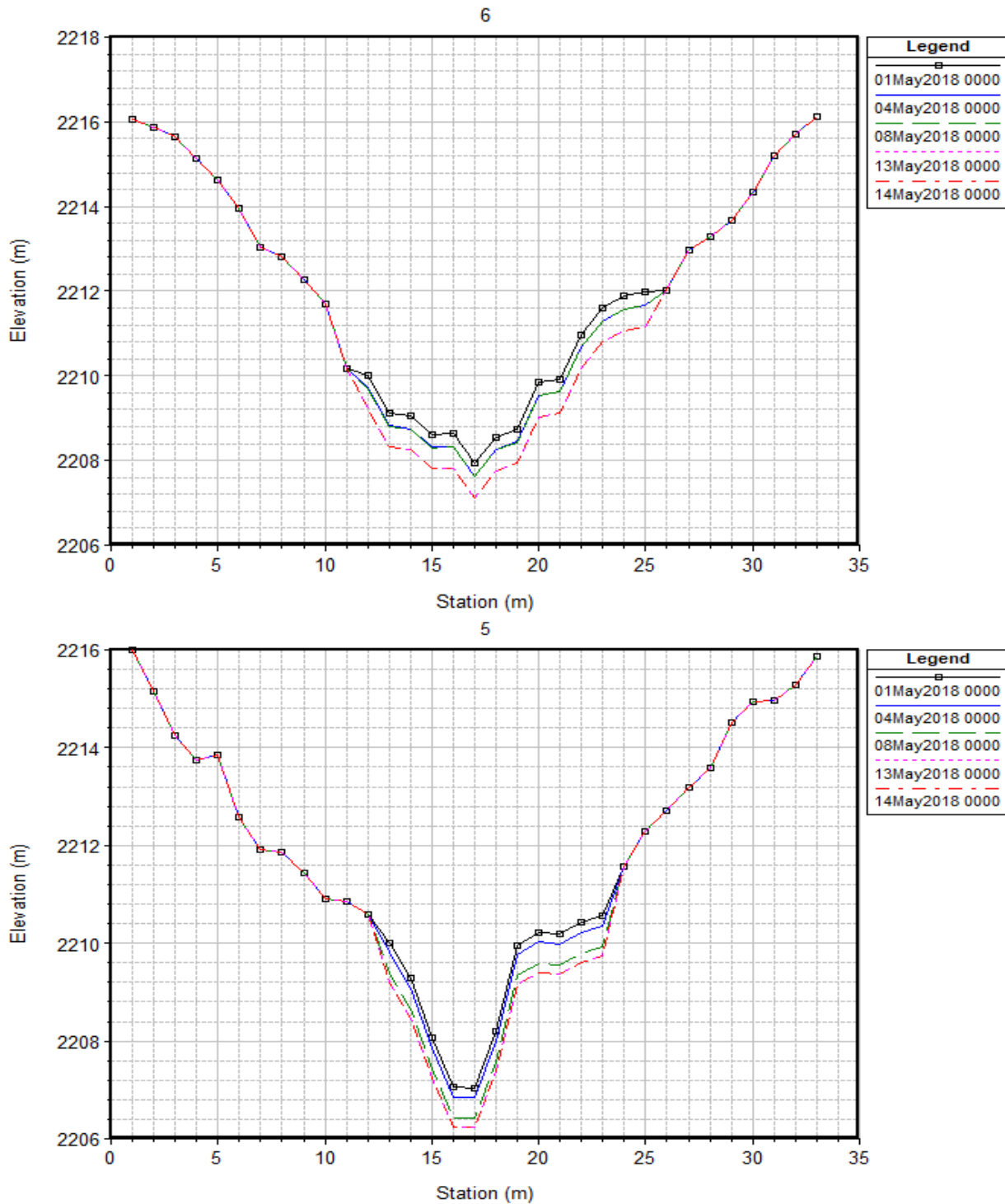


Figure 5-9: HEC-RAS Bed change profile at cross section 6 (upstream) and 5 (downstream) of the Bridge crossing structure for Q2.

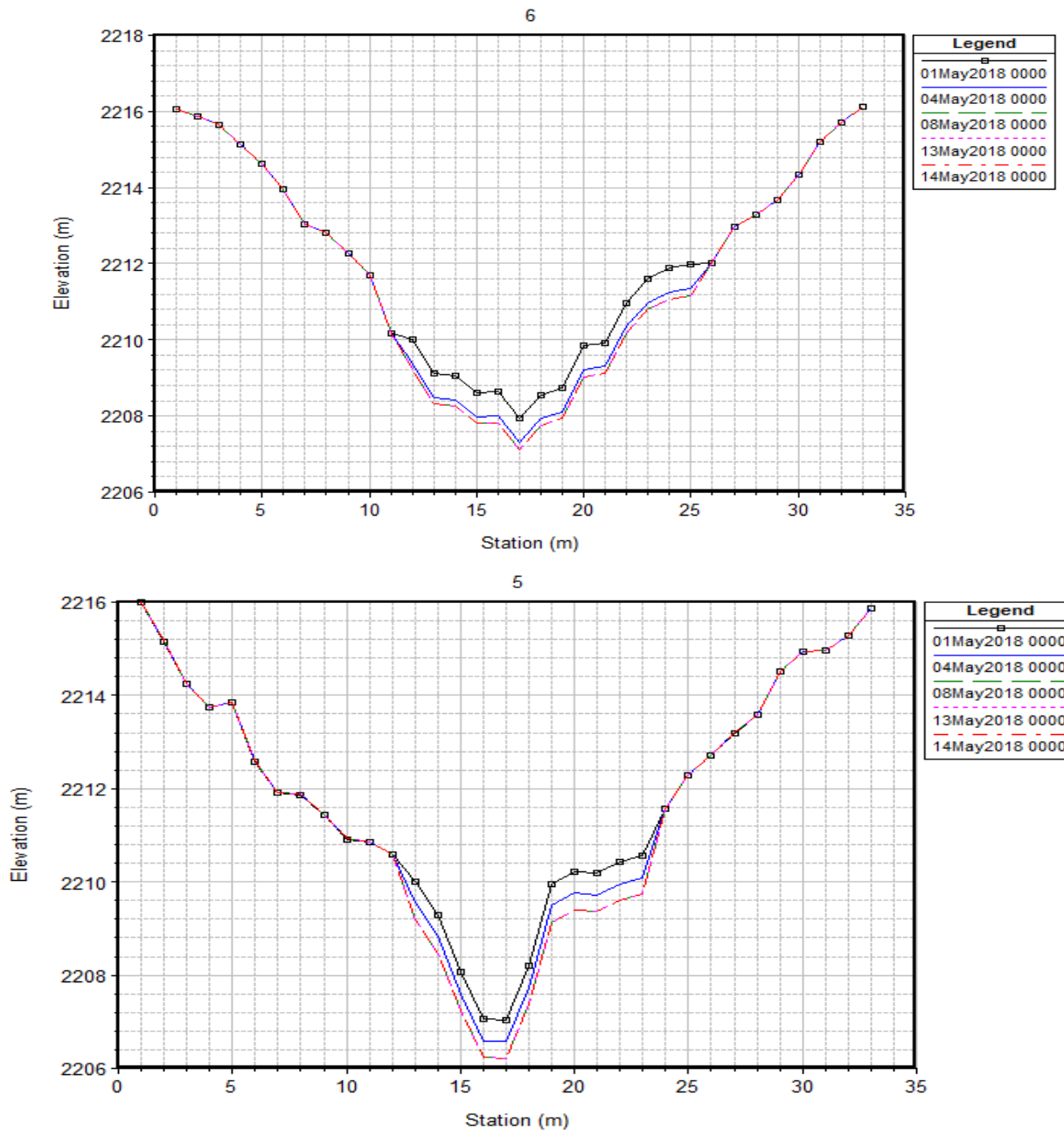


Figure 5-10: HEC-RAS Bed change profile at cross section 6 (upstream) and 5 (downstream) of the Bridge crossing structure for Q100.

Generally the causes for reduction of bed elevation at the bridge cross section and near to the bridge upstream and downstream cross section is increasing in velocity due to the constriction of opening size of the bridge and the fall velocity of the grain particles in the river is less than the shear force and velocity of water at Bridge location.

5.4.3. Bridge Scour Depth Result

Based on the flow in the main channel and flood plain, average depth and average velocity before contracted and in the approach section and the material particle size of which 50 percent finer (D50) for left overbank, main channel and right overbank HEC-RAS computed the Bridge scouring depth. For the purpose of comparison, the less frequent 100 year peak flood and the most frequent 2 year peak flood were used as an input and the following total scour depth results were determined.

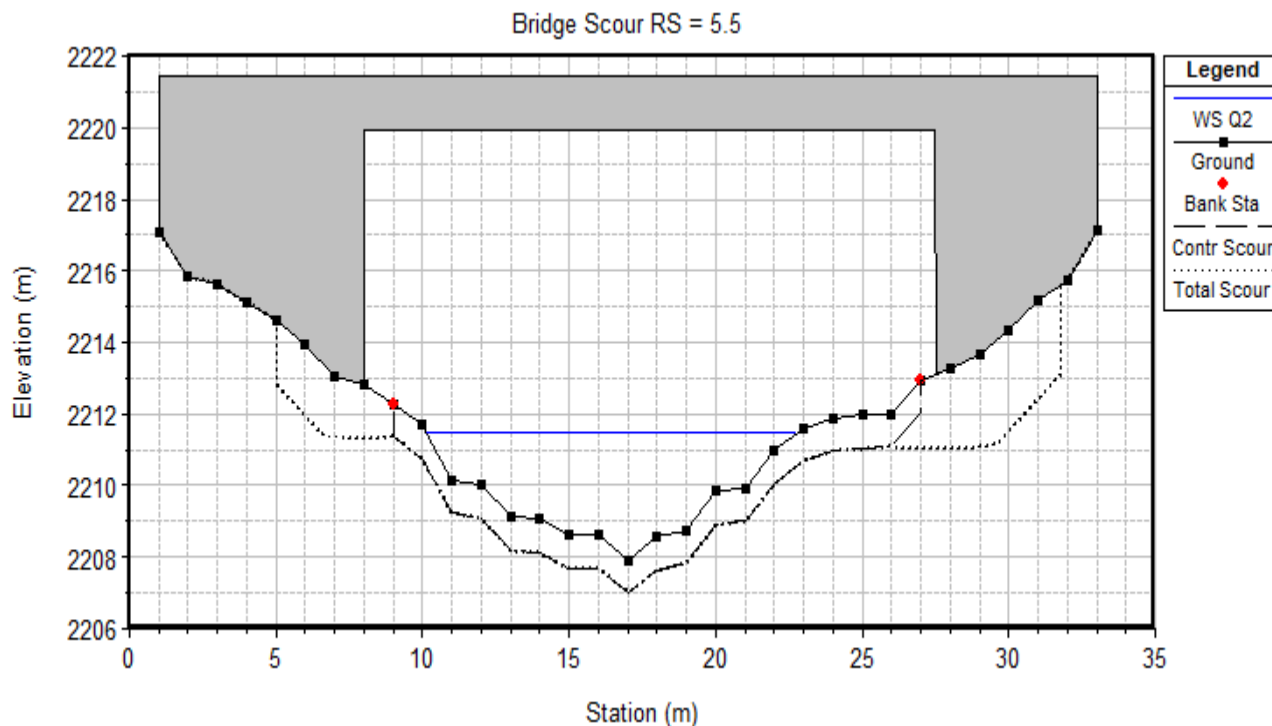


Figure 5-11: Total scour depicted for design flood of 2 year return period.

From the above figure of bridge cross section for the most frequent flood (Q2) we can see that there is bed elevation change at abutment of the Bridge due to abutment local scouring and at right side of the flood plain due to the contraction scouring which results the following total scouring depth.

Table 5-6: Combined local and contraction Bridge scour depth result at main channel and flood plain of Bridge crossing for design flood of 2 year return period.

<u>Contraction scour</u>			
<u>Input Data</u>	Left	Channel	Right
Average Depth (m):	0.73	3.00	0.98
Approach Velocity (m/s):	0.53	2.38	1.03
Bridge Average Depth (m):	-	2.05	-
Bridge Opening Flow (m ³ /s):	-	88.24	-
Bridge Top Width (m):	-	12.63	-
Grain size D50 (mm):	0.95	3.00	0.95
Approach Flow (m ³ /s):	0.57	85.70	1.97
Approach Top Width (m):	1.46	12.00	2.78
K1 Coefficient:	0.59	0.64	0.59
<u>Results</u>			
Scour Depth Ys (m):	0.00	0.93	0.00
Equation	Live	Live	Live
<u>Local Scour</u>			
<u>Abutment Scour</u>	Left	Right	
Station at Toe (m):	7.98	27.51	
Toe Station at approach (m):	9.98	22.51	
Abutment Length (m):	1.46	2.78	
Depth at Toe (m):	-01.30	-1.61	
K1 Shape Coefficient :	0.82-Vert.with wing walls		
Degree of Skew (degrees):	42.00	42.00	
K2 Skew Coefficient:	0.91	0.91	
Projected Length L' (m):	2.28	3.19	
Average Depth Obstructed ya (m):	0.73	0.98	
Flow Obstructed Qe (m ³ /s):	0.57	1.97	
Area Obstructed Ae(m ²):	1.06	2.71	
<u>Results</u>			
Scour Depth Ys (m):	1.49	2.12	

Qe/Ae = Ve (m/s):	0.54	0.73
Froude #:	0.20	0.23
Equation:	Froehlich	Froehlich
Combined Scour Depth		
Left Abutment Scour + Contraction Scour (m):	1.49	
Right Abutment Scour + Contraction Scour (m):	2.12	

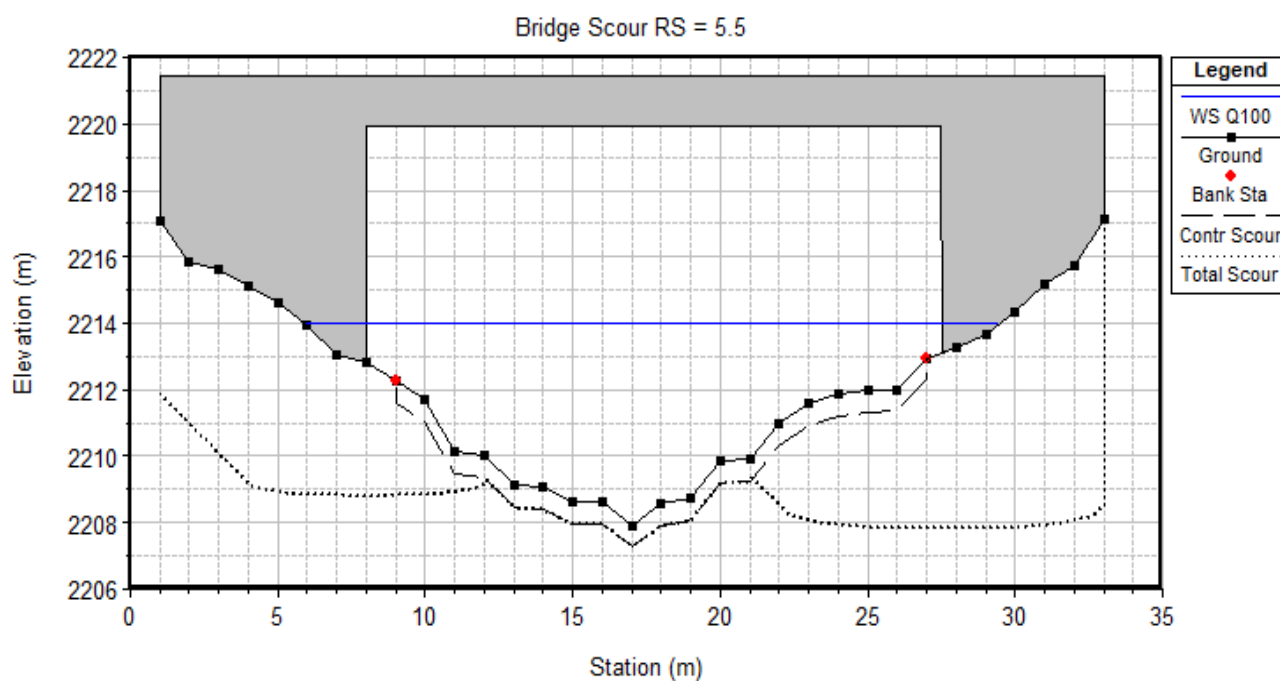


Figure 5-12: Total scour depicted for design flood of 100 year return period.

From Figure 5-12 we can see that the bridge cross section for less frequent flood (Q100) is correspondingly there is bed elevation change as that of the 2 year return period at abutment of the Bridge due to abutment local scouring and both at right and left of the main channel due to the contraction scouring but the difference is the damage of the Q100 is more than Q2 which is due to the difference in amount of input flood flow. Total scour depth result of 100Q is tabulated in the following table.

Table 5-7: Combined local and contraction Bridge scour depth result at main channel and flood plain of Bridge crossing for design flood of 100 year return period.

<u>Contraction scour</u>			
<u>Input Data</u>	Left	Channel	Right
Average Depth (m):	1.68	5.32	2.26
Approach Velocity (m/s):	1.22	4.11	1.43
Bridge Average Depth (m):	1.44	3.80	0.95
Bridge Opening Flow (m ³ /s):	1.34	289.39	0.30
Bridge Top Width (m):	1.02	17.00	0.51
Grain size D50 (mm):	0.95	3.00	0.95
Approach Flow (m ³ /s):	9.81	207.56	18.86
Approach Top Width (m):	4.82	12.00	5.82
K1 Coefficient:	0.590	0.640	0.590
<u>Results</u>			
Scour Depth Ys (m):	0.00	0.66	0.00
Equation	Live	Live	Live
<u>Local Scour</u>			
<u>Abutment Scour</u>	Left	Right	
Station at Toe (m):	7.98	27.51	
Toe Station at approach (m):	8.98	22.51	
Abutment Length (m):	4.82	5.82	
Depth at Toe (m):	1.30	0.98	
K1 Shape Coefficient :	0.82-Vert.with wing walls		
Degree of Skew (degrees):	45.00	45.00	
K2 Skew Coefficient:	0.91	0.91	
Projected Length L' (m):	5.79	7.16	
Average Depth Obstructed ya (m):	1.68	2.26	
Flow Obstructed Qe (m ³ /s):	9.81	18.86	
Area Obstructed Ae(m ²):	8.07	13.16	
<u>Results</u>			
Scour Depth Ys (m):	4.00	5.30	
Qe/Ae = Ve (m/s):	1.22	1.43	
Froude #:	0.3	0.3	
Equation:	Froehlich	Froehlich	
<u>Combined Scour Depth</u>			
Left Abutment Scour + Contraction Scour (m):	4.00		
Right Abutment Scour + Contraction Scour (m):	5.30		

From the above Table 5-6 and Table 5-7 it is feasible to conclude that the calculated total scour depth at the right and left abutment and contraction scouring by using HEC-RAS software for 100 year return period of peak discharge (less frequent flood) is more hazardous than 2 year return period of peak discharge (the most frequent flood).

For the design flood of 100 year return period case the local abutment scour depth at left abutment and right abutment is 4.12m and 4.84 respectively and the contraction scour depth is 1.66m. Similarly for design flood of 2 year the local abutment scour depth at left abutment and right abutment is 1.42m and 2.35m respectively and the contraction scour depth is 0.85m. The total scour depth at right abutment and left abutment is the sum of local abutment scour depth and the contraction scour depth. So the total scour at left abutment and right abutment for design flood of 100 year return period is 5.78m and 6.50m respectively and the case of 2 year design flood is 2.27 and 3.20m respectively.

The elevation difference between the scour depth calculated for less frequent flood and the most frequent flood is 3.51m at the Left abutment and 3.30m at right abutment. This clearly shows that the sediment erosion at the flood plain and the main channel of Bridge crossing area is increases as the flood amount transported under the bridge opening is increases which can result the failure of the bridge without any preparation.

5.5. Mitigation Measurements

Admissible Mitigation measures are required to lengthen the life of the bridge and reduce the impact of the failure of the Bridge. Scour countermeasures at bridges are aimed at reducing the negative impact of shear stresses and turbulence and velocity variations near the boundary of the structure-water interface, and of micro turbulence flow processes, such as eddies and helicoidal flows around bends. The following mitigation measures are recommended for the bridges in areas where bridges are lying in order to mitigate the bridge scour problem due to enhanced sediment degradation caused by both natural and human activities.

In HEC-RAS model we can arrange and take numerous bridge scour mitigation techniques and possible combinations. This is done for the riprap case by changing the manning roughness coefficient of the flood plain from 0.035 to 0.018, for dredging case by changing the manning

roughness coefficient of the main channel from 0.045 to 0.015 and for the case of approach channel by changing the skew angle from 42° to 5° . For the analysis of peak discharge used for 2 year and 100 year return period different mitigation measures are used and summarized in the table below.

Table 5-8: Scour depth of Ginchi Awash Bridge at Original case and including different mitigation methods.

Types of scour Mitigation measurement Methods	Scour Depth (m) For Q2		Scour Depth (m) For Q100	
	Abutment Scour Left	Abutment Scour Right	Abutment Scour Left	Abutment Scour Right
Vertical Abutment with wing wall (Original)	2.27	3.20	5.78	6.50
Vertical Abutment with wing wall + Side Riprap + Dredging River bed + approach channel	1.37	1.93	3.53	3.86
Spill-through abutment	1.92	2.68	4.82	5.38
Spill-through abutment + Side Riprap	1.54	2.40	4.55	5.71

In order to compare each scour mitigation method, the scour depth of the bridge without any protection (Original) is subtracted by the scour depth of each of the methods.

Table 5-9: Improvement of Scour mitigation methods.

Types of scour Mitigation measurement Methods	Scour Depth improvement (m) For Q2		Scour Depth improvement (m) For Q100	
	Abutment Scour Left	Abutment Scour Right	Abutment Scour Left	Abutment Scour Right

Vertical Abutment with wing wall + Side Riprap+ Dredging River bed + approach channel	0.90	1.27	2.25	2.64
Spill-through abutment	0.35	0.52	0.96	1.12
Spill-through abutment + Side Riprap	0.73	0.8	1.23	0.79

From the above table the spill-through abutment with constructing side riprap is the most scour improvement mitigation method but in economic aspect wing wall with side riprap mitigation method is effective. Comparing the methods by its own and ignoring the combinations, riprap decreases the scour depths all abutment scour. The wing wall and the spill-through abutment decrease both abutment scours on the left and right side.

Taking into account the possible geotechnical failure repair and construction costs of spill-through abutments, and the economic advantages of wing walls, the combination of riprap and wing walls ultimately is the most cost effective solution. Furthermore, Riprap is, and will remain, one of the primary scour countermeasures to resist local scour forces at abutments of typical bridges. Riprap is generally abundant, inexpensive and requires no special equipment. However, proper design and placement is essential. So side riprap is adequate in preventing abutment scour when subjected to the average flow rate of Ginchi Awash River. Therefore finally Based on both economic advantage and decreasing the scour depth of the bridge constructing, wing wall with riprap at abutment of typical bridge is the most efficient mitigation method to be used.

Additionally since there is channel bank erosion constructing approach channel over the bank upstream and downstream of the Bridge crossing is needed and it can reduce the local abutment scour effectively.

Finally the improvement of scour depth of the left and right abutment of the Bridge after the scour mitigation methods are modeled in HEC-RAS model is as follows.

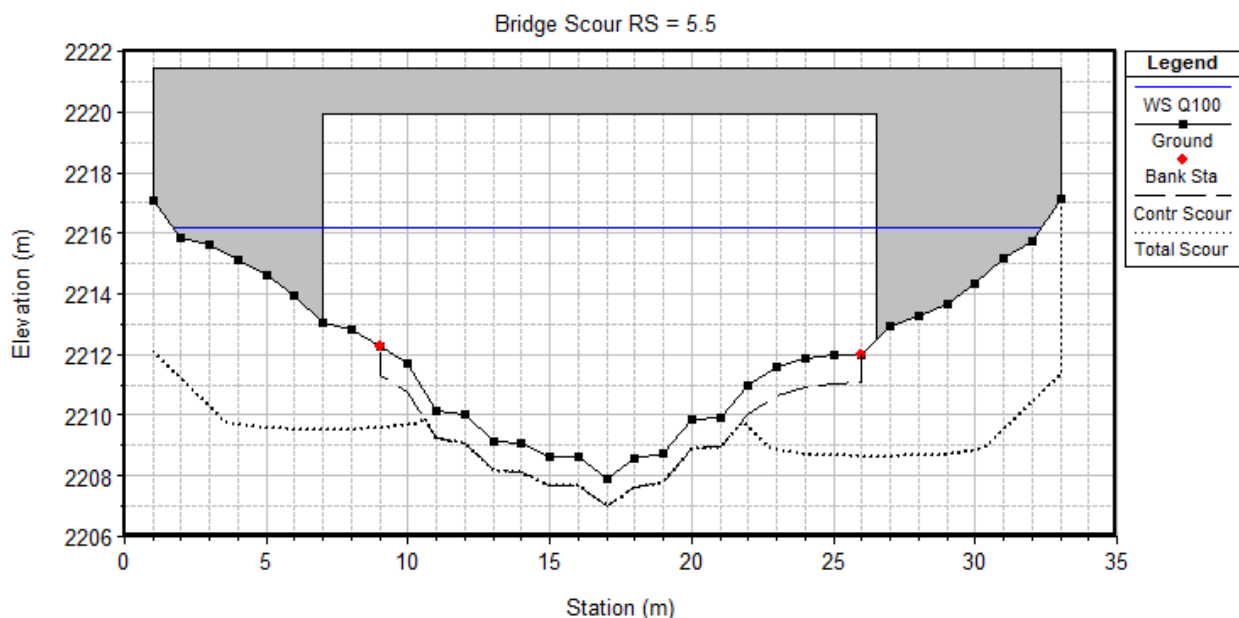


Figure 5-13: Total scour depth depicted for design flood of 100 year return period.

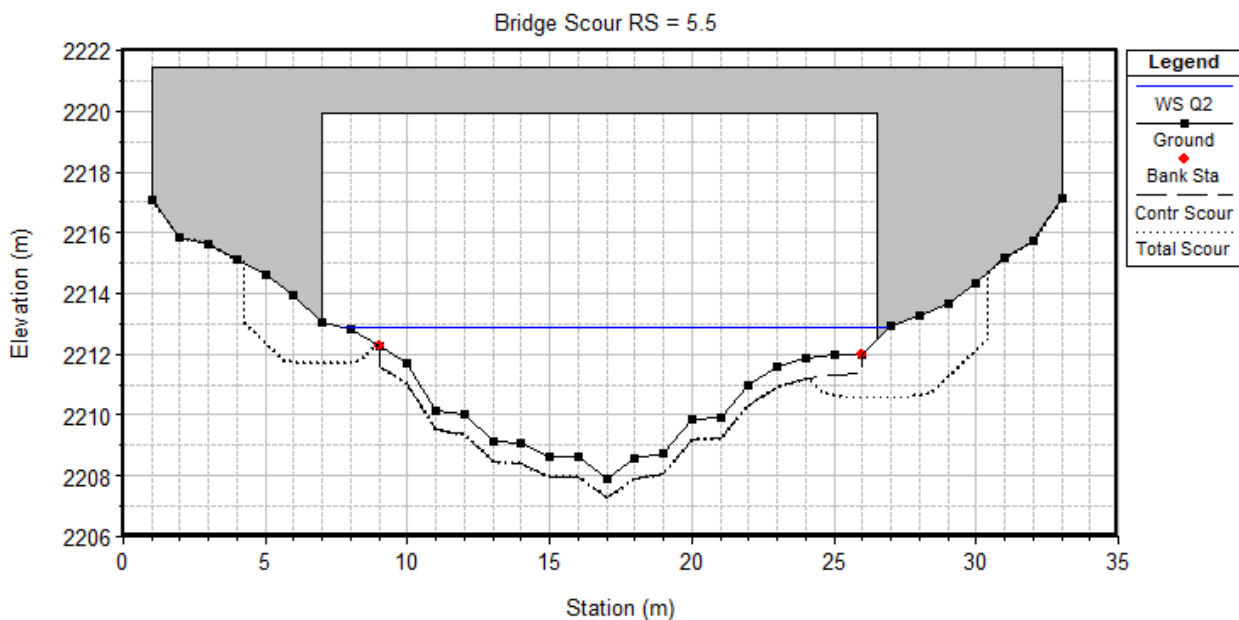


Figure 5-14: Total scour depth depicted for design of 2 year return period.

Generally constructing Riprap both at left and right abutment, dredging the riverbed both upstream and downstream of the Bridge crossing and constructing the approach channel both

upstream and downstream of the bridge crossing will mitigate the scour depth with some extent.

5.5.1. Design of Riprap at Abutment

The first step in riprap design is sizing the stone in the compression design. To determine the riprap size the following two equations are recommended by FWHA for the sizing of riprap at abutment.

$$D50 = Y * \left(\frac{K}{S_s - 1} \right) * (F_r)^2 \text{ For Froude number less than or equal to } 0.8 \quad 6.1$$

$$D50 = Y * \left(\frac{K}{S_s - 1} \right) * (F_r)^{0.14} \text{ For Froude number greater than } 0.8 \quad 6.2$$

Where

D50 = Riprap size

K = shape factor of the abutment = 1.02 for Vertical abutment.

Y = flow depth in contracted section, m.

S_s = Specific gravity = 2.65

F_r = Froude number in the contracted section = $\frac{V}{\sqrt{gY}}$

V = velocity at contracted section, m/s.

g = acceleration due to gravity, m/s².

Right side Abutment Riprap Design

Flow depth, Y = 3.91m

V₁₀₀ at abutment = 1.47 m/s

Specific gravity of stone = 2.65

1. Calculating the Froude number.

$Fr = \frac{V}{\sqrt{gY}} = 1.47 / ((9.81 * 3.91)^{0.5}) = 0.28$ which is less than 0.8 so equation 6.1 is used to calculate the riprap sizing D50.

2. Calculating the riprap size, D50.

$$D50 = Y * \left(\frac{K}{S_s - 1} \right) * (F_r)^2 = 3.91 * (1.02 / (2.65 - 1)) * (0.28)^2$$

$$= 0.19\text{m} = 190\text{mm}$$

3. Calculating riprap layer thickness.

$$T = 1.5 * D50$$

$$= 1.5 * 190 = 285\text{mm}$$

which is less than 300mm since ERA Drainage recommend,

$$= 300\text{mm}$$

as minimum thickness value 300mm is used.

4. Calculating the Horizontal dimensions.

The horizontal dimensions of the riprap into the overbank area of the river should be twice the flow depth on the overbank area in the vicinity of the abutment, but not less than 1.5 m or more than 7.5m (ERA Drainage manual 2013).

$$= 2 * \text{Flow depth}$$

$$= 2 * (3.91\text{m})$$

$$= 7.82\text{m},$$

but since 7.5m is the maximum recommended value 7.5m is used.

Left side Abutment Riprap Design

Flow depth, Y = 3.35m

V100 at abutment = 1.41 m/s

Specific gravity of stone = 2.65

1. Calculating the Froude number.

$Fr = \frac{V}{\sqrt{gY}} = 1.47 / ((9.81 * 3.91)^{0.5}) = 0.27$ which is less than 0.8 so equation 6.1 is used to calculate the riprap sizing D50 .

2. Calculating the riprap size, D50.

$$D50 = Y * \left(\frac{K}{S_s - 1} \right) * (F_r)^2 = 3.35 * (1.02 / (2.65 - 1)) * (0.27)^2$$
$$= 0.15\text{m} = 150\text{mm}$$

3. Calculating riprap layer thickness.

$$T = 1.5 * D50$$
$$= 1.5 * 150 = 225\text{mm}$$

which is less than 300mm since ERA Drainage recommend,

$$= 300\text{mm}$$

as minimum thickness value 300mm is used.

4. Calculating the Horizontal dimensions.

The horizontal dimensions of the riprap into the overbank area of the river should be twice the flow depth on the overbank area in the vicinity of the abutment, but not less than 1.5 m or more than 7.5m (ERA Drainage manual 2013).

$$= 2 * \text{Flow depth}$$
$$= 2 * (3.35\text{m})$$
$$= 6.7\text{m}$$

The following rough sketch is determined based on the above design calculations. It should be placed both on left and right, both at upstream and downstream of the bridge abutment.

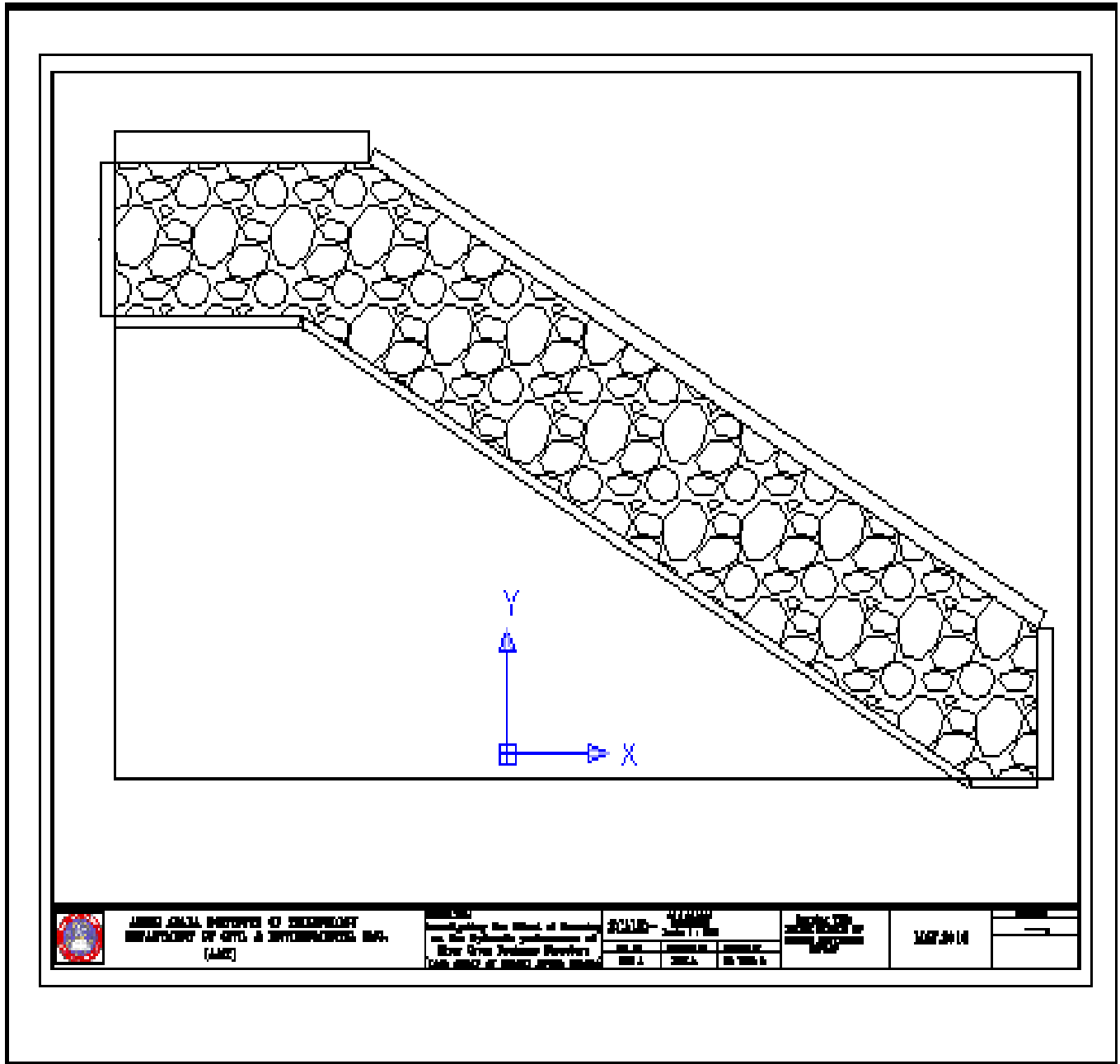


Figure 5-15: Rough Sketch of Riprap on Abutment of Ginchi Awash Bridge.

6.0. CONCLUSION AND RECOMMENDATION

6.1. Conclusion

This thesis work tries to cover the basic hydrological and hydraulic analysis perspectives of Ginchi Awash Bridge to investigate the causes of bridge scour and scour mitigation methods. In order to investigate the hydrological analysis General Extreme Value1 (Gumbell's) is used and the maximum design flood for 100 year return period and 2 year return period is estimated being **291.037m³/s** and **88.237m³/s** respectively. This result is used in HEC-RAS in order to analysis the hydraulic of the Bridge.

Based on the design flood of different return period, geometric data of the river channel and Deck dimensions are used in HEC-RAS software to calculate the steady flow water profile through the channel and under the bridge. The sediment series and soil gradient data are used in sediment transport capacity and Hydraulic Bridge scour depth computations. From the HEC-RAS model results for 100 year design flood the scour depth at the left abutment and right abutment are **5.78 m** and **6.50 m** respectively and additionally 2 year design flood is considered in order to comparison of the most frequent (Q2) and lest frequent (Q100) Design flood. Accordingly the scour depth at left abutment and right abutment for 2 year design flood is **2.27m** and **3.20m** respectively. Therefore when we compare the scour depth result of Q100 and Q2 it is realistic that in case of Q100 the scour depth both at left and right is high and can consequences a disaster if mitigation measurement is not practicable.

As described in result and discussion part of this thesis work sediment erosion of the river bank and bridge abutment and foundation (River bed) scouring is the major problem in reducing of the bridge hydraulic performance. From the study the main causes for bridge scouring is the soil type of the study area is Eurtic Cambisol which is the most erodible soil and the contraction of the river channel which results the increment of velocity of flow.

From site observation and some interviewees with communities around that area the bridge is started to be scoured at 1996 E.C where the recorded instantaneous daily flow is **196.15m³/s** which is maximum record among 17 years instantaneous annual daily stream flow data. So from this we can conclude that the bridge will be in danger with the 100 year design flood

291.037m³/s. For hydraulic Bridge scouring problem various mitigation measures are investigated and from HEC-RAS result constructing wing wall with riprap both on left and right abutment at upstream and downstream of the bridge as per to the design above is recommended. Additionally since the bank of the channel is eroded constructing approach channel upstream and downstream of the bridge crossing is needed.

6.2. Recommendation

The following recommendations are drawn from the investigation on Ginchi Awash Bridge.

- ✓ Based on the result from HEC-RAS model of hydraulic bridge scour analysis the main concern should be how to reduce the effect of scouring problem or sediment degradation from the abutment and foundation of the bridge and from bank of the channel so constructing the wing wall with riprap is recommended.
- ✓ Dredging river bed both upstream and downstream of the bridge crossing structure could also be considered after its impact on the environment and the cost is considered. Dredging may not bring permanent solution to the problem but it will provide temporary solution based on the severity of the problem.
- ✓ There is a limitation of organized data's of Bridge deck structure which are very essential to HEC-RAS model in Ethiopia. Since the limitation of data in investigation of the hydraulic analysis makes the result from the model inadequate, Ethiopian Road Authority (ERA) should give some consideration to this area.
- ✓ The River series data used in the hydrological analysis is older than 2011 which is eight years back, so the flood estimated based on these data are not representative of the current situation of the study area.
- ✓ The sediment concentration flow data around the study area is not sufficient to investigate more so additional sediment concentration flow data is also required to estimate the exact amount of sediment transport capacity in the channel around the bridge.
- ✓ From field visit there is also bank erosion upstream of the bridge crossing area to diminish this problem irrigation practice on the over bank of the river should be carried out in a controlled way. Agricultural activity generally looseness the soil surface and facilitate surface erosion during high flood seasons. If possible create means by which the soil is covered with plants or grass throughout the year. This will protect the top most layer of the soil from erosion during peak flood seasons of the year.
- ✓ Lastly, practical experiments in a short time could be conducted by applying the recommended mitigation measurements in chapter 6 to further testify the method's feasibility and those mitigation measures are not be a permanent solution so replacement of the bridge and further study of this Bridge is essential.

REFERANCE

1. ERA, (2013) Drainage Design Manual, Addis Ababa Ethiopian Road Authority.
2. HEC 18, Second Edition Evaluating Scour at Bridges, February 1993.
3. Department of Transport and Main Roads April 2013 Hydraulic Guidelines for Bridge Hydraulic Guidelines for Bridge Design Projects, April 2013.
4. Natnael Hailu., 2016 Effect of Sediment Aggradation on the Hydraulic Performance of River Cross Drainage Structures (The case of Mekanisa Bridge on Mekanisa to Gofa Highway Road) MSc. Thesis, Addis Ababa University.
5. Gary, W. B. (2010). HEC-RAS Hydraulic Reference Manual.
6. Les Hamill, 2004. Bridge Hydraulics, University of Plymouth, New York.
7. L.W.Zevenbergen, L.A.Arneson, J.H.Hunt, A.C.Miller., 2012, Hydraulic Design of Safe Bridges, Colorado.
8. Jean-Louis Briaud, Hamn-Ching Chen, Billy Edge, Siyoung Park, and Adil Shah, 2001. Guidelines for bridges over degrading and migrating streams, Project No. 7-2105.
9. Hydraulic Engineering Circular No. 23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection and Design Guidance-Third Edition, September 2009.
10. Dawit Hadera^{1*} and Bizuneh Asfaw², J Civil Environ Eng 2016, Investigating Causes of Highway Crossing (Bridges and Culverts) Outlet Erosion in Selected Structures of Mekelle to Adigrat Highway - Tigray, Ethiopia.
11. K Subramanya, Professor of Indian Institute of Technology Kanpur, Engineering Hydrology, 1982.
12. Chow, V.T., Maidment, D.R., Mays, L.W. 1988, Applied Hydrology, McGraw-Hill, New York.
13. Laursen, E. M. (1963). An analysis of Relief Bridge scour. *J. Hydraul. Div., Am. Soc. Civ. Eng*, 89(3), 93-118.
14. Beza Nigussie., 2010. Investigation of cause of failures of Highway cross drainage Structures (Case study on Raya River Bridge), MSc. Thesis, Addis Ababa University.
15. A Ramachandra Rao, Khaled H. Hamed. Flood Frequency Analysis 2000.
16. Federal Highway Administration, "Drainage at Highway Pavements", HEC-12, 1984.

17. Federal Highway Administration "Stream Stability at Highway Structures", HEC-20, 1991.
18. Bradley, J.N. (1973).Hydraulics of bridge waterways. Hydraulic Design Series No 1. Second edition. FHWA. Washington DC.
19. Brown, S.A. and Clyde, E.S. (1989). Design of riprap revetment. Report No FHWA-IP-89-016. HEC 11. Georgetown: Federal Highway Administration.
20. U.S. Army Corps of Engineers, "HEC-2 Water Surface Profiles," User's Manual, September 1982.

APPENDICES

Appendix A: Data outlier Test.

DATA OUTLIER TEST							
XH = exp($\bar{X}+K_N S$)1							
XL = exp($\bar{X}-K_N S$)2							
KN=-3.62201+(6.28446N)^(1/4)-(2.49835N)^(1/2)+ (0.491436N^(3/4)-(0.037911N)3							
N = 17							
Standard Error = (Cof.Var./(N^0.5))*100							
Kn = 2.31							
Stand. Error = {Cof. Vari/(N^0.5)}*100							
Coff. Var= (Sd-1)/Mean							
Test = IF(St.Er <=, "Adequate Data","Inadequate Data")							
Skew= (N*(Y-y))/((N-1)*(N-2)*((Sd)^3))							
= -0.30							
Test dequate Data							
	Outerlier	X					
Higher	2.24	173.39					
Lower	1.13	13.56					
No Rec	Year	X(Max Q)	Y=LogX	(Y-y) ²	(Y-y) ³	Outerlier	
						Higher	Lower
1	1994	82.79	1.92	0.05394	0.012529	Ok	Ok
2	1995	46.44	1.67	0.00036	-0.000007	Ok	Ok
3	1996	113.87	2.06	0.13741	0.050934	Ok	Ok
4	1997	63.88	1.81	0.01432	0.001713	Ok	Ok
5	1998	75.46	1.88	0.03687	0.007080	Ok	Ok
6	1999	89.41	1.95	0.07059	0.018753	Ok	Ok
7	2000	24.16	1.38	0.09156	-0.027707	Ok	Ok
8	2001	30.59	1.49	0.04008	-0.008023	Ok	Ok
9	2002	20.40	1.31	0.14152	-0.053236	Ok	Ok
10	2003	18.24	1.26	0.18034	-0.076582	Ok	Ok
11	2004	30.01	1.48	0.04342	-0.009049	Ok	Ok
12	2005	38.66	1.59	0.00970	-0.000956	Ok	Ok
13	2006	47.19	1.67	0.00014	-0.000002	Ok	Ok
14	2007	49.49	1.69	0.00008	0.000001	Ok	Ok
15	2008	95.178	1.98	0.08575	0.025109		
16	2009	61.432	1.79	0.01054	0.001083		
17	2010	55.25	1.74	0.00321	0.000182		
Total	=	942.44	28.66	0.919815	-0.058178		
Mean	=	55.44	1.69				
St.Dev	=	28.20	0.24				
Skew	=		-0.30				
Coff. Var.	0.491						
Stan.Error	11.90						

Appendix B: Flood frequency Analysis of Peak Discharge Estimation Result.

Table B.1: Calculation of Annual Instantaneous daily flow from maximum mean annual daily stream gauged flow of 17 years recorded data (from 1994 to 2010).

$$Q_{Inst} = Q_{Avr} + Q_{Avr} * (2A^{-0.3})$$

Catchment Area
A= 47.9 Km2
= 29.764 mi2

Year	Q _{avr} max(m ³ /s)	2*A ^{-0.3}	Q _{avr} *2*A ^{-0.3}	Q _{Inst} (m ³ /s)
1994	82.788	0.7226	59.826	142.614
1995	46.436	0.7226	33.557	79.993
1996	113.866	0.7226	82.285	196.151
1997	63.879	0.7226	46.162	110.041
1998	75.462	0.7226	54.532	129.994
1999	89.411	0.7226	64.612	154.023
2000	24.161	0.7226	17.460	41.621
2001	30.586	0.7226	22.103	52.689
2002	20.395	0.7226	14.738	35.133
2003	18.241	0.7226	13.182	31.423
2004	30.014	0.7226	21.689	51.703
2005	38.655	0.7226	27.934	66.589
2006	47.189	0.7226	34.101	81.290
2007	49.492	0.7226	35.765	85.257
2008	95.178	0.7226	68.780	163.958
2009	61.432	0.7226	44.393	105.825
2010	55.25	0.7226	39.926	95.176

Appendix B.1: General Extreme Value 1 Gumbell’s Distribution method

$$X_T = \bar{X} + K \sigma_{n-1}$$

Where σ_{n-1} = standard deviation of the sample of size N = $\sqrt{\frac{\sum(X-\bar{X})^2}{N-1}}$

K = frequency factor expressed as $K_T = \frac{Y_T - \bar{Y}_n}{S_n}$

Y_T = reduced variate, a function of T and given by $Y_T = -\left[\ln. \ln \frac{T}{T-1}\right]$

Or $Y_T = -\left[0.834 + 2.303 \log. \log \frac{T}{T-1}\right]$

$Y_n =$	0.5181	} from the table 7.3 and 7.4 of Appendix B.2.1
$s_n =$	1.01411	

Table B.2: General Extreme Value 1 (Gumbell's) Distribution Method

Year	Q max (m ³ /s)	Q-Q _{mean} (m ³ /s)	(Q-Q _{mean}) ² (m ³ /s)
1994	142.6142	47.12	2219.86
1995	79.9927	-15.51	240.44
1996	196.1505	100.65	10130.77
1997	110.0407	14.54	211.47
1998	129.9941	34.50	1189.93
1999	154.0233	58.52	3425.11
2000	41.6208	-53.88	2902.84
2001	52.6888	-42.81	1832.70
2002	35.1333	-60.37	3643.99
2003	31.4227	-64.08	4105.74
2004	51.7034	-43.80	1918.03
2005	66.5888	-28.91	835.79
2006	81.2898	-14.21	201.89
2007	85.2571	-10.24	104.89
2008	163.9577	68.46	4686.63
2009	105.8254	10.33	106.64
2010	95.1760	-0.32	0.10
Total Q_{Ins} =	1623.479381	$\Sigma(x-x_{mean})^2 =$	37756.83172
X_{mean} =	95.49878712		
$\sigma_{n-1} =$	48.5777931		

RP T(year)	YT	Yn	Sn	K	σ_{n-1}	\bar{X}	XT(m ³ /s)
2	0.36651	0.5181	1.01411	-0.1495	48.5778	95.4988	88.237
10	2.25037	0.5181	1.01411	1.70817	48.5778	95.4988	178.478
25	3.19853	0.5181	1.01411	2.64314	48.5778	95.4988	223.897
50	3.90194	0.5181	1.01411	3.33676	48.5778	95.4988	257.591
100	4.60015	0.5181	1.01411	4.02525	48.5778	95.4988	291.037

Table B.2.1 Value of Reduced Mean (y_n) and Reduced Standard Deviation (s_n).

Table 7.3 Reduced mean \bar{y}_n in Gumbel's Extreme Value Distribution

N = sample size

<i>N</i>	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5035	0.5070	0.5100	0.5128	0.5157	0.5181	0.5202	0.5220
20	0.5236	0.5252	0.5268	0.5283	0.5296	0.5309	0.5320	0.5332	0.5343	0.5353
30	0.5362	0.5371	0.5380	0.5388	0.5396	0.5402	0.5410	0.5418	0.5424	0.5430
40	0.5436	0.5442	0.5448	0.5453	0.5458	0.5463	0.5468	0.5473	0.5477	0.5481
50	0.5485	0.5489	0.5493	0.5497	0.5501	0.5504	0.5508	0.5511	0.5515	0.5518
60	0.5521	0.5524	0.5527	0.5530	0.5533	0.5535	0.5538	0.5540	0.5543	0.5545
70	0.5548	0.5550	0.5552	0.5555	0.5557	0.5559	0.5561	0.5563	0.5565	0.5567
80	0.5569	0.5570	0.5572	0.5574	0.5576	0.5578	0.5580	0.5581	0.5583	0.5585
90	0.5586	0.5587	0.5589	0.5591	0.5592	0.5593	0.5595	0.5596	0.5598	0.5599
100	0.5600									

Table 7.4 Reduced Standard Deviation S_n in Gumbel's Extreme Value Distribution

N = sample size

<i>N</i>	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	1.0095	1.0206	1.0316	1.0411	1.0493	1.0565
20	1.0628	1.0696	1.0754	1.0811	1.0864	1.0915	1.0961	1.1004	1.1047	1.1086
30	1.1124	1.1159	1.1193	1.1226	1.1255	1.1285	1.1313	1.1339	1.1363	1.1388
40	1.1413	1.1436	1.1458	1.1480	1.1499	1.1519	1.1538	1.1557	1.1574	1.1590
50	1.1607	1.1623	1.1638	1.1658	1.1667	1.1681	1.1696	1.1708	1.1721	1.1734
60	1.1747	1.1759	1.1770	1.1782	1.1793	1.1803	1.1814	1.1824	1.1834	1.1844
70	1.1854	1.1863	1.1873	1.1881	1.1890	1.1898	1.1906	1.1915	1.1923	1.1930
80	1.1938	1.1945	1.1953	1.1959	1.1967	1.1973	1.1980	1.1987	1.1994	1.2001
90	1.2007	1.2013	1.2020	1.2026	1.2032	1.2038	1.2044	1.2049	1.2055	1.2060
100	1.2065									

Appendix B.2: Log-Pearson Type III Distribution Method.

$$z_T = \bar{z} + K_z \sigma_z$$

$$z = \log x \quad x_T = \text{antilog}(z_T)$$

where K_z = a frequency factor which is a function of recurrence interval T and the coefficient of skew C_s ,

σ_z = standard deviation of the Z variate sample

$$= \sqrt{\Sigma(z - \bar{z})^2 / (N - 1)} \tag{7.25a}$$

and

C_s = coefficient of skew of variate Z

$$= \frac{N \Sigma(z - \bar{z})^3}{(N - 1)(N - 2)(\sigma_z)^3} \tag{7.25b}$$

\bar{z} = mean of the z values

N = sample size = number of years of record

$$CS = \frac{N \Sigma(\log Q - \text{meanlog} Q)^3}{(N - 1)(N - 2)\delta Q^3}$$

Year	Q max (m ³ /s)	Z=log(Q _{peak})	(Z-Z _{mean}) ²	(Z-Z _{mean}) ³
1994	142.614	2.154	0.054	0.013
1995	79.993	1.903	0.000	0.000
1996	196.151	2.293	0.137	0.051
1997	110.041	2.042	0.014	0.002
1998	129.994	2.114	0.037	0.007
1999	154.023	2.188	0.071	0.019
2000	41.621	1.619	0.092	-0.028
2001	52.689	1.722	0.040	-0.008
2002	35.133	1.546	0.142	-0.053
2003	31.423	1.497	0.180	-0.077
2004	51.703	1.714	0.043	-0.009
2005	66.589	1.823	0.010	-0.001
2006	81.290	1.910	0.000	0.000
2007	85.257	1.931	0.000	0.000
2008	163.958	2.215	0.086	0.025
2009	105.825	2.025	0.011	0.001
2010	95.176	1.979	0.003	0.000
SUM OF Z =		32.67239	0.91982	-0.05818
$\bar{Z} =$	1.92			
$\sigma =$	0.24			
$\sigma^3 =$	0.014			
$C_s =$	-0.30			

$K_z = F(C_s, T)$ from the table 7.6 of Eng' Hydrology, Subramanya

RP T(yr)	σ	σ_3	C_s	K_z	Z_{mean}	ZT	XT(m ³ /s)
2	0.240	0.014	-0.30	0.05	1.92	1.934	85.880
10	0.240	0.014	-0.30	1.245	1.92	2.220	166.118
25	0.240	0.014	-0.30	1.643	1.92	2.316	206.940
50	0.240	0.014	-0.30	1.890	1.92	2.375	237.173
100	0.240	0.014	-0.30	2.104	1.92	2.426	266.917

Table B.3.1: $K_z=F(C_s,T)$ for use in Log-Pearson Type III Distribution

Table 7.6 $K_z = F(C_s, T)$ for Use in Log-Pearson Type III Distribution

Coefficient of skew, C_s	Recurrence interval T in years						
	2	10	25	50	100	200	1000
3.0	-0.396	1.180	2.278	3.152	4.051	4.970	7.250
2.5	-0.360	1.250	2.262	3.048	3.845	4.652	6.600
2.2	-0.330	1.284	2.240	2.970	3.705	4.444	6.200
2.0	-0.307	1.302	2.219	2.912	3.605	4.298	5.910
1.8	-0.282	1.318	2.193	2.848	3.499	4.147	5.660
1.6	-0.254	1.329	2.163	2.780	3.388	3.990	5.390
1.4	-0.225	1.337	2.128	2.706	3.271	3.828	5.110

(Contd.)

(Contd.)

1.2	-0.195	1.340	2.087	2.626	3.149	3.661	4.820
1.0	-0.164	1.340	2.043	2.542	3.022	3.489	4.540
0.9	-0.148	1.339	2.018	2.498	2.957	3.401	4.395
0.8	-0.132	1.336	1.998	2.453	2.891	3.312	4.250
0.7	-0.116	1.333	1.967	2.407	2.824	3.223	4.105
0.6	-0.099	1.328	1.939	2.359	2.755	3.132	3.960
0.5	-0.083	1.323	1.910	2.311	2.686	3.041	3.815
0.4	-0.066	1.317	1.880	2.261	2.615	2.949	3.670
0.3	-0.050	1.309	1.849	2.211	2.544	2.856	3.525
0.2	-0.033	1.301	1.818	2.159	2.472	2.763	3.380
0.1	-0.017	1.292	1.785	2.107	2.400	2.670	3.235
0.0	0.000	1.282	1.751	2.054	2.326	2.576	3.090
-0.1	0.017	1.270	1.716	2.000	2.252	2.482	2.950
-0.2	0.033	1.258	1.680	1.945	2.178	2.388	2.810
-0.3	0.050	1.245	1.643	1.890	2.104	2.294	2.675
-0.4	0.066	1.231	1.606	1.834	2.029	2.201	2.540
-0.5	0.083	1.216	1.567	1.777	1.955	2.108	2.400
-0.6	0.099	1.200	1.528	1.720	1.880	2.016	2.275
-0.7	0.116	1.183	1.488	1.663	1.806	1.926	2.150
-0.8	0.132	1.166	1.448	1.606	1.733	1.837	2.035
-0.9	0.148	1.147	1.407	1.549	1.660	1.749	1.910
-1.0	0.164	1.128	1.366	1.492	1.588	1.664	1.880
-1.4	0.225	1.041	1.198	1.270	1.318	1.351	1.465
-1.8	0.282	0.945	1.035	1.069	1.087	1.097	1.130
-2.2	0.330	0.844	0.888	0.900	0.905	0.907	0.910
-3.0	0.396	0.660	0.666	0.666	0.667	0.667	0.668

[Note: $C_s = 0$ corresponds to log-normal distribution]

Appendix B.3: Log-Normal Distribution Method.

$XT = \text{antilog}(Y_T)$

$YT = \bar{Y} + K_v * \delta_v$

$$KY = w - \left[\frac{2.516 + 0.8028w + 0.0103w^2}{1 + 1.4328w + 0.1893w^2 + 0.0013w^3} \right]$$

Where KY is a function of w

$$w = \left[\ln\left(\frac{1}{p^2}\right) \right]^{\frac{1}{2}} \quad 0 \leq p \leq 0.5$$

When $p > 0.5$, $1-p$ is substituted for p

$A = 2.516 + 0.8028W + 0.0103W^2$

$B = 1 + 1.4328W + 0.1893W^2 + 0.0013W^3$

$\bar{Y} = \Sigma Y / 17$

$\sigma = \sqrt{\Sigma(Y - Y \text{ mean})^2 / (N-1)}$

Year	Q max (M3/S)	Y=log(Qpeak)	(Y-Ymean)	(Y-Ymean)^2
1994	142.6141973	2.154	0.232	0.0539
1995	79.99266638	1.903	-0.019	0.0004
1996	196.1505072	2.293	0.371	0.1374
1997	110.0407343	2.042	0.120	0.0143
1998	129.9941121	2.114	0.192	0.0369
1999	154.0232641	2.188	0.266	0.0706
2000	41.62078587	1.619	-0.303	0.0916
2001	52.68876936	1.722	-0.200	0.0401
2002	35.13331103	1.546	-0.376	0.1415
2003	31.42273726	1.497	-0.425	0.1803
2004	51.70341737	1.714	-0.208	0.0434
2005	66.58877851	1.823	-0.099	0.0097
2006	81.28981682	1.910	-0.012	0.0001
2007	85.25706444	1.931	0.009	0.0001
2008	163.9577483	2.215	0.293	0.0857
2009	105.825426	2.025	0.103	0.0105
2010	95.17604483	1.979	0.057	0.0032
	Total sum =	32.672		0.9198
$\bar{Y} =$	1.922			
$\sigma =$	0.240			

RP, T(yr)	P	T	1/p*p	ln(1/P ²)	W=(ln(1/p ²)) ^{0.5}	W ²	W ³	A	B	A/B	KT	Mean (\bar{y})	SD (σ)	YT	XT(m ³ /s)
2	0.500	2.00	4.00	1.39	1.18	1.39	1.63	3.48	2.95	1.18	0.00	1.92	0.24	1.92	83.54
10	0.100	10.00	100.00	4.61	2.15	4.61	9.88	4.29	4.96	0.86	1.28	1.92	0.24	2.23	169.52
25	0.040	25.00	625.00	6.44	2.54	6.44	16.33	4.62	5.88	0.79	1.75	1.92	0.24	2.34	219.66
50	0.020	50.00	2500.00	7.82	2.80	7.82	21.89	4.84	6.52	0.74	2.05	1.92	0.24	2.41	259.68
100	0.010	100.00	10000.00	9.21	3.03	9.21	27.95	5.05	7.13	0.71	2.33	1.92	0.24	2.48	301.85

Appendix C: River cross section data at Downstream and Upstream of Ginchi Awash Bridge (Surveying Data).

Table C.1: Cross section data at Downstream of the Bridge

Station1. Downstream		Station 2. Dowsream		Station 3. Downstream	
Station	Elevation	Station	Elevation	Station	Elevation
1	2216.56	1	2216.83	1	2216.69
2	2214.95	2	2215.12	2	2215.84
3	2214.73	3	2214.49	3	2215.26
4	2214.69	4	2214.37	4	2214.85
5	2214.28	5	2213.36	5	2213.33
6	2211.09	6	2212.58	6	2212.69
7	2210.91	7	2212.39	7	2211.96
8	2211.39	8	2211.32	8	2211.28
9	2210.65	9	2210.33	9	2211.02
10	2210.36	10	2210.01	10	2210.98
11	2210.39	11	2209.65	11	2209.98
12	2209.85	12	2209.57	12	2209.23
13	2209.66	13	2209.18	13	2208.81
14	2208.62	14	2209.16	14	2207.35
15	2206.87	15	2207.77	15	2207.06
16	2205.95	16	2206.83	16	2207.28
17	2206.79	17	2207.75	17	2206.25
18	2206.86	18	2209.85	18	2206.04
19	2208.309	19	2210.63	19	2207.29
20	2210.95	20	2210.66	20	2208.86
21	2210.93	21	2210.64	21	2208.87
22	2211.21	22	2210.73	22	2209.38
23	2211.96	23	2209.75	23	2209.65
24	2212.84	24	2209.87	24	2209.78
25	2213.98	25	2210.02	25	2210.22
26	2214.01	26	2211.96	26	2211.03

27	2214.58	27	2212.48	27	2211.92
28	2214.21	28	2213.02	28	2212.29
29	2215.15	29	2213.94	29	2213.59
30	2215.85	30	2214.29	30	2214.62
31	2215.92	31	2214.63	31	2214.67
32	2216.12	32	2215.26	32	2215.2
33	2216.73	33	2216.69	33	2216.58

Station 4. Downstream

<u>Station</u>	<u>Elevation</u>
1	2216.86
2	2215.35
3	2214.28
4	2214.09
5	2213.59
6	2212.66
7	2211.59
8	2211.39
9	2210.38
10	2210.21
11	2209.69
12	2209.68
13	2209.05
14	2209.01
15	2208.26
16	2207.25
17	2207.93
18	2208.06
19	2208.16
20	2208.56
21	2209.62
22	2210.15
23	2210.64
24	2210.79
25	2210.92
26	2211.91
27	2212.28
28	2213.08
29	2213.93
30	2214.35

Station 5. Downstream

<u>Station</u>	<u>Elevation</u>
1	2216.99
2	2215.16
3	2214.24
4	2213.75
5	2213.85
6	2212.59
7	2211.92
8	2211.87
9	2211.44
10	2210.92
11	2210.85
12	2210.59
13	2210.01
14	2209.29
15	2208.06
16	2207.05
17	2207.04
18	2208.19
19	2209.96
20	2210.22
21	2210.18
22	2210.42
23	2210.55
24	2211.56
25	2212.29
26	2212.7
27	2213.2
28	2213.59
29	2214.52
30	2214.93

31	2214.89	31	2214.96
32	2215.69	32	2215.28
33	2216.84	33	2216.89

Table C.2: Cross section data at Upstream of the Bridge

Station 6. Upstream		Station 7. Upstream		Station 8. Upstream	
Station	Elevation	Station	Elevation	Station	Elevation
1	2217.05	1	2217.29	1	2216.98
2	2215.86	2	2216.08	2	2216.41
3	2215.63	3	2215.28	3	2215.92
4	2215.14	4	2214.68	4	2214.25
5	2214.62	5	2214.05	5	2213.83
6	2213.95	6	2214.22	6	2213.59
7	2213.05	7	2212.56	7	2212.65
8	2212.81	8	2212.39	8	2211.3
9	2212.28	9	2211.37	9	2210.92
10	2211.69	10	2210.41	10	2210.21
11	2210.16	11	2210.256	11	2210.08
12	2210.02	12	2208.85	12	2210.09
13	2209.13	13	2208.3	13	2209.87
14	2209.06	14	2208.32	14	2209.63
15	2208.62	15	2207.33	15	2209.62
16	2208.63	16	2207.26	16	2209.12
17	2207.93	17	2208.27	17	2208.56
18	2208.56	18	2208.96	18	2208.22
19	2208.75	19	2208.76	19	2208.69
20	2209.85	20	2209.53	20	2209.432
21	2209.93	21	2210.08	21	2209.65
22	2210.98	22	2210.09	22	2210.19
23	2211.61	23	2210.64	23	2210.82
24	2211.88	24	2211.13	24	2211.99
25	2211.97	25	2212.05	25	2211.79
26	2212.01	26	2212.18	26	2212.42
27	2212.96	27	2213.65	27	2213.62
28	2213.29	28	2214.28	28	2213.86
29	2213.68	29	2214.3	29	2215.365
30	2214.35	30	2215.76	30	2216.89
31	2215.19	31	2215.88	31	2217.02
32	2215.72	32	2216.15	32	2217.18
33	2217.11	33	2217.54	33	2217.23

Station 9. Upstream

<u>Station</u>	<u>Elevation</u>
1	2217.95
2	2217.59
3	2217.15
4	2216.28
5	2215.14
6	2214.83
7	2213.58
8	2212.51
9	2211.64
10	2210.42
11	2210.327
12	2209.923
13	2209.85
14	2209.24
15	2209.32
16	2209.12
17	2208.58
18	2208.61
19	2208.62
20	2209.56
21	2209.73
22	2210.46
23	2210.48
24	2210.91
25	2211.95
26	2212.23
27	2213.05
28	2213.86
29	2214.58
30	2215.65
31	2216.82
32	2217.55
33	2217.64

Station 10. Upstream

<u>Station</u>	<u>Elevation</u>
1	2218.23
2	2218.15
3	2217.89
4	2216.59
5	2215.38
6	2214.95
7	2214.08
8	2213.29
9	2212.59
10	2211.62
11	2211.45
12	2210.38
13	2210.2
14	2210.18
15	2209.52
16	2209.43
17	2208.99
18	2208.99
19	2209.26
20	2209.78
21	2209.9
22	2210.28
23	2210.25
24	2210.39
25	2211.65
26	2212.27
27	2213.65
28	2214.06
29	2214.83
30	2215.64
31	2216.86
32	2217.29
33	2217.52

Station 11. Upstream

<u>Station</u>	<u>Elevation</u>
1	2218.75
2	2218.26
3	2217.63
4	2216.14
5	2215.62
6	2214.95
7	2214.05
8	2213.81
9	2213.28
10	2212.5
11	2211.16
12	2211.02
13	2210.13
14	2210.06
15	2210.62
16	2210.63
17	2210.39
18	2210.56
19	2210.75
20	2211.85
21	2211.93
22	2211.98
23	2212.61
24	2212.88
25	2212.97
26	2213.01
27	2213.96
28	2214.29
29	2214.68
30	2215.35
31	2216.19
32	2217.72
33	2218.11

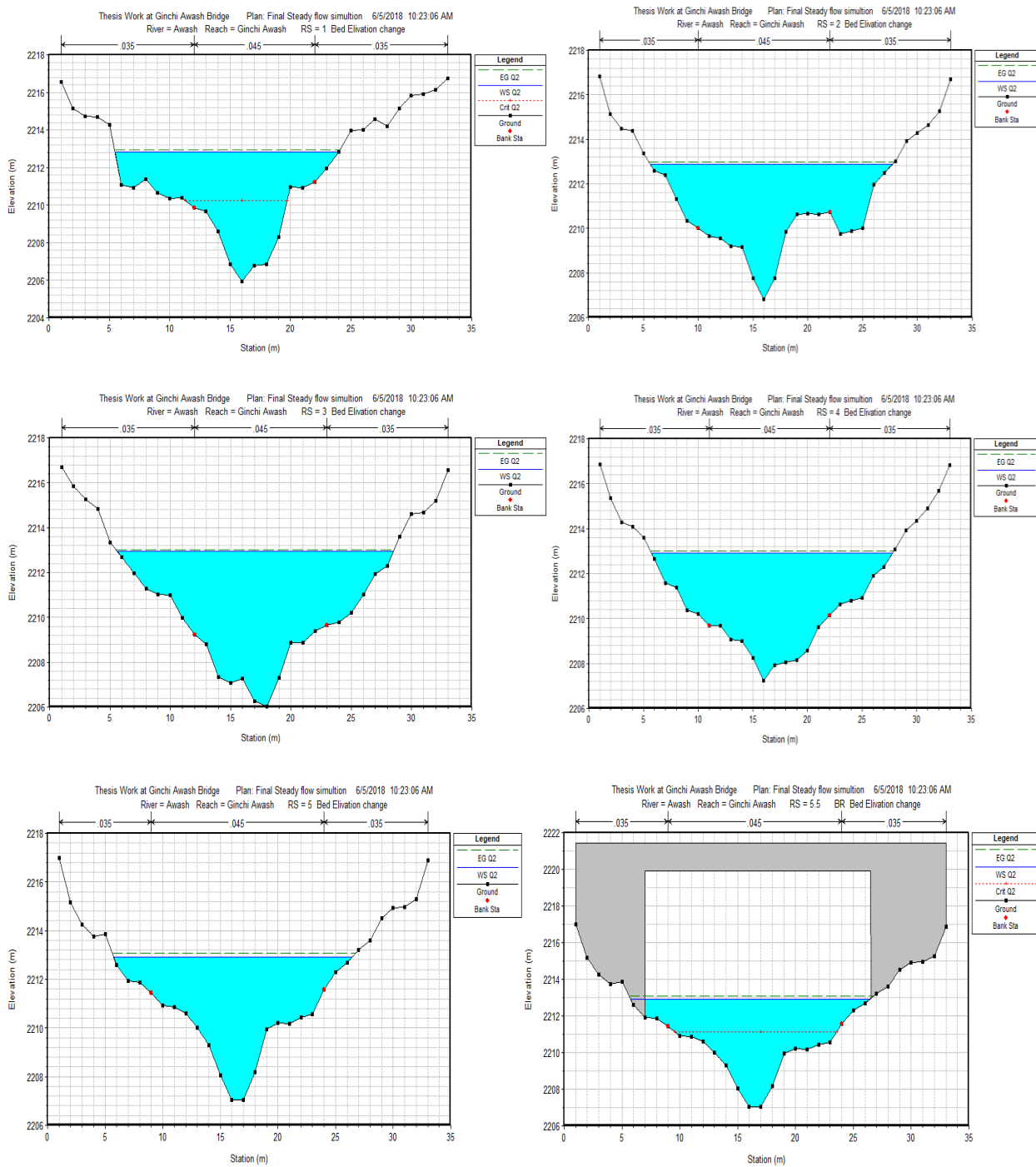


Figure C0-1: River cross section profile Downstream of the Bridge crossing

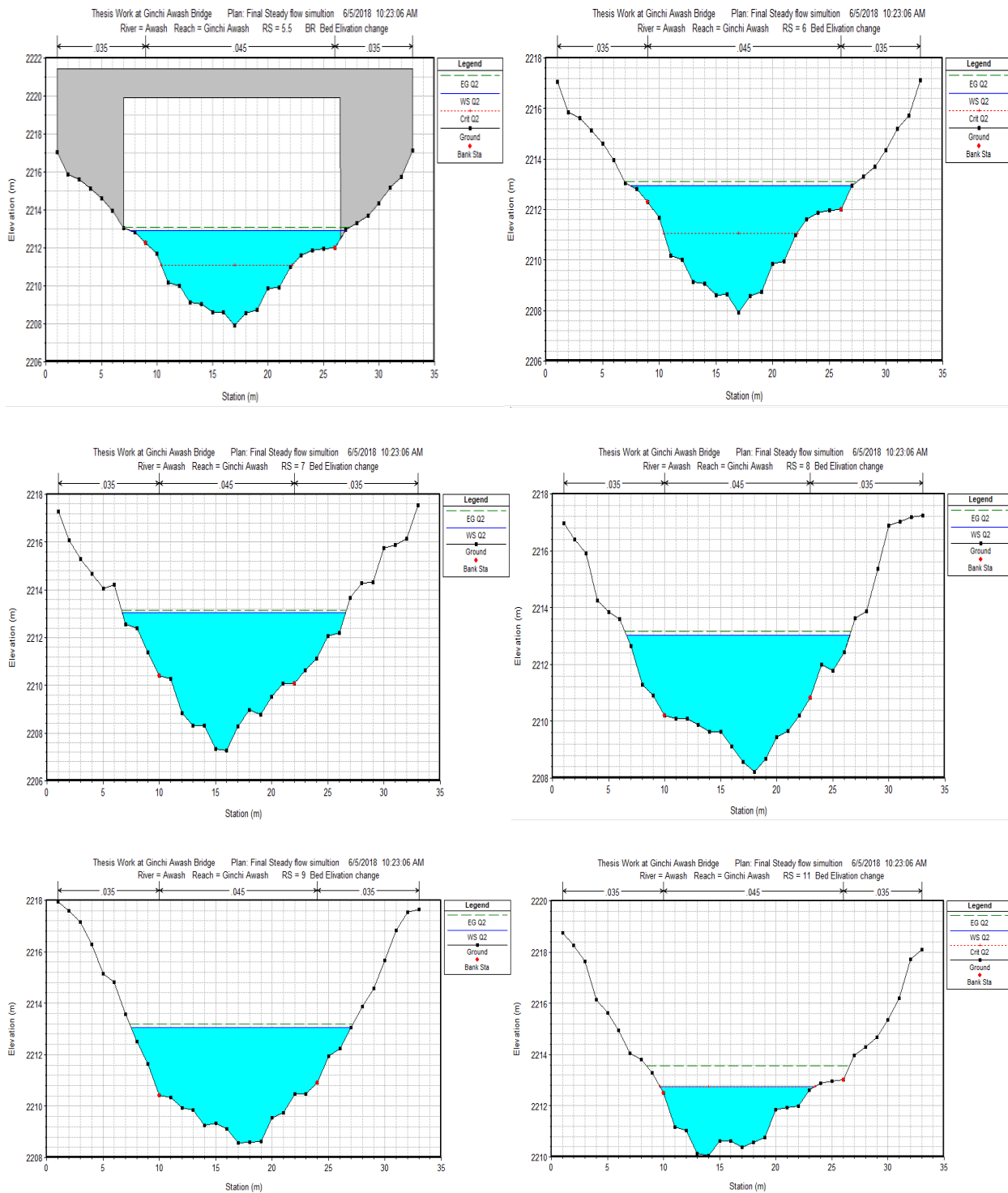


Figure C0-2: River cross section profiles Upstream of the Bridge crossing.

Appendix D.1: Grain Size Analysis of River Bed Soil Sample Data.

Date Tested: 4/5/2018
 Tested By: Hawi Adicho
 Project Name: Ginchi Awash Bridge
 Visual Classification of soil: Black Cotton

Total Mass of soil Before socking : 1500 (gm)
 Wt. of soil removed by socking : 443.5 (gm)
 Weight of container : 93 (gm)
 Wt. Container + Dry soil : 1.1495 (gm)
 Wt. of Dry soil: 1056.5 (gm)

Table: D.1.1: Soil Sample gradation Data for Sieve analysis for the case of River Bed.

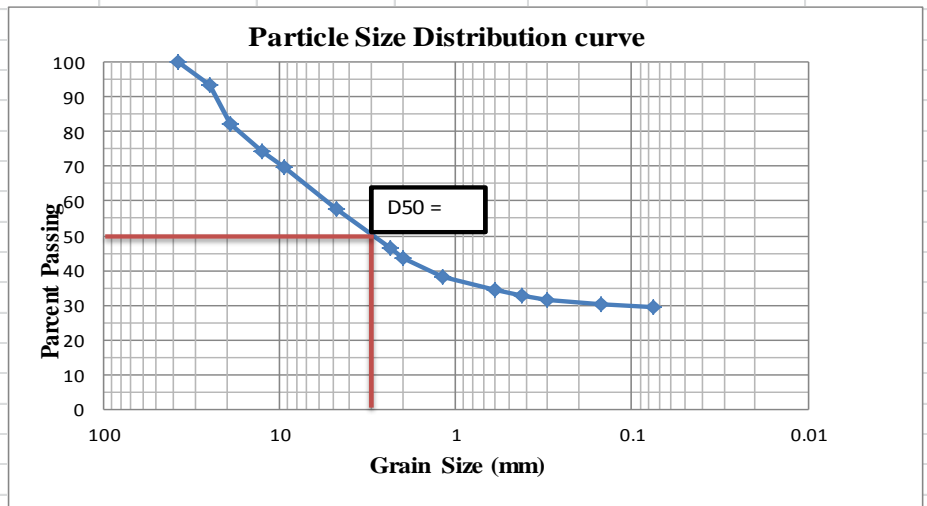
Sieve Di- am(mm)	M of Empty Sieve in (gm.)	M of Soil +M of sieve(gm.)	M of soil re- tained(gm.)	% Re- tained	Cumm % Re- tain	% Passing
37.5	440.5	440.5	0	0.00	0.00	100.00
25	453.5	554	100.5	6.69	6.69	93.31
19	443.5	611.5	168	11.18	17.86	82.14
12.5	447	562	115	7.65	25.52	74.48
9.5	585	653	68	4.52	30.04	69.96
4.75	431.5	613.5	182	12.11	42.15	57.85
2.36	387.5	561.5	174	11.58	53.73	46.27
2	555.5	599	43.5	2.89	56.62	43.38
1.18	354.5	432	77.5	5.16	61.78	38.22
0.6	323.5	383	59.5	3.96	65.74	34.26
0.425	293	316	23	1.53	67.27	32.73
0.3	302.5	320	17.5	1.16	68.43	31.57
0.15	282.5	302	19.5	1.30	69.73	30.27
0.075	256	267.5	11.5	0.77	70.49	29.51
Pan	254	697.5	443.5	29.51	100.00	0.00
Total Mass =			1503			

***Percent Passing = 100-Cummulative Percent Retained**

Table D.1.2: Classification of soil grain size and Particle Size distribution curve for the case of River bed material.

Class Name	Millimeters	Micrometers	Phi Values
Boulders	> 256		< -8
Cobbles	256 - 64		-8 to -6
Gravel	64 - 2		-6 to -1
Very coarse sand	2.0 - 1.0	2000 - 1000	-1 to 0
Coarse sand	1.0 - 0.50	1000 - 500	0 to +1
Medium sand	0.50 - 0.25	500 - 250	+1 to +2
Fine sand	0.25 - 0.125	250 - 125	+2 to +3
Very fine sand	0.125 - 0.062	125 - 62	+3 to +4
Coarse silt	0.062 - 0.031	62 - 31	+4 to +5
Medium silt	0.031 - 0.016	31 - 16	+5 to +6
Fine silt	0.016 - 0.008	16 - 8	+6 to +7
Very fine silt	0.008 - 0.004	8 - 4	+7 to +8
Coarse clay	0.004 - 0.0020	4 - 2	+8 to +9
Medium clay	0.0020 - 0.0010	2 - 1	+9 to +10
Fine clay	0.0010 - 0.0050	1 - 0.5	+10 to +11
Very fine clay	0.0005 - 0.00024	0.5 - 0.25	+11 to +12
Colloids	< 0.0024	< 0.24	> +12

Percent Passing(%)	Grain Size(mm)
100.00	37.5
93.31	25
82.14	19
74.48	12.5
69.96	9.5
57.85	4.75
46.27	2.36
43.38	2
38.22	1.18
34.26	0.6
32.73	0.425
31.57	0.3
30.27	0.15
29.51	0.075



From Graine Size Distributio Curve

- 1. % Gravel = 53.726
- 2. % Sand = 16.766
- 3. % Fines = 29.508

From the Particle size distribution curve D50= 3mm

Appendix D.2: Grain Size Analysis of left and right Bank Soil Sample Data

Date Tested: 4/5/2018
 Tested By: Hawi Adicho
 Project Name: Ginchi Awash Bridge
 Visual Classification of soil: Black Cotton

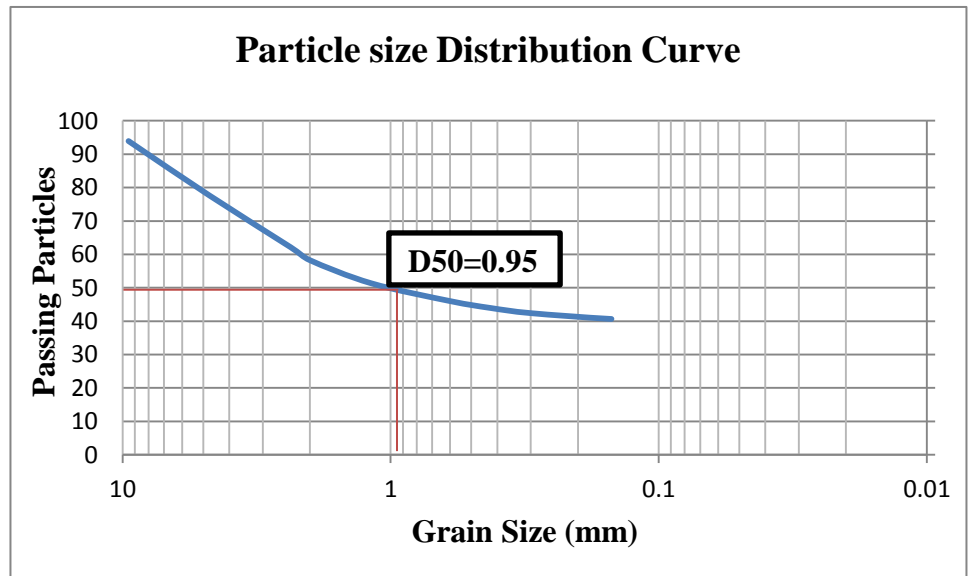
Total Mass of soil Before soaking: 1119.5 (gm)
 Wt. of soil removed by soaking: 443.5 (gm)
 Weight of container: 93 (gm)
 Wt. Container + Dry soil: 769 (gm)
 Wt. of Dry soil: 676 (gm)

Table: D.2.1: Soil Sample gradation Data for Sieve analysis for the case of left and right Bank.

Seive Di-am(mm)	M of Empty Sieve in (gm)	M of Soil+M of sieve(gm)	M of soil re-tained(gm)	% Re-tained	Cumm % Retain	% Pass-ing
9.5	585	653	68	6.07	6.07	93.93
4.75	431.5	613.5	182	16.26	22.33	77.67
2.36	387.5	561.5	174	15.54	37.87	62.13
2	555.5	599	43.5	3.89	41.76	58.24
1.18	354.5	432	77.5	6.92	48.68	51.32
0.6	323.5	383	59.5	5.31	53.99	46.01
0.425	293	316	23	2.05	56.05	43.95
0.3	302.5	320	17.5	1.56	57.61	42.39
0.15	282.5	302	19.5	1.74	59.35	40.65
0.075	256	267.5	11.5	1.03	60.38	39.62
0	254	697.5	443.5	39.62	100.00	0.00
Total Mass of Soil =			1119.5			

Table D.2.2: Classification of soil grain size and Particle Size distribution curve for the case of left and right Bank.

Percent Passing (%)	Grain Size(mm)
93.93	9.5
77.67	4.75
62.13	2.36
58.24	2
51.32	1.18
46.01	0.6
43.95	0.425
42.39	0.3
40.65	0.15
39.62	0.075
0.00	0



From Grain Size Distribution Curve

- 1. % Gravel = 37.870
- 2. % Sand = 22.510
- 3. % Fines = 39.620

Appendix H: Sediment data.

Table H.1: Sediment Data

Ministry of Water, Irrigation and Electric / Hydrology and Water Quality Directorate											
Laboratory Result of Sediment Suspension Analysis											
Sampled by		Hydrology Technicians									
Conducted by		Lab technician									
Checked by											
Lab.	River / Stream	Station / Near	Basin	Region Office	Time Taken (Sec)	G/height (m)	Flow (m3/s)	Depth (m)	Direction	Width (m)	Sediment Conc.(mg/l)
	Awash	Ginchi	Awash	Addis Ababa	14	0.94	0.57	0.22	Left	2.4	378.2
	Awash	Ginchi	Awash	Addis Ababa	8	0.94	0.57	0.22	Left	3.6	349.4
	Awash	Ginchi	Awash	Addis Ababa	13	0.94	0.57	0.12	Left	5.6	469
	Awash	Ginchi	Awash	Addis Ababa	25	0.73	0.128		Left	1.1	331.3
	Awash	Ginchi	Awash	Addis Ababa	30	0.73	0.128		Left	2.3	238.12
	Awash	Ginchi	Awash	Addis Ababa	35	0.73	0.128		Left	3.2	345.45
026/04	Awash	Ginchi	Awash	Addis Ababa	18	0.89	0.48	0.36	Left	2	185.76
	Awash	Ginchi	Awash	Addis Ababa	20	0.89	0.48	0.34	Left	4	189.39
	Awash	Ginchi	Awash	Addis Ababa	22	0.89	0.48	0.1	Left	6	184.85
194/00	Awash	Ginchi	Awash	Addis Ababa	8	0.84	0.6	0.4	Left	2.35	651.56
	Awash	Ginchi	Awash	Addis Ababa	10	0.84	0.6	0.82	Left	4.7	620.63
	Awash	Ginchi	Awash	Addis Ababa	12	0.84	0.6	0.53	Left	7.05	621.52
	Awash	Ginchi	Awash	Addis Ababa	23	0.5	1.014	0.24	Left	3	1075.4286
	Awash	Ginchi	Awash	Addis Ababa	21	0.5	1.014	0.34	Left	7	1078.7302
	Awash	Ginchi	Awash	Addis Ababa	25	0.5	1.014	0.42	Left	11	1082.5

Time(hr)	Flow(Q)(m3/s)	Volume(m3)	V(l)	Sediment(mg/l)	Sediment (mg)	Sediment(tonne)	Sediment(tonne)
14	0.57	7.98	7980	378.2	3018036	0.003	18.626
8	0.57	4.56	4560	349.4	1593264	0.002	17.207
13	0.57	7.41	7410	469	3475290	0.003	23.097
25	0.128	3.2	3200	331.3	1060160	0.001	3.664
30	0.128	3.84	3840	238.12	914380.8	0.001	2.633
35	0.128	4.48	4480	345.45	1547616	0.002	3.820
18	0.48	8.64	8640	185.76	1604966.4	0.002	7.704
20	0.48	9.6	9600	189.39	1818144	0.002	7.854
22	0.48	10.56	10560	184.85	1952016	0.002	7.666
8	0.6	4.8	4800	651.56	3127488	0.003	33.777
10	0.6	6	6000	620.63	3723780	0.004	32.173
12	0.6	7.2	7200	621.52	4474944	0.004	32.220
23	1.014	23.322	23322	1075.428571	25081145.14	0.025	94.218
21	1.014	21.294	21294	1078.730159	22970480	0.023	94.507
25	1.014	25.35	25350	1082.5	27441375	0.027	94.837