Addis Ababa University

Addis Ababa Institute of Technology

School of Civil and Environmental Engineering

HYDRAULIC PERFORMANCE of RIVER CROSSING STRUCTURE AGAINST SEDIMENT AGGRADATION and TRASH ACCUMULATION

(The Case of Arsema Bridge from German round about to Lebu Mebrate Haile)

A thesis Submitted to Addis Ababa institute of Technology, School of graduates Studies,

Addis Ababa University

In Partial Fulfillment of the Requirement for Master of Science Degree in Civil and Environmental Engineering

Hydraulic Engineering Stream

By

Wendmagegnehu Taye

Advisor: Dr. Yilma Seleshi

Addis Ababa, Ethiopia

May 2019

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Declaration

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

Wendmagegnehu Taye

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Wendmagegnehu T.

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Addis Ababa, Ethiopia.

Abstract

Arsema Bridge was constructed on Jemo River, Jemo River is tributary of Little Akaki River which is the sub catchment of Akaki basin and study area has an inclusive area of **73.3 Square Kilometer**, Arsema Bridge built as River crossing structure to connect the ring road from **German round about to Lebu Mebrate Haile under Addis Ababa city Administration.**

It had been built just before 15 years ago and currently the Bridge faces a serious problem of sediment aggradations both at the left and right side of the abutment, and trash accumulation on the entrance of the Bridge. Due to this; overtopping of the Bridge and flooding of residential house were observed frequently.

HEC- HMS modeling was used to determine peak discharge of **2, 5, 10, 25, 50** and **100** year return periods and then **67, 97.6, 117, 136.1, 154.5** and **174.9** m^3/sec results were found, subsequence HEC-RAS Modeling was used to determine water surface profile level, performance of existing bridge when difference return period peak discharge pass through it and sediment transport at the bridge location based on the current river sectional profiles however the bridge accommodate only up to 10 year return flow.

Then Q_{50} was used to analysis before and after bridge constructed, then after the water surface elevation is **2214.36m** and **2214.57m** amsl respectively however existing bridge high chord is **2213.684m** amsl i.e. **0.676m** and **0.886m** difference before and after the bridge constructed, this show us the existing bridge is submerged as per previously estimated flood.

After detail analysis of the result and discussion I recommend the appropriate remedial measure for sedimentation aggradations and trash accumulation as well as flooding of the residential house, HEC-RAS was used with combination of **HEC- RAS Mapping**: **to redesign of Bridge with Trash Trap Screen** and **Flood Hazard map,** the result show us road surface finishing level has to increase at least **0.766 m high at the bridge and refill** both left and right side of the bridge up to **150 m length** as per road profile; to accommodate the redesigned bridge high chord of **2214.45m** amsl, residential house upstream to bridge has to consider flood protection or guide wall up to **2.0m** high depending on their **OGL** against the **flood hazard map**, **relocate of all utilities line** which are pass through bridge and period clearing of trash trap screen is highly recommended as main mitigation measure.

Key Words: **Bridge, HEC Geo HMS, HEC _HMS, flood frequency analysis, HEC_RAS, RAS mapping, trash accumulation and sediment aggradations.**

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List of Abbreviations and Glossary

AACRA= Addis Ababa City Road Authority.

Amsl = Above Mean Sea level.

DS= Downstream

EMA= Ethiopia Meteorological Agency.

ERA=Ethiopia Road Authority.

ERA=Ethiopia Road Authority Drainage Design Manual.

HEC- HMS= Hydrologic Engineering Center_ Hydrologic Modeling System.

HEC- RAS=Hydrologic Engineering Center_ River Analysis System.

K_T = Frequency Factor

MoWIE=Ministry of Water, Irrigation and Electricity

OGL=Original Ground Level

Q_n = Peak discharge at n-year return period.

P= Exceedence Probability

W= intermediate variable

UH =Unit Hydrograph

US= Upstream.

Z= standard normal variable

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1. General

1.1. Background

Bridge is structure that transport traffic over waterways or other obstructions: stream crossing system that includes approach roadway over the flood plain and relief openings, with structural centerline span of 6 m or more, even a large box culverts of multi-plate arches, designed hydraulically as bridges, accord to ERA DDM, (2013).

Generally bridge design comprises two aspects: Hydraulic and Structural, different studies conducted in US and UK and the results show that many bridge failures are due to hydraulic effects than structural.

Unfortunately engineers give great attention for structural design than the hydraulic. Undermining the bridge opening size, sediment aggradations, trash accumulation and scouring are major problems in bridge hydraulic design and these factors play a great roll behind failures of bridge structure.

Hydraulic problem on bridge structures is also a great problem in Ethiopia. Many bridge and culvert failed due to flooding and scouring, northern part of the country bridge failures are mostly by **scouring effects** whereas southern and eastern: flooding and overtopping are the major cause. Arsema Bridge is an example to show bridge hydraulic performance reduction due to sediment aggradations at downstream and upstream of the bridge as well as trash accumulation in the inlet, further more layout of the bridge with respect to the flow direction contributes additional problems on the bridge performance.

The main objective of this thesis work is to show the effects of sediment aggradations and trash accumulation on the hydraulic performance of bridge structures.

Different software's were used to model the actual field events. This would helped us to recognize the problem of the bridge very easy way.

1.2. Statement of the Problem

Arsema Bridge is built on Jemo River, the Jemo River is one the tributary of little Akaki river in Awash catchment basin, this bridge connect the ring road between Lebu and German round about. Currently the bridge is insufficient to pass the incoming flood, the main observed causes are: accumulation of trash in the entry of the bridge, development of silt in the approach channel of bridge as well as narrow opening of the bridge. The main defects observed is overtopping of the bridge, clogging of runoff collector and flooding of road surface which is hindering traffic movement of light vehicles and flooding of buildings at upstream side of bridge.

During rainy season: it is common to see the accumulation of logs, trashes and silt. See the below figure.

Figure 1 Trash accumulation and blockage of the bridge

1.3. Significant of the Study

The study of sedimentation and trash accumulation helps us to plan sustainable design, construction, and maintenance technique such as:

- To plan mitigation measure for flood prone hydraulic structure.
- How to minimize the clogging of the bridge entry by trash.
- It can be used as early warning flood forecasting tool.
- It inspires other to do similar study.

1.4.Research Questions

- What was the design flood of the bridges?
- Is the bridge cross section is sufficient enough to pass the design flood?
- Does the bridge have a direct effect on sedimentation, trash accumulation and river bed change?
- What are mitigation measures that improve the drainage problems?

1.5. Objective

1.5.1. General Objective

The general objective of this study is investigating the effect of sedimentation and trash accumulation against the hydraulic performances of Arsema Bridge from hydrologic and hydraulic perspective.

1.5.2. Specific Objective

- To evaluate the design discharge and current Bridge opening capacity.
- To determine the source and cause of sedimentation, trash accumulation and then to provide possible mitigation measure.

1.6. Limitation of the Study

I couldn't find the hydrologic analysis report of the bridge as well as future land use and land cover plan of the study area: So the hydrologic analysis report wasn't triangulated, and satellite image of the Google earth was used as input data for CN determination.

1.7. Thesis Work Outline and Organization

This thesis paper generally organized into five chapters. Chapter one: general introduction, statement of the problem, objectives, significant and limitation of study area are included. Chapter two: other scholar's paper works are sited, discussed and summarized in related to the scope and objective of this thesis works. Chapter three: project location description, data collection and analysis with different methodology were compiled. In Chapter four: results and discussions presented and mitigation measures are proposed. In the Last chapter conclusion and recommendation is articulated.

2. Literature Review

2.1.Bridge Hydraulics

2.1.1. Overview on Hydraulic Design of Bridges

Bridges that cross rivers and valleys are vital components of the road network that contributes greatly to the national development and public daily life. According to Yonas B. (2015)

Any collapse of bridge structure can cause the risk of the lives of road users as well as create serious influence to the entire country; furthermore, the construction of these road drainage structures needs considerable amount of skilled force, money and time. Drainage Structure intended to allow the runoff of any flow of water with limited drainage damages and disturbances to the road and to the surrounding areas. Yifred K., (2013)

In fact, 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.

Presently, there are 2,955 bridges along the federal road network, 4.7% of the bridges are located on the trunk roads of A1 (A-A – Djibouti), A2 (A-A – Axum), A3(AA- Gondar), A4 (AA- Gimbi) and A5 (AA- Metu) 33.61 % are rated poor conditions that require interventions 30% needs repair and 3.6% replacement. The fact that 35.2% of all the bridges on the A1 road need repair or replacement indicates the seriousness of the bridge conditions in Ethiopia. Some bridges are in good condition but they need repair or rehabilitation due to hydrological problems. Beza N. (2010)

The two main types of water flows that can be considered are the flows that usually crossing the area that could be diverted by the presence of the road and the flows generated by the runoff of the rainwater falling on the carriageway and its surroundings. The basic design techniques in roadway drainage system should be developed for economic design of surface drainage structures including ditches, culverts and bridges. (ERA, 2002), the most common cause of highway bridge failure is due to the adverse hydraulic action of the water that passes through it.

It is therefore essential that sufficient attention has to pay to the prevention of such failures when designing bridges over rivers, or flood-plains. It is clear that the design of bridges across watercourses requires a multi-disciplinary approach, such as highway, structural, geotechnical, hydrology and hydraulic expertise.

According to Les Hamill, (2004) the two overall objectives for the Hydraulic design of Bridges are:

- The effect of constructing the bridge on the existing water regime should be kept to the minimum.
- The Structural design should aim to prevent failure under the various types of hydraulic actions.

2.1.2. Bridge Site Characteristics

A complete understanding of the physical nature of stream reach is importance to have a good hydraulic design: particularly at the site of interest. Any work being performed, proposed or completed, that changes the hydraulic efficiency of a stream reach, must be studied to determine its effect on the stream flow.

The stream classified as: rural or urban, improved or unimproved, narrow or wide, shallow or deep, rapid or sluggish, stable, transitional, or unstable, sinuous, straight, braided, alluvial, or incised and perennial or intermittent flow.

The location, size, description, condition, observed flood stages, and channel section relative to existing structures on the stream reach and near the site must be secured in order to determine their capacity and effect on the stream flow. For bridge, this data should include span lengths, type of piers, and substructure orientation which can usually be obtained from existing structure plans. Photographs and high water profiles or marks of flood events at the structure and past flood scour data can be valuable in assessing the hydraulic performance of the existing facility, ERA Drainage Design Manual, 2013.

2.1.3. Outcome of Bridge Construction on the River

When a bridge is placed in a river it will reduce the natural channel capacity and acts as an obstacle to the flow. This results a loss of energy as the flow contracts, passes through the bridge and then, most significantly, re-expands back to the full channel width. To provide the additional head necessary to overcome the energy loss the upstream water level increases above which would not usually experienced without the Bridge. This additional head is called the afflux, and its variation with distance upstream is called the backwater profile. (Les Hamill, 2004). Hydraulic engineers and designers are faced with a wide variety of choices when determining the capacity or location of a new bridge or an existing bridge that is to be replaced (Hydraulic Design of Safe Bridges, 2012). Sometimes it becomes very difficult to decide whether the construction of a Bridge has a direct effect on the rise in back water level or not. First appearances will lead to a total wrong conclusion.

In many situations flooding, in the form of overbank flow, would occur even if the bridge was not there. According to Les Hamill (2004), a rough guide can be developed to investigating whether or not a bridge is the primary cause of flooding by comparing the hydraulic capacity of the main river channel without the bridge (Q_R), with the capacity of the bridge waterway (Q_W) and the design flood (Q_{DF}).

- If $Q_R < Q_W$ the bridge is relatively blameless;
- If $Q_R < Q_{DF}$ inundation of the floodplains would occur without the bridge;
- If $Q_W < Q_R$ the Bridge forms an obstacle to flow and may cause or exacerbate flooding;
- If $Q_W < Q_{DF}$ the waterway is under designed;
- If $Q_W > Q_{DF}$ the waterway is overdesigned or has a margin of safety.

The distance between the constriction and section 0 depends upon such factors as the geometry, roughness and slope of the channel, and can be calculated using a backwater analysis. In the reach affected by the backwater the depth will be greater than normal, so the velocity and energy loss are less than would otherwise occur (Les Hamill, 2004). This will in turn facilitates the accumulation of trash and sediment materials, if there is excess supply of trash and sediment on the upstream side of the river.

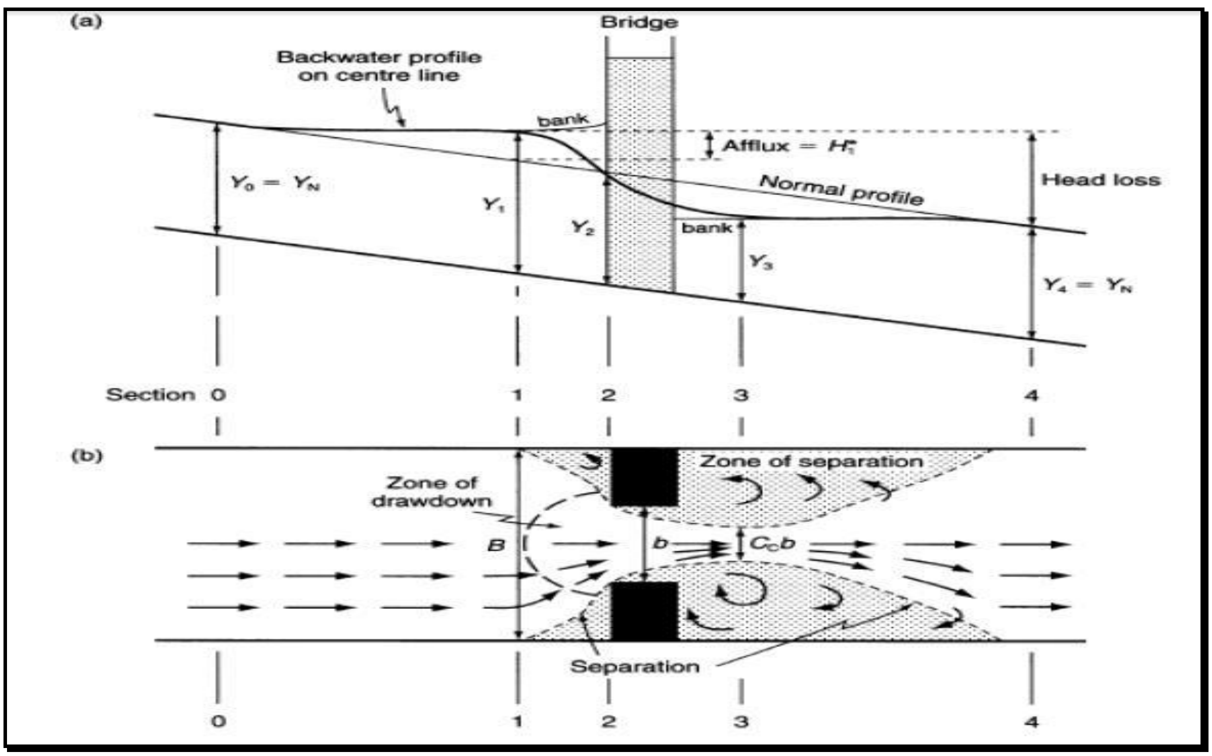


Figure 2 Outcome of bridge construction on the river

2.2. Trash Accumulation, Sediment Aggradations and Human Activities

2.2.1. Trash Accumulation

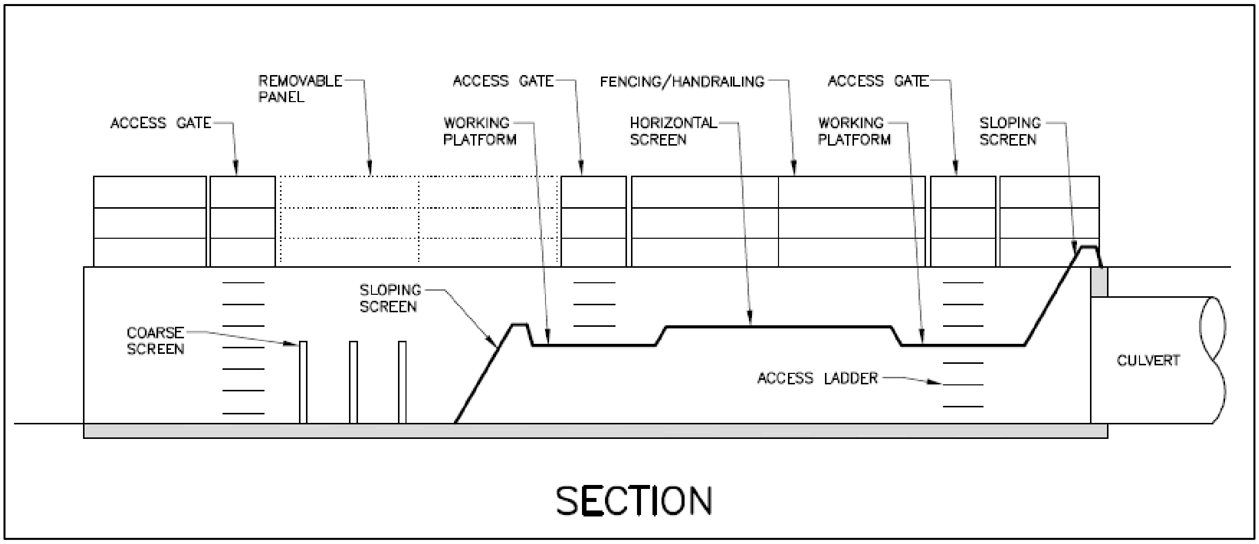
Trash build-up against bridge components can significantly affect the hydraulic performance of bridges; problem of blockage is normally associated with small single span bridges than large multi-span. It create water level rise on upstream and associated flooding. **Trash may partially restrict the flow leading to significant scour around piers or abutments threatening the safety of the structure,** as per Debris control structures evaluation and countermeasure, 2005.The likelihood of blockage should be considered for all culverts, even it can occur through siltation or vegetation, however blockage siltation by is more temporary in nature.

Culvert entrance in urban area should be protected with a **rack or grate** because urban flood flows are **quick**, **concentrated**, and **fast**, Guo, J. C.Y., Jones, J and Earles, A. 2008.

Blockage causes a reduction in the capacity of hydraulic structures and drainage systems in which they lie, especially if the blockage occurs prior to or around the flood peak. Blockage of structures also has the potential to divert flows into adjacent streams or catchments, thereby altering flood levels and flood-related damage as per Australia Rainfall Runoff Project blockage of hydraulic structures, 2013.

There is no clearly defined distinction between debris types and trash screen, the screen can causes several problems, most notably local flooding due to blockage of the screen. Decision to provide a screen at any location must be based on a full appreciation of the risk and benefits. It is essential that all practical alternatives are investigated and eliminated before reaching a decision to provide the screen, decision making may be aided by benefits cost analysis though, in the case of security screen. TSSG (EA, 2009.)

Determination of Screen Area- main component of trash screens are set out in the below figure.



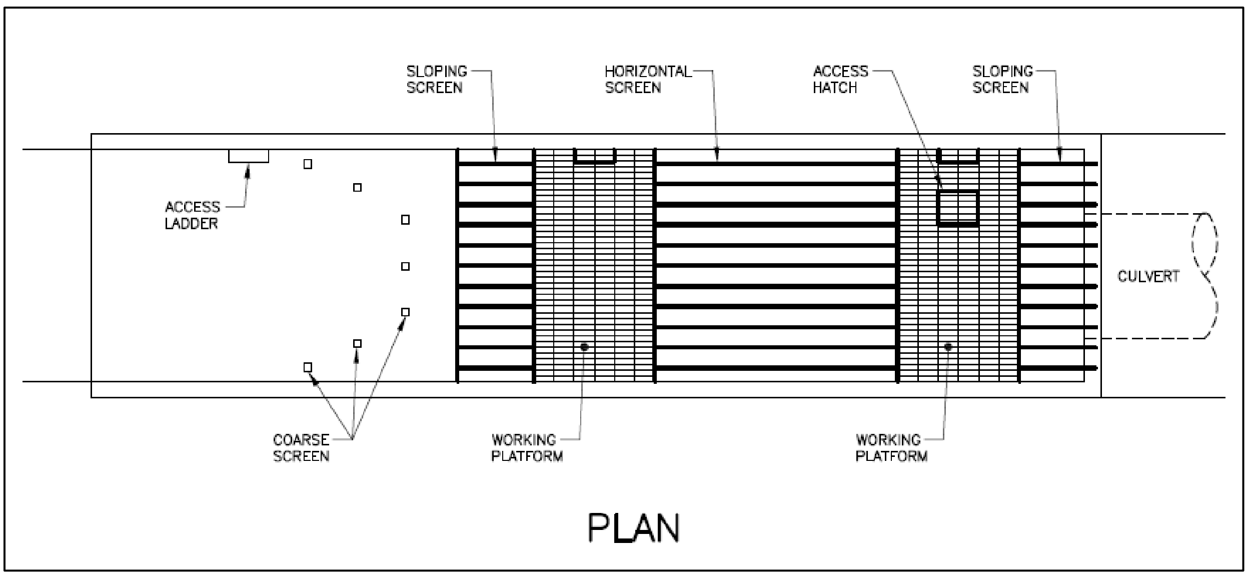


Figure 3 Trash trap screen photo, section and plan (source: TSSG, 2010).

Sizing of Screen Area is depending of blinded depth factor (Bdf) and design debris amount (Dda)

$$Screen\ Area(m^2) = \frac{\text{Design debris amount (Dda)}}{\text{No. of significant events} * \text{blinded depth factor (Bdf)}}$$

Blinded depth factor (Bdf) is based on the predominant catchment types whereas Design debris amount is the contributing measuring length of each five catchment type.

Thus the screen area determine by evidence based method should be in the range of three times to thirty times of area of culvert, According to TSSG (EA, 2009.)

Bar spacing (Trash screens)

Trash screens placed upstream of culverts and inverted siphons should have a **minimum clear spacing of 150 mm between bars.** The spacing should prevent the passage of material of the type and size likely to pose a significant risk at the site. In **urban locations** where larger debris needs to be excluded but smaller debris should be allowed to pass, spacing of **300 mm between bars may be appropriate**, According to TSSG (EA, 2009).

2.2.2. Sediment Aggradations Problems on Bridges

Sediment aggradations occurs whenever there is excess supply of sediment from the upstream side of the river and existence of any obstruction like **bridge** structures which have a direct effect on the flow velocity, According to **Brian P. Schultz, (2012).** **Bed aggradations** occur when flow is slower: before, under, and after the Bridge section compared to the rest of the stream channel. The transported sediment is comprised of bed load; suspended load, and wash load, **vanrijn (1993)** defines them as:

Suspended load: - that part of the total sediment transport which is maintained in suspension by turbulence in the flowing water for considerable periods of time without contact with the Streambed. It moves with practically the same velocity as that of the flowing water.

Bed load: - the sediment in almost continuous contact with the bed carried forward by rolling, sliding or hopping.

Wash load: - that part of the suspended load which is composed of particle sizes smaller than those found in appreciable quantities is transported though the stream without deposition.

Different site observations show that, “no hydrological and hydraulics analysis including sediment transport analysis has been undertaken to ensure that drainage structures are designed to minimize future maintenance requirements, **(Manaye 2013).**

Flowing water over sediment exerts forces on streambed sediments that tend to move the sediments, these forces have components: the tangential force, drag, and the normal force, lift.

Drag results from viscous stresses at low velocities but at high pressure differential between the upstream and downstream face of the particle is the principle force moving the particles **(Leopold, 1994).**

Data available on critical shear stress are based on what seem to be subjective definitions of critical conditions. However, observers asked to decide when general movement has occurred, will pick a point that is within a few percent of the same velocity, (Henderson 1966).

Early Shear Stress Studies: flume experiments on critical shear stress for non-cohesive sediments show that the motion of sediment grains on the bed is highly unsteady and non-uniformly distributed over the bed area.

The drag force is predominant in turbulent flows when the Reynolds number (D_sV/ν) is high. In laminar flow the shear force is predominant and the Reynolds number is small. The ratio of the forces that move a particle to that of the forces that resist movement is:

$$\frac{\tau_o}{(\gamma_s-\gamma)} Ds \text{ --- } (2.1)$$

Where: τ_o = average shear stress, γ = specific weight of water γ_s = specific weight of the sediment

D_s = diameter of sediment particle

Shields (1935) experiments of incipient motion determined the relationship between the Reynolds number, V Ds/υ and τo/ (γs-γ) Ds, known as the Shields relation. His experiments led to the development of the widely-accepted Shields diagram to determine the incipient motion shear stress.

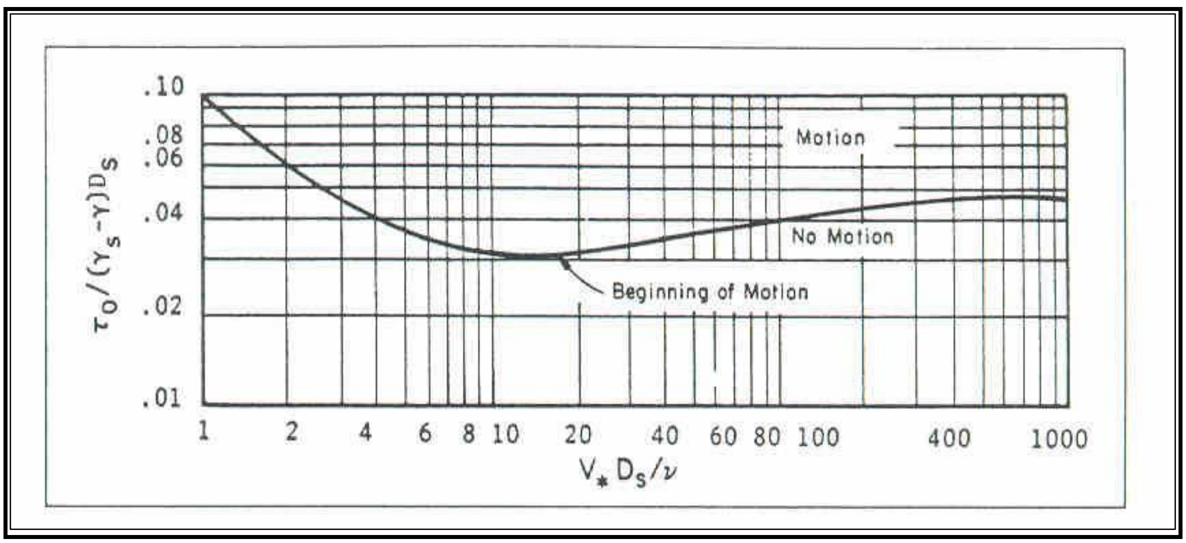


Figure 4 Shields Diagram (FHWA, Hd-6 2001, After Gessler 1971.

2.2.3. Human Activities at the Bridge Location or along the River

Humans have made many land-use changes such as agricultural, urbanization, and commercial development of land resources. Among those agriculture practices is the most significant. The removal of the protective vegetation and the loosening of the soil during cultivation results in a speeding up of the erosion process. The streams and rivers draining through these agricultural areas suddenly overloaded with sediment, producing an imbalance in the equilibrium between sediment and water discharge, Natnael H. 2016.

Another important land-use change is urbanization. The long-term urbanization effect on river morphology and system development is the resultant increase in peak discharges. The increase in discharge comes from the reduction in overland flow resistance produced by the greatly increased impervious land-area.

The response of the river system to these changes is degraded and reshapes those channels immediately downstream of the urbanized area (S. A. Brown, 1981).

2.3.Cause of Bridge Failure in Ethiopia

The main causes of the failure of the drainage structures are due to the following factors:

- Basin characteristics: size, shape, land use, geology, soil type, surface infiltration and storage etc.
- Stream channel characteristics: geometry and configuration, natural and artificial controls, channel modifications, aggradations, degradation and debris.
- Flood plain characteristics
- Meteorological characteristics: precipitation amount and type, storm cell size and distribution, storm direction and time of precipitation (hyetographs).

Based on the above factors we can categorize the causes of the failures of the drainage structures in the following groups:

- Hydrological failure
- Hydraulic failure
- Failure due to aggradations or degradation
- Orientation or location of structure

The bridge failures presented below are some of different types of bridge failures observed in our country.

A) Fafem River Bridge

It is located on Harar to Jigjiga road at 68km from Harar. The bridge was constructed in 2007 and failed a year later due to flooding, it has inadequate opening to accommodate the coming flood. Beza N. 2010.

Figure 5 Fail Fafem River Bridges and Fafem River Flooding Condition.

B) Sedimentation

Sedimentation is another common cause of failure, often occurs in culvert inlets and outlets as well as creeks.

Figure 6 Sedimentation (Source: ERA Drainage Manual 2013)

C) Blockage of bridge

Accumulated trash on the entrance or inlet of culvert or bridge, consequently it can bring obstruction of the incoming flood.

Figure 7 Accumulated Trash on inlet of Bridge

2.4. Span Length and Clear Height of the Bridge

2.4.1. Span Length

With regard to the opening size of the bridge, the Span Length should be selected in such a way that the bridge spans from the edges of one bank to another and in no case, the flow should be constricted. If the flow is constricted, backwater affects causing sedimentation that will aggravate progressively with each previous siltation.

2.4.2. Clear Height of the Bridge

Once the Span is fixed, the detail full-fledged hydrology must be carried out to determine the high water mark for the design and review peak discharges for the return periods recommended. The amount of unavoidable expected sedimentation in the channel and the resulting channel aggradations should be determined including sufficient free-board. Niguse , 2010.

Bridge opening has a significant effect on the performance of the bridge structure. Inadequate size of bridge opening may be a result of error hydrologic analysis or river morphology change due to the construction on the main channel, inefficient opening can cause flooding upstream, or it magnifies the problems which already exist. Consequences of inefficient opening can be extremely damaging and expensive if a large number of properties or factories were flooded, so it is important to make sure that a hydraulically efficient structure is designed and the backwater level calculated accurately, Natnael H. 2015

Bridge events including, flood and scour were the biggest causes of bridge problems in the USA, 32.5 % of failure is due to flood and 15.5% is due to scour, McLinn, 2009.

The bridges failure in Iran is very common, due to neglecting the hydraulics aspects of bridge design during a flood, the scouring and pier foundation are origin of the collapse.

The hydraulic opening of a waterway bridge is determined based on a combination of the following:

- The contributing watershed area.
- The magnitude and frequency of the design flood.
- The roadway alignment, profile, physical and environmental constraints.

The sizing of waterway shall include evaluation of water surface elevation for existing conditions and for proposed once. A comparison of the two results shall be made to identify the effects of the waterway. These results shall clearly identify and mitigate any backwater effects caused by the project.(Adot 2012)

2.5.Computer Applications and Models

In modern practice the hydrology and hydraulic of waterways at bridges is analyzed by using mathematical simulation models with help of computer applications. The simulation models incorporate site and event-specific data supplied by the Hydrologist and Hydraulic engineer into approximate solutions of the governing equations. John H. Hunt 1995.

2.5.1. Hydrologic Modeling (HEC-HMS)

HEC-HMS is designed to simulate the rainfall-runoff processes of watershed systems. This includes large river basins water supply and flood hydrology, and small urban or natural watershed. The runoff hydrographs produced by the program used directly or in conjunction with other software for studies of water availability, urban drainage, flow forecast, future urbanization impact, reservoir spillway design, flood damage reduction, flood plain regulation, and system operation, HEC-HMS user’s manual, and 2010. This model enables the user to perform hydrological modeling based on a wide selection of common mathematical models used in Hydrology.

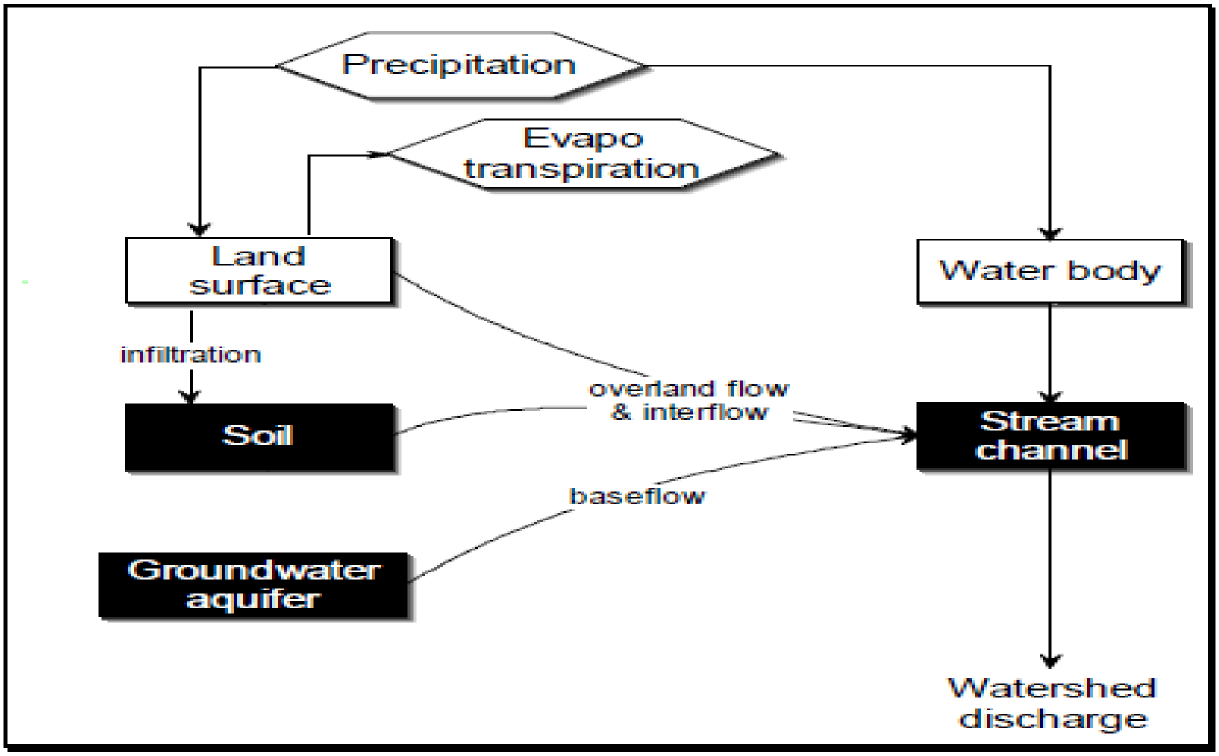


Figure 8 Typical HEC –HMS Representation of Watershed, HEC_HMS Users' Manual 2000.

HEC- GeoHMS has been developed as geospatial hydrological toolkits for Hydrologist and Engineers with limited GIS Experience. The program allow the user to visualize a spatial information, document watershed characteristics, perform spatial analysis, delineate sub basin and streams, construct inputs for hydrologic models, and assist in report preparation. HEC-GeoHMS creates background map files, basin model files, meteorological models files, grid cell parameters file which can be use by HEC HMS to develop hydrological model, HEC-GeoHMS user manual February 2013.

Hydrologic cycle is divided into four major components,

Loss Method: A model used to compute the runoff volume.

Transform Method: models of direct runoff, it accounts surface roughness and geometry of the watershed.

Base flow Method: used to simulate the fraction of the runoff contributed by groundwater.

Routing Method: the flow at the outlet of a certain upstream watershed has to be routed through the river channel in the downstream watershed, the process known as **Routing Method**.

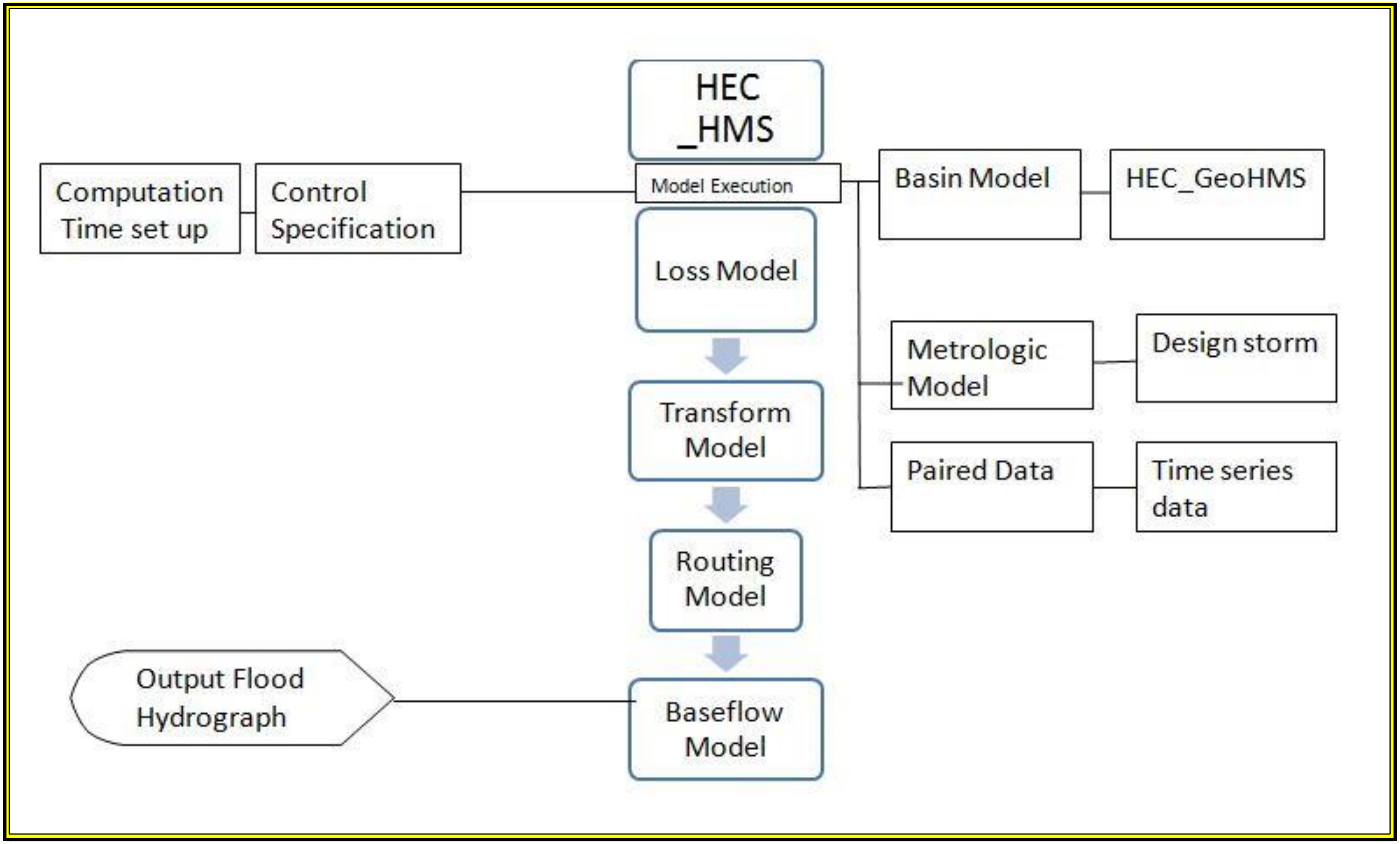


Figure 9 HEC HMS Components: HEC –HMS User Manual August 2016

As it is stated in the user manual, there is five major component of HEC-HMS, such as: **basin model, meteorological model, control specification, paired data** and **computation time step.** Different condition needed to be taken, to select the hydrological model, however the bridge located in un-gauged catchment, so four components of the runoff process is chosen, as it is summarized in the below table.

Methods	Selected Model
Loss method	NRCS curve number (CN) method
Transform method	NRCS dimensionless UH method
Routing method	Muskingum method
Base flow method	Not considered

Table 1 Methods and Selected model.

Loss method: NRCS Curve Number (CN) method

NRCS Curve Number (CN) method is requiring only one unknown input data which is Curve Number. The hydrologic data used to estimate CN are normally available in most non-gauged watersheds toting to that, the equation can be applied to a wide range of watershed conditions. The basic runoff equation of CN method is shown in Equation 1. According to SCS (1985), Relationship in mathematical form is:

$$\frac{F}{S} = \frac{Q}{P-I} \text{ --1}$$

Where: F = accumulated infiltration, S = watershed storage,

P = total rainfall, I = initial abstraction, Q = actual direct runoff,

The amount of rainfall infiltrated after runoff begins can be expressed as:

$$\mathbf{F} = (P - I) - Q \text{ --------------------------------------2}$$

By substituting (2) into (1) and solving for Q in terms of P, I, and S, then (1) become:

$$Q = \frac{(P-I)^2}{(P-I)+S} \qquad P > 1 \text{-----------------------------------3}$$

The empirical relationship between ‘I’ and ‘S’ for many small watersheds is:

$$\mathbf{I} = 0.2\,\mathbf{S} \text{ --- 4}$$

Substituting (4) into (3) yields:

$$Q = \frac{(P-0.2S)2}{(P+0.8S)} \text{-- 5}$$

Finally S is related to CN using equation:

$$S = \frac{25400}{CN} - 254 \text{---6}$$

Transform method: NRCS Unit Hydrograph method

As per Sherman (1932); UH (originally known as unit-graph) of a watershed is a direct runoff hydrograph (DRH) resulting from 1 unit of excess rainfall generated uniformly over the drainage area at a constant rate for an effective duration (Chow, 1988). To compute DRH with a UH, a discrete of excess precipitation used to solves the discrete convolution equation of linear system:

$$Q_n = \sum_{m=1}^{n\leq M} P_m U_{n-m+1} \qquad \text{----------------------------------7}$$

Where: Q_n= storm hydrograph ordinate at time n*Δt, P_m=rainfall excess depth in time interval m*Δt to (m+1)*Δt; m=total number of discrete rainfall pulses; U_{n-m+1} = UH ordinate time (n-m+1)*Δt.
Q_n and P_m: flow rate and depth respectively, and Un-m+1 have dimensions of flow rate per unit depth.

For the case of this study paper the catchment where the bridge located is un-gauged watershed. So it is impossible to develop UH from observed rainfall-runoff relationship. Ever rainfall-runoff data is not available a synthetic UH technique is employed as per *SCS* runoff procedures, 1985.

A synthetic UH relates the parameters of UH model to watershed characteristics. In this model can be generated in the absence of the precipitation and runoff data necessary to derive the UH *(HEC, 2010).* HEC-HMS provides different synthetic UH models for un-gauged watershed; in this thesis work dimensionless SCS synthetic unit hydrograph is used. The derived UHs were then made dimensionless and averaged to obtain a standard dimensionless unit hydrograph *(DUH)* as shown in *Figure.*

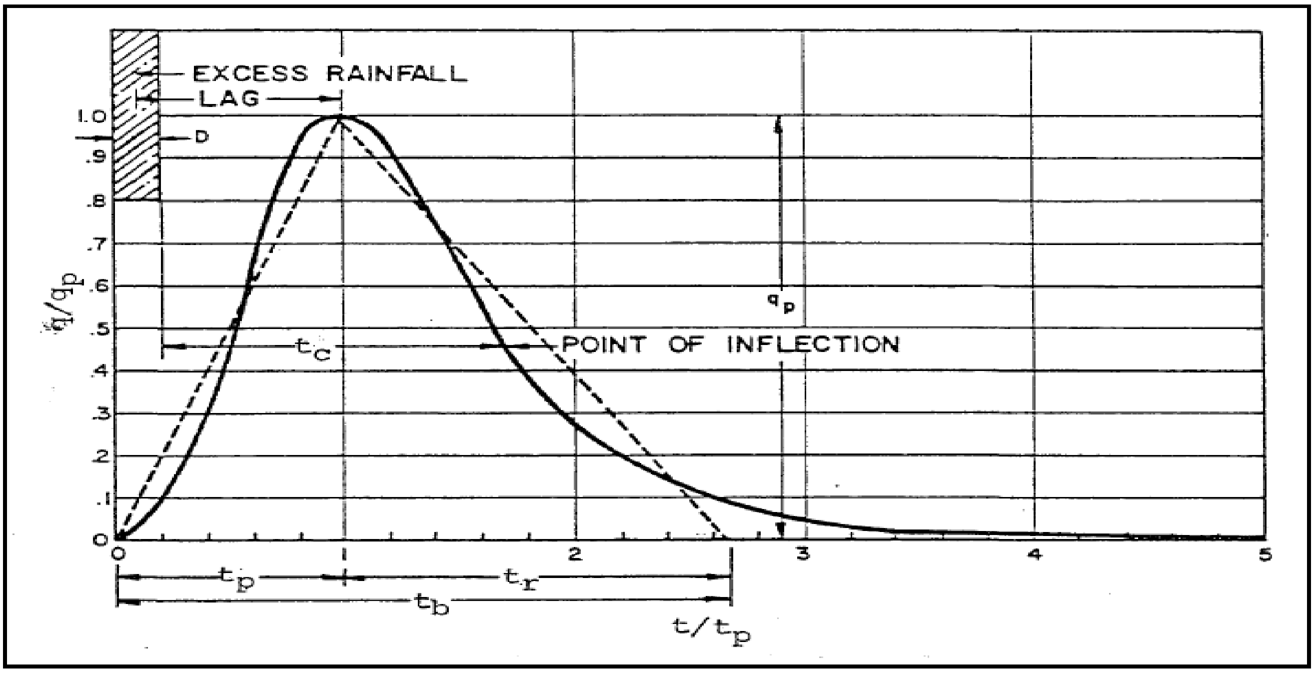


Figure 10 SCS suggests that UH peak and Time of UH Peak.

SCS suggests that UH peak and time of UH peak to be related by:

$U_P = C\frac{A}{T_P}$---.8

Where: U_p= UH peak discharge, C=conversion constant (2.08 in SI), A=watershed area

T_p= time to peak UH then, time of peak UH (Tp) can be determined using:

$T_p = \frac{\Delta t}{2} + t_{lag}$--9

Where: Δt =excess precipitation duration, t_{lag} = the basin lag, a computation interval, Δt, is less than 29% of t_{lag} must be used, HEC, 2010. Lag time is calculated using the SCS lag time equation as listed below.

$$t_{lag} = \frac{L^{0.8} * (S+1)^{0.7}}{1900 * {S_b}^{0.5}}$$ ---10

Where: t_{lag}=lag time in Hr. L=longest flow path in the watershed, S= watershed storage, S_b= basin slop When the lag time specified, HEC-HMS solves (9) to find T_p, and (8) to find Up, knowing U_p and T_p, UH can be found by multiplying the dimensionless UH ordinates, included in HEC-HMS. All above methods are merged in HEC-HMS, and applied the put data, result will be created in graphs and tables.

Routing method: Muskingum method

HEC-HMS provides six different models to estimate flow through river channels, all of the models require input parameters even if each of them uses the concept of continuity and momentum equations.

2.5.2. Hydraulic Modeling (HEC-RAS)

HEC-RAS is capable of performing one-dimensional steady and unsteady flow simulations and comprises a graphical user interface, separate hydraulic analysis components, data storage and management capabilities, graphics and reporting facilities, Hydraulic Reference Manual, 2010.

HEC-RAS has four components: steady flow water surface profile computation, unsteady flow simulation, movable boundary sediment transport computation, and water quality analysis, Hydraulic Reference Manual, 2010. For the case of this thesis work, steady flow water surface profile computation and movable boundary sediment transport computation were determined using HEC-RAS 5.0.5.

2.5.2.1.Steady Flow Water Surface Profile Computation in HEC-RAS

Gradually varied flow was calculated by HEC -RAS profiles. 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.

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 is written as follows:

$$Z_2 + Y_2 + \frac{a_2 V_2^2}{2g} = Z_1 + Y_1 + \frac{a_1 V_1^2}{2g} + h_e \text{-------------------11}$$

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;

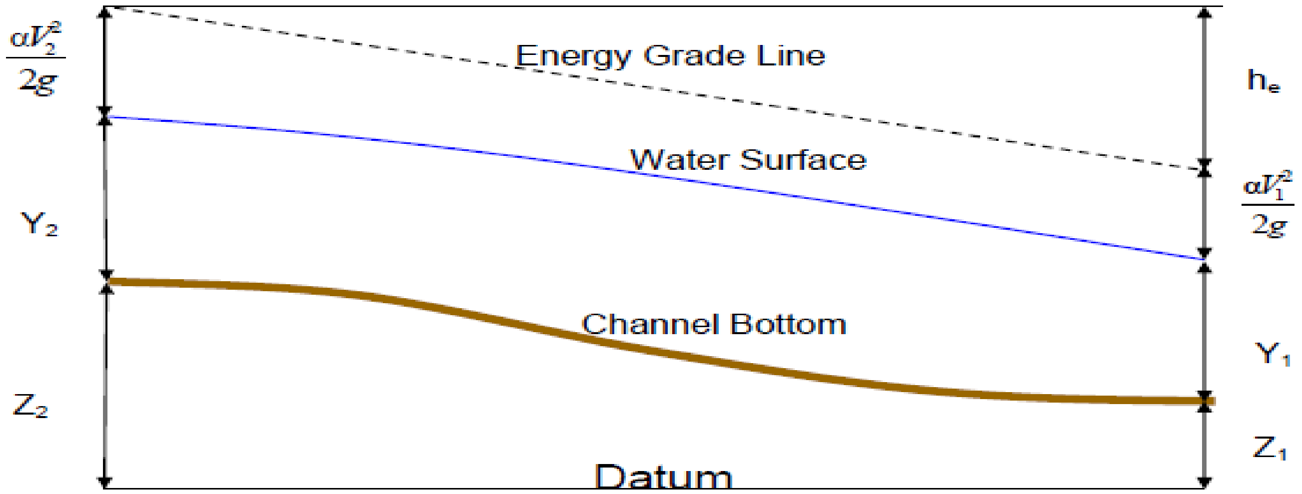


Figure 11 Energy Equation as per Hydraulic Reference Manual, 2010

The ***he*** is estimated by adding friction head loss (h_f) and contraction or expansion head loss (h_{ce}). H_f 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 12*;

$$h_e = LS_f + c[\frac{a_2 V_2^2}{2g} - \frac{a_1 V_1^2}{2g}] \qquad \text{------------------------------12}$$

Stream cross sections may not always taken perpendicular to the flow direction, maybe left and right side of the channel will have different length, HEC-RAS considers weighted channel length during computation process and expressed in Equation.13 below.

$$L = \frac{L_{lob}\bar{Q}_{lob}+L_{ch}\bar{Q}_{ch}+L_{rob}\bar{Q}_{rob}}{\bar{Q}_{lob}+\bar{Q}_{ch}+\bar{Q}_{rob}} \text{ ---------------------13}$$

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 velocity head can be determined from manning's equation:

$$Q = KS_f^{\frac{1}{2}}, \ K=\frac{1}{n}AR^{2/3} \text{ ---14}$$

Where: Q=discharge in the channel, K=conveyance constant, S_f=friction slop

Contraction and expansion losses assume that as contraction is occurring whenever the velocity head downstream is greater than upstream, whereas the velocity head upstream is greater than downstream, assumes that a flow expansion is occurring, Hydraulic Reference Manual 2010.

Energy equation is not applicable in rapidly varied flow condition which happens when flow changes from subcritical to supper critical or vice versa. This condition generally occur when there is hydraulic jump, that case momentum equation is applicable rather than energy equation.

$$F = ma \text{ ---15}$$

After considering all horizontal forces acting on a block of water along the bed of the river which are presented in figure, the following equation is developed.

$$P_2 - P_1 + W_x - F_f = Q\rho\Delta V_x \text{ --16}$$

Where: P2, P1= hydrostatic pressure force at location 1 and 2 respectively, W x = weight component along x direction, F_f = drag force, Q = discharge between the section, ρ = density of water, ΔV_x= change in velocity between the section

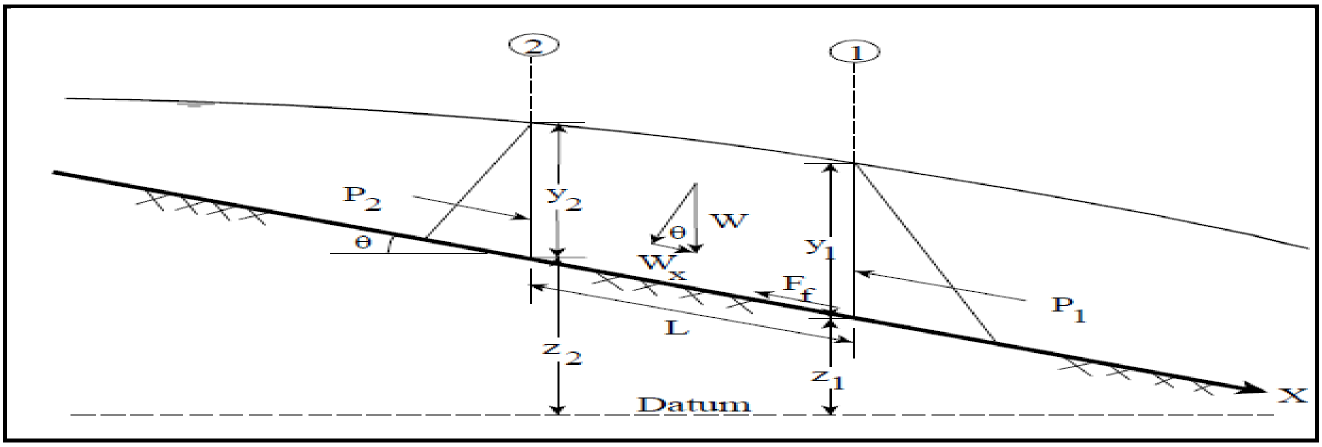


Figure 12 Application of the momentum principle: Hydraulic Reference Manual 2010

A simplified general momentum equation is developed as shown below.

$$\frac{Q_2^2\beta_2}{gA_2} + A_2\bar{Y}_2 + \left(\frac{A_1+A_2}{2}\right)L\bar{S}_0 - \left(\frac{A_1+A_2}{2}\right)L\bar{S}_f = \frac{Q_1^2\beta_1}{gA_1} + A_1\bar{Y}_1 \text{ ----------------17}$$

This equation will be used throughout HEC RAS to solve problems with or without modification.

2.5.2.2.Sediment Transport Modeling in HEC-RAS

The sediment transport event in 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. The flow durations are further broken down into computation increments, in which the bed elevation and hydrodynamics are updated after each increment. Finally, computational increment is 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.

$$(1-\lambda_p)B\frac{\partial \eta}{\partial t} = -\frac{\partial Q_s}{\partial x} \text{----------------------}\mathbf{16}$$

Where: B= channel width, η = channel elevation, λ_p= active layer porosity, t= time, x= distance and,

Qs= transported sediment load

The change of sediment volume in a controlled section is equal to the difference between the inflowing and out flowing loads. HEC-RAS determines transport potential for a single representative soil grain.

This is done by multiplying the transport potential for each grain class and percentage of that grain class in the bed and summarized in *Equation19* below.

$$T_c = \sum_{j=1}^{n} \beta_j T_j \text{-----------------------------19}$$

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.

3. Methodology and Data Collection

3.1. Introduction

This chapter provides a detailed project area description, data collection and methodology to meet the objectives of the study.

3.1.1. Description of the Project Area

3.1.1.1.Location of the Project Area

It is located in the little Akaki River which is the part of Akaki river catchment with an **inclusive area of 73.3 km^2 and bounded between the geographic coordinate of 38° 36' 50.9" E to 38° 43' 54.6" E longitude and 8° 53' 53.5" N to 9° 00' 15.3" N latitude**.

Akaki River basin is located in the north western part of Awash river basin and its boundary divide the Awash and Abay river basin, it is divided into two major sub-catchment: large Akaki River (Eastern) locally known as Tiliku Akaki with an area of 1008.68 km^2 and little Akaki River (Western) locally known as Tinishu Akaki with an area of 396.06 km ^{2}. The large Akaki and little Akaki join together then flow to the Awash River as a tributary.

The capital city Addis Ababa and other small towns such as Akaki, Sendafa, Legatefa, Burayu and small farm villages are also found in this catchment.

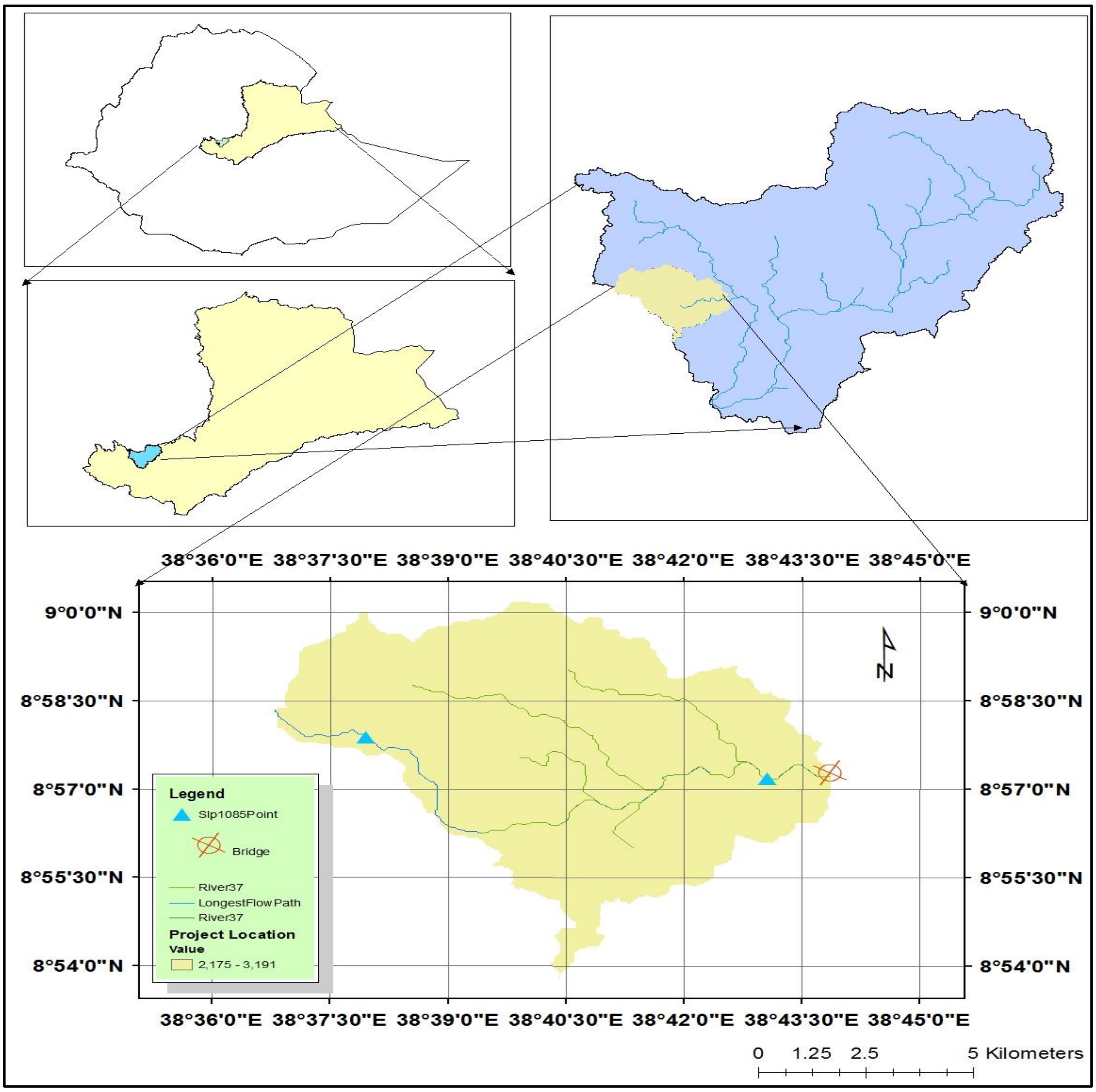


Figure 13 Location of Jemo (Study Area) Catchment

3.1.1.2.Location of the Bridge

Arsema Bridge is found in City administration of Addis Ababa, Nifas Silk Lafto Sub City, **it is built on Jemo River as the river cross drainage structure to connect Akaki to Ayer Tena that connect German to Lebu Mebrat Hail round about,** Jemo River originates from highlands of Central Shewa plateau located on the northern western part of Addis Ababa City having an elevation of 3191m above sea level and final at the bridge crossing site, the elevation becomes 2209 amsl. The Bridge is located at **8° 57' 12.57" N latitude and 38° 43' 56.53" E longitude as shown in the figure.**

Figure 14 Location of Arsema Bridge.

3.1.1.3.Climate of the Study Area

High annual rainfalls in the region of Addis Ababa occur when moist wind is forced to rise in order to pass over mountains; such conditions prevail especially along the mountain ranges that extend early at right angles to the prevailing storm movements. The rainfall patterns in the catchment areas have a bimodal profile with strong peaks in the summer months and minor rainfalls in the months of March and April.

The aerial average annual catchment rainfall and potential evaporation- transpiration are **1100 and 1226 mm,** respectively. The seasonal variation of rainfall distribution within the study area is due to the annual migration of the **intertropical convergence zone**, **a low pressure zone marking the convergence of dry tropical easterlies** and **moist equatorial westerly across the catchment.** The highest and lowest mean maximum temperature over record periods is 25^0c in dry season and 20^0c in wet season, while the variation of mean monthly temperature values fall in the range of 7^0C to 12^0C throughout the year, the study area shares the same climatic condition as that off Addis Ababa watershed, therefore the rainfall patterns of the study catchment area has a bimodal profile with strong peaks in the summer months and minor rainfalls in the months of March and April.

3.1.1.4.Topography

The topography of the study area is undulating and form plateau in the Northern, Eastern and Southwestern while gentle morphology and flat in the Southern and South-Eastern part. It has **73.3 km2** watershed areas.

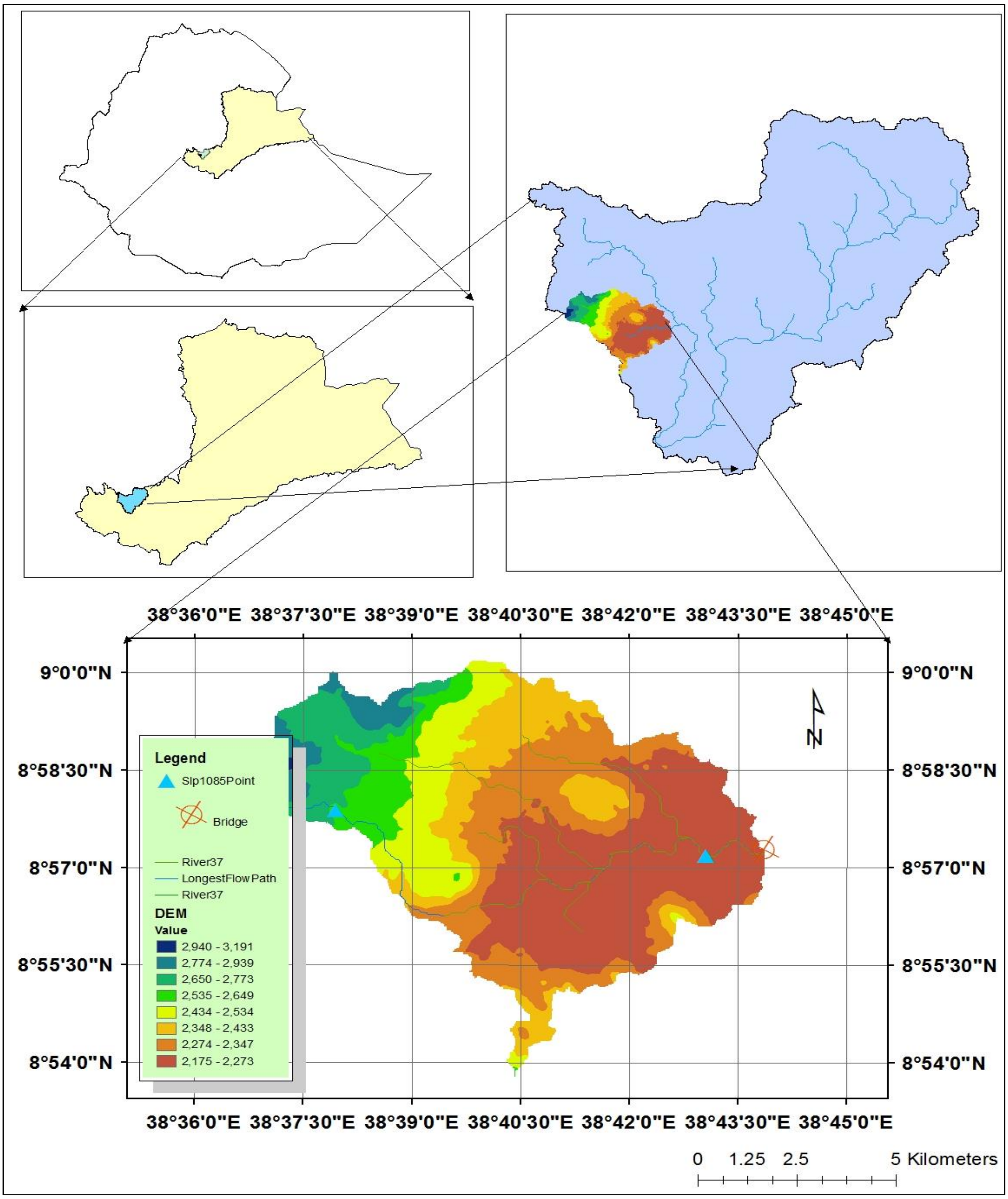


Figure 15Topographic condition of the study area.

3.1.1.5.Land Use and Land Cover

Arc GIS 10.3, is used to analyze the general land use and cover of the project area and classified as grassland, cultivated land, urban and forest or plantation. In the North-western part of the catchment which is on the Furi Mountain the land is covered by forest, dominantly eucalyptus trees and the top of the mountain range is relatively gentle slope that increase run off towards the catchment outlet. Residential areas are found in and around the city which is generally characterized as paved surface that cause small infiltration and most of the rainfall that comes is converted into surface runoff and drains into networks of rivers. The rest of the catchment is dominantly covered by woods-grass combinations and small grain straight row crop (cereal crop like teff.)

LC1	LC1LU_DES	LC2LU-DES	LC1_MASDES	LC2_MASDES
GL	Grassland	Cultivation	Grassland: lightly stoked	Cultivated Land; Rain fed; Cereal Land Cover System; lightly stocked
U	Urban		Urban	
FPC	Plantation		Forest: plantation forest Closed (>80% crown cover)	
FPO	Plantation		Forest; Plantation forest; Open (20-50% crown cover)	

Table 2 Land Use and Land Cover

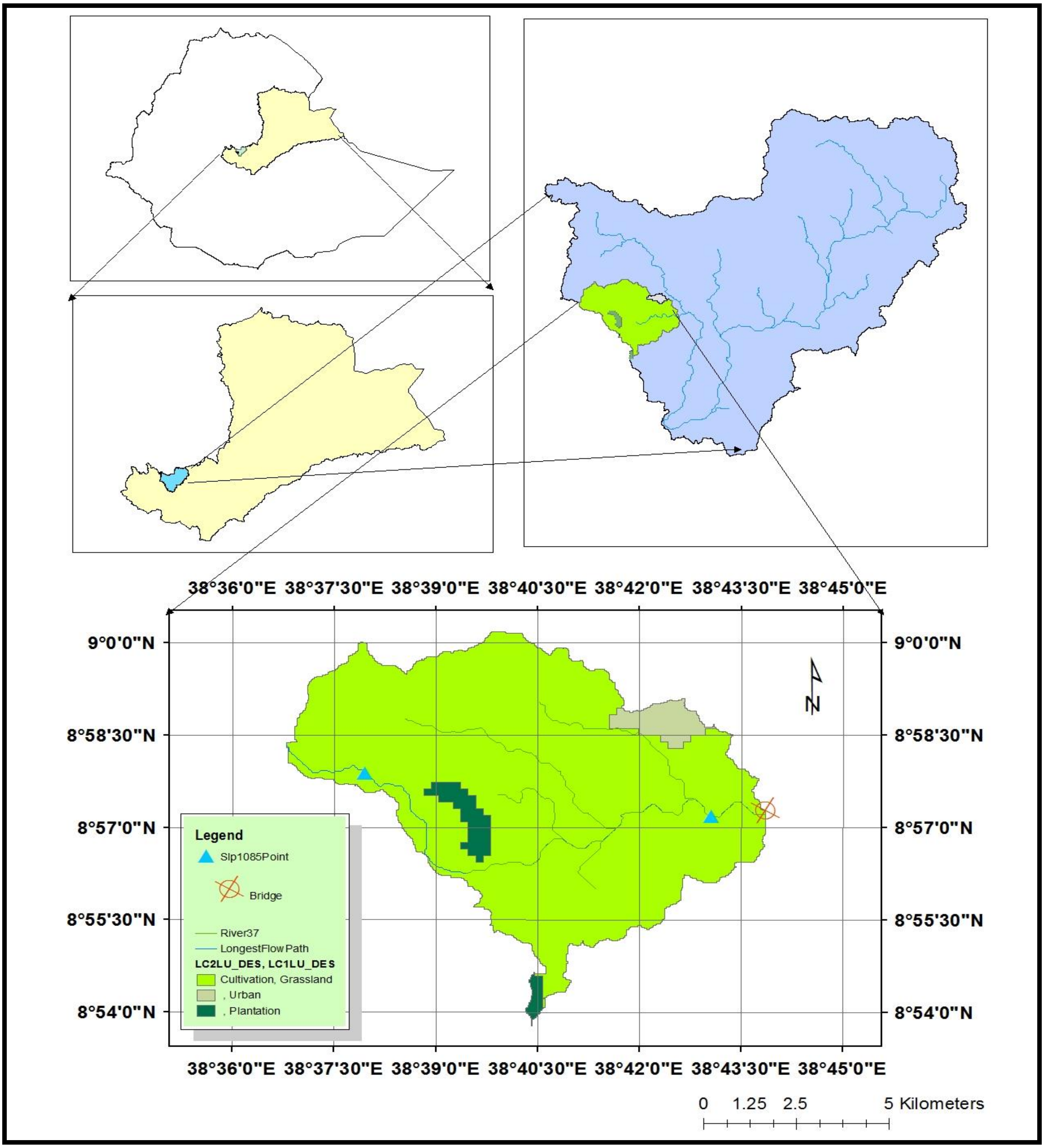


Figure 16Land Cover and land Use

3.1.1.6.Soil Type

After analyzing the shape file of the soil in GIS, the general soil type of the study area is categorized as

Group B and Group D. According to ERA DDM, (2013) **Group B** soils are categorized as Silt loam or loam and this type of soils have a moderately low runoff potential due to moderate infiltration rates. Group D soils are categorized as Clay loam, silty clay loam, sandy clay, silty clay or clay. It has high runoff potential due to very slow infiltration rates.

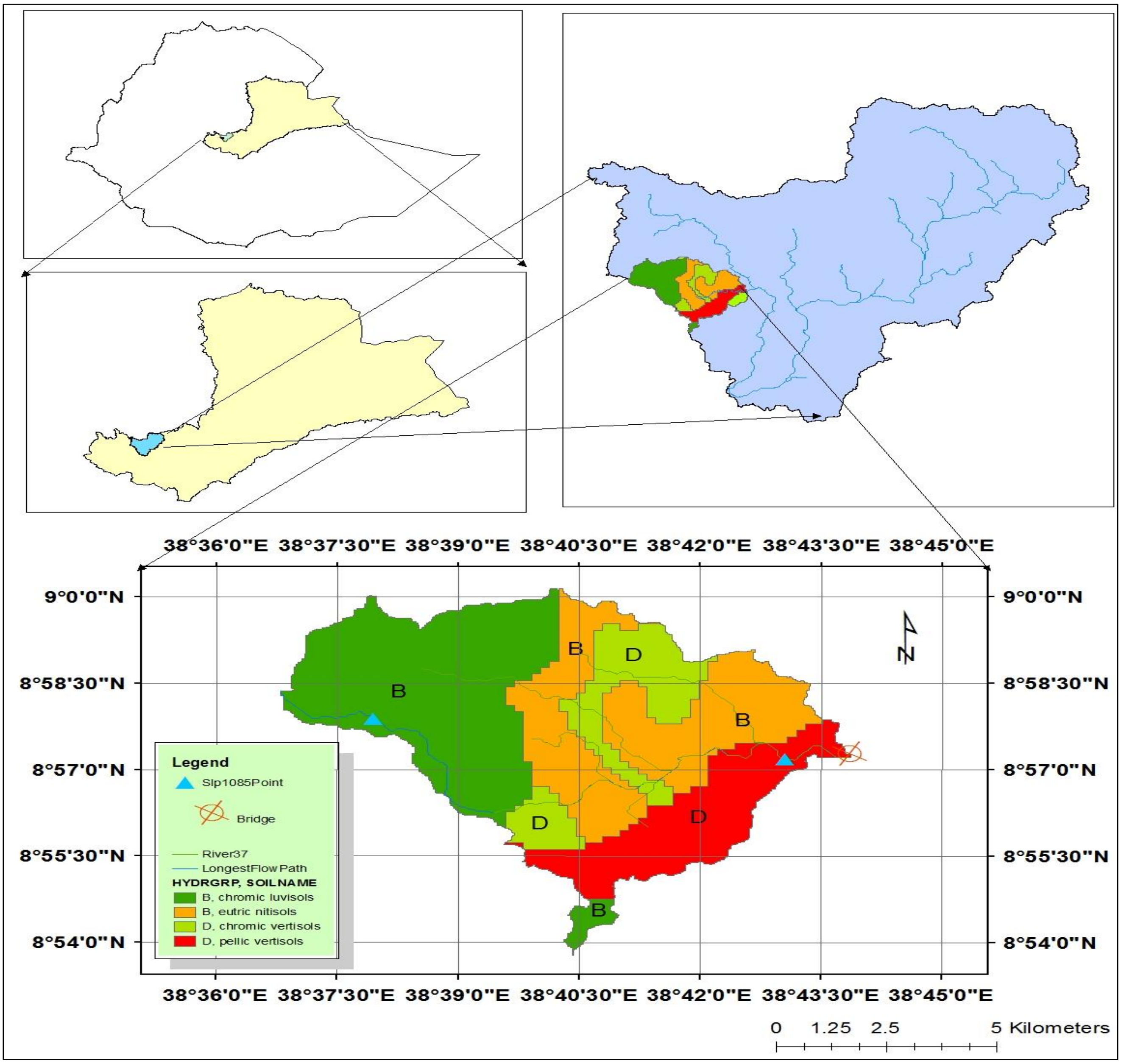


Figure 17Soil Type

3.1.1.7.Hydrology and Drainage

The two major Rivers in the Akaki catchment are the Little Akaki River, which flows through the Western part of the city and Large Akaki River, which flows through the Eastern part of the city.

Major perennial rivers such as **Large** and **Small Akaki** and **Kebena River** start from the **Entoto ridge** and drains to the South by passing through the center of the city of Addis Ababa.

In the project area the streams drain towards northern-west from the Furi ridge; Southeast direction from Mt. Wechecha; and other elevated areas of the central parts of the city. The perennial streams in the Akaki catchment are Little Akaki, Large Akaki, Bantyiktu, Kurtume, Kebena and Ginfile. Other streams are intermittent in nature. On the top of the mountain streams are dense forming radial drainage pattern, whereas in most parts of the city core they form dendrite drainage pattern. Towards the South almost all streams or big tributaries crossing the city in different direction join either Little Akaki or Large Akaki River. The two rivers flow on either side of Addis Ababa and complete their courses by entering Lake Abba Samuel.

3.2. Data Collection

3.2.1. Primary Data Collection

Jemo river cross section, soil sample and bridge structure survey data is collected at field level as of primary data.

Site observation and interviewing with local people about flooding problems and water Level marks during summer is conducted.

3.2.1.1. River Cross Section Data

Field survey was accompanied to collect: cross section of bridge, 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.

Figure 18 River Cross Section Data Collection

Point data were collected as shown in the figure above using by total station both at upstream and downstream of the bridge within 20 meter interval and high flood mark of surrounding area also collected. The point data was exported to EXCEL and Auto CAD then used for further analysis in HEC-RAS model.

3.2.1.2. Soil Gradation

Particular soil conditions, such as mud or acid sulphate soils for example, may be a problem and this can affect the selection of drainage structures, sedimentation or deposition of soil particles that have been carried by flood waters is one of bottle neck of the hydraulic performance of river crossing structure in Ethiopia. Arsema Bridge is one of the victims by deposition of sediment as well as blockage of trash. So in order recommend a remedial measure of sedimentation and blockage of trash, soil sample collected during field survey and gradation sieve analysis was carried out at Addis Ababa University Highway lab then sieve analysis result attached in the appendix C. The result of lab is one of the basic input data in HEC –RAS analysis of movable bed load transport simulation phase.

3.2.2. Secondary Data Collection

Rainfall records of different stations were collected from the Ethiopian Meteorological Services Agency, (Ayer Tena, and Bole, Akaki, Kality, Kolfe, Medhaniyalem, Yakutat 23 gauged rainfall data).

Stream **flow data of Akaki River**, **DEM,** and **land use** and **soil characteristics of the catchment** collected from MoWIE whereas **Bridge Structural as built drawing** collected from AACRA.

3.3. Hydrologic and Hydraulic Modeling

The hydrologic modeling was undertaken in order to evaluate peak discharges of flood whereas hydraulic modeling is to determine water surface level of peak flood.

Modeling and analysis of this project was conducted with help of: **Global Mapper 12.0, 3DEM, Arc Hydro and HEC- GeoHMS tools on Arc GIS 10.3, HEC-HMS 4.2, and HEC-RAS and RAS Mapping programs.**

3.3.1. Hydrologic and Hydraulic Process

The catchment area of study which is extends from Furi Mountains to Little Akaki main river basin. 30mx30m resolution Digital Elevation Model (DEM) with topographical maps was used by Global Mapper to delineate **Raw DEM** then it is used as input of Arc GIS 10.3.

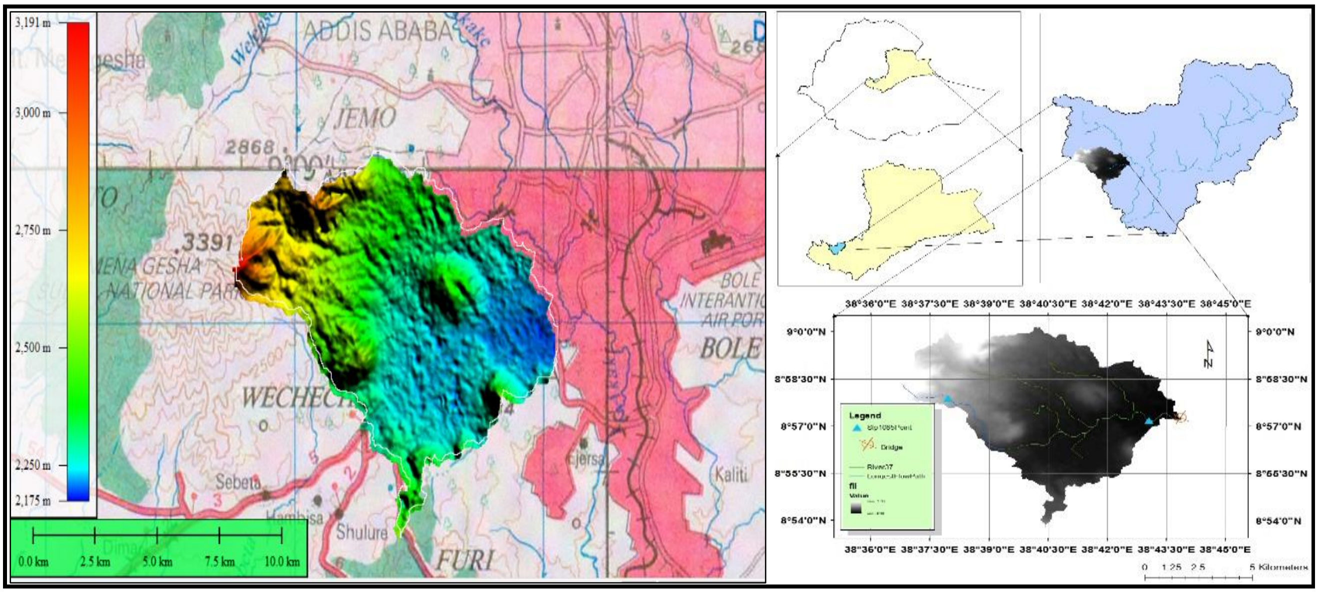


Figure 19 terrain Processing

On Arc GIS 10.3 interface Arc Hydro and HEC Geo- HMS extent ion tools was applied to extract watershed boundary, river networks and drainage outlets. Subsequently output of HEC-GeoHMS was exported to HEC-HMS to complete hydrological analysis section. On HEC –HMS: hydrologic analysis was computed for 2, 5, 10, 25, 50 and 100 year return period. The HEC-HMS model calibrated before its output used in HEC-RAS for hydraulic analysis. HEC-RAS was used to analyze water surface profile and movable bed load transport simulation.

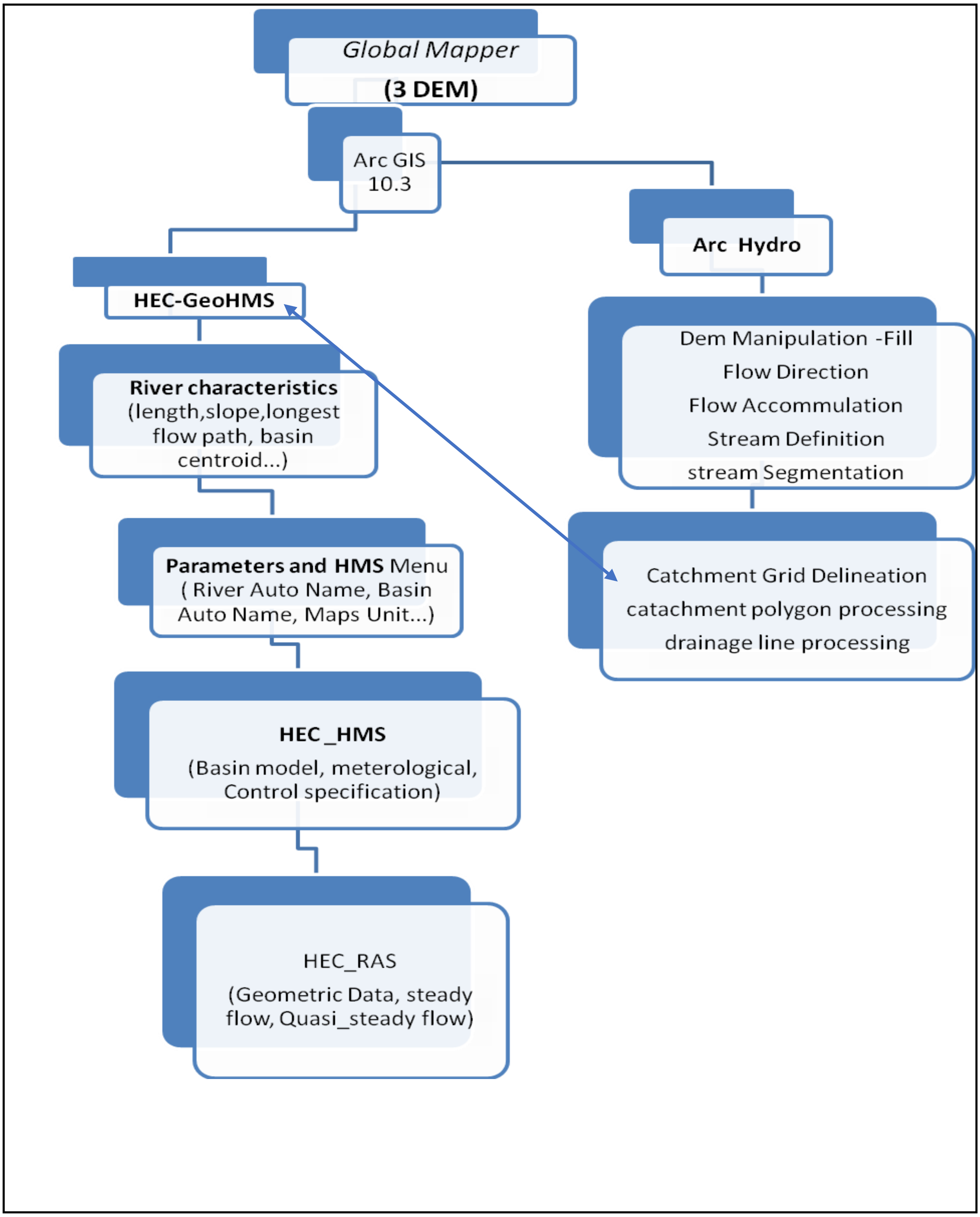


Figure 20 Log frame for Software Usage and procedure

HEC-GeoHMS –final output used as in put of HEC-HMS, then the final out of HEC-Geo HMS from Arc-Map shown in below figure.

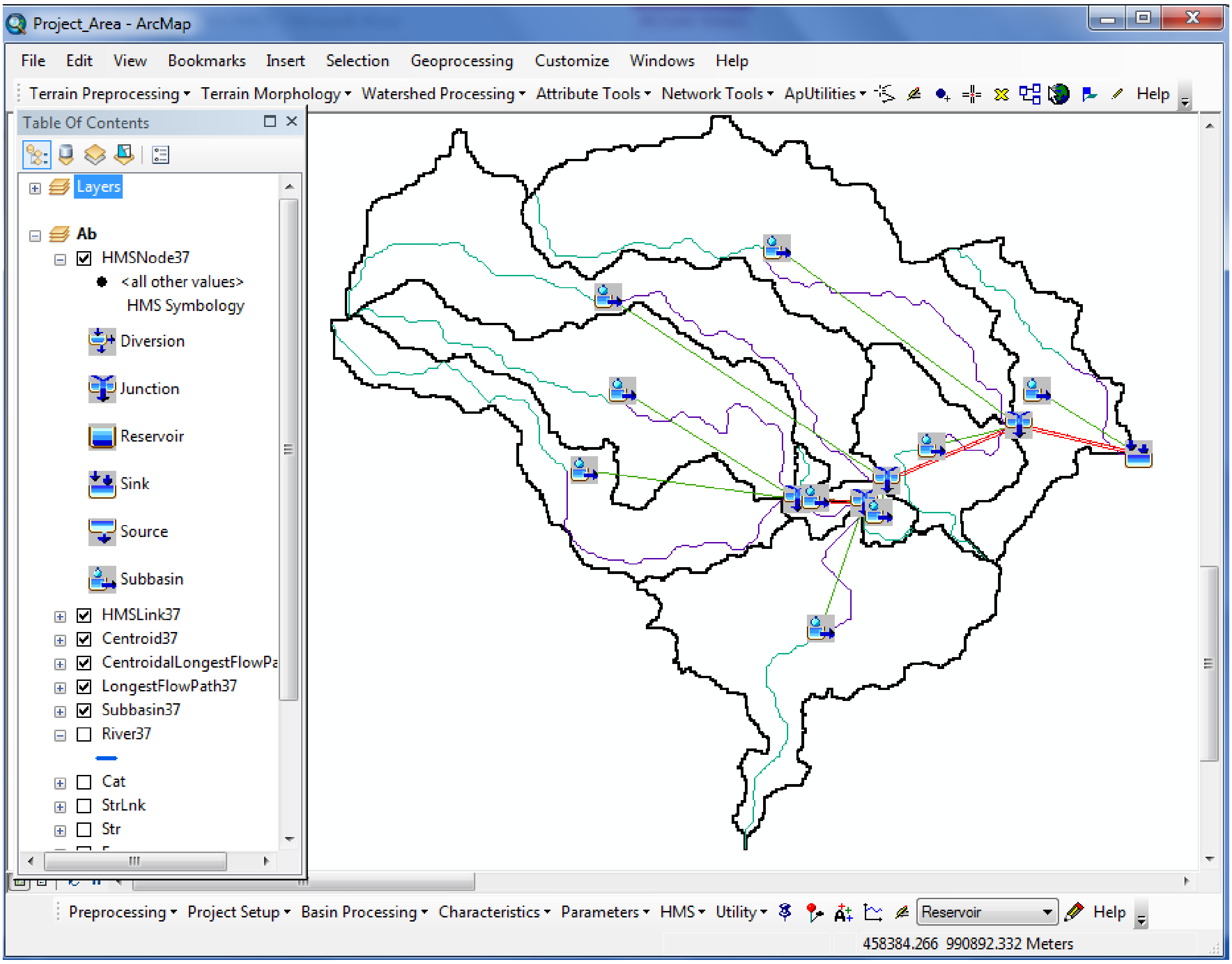


Figure 21 Parameters and HMS Menu

Table

Subbasin37

Name	gridcode	Shape_Length	Shape_Area	BasinSlope	BasinCN	BasinLag	Area_HMS
W100	1	30227.868	15449682.880722	1	1	291.769424	5.965156
W110	2	35418.512	13187980.571487	2	2	133.595782	5.091908
W120	3	26258.552	10956111.103924	3	3	70.560613	4.230178
W130	4	21312.1736	6009453.044239	4	4	30.594899	2.320263
W140	5	14900.2016	4821733.035185	5	5	22.797084	1.861682
W150	6	5312.7768	840913.22462	6	6	7.871478	0.324678
W160	7	6228.7728	799893.067322	7	7	7.529435	0.30884
W170	8	28762.2744	12995931.653225	8	8	16.64836	5.017757
W180	9	27724.1456	8222676.985758	9	9	19.808826	3.174793

(0 out of 9 Selected)

Subbasin37 River37

Figure 22 Physical parameters of River network.

3.3.2. Hydrologic Modeling (HEC-HMS)

A. Basin Model

HEC-HMS software combines nine basin elements which include sub basin creation, reach creation tool, reservoir, junction tool, diversion tool, source tool and sink tool. All basic physical input parameters used in basin modeling including area of sub catchments, longest flow path, river length, basin slope, river slope were imported directly from HEC-Geo HMS software.

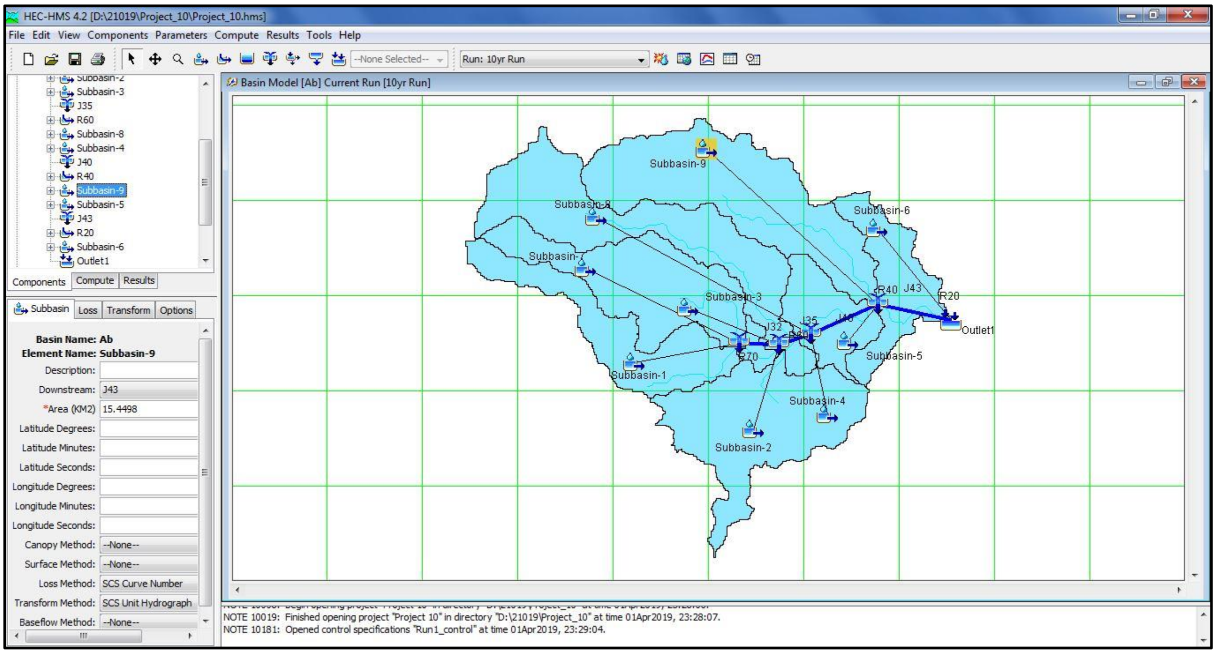


Figure 23 Basin Model representation of the project area using HEC- HMS

B. Curve Number (CN)

Basically the curve number of the project area derived from soil type, land cover and land use. As per **ethio01** data collected from MoWIE such as: land use and cover, and soil type, delineated with Arc-Map on Arc-GIS. As it is clearly shown in the figure 13 and 14 result of Soil Hydrological Group realistic which is grouped into B and D as major Group, even B-Group further grouped: chromic luvisols and eutric nitirols, D –Group chromic and pellic vetrisols however land use and land cover need further examination and embellishment with help of more recent data.

The Google earth satellite image of 2019 and 2018 used to approximate accurate estimation of land use and cover as it is referred the figure below, and ERADD, 2002 attached on the appendix –B, and then it is classified as into three main groups.

1. Residential (roof and parking lots, paved and gravel)
2. Small grain straight crops
3. woods-grass combinations

Then the result of the CN summarized the table below.

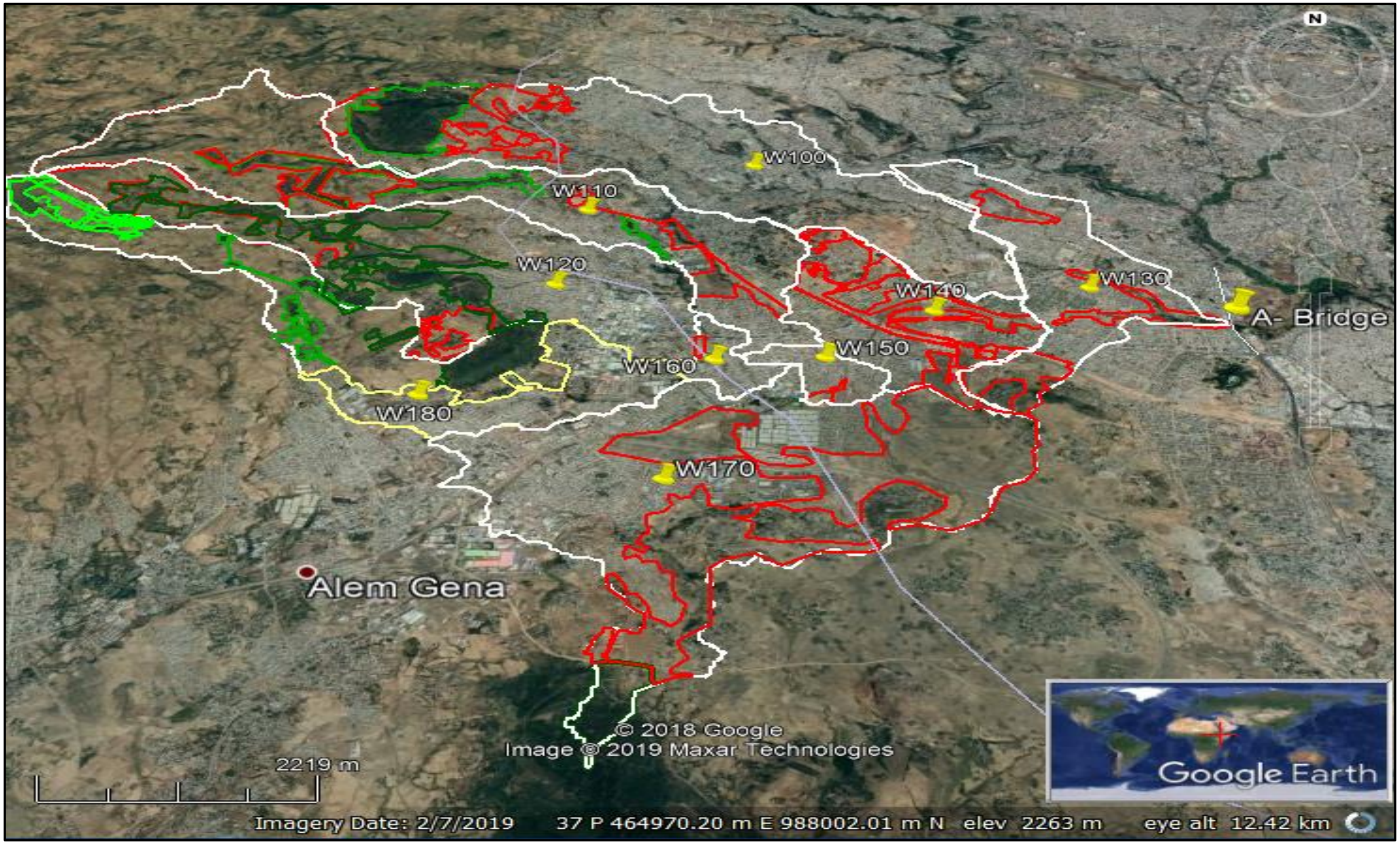


Figure 24 Google earth 2018 and 2019 satellite image

Sub basin	Area of Sub basin	small grain straight row crop		Woods-grass combinations		Residential						
		Hydrological Soil groups		Hydrological Soil groups		roof and parking lots	Paved Hydrological Soil groups		Gravel Hydrological Soil groups		CN
		B (CN=76)	D (CN=88)	B (CN=65)	D (CN=82)	CN=98	B (CN=89)	D (CN=93)	B (CN=85)	D (CN=91)	
W100	15.45	0.83	0.42	1.02	0.51	9.50	1.37	0.91	0.53	0.35	92.14
W110	13.19	4.08	2.04	0.80	0.40	4.40	0.63	0.42	0.25	0.16	86.24
W120	10.96	1.28	0.64	1.71	0.85	4.86	0.70	0.47	0.27	0.18	87.24
W130	6.01	0.94	0.47	0.00	0.00	3.45	0.50	0.33	0.19	0.13	92.19
W140	4.82	1.01	0.51	0.00	0.00	2.48	0.36	0.24	0.14	0.09	90.90
W150	0.84	0.04	0.02	0.00	0.00	0.59	0.08	0.06	0.03	0.02	94.79
W160	0.80	0.02	0.01	0.00	0.00	0.58	0.08	0.06	0.03	0.02	95.33
W170	13.00	3.55	1.78	0.25	0.13	5.47	0.79	0.52	0.31	0.20	88.66
W180	8.22	2.55	1.27	1.05	0.53	2.12	0.30	0.20	0.12	0.08	83.67

Table 3 Curve number value of the project sub-basin

C. Storm Design

The SCS method is based on a 24-hour storm event which has a Type II time distribution. The Type II storm distribution is a typical time distribution which the SCS has prepared from rainfall records. It is applicable for interior rather than the coastal regions. So it is ideal for Ethiopia.

To apply distribution method, I should have 24Hr. point rainfall design frequency, to determine the point rain fall of 24 Hr. with different design frequency period, the climate data collected from EMA. Even if the collected data is more than seven stations with 20 years recorded data, except two station, the rest has less than 13 year recorded rain fall data, the two station namely: Ayer Tena and Bole International Airport consider for this analysis, then point rainfall computed by using frequency analysis method, and each station result compared with each other's and ERA DD of 24Hr storm duration, peak point rainfall is selected to minimize the risk factor, accordingly ERA DD, 2013 of 24Hr. (Region A2) is adopted.

$$RR_t = \frac{t}{24}\left[\frac{(b+24)^n}{(b+t)^n}\right]$$

RR_t=Rainfall Ration R_t: R_{24}, $R_{t=}$ Rainfall in a given duration't' in hours.

R_{24}= Rainfall in 24 hours, t= time in hours, b= 0.3, n=0.9.

Based on studies of a large number of gauges in East Africa, ($0.78 \leq n \leq 1.09$)

Best fit frequency analysis result.

For GEV K=0.06328, μ=40.577, s=10.218.

1. Ayer Tena Station.

Ayer Tena Station			
Year	Max	Log of Max	Rank
2006	60.1	1.78	1
2010	59.4	1.77	2
2015	51.2	1.71	3
2016	43.5	1.64	4
2012	43.4	1.64	5
2013	42.6	1.63	6
2017	41.6	1.62	7
2014	39.6	1.60	8
2011	38.1	1.58	9
2005	37.5	1.57	10
2008	34.7	1.54	11
2009	34.2	1.53	12
2007	32.4	1.51	13
2004	17.4	1.24	14
Mean	41.12	1.60	
Standard	10.96	0.13	
Skew Coefficient	-0.05	-1.33	

T(yr)	(P)	W	Z	Log Normal			Log Pearson Type III			EVI (Gumbel)		GEV	
				K_T	Y_T	X_T	K_T	Y_T	X_T	K_T	X_T	Z'	X_T
500	0.002	3.53	2.88	2.88	1.98	94.53	1.46	1.79	61.53	4.39	89.29	5.14	93.07
200	0.005	3.26	2.58	2.58	1.94	86.27	1.42	1.78	60.85	3.68	81.44	4.50	86.56
100	0.010	3.03	2.33	2.33	1.90	80.00	1.38	1.78	60.09	3.14	75.50	3.99	81.36
50	0.020	2.80	2.05	2.05	1.87	73.67	1.32	1.77	59.02	2.59	69.53	3.46	75.91
25	0.040	2.54	1.75	1.75	1.83	67.22	1.24	1.76	57.52	2.04	63.52	2.90	70.16
10	0.100	2.15	1.28	1.28	1.77	58.32	1.06	1.74	54.51	1.30	55.42	2.10	62.01
5	0.200	1.79	0.84	0.84	1.71	51.05	0.83	1.71	50.90	0.72	49.01	1.43	55.20
2	0.500	1.18	0.00	0.00	1.60	39.58	0.21	1.63	42.18	-0.16	39.32	0.36	44.28

Probability =P, Return Period (T (yr))

D (Min)	D (Hr.)	RR	Depth for given Return periods(mm)						Intensity for given return periods (mm/hr.)					
			2 years	5 years	10 years	25 years	50 years	100 years	2 years	5 years	10 years	25 years	50 years	100 years
5	0.08	0.15	6.44	8.02	9.01	10.20	11.03	11.83	77.23	96.28	108.16	122.39	132.40	141.91
10	0.17	0.24	10.78	13.44	15.10	17.09	18.49	19.81	64.70	80.66	90.61	102.53	110.92	118.89
20	0.33	0.37	16.38	20.43	22.95	25.96	28.09	30.11	49.15	61.28	68.84	77.89	84.26	90.32
30	0.50	0.45	19.92	24.83	27.89	31.56	34.14	36.60	39.83	49.66	55.78	63.12	68.29	73.19
45	0.75	0.53	23.39	29.16	32.76	37.06	40.10	42.98	31.19	38.88	43.67	49.42	53.46	57.30
60	1.00	0.58	25.73	32.08	36.04	40.78	44.11	47.28	25.73	32.08	36.04	40.78	44.11	47.28
75	1.25	0.62	27.46	34.23	38.45	43.51	47.07	50.45	21.97	27.38	30.76	34.81	37.65	40.36
90	1.50	0.65	28.80	35.90	40.33	45.63	49.37	52.92	19.20	23.93	26.89	30.42	32.91	35.28
120	2.00	0.70	30.80	38.39	43.13	48.80	52.79	56.59	15.40	19.20	21.56	24.40	26.40	28.29
150	2.50	0.73	32.25	40.20	45.16	51.10	55.29	59.26	12.90	16.08	18.07	20.44	22.11	23.70
180	3.00	0.75	33.38	41.61	46.75	52.89	57.22	61.33	11.13	13.87	15.58	17.63	19.07	20.44

Duration (D), Rainfall Ratio (RR)

Table 4 24 hr. Rainfall Depth from Ayer Tena Meteorological Station

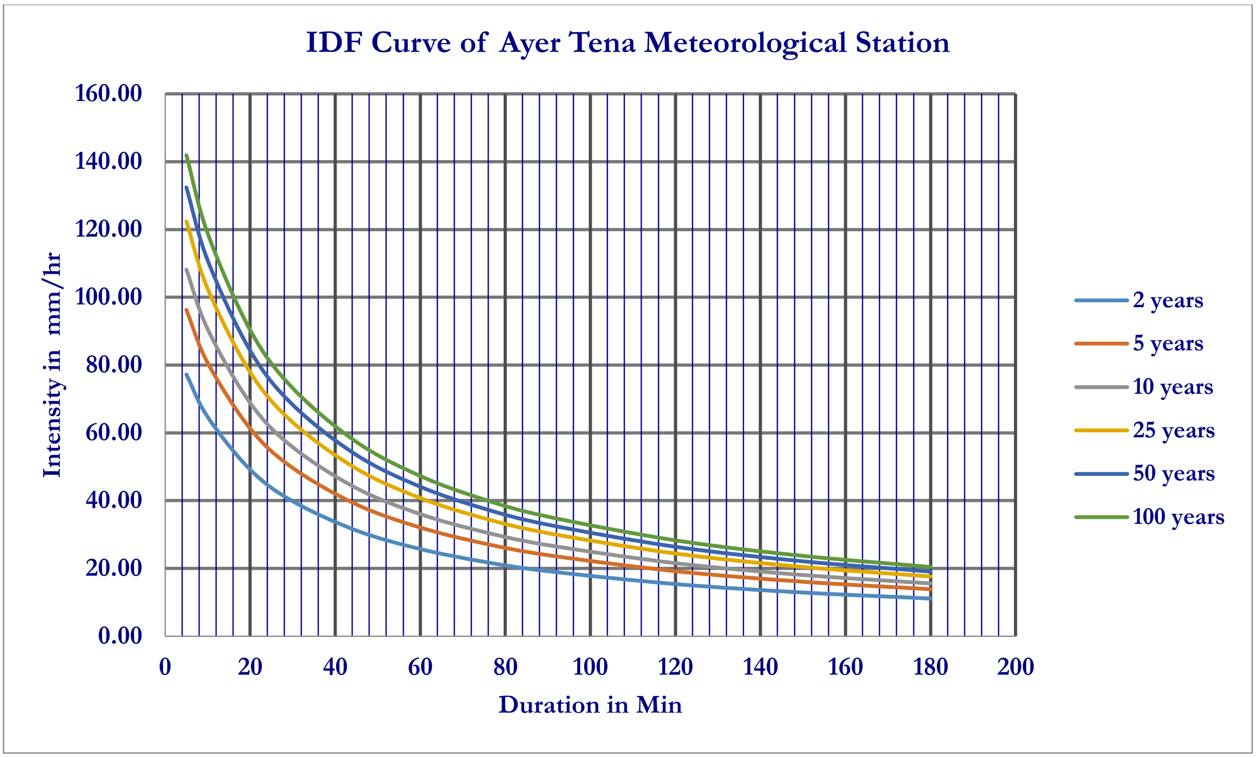


Figure 25 IDF Curve of Ayer Tena Meteorological Station

2. Bole International Airport Station

Ayer Tena Station			
Year	Max	Log of Max	Rank
2007	71.2	1.85	1
2012	64.7	1.81	2
2006	61.7	1.79	3
1998	60.1	1.78	4
2010	54.4	1.74	5
2009	51.2	1.71	6
2000	47.0	1.67	7
2005	44.5	1.65	8
2013	42.6	1.63	9
2011	40.0	1.60	10
1999	37.8	1.58	11
1997	37.3	1.57	12
2008	37.2	1.57	13
2003	34.6	1.54	14
2004	34.6	1.54	15
2001	32.4	1.51	16
2002	28.6	1.46	17
Mean	45.88	1.65	
Standard	12.61	0.12	
Skew Coefficient	0.63	0.28	

T(yr)	(P)	W	Z	Log Normal			Log Pearson Type III			EVI (Gumbel)		GEV	
				K_T	Y_T	X_T	K_T	Y_T	X_T	K_T	X_T	Z'	X_T
500	0.002	3.53	2.88	2.88	1.98	95.81	3.22	2.02	104.97	4.39	101.32	5.14	93.07
200	0.005	3.26	2.58	2.58	1.95	88.36	2.84	1.98	94.77	3.68	92.29	4.50	86.56
100	0.010	3.03	2.33	2.33	1.92	82.65	2.53	1.94	87.27	3.14	85.45	3.99	81.36
50	0.020	2.80	2.05	2.05	1.89	76.83	2.20	1.90	79.90	2.59	78.58	3.46	75.91
25	0.040	2.54	1.75	1.75	1.85	70.85	1.84	1.86	72.61	2.04	71.66	2.90	70.16
10	0.100	2.15	1.28	1.28	1.80	62.48	1.31	1.80	62.91	1.30	62.33	2.10	62.01
5	0.200	1.79	0.84	0.84	1.74	55.53	0.82	1.74	55.29	0.72	54.95	1.43	55.20
2	0.500	1.18	0.00	0.00	1.65	44.33	-0.05	1.64	43.79	-0.16	43.80	0.36	44.28

D (Min)	D (Hr.)	RR	Depth for given Return periods(mm)						Intensity for given return periods (mm/hr.)					
			2 years	5 years	10 years	25 years	50 years	100 years	2 years	5 years	10 years	25 years	50 years	100 years
5	0.08	0.15	6.44	8.02	9.01	10.20	11.03	11.83	77.23	96.28	108.16	122.39	132.40	141.91
10	0.17	0.24	10.78	13.44	15.10	17.09	18.49	19.81	64.70	80.66	90.61	102.53	110.92	118.89
20	0.33	0.37	16.38	20.43	22.95	25.96	28.09	30.11	49.15	61.28	68.84	77.89	84.26	90.32
30	0.50	0.45	19.92	24.83	27.89	31.56	34.14	36.60	39.83	49.66	55.78	63.12	68.29	73.19
45	0.75	0.53	23.39	29.16	32.76	37.06	40.10	42.98	31.19	38.88	43.67	49.42	53.46	57.30
60	1.00	0.58	25.73	32.08	36.04	40.78	44.11	47.28	25.73	32.08	36.04	40.78	44.11	47.28
75	1.25	0.62	27.46	34.23	38.45	43.51	47.07	50.45	21.97	27.38	30.76	34.81	37.65	40.36
90	1.50	0.65	28.80	35.90	40.33	45.63	49.37	52.92	19.20	23.93	26.89	30.42	32.91	35.28
120	2.00	0.70	30.80	38.39	43.13	48.80	52.79	56.59	15.40	19.20	21.56	24.40	26.40	28.29
150	2.50	0.73	32.25	40.20	45.16	51.10	55.29	59.26	12.90	16.08	18.07	20.44	22.11	23.70
180	3.00	0.75	33.38	41.61	46.75	52.89	57.22	61.33	11.13	13.87	15.58	17.63	19.07	20.44

Table 5 24 hr. Rainfall Depth from Bole International Meteorological Station

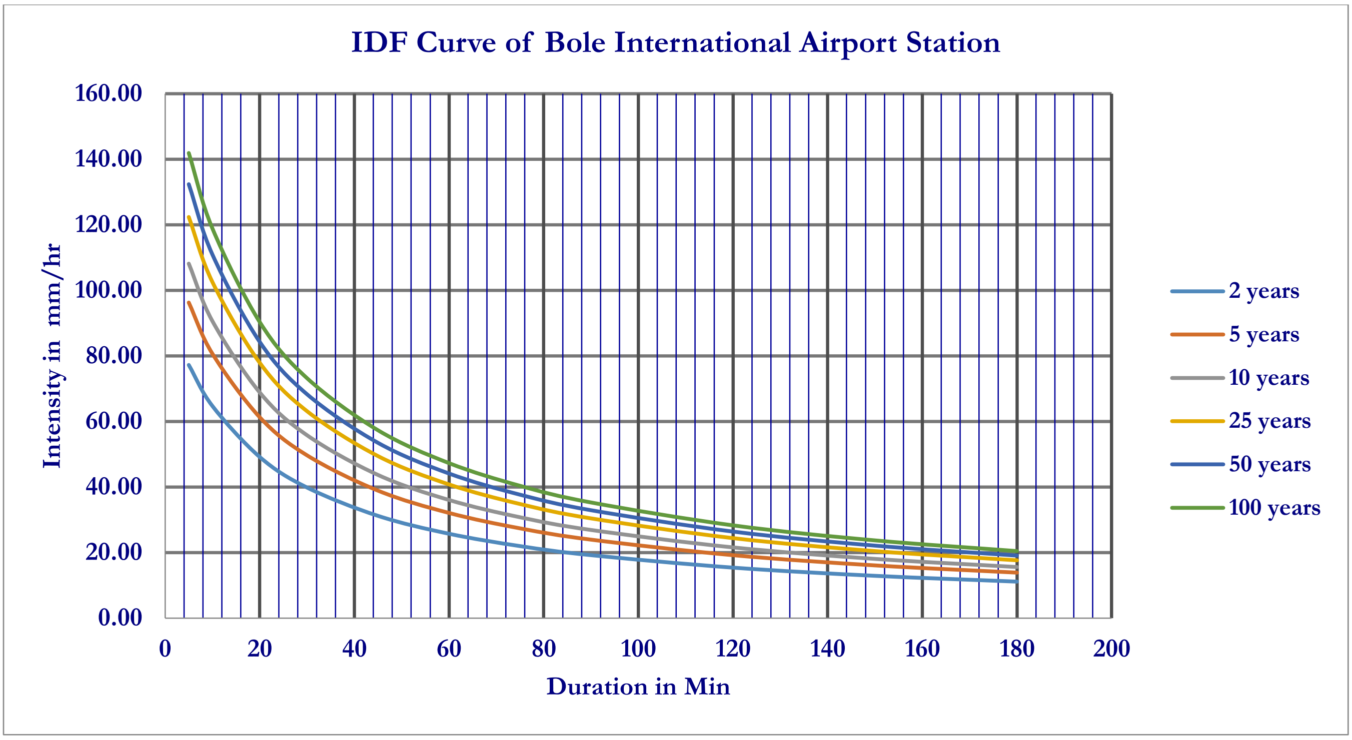


Figure 26 IDF Curve of Bole International Airport Meteorology Station

3. ERA design drainage manual

ERA, (2013) developed a 24 hour rainfall depth for different hydrological regions in the country displaying similar rainfall pattern as shown in Figure 27. The project area is found in Region A2 and the design storms for different return periods of 24 hour rainfall depth was derived from it.

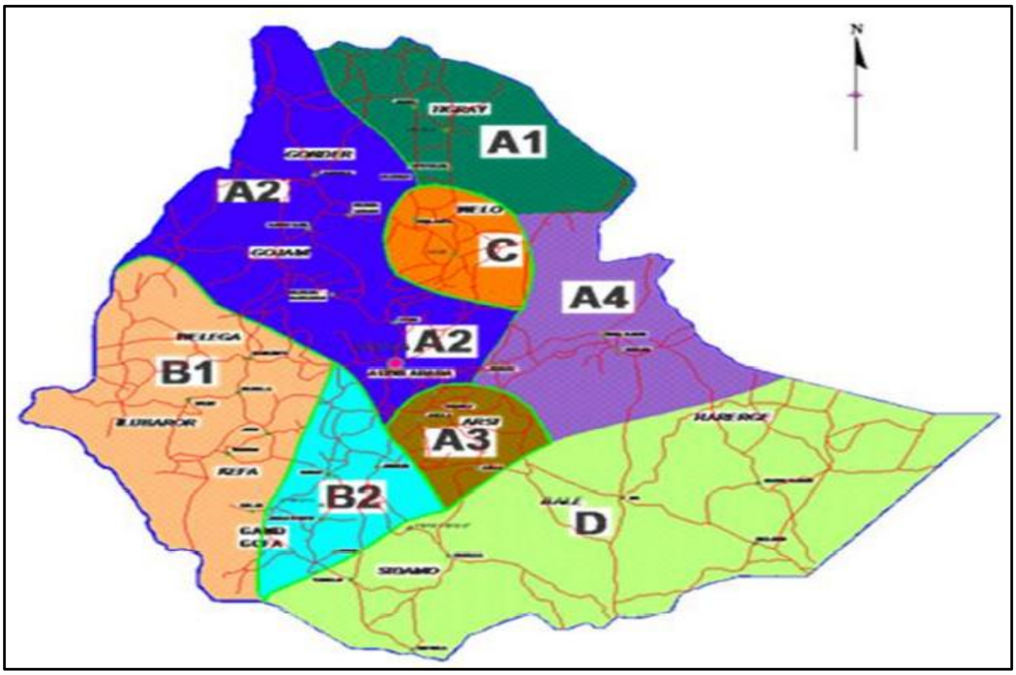


Table 5-19: 24hr Rainfall Depth Vs Frequency

24 hr Rainfall Depth (mm) vs Frequency (yr)								
Return Period Years	**2**	**5**	**10**	**25**	**50**	**100**	**200**	**500**
RR-A1	50.30	66.02	76.28	89.13	98.63	108.06	117.48	130.00
RR-A2	51.92	65.52	74.45	85.70	94.07	102.45	110.91	122.27
RR-A3	47.54	59.61	67.66	77.92	85.62	93.34	101.13	111.58
RR-A4	50.39	63.83	72.28	82.55	89.97	97.20	104.32	113.63
RR-B1	58.87	71.26	79.29	89.35	96.84	104.37	112.02	122.41
RR-B2	55.26	69.95	79.68	92.03	101.29	110.61	120.07	132.87
RR-C	56.52	71.04	80.54	92.52	101.48	110.50	119.66	132.06
RR-D	56.23	76.84	90.37	107.46	120.23	133.05	146.00	163.44

Note: RR- Rainfall Region

Figure 27 Rainfall Region A2

From the table, for region A2 the **2**, **5**, **10**, **25, 50 and 100** year return periods were selected and their respective design storm depths of 24 hour duration are **51.92mm**, **65.52mm**, **74.45mm**, **85.7mm**, **94.07mm** and **102.45 mm.** The other 1Hr, 2Hr and 3Hr duration of storm was extracted from IDF curve of region A2 and tabulated in the below table, then after for 6Hr and 12Hr duration was determined by interpolation.

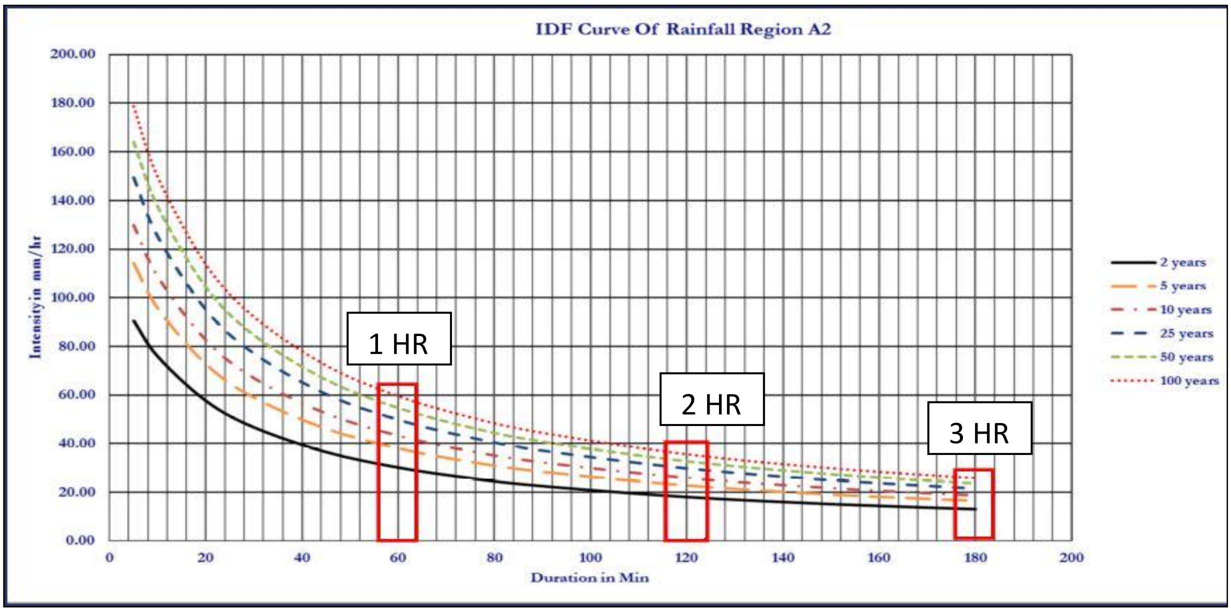


Figure 28 IDF Curve of Rainfall Region A2

Precipitation Depths in mm for different durations and return periods							
		Frequency in yrs.					
		2	5	10	25	50	100
Duration in Hrs.	1	27.1	35.3	41.5	49.6	55.8	62.0
	2	32.6	42.4	49.8	59.4	66.8	74.2
	3	40.6	54.0	58.8	63.0	70.2	79.2
	6	48.1	61.0	68.7	72.0	78.3	84.7
	12	49.1	63.0	70.6	79.0	84.6	94.8
	24	51.92	65.52	74.5	85.5	94.1	102.5

Table 6 Precipitation Depths in mm for different durations and return periods.

4. Comparison of stations data and ERA.

24hr Rainfall Depth from Bole International Airport Meteorological Station

Return Period T(yr)	Log Normal	Log Pearson Type III	Log Pearson Type III	GEV	RR A2
500	95.81	104.97	101.32	93.07	122.27
200	88.36	94.77	92.29	86.56	110.91
100	82.65	87.27	85.45	81.36	102.45
50	76.83	79.90	78.58	75.91	94.07
25	70.85	72.61	71.66	70.16	85.70
10	**62.48**	**62.91**	**62.33**	**62.01**	**74.45**
5	**55.53**	**55.29**	**54.95**	**55.20**	**65.52**
2	**44.33**	**43.79**	**43.80**	**44.28**	**51.92**
24hr Rainfall Depth from Ayertena Meteorological Station					
Return Period T(yr)	**Log Normal**	**Log Pearson Type III**	**EVI(Gumbel)**	**GEV**	**RR A2**
500	**94.53**	**61.53**	**89.29**	**93.07**	**122.27**
200	**86.27**	**60.85**	**81.44**	**86.56**	**110.91**
100	**80.00**	**60.09**	**75.50**	**81.36**	**102.45**
50	**73.67**	**59.02**	**69.53**	**75.91**	**94.07**
25	**67.22**	**57.52**	**63.52**	**70.16**	**85.70**
10	**58.32**	**54.51**	**55.42**	**62.01**	**74.45**
5	**51.05**	**50.90**	**49.01**	**55.20**	**65.52**
2	**39.58**	**42.18**	**39.32**	**44.28**	**51.92**

Hence the 24hr. Rainfall depth is taken from ERA Rainfall Region A2 is used for the subsequent hydrologic analysis.

Hence the 24hr. Rainfall depth is taken from ERA Rainfall Region A2 is used for the subsequent hydrologic analysis.

Time series data

Time series data includes both flow and precipitation data**.** In the HEC-HMS model no time series data was used for modeling, since no gauging station available in the project area and it is also difficult to find flow data.

Control specification

Control specification is the part of the software where the computation time interval *(Δt)*, the Start and End time of simulation specified. After that the program runs and the simulated results collected from the result tab both in the form of Table and Graphs.

3.4 Hydraulic Modeling (HEC_RAS)

HEC-RAS is a computer program which is one of the best performing hydraulic analysis of open channel problems. The Program was specifically developed for analysis of Highway Bridge and culvert backwater analysis.

HEC-RAS is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels.
HEC-RAS is capable of importing GIS/CAD data. Source ERADDM 2013.

In this thesis works there are different input data for HEC-RAS model and it involves some procedures, the followings are some input data in HEC-RAS model.

- Define reach and name of the river
- Enter geometry cross section data
- Identify and enter manning's roughness coefficient, expansion and contraction of the bridge.
- The bridges cross section and layout inserted in the bridge and culvert data editor.

Once river and reach name defined simple layout of the river, plotted on HEC-RAS, and geometry of the cross sections were determined with Auto CAD then the cross section output transferred to excel and cross section data exported to HEC-RAS geometric data editor.

The elevation of cross sections, the distance between the banks, the manning's roughness coefficient, expansion and contraction coefficients was applied. Manning's values 0.035 and 0.04 used for the flood plain and main channel respectively, for contraction coefficient 0.3 and 0.1 was used at the bridge section and for the rest of the river section respectively. In the same way 0.4 and 0.3 was used for expansion coefficient based on the hydraulic reference manual recommendation, the bridge cross section and layout is inserted in the bridge data editor window. The bridge length, height, high and low chords, width and finally the abutments were inserted in this editor.

The bridge geometric data and additional figures are annexed at the end of the paper.

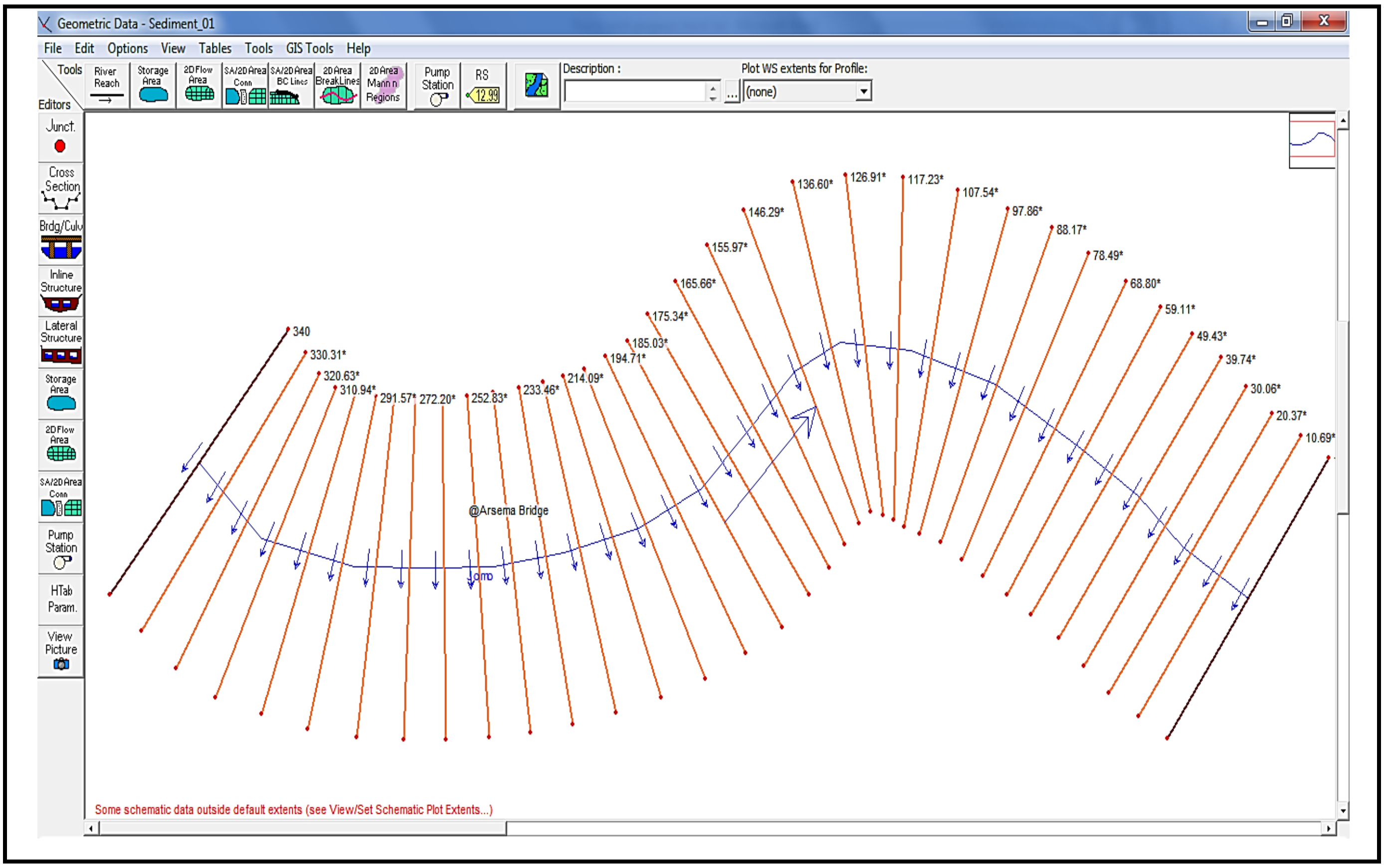


Figure 29 Geometric Data

4. Result and Discussion

In this section the result of the models described in detail using graphs, tables and figures. Both hydrologic and hydraulic results were assembled here.

4.1. Peak Discharge Result Using HEC-HMS at Arsema Bridge (Study Area)

Based on ERA drainage design manual 2002 and 2013, **24 Hr., 12 Hr., 6 Hr., 3Hr and 1Hr duration of rainfall depth for 2, 5, 10, 25, 50, 100** years return period was taken as input in HEC-HMS software for this study area analysis and the results are summarized in the following table and graphs.

	Return Period (Year)					
Q_{max} (m^3/s)	**2**	**5**	**10**	**25**	**50**	**100**
	67	**97.6**	**117**	**136.1**	**154.5**	**174.9**

Table 7 Peak Flood at Different Return Period

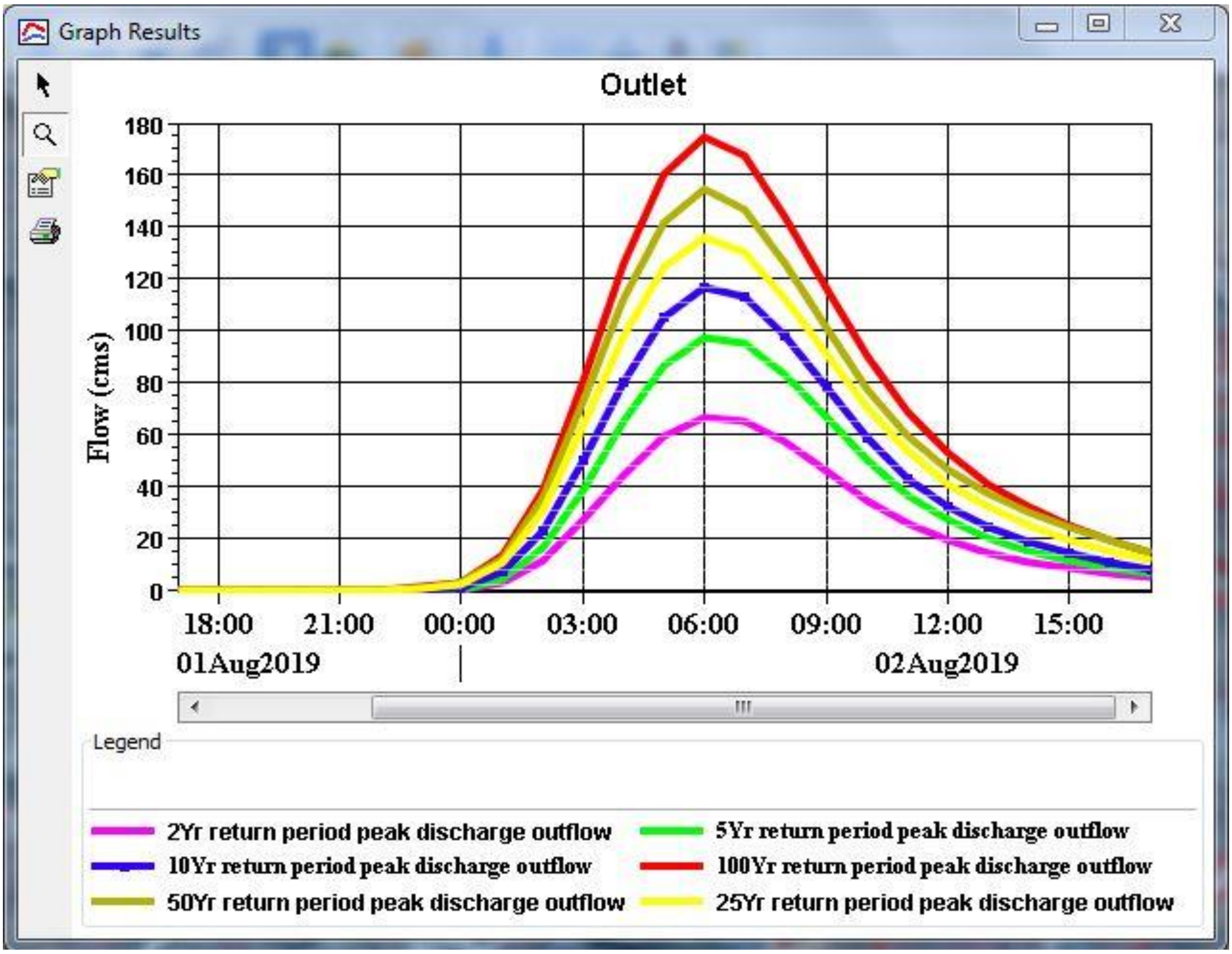


Figure 30 Peak Flood Graph at different return period 2, 5, 10, 25, 50 and 100 Yr.

4.2.Model Validation and Calibration

Study area is located in Akaki sub-basin , large Akaki is one the component of Akaki basin which share similar physical and hydrological parameters with the study area, as it is shown in the below maps of soil and land use, both basin share similar feature.

That means both large Akaki and study area found in the same hydrologic region, since they share similar physical and hydrological parameters, so it is possible to transfer the hydrological data from large Akaki to the study area. Large Akaki has a gauged river flow data in the outlet of the catchment, then its' peak flow determined by frequency analysis method, and then peak flow transferred to the study area to estimate peak discharge by using Dr. Admasu G. formula (source ERADDM, 2002).

Both peak discharge results i.e. HEC-HMS and transferred method is plotted in line of fit method in excel, and then regression result is $0.7 \leq R^2 \leq 1$. It shows us HEC-HMS model is ideal for this study area.

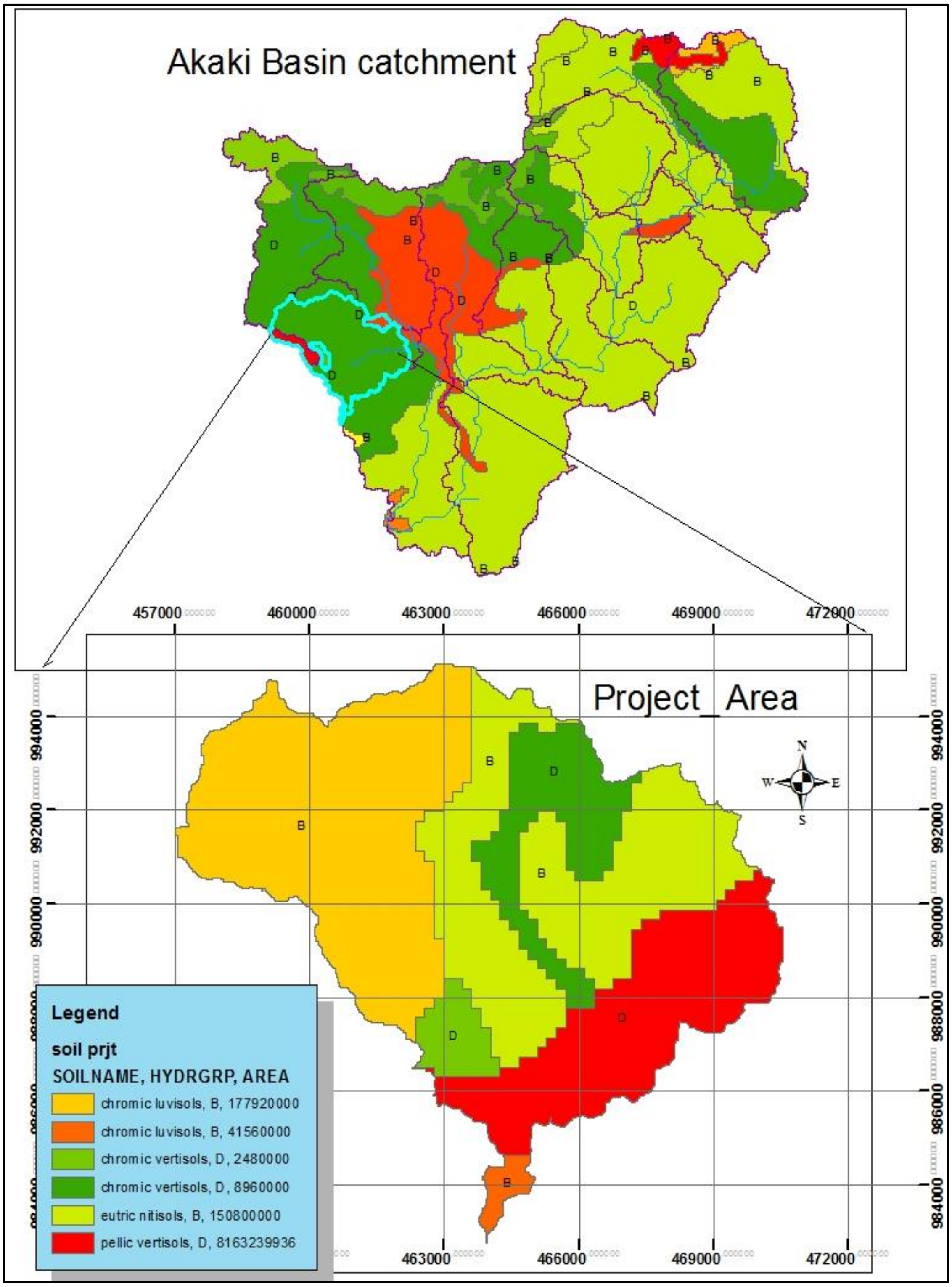


Figure 31 Soil map of Akaki catchment including study area

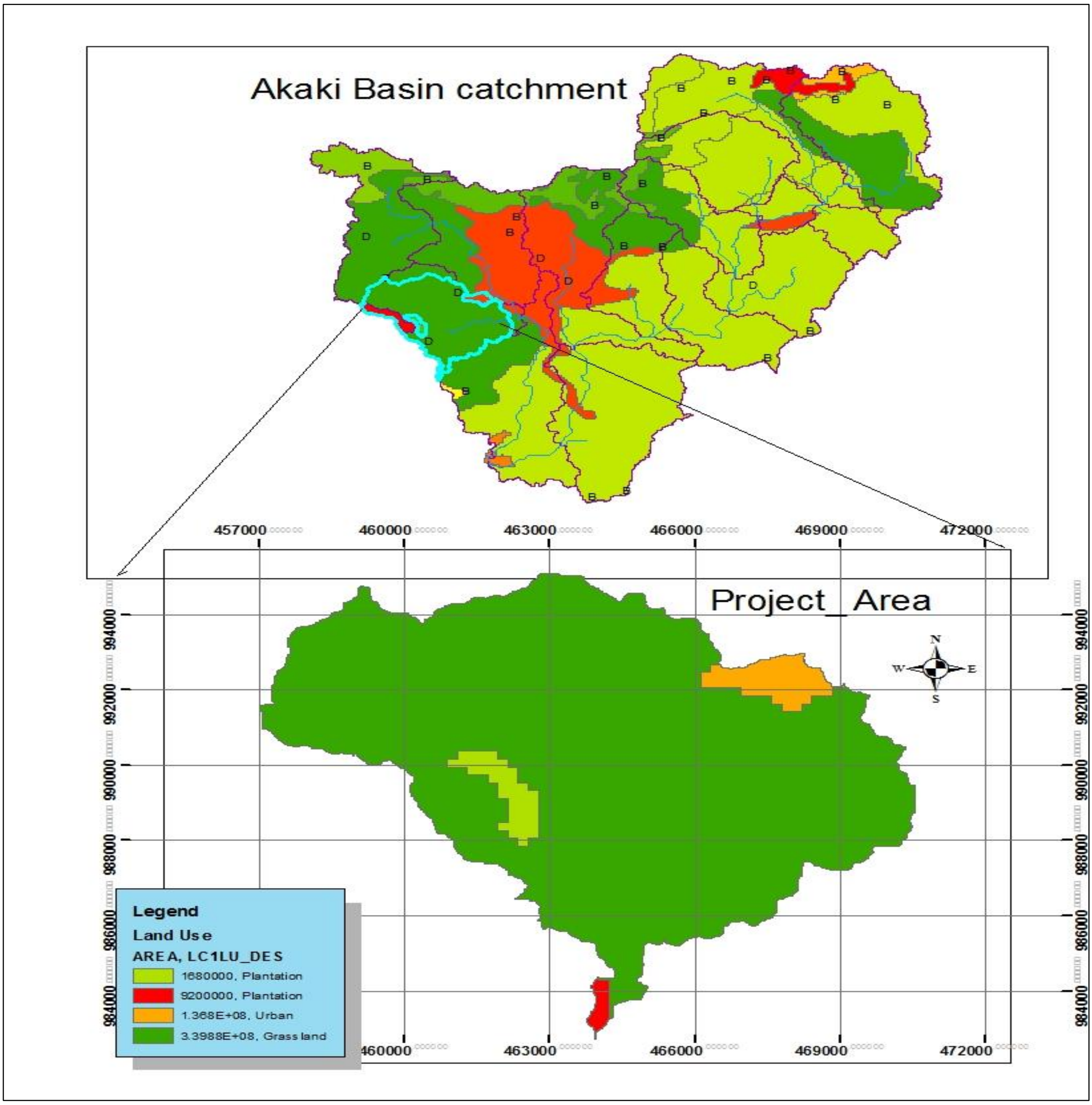


Figure 32 Land use and cover map of Akaki catchment including study area.

4.2.1. Flood Frequency Analysis Method

The stream flow data of Akaki River was acquired from Ministry of Water, Irrigation and Electricity to estimate design flood at Akaki stream flow gauge station Log –Normal, log Pearson Type III and Gumbel (Extreme Value I), methods are applied to estimate the peak flood.

Akaki River Flow Gauge Station			
Year	Max	Log of Max	Rank
1999	693.10	2.84	1
1996	615.76	2.79	2
1993	573.57	2.76	3
2001	435.34	2.64	4
1998	421.52	2.62	5
2003	420.06	2.62	6
1990	277.22	2.44	7
1997	276.32	2.44	8
1995	257.98	2.41	9
2000	255.78	2.41	10
2004	250.47	2.40	11
1989	233.77	2.37	12
2002	219.87	2.34	13
1991	215.22	2.33	14
1981	201.31	2.30	15
1984	189.38	2.28	16
1982	172.77	2.24	17
1985	165.65	2.22	18
1994	162.58	2.21	19
1992	153.07	2.18	20
1988	148.35	2.17	21
1983	138.72	2.14	22
1986	68.78	1.84	23
1987	36.55	1.56	24
Mean	274.30	2.36	
Standard	168.69	0.29	
Coefficient of Skew	1.16	-0.70	

Return Period T(yr.)	P	W	Z	Log Normal			Log Pearson Type III			EVI (Gumbel)	
				K_T	Y_T	X_T	K_T	Y_T	X_T	K_T	X_T
500	0.002	3.53	2.88	2.88	3.18	1530.22	2.07	2.95	897.39	4.39	1015.65
200	0.005	3.26	2.58	2.58	3.10	1252.68	1.94	2.91	820.46	3.68	894.94

100	0.010	3.03	2.33	2.33	3.03	1061.97	1.81	2.88	756.46	3.14	803.44
50	0.020	2.80	2.05	2.05	2.95	886.61	1.67	2.84	686.80	2.59	711.60
25	0.040	2.54	1.75	1.75	2.86	725.39	1.49	2.79	610.66	2.04	619.08
10	0.100	2.15	1.28	1.28	2.73	531.64	1.18	2.70	497.99	1.30	494.37
5	0.200	1.79	0.84	0.84	2.60	397.21	0.86	2.60	400.79	0.72	395.66
2	0.500	1.18	0.00	0.00	2.36	227.55	0.11	2.39	245.51	-0.16	246.58

Table 8 Flow Frequency Analysis at Akaki River

4.2.2. Transposing or Transferring Method

Transferring Gauged Data - Gauged data may be transferred to an ungauged site of interest provided such data are nearby (i.e., within the same hydrologic region, and there are no major tributaries or diversions between the gage and the site of interest).

These procedures make use of the constants obtained in developing the regression equations.

These procedures are adopted from the work of Dr. Admasu G. as it is stated ERADDM, 2002:

$$\frac{Q_u}{Q_g} = \left[\frac{A_u}{A_g}\right]^{0.7}$$

Where: Q_u = Flow rate at un-gaged site

Q_g = Flow rate at gaged site

Au = Ungauged drainage basin area

Ag = Gauged drainage basin area

Return period (Year)	Area of Large Akaki (KM^2)	Study Area (KM^2)	Q_{peak} of Large Akaki Basin	Q_{peak} of the Study Area using transfer method.
2	901.7	73.3	227.5	39.3
5	901.7	73.3	397.2	68.6
10	901.7	73.3	531.6	91.8
25	901.7	73.3	725.4	125.2
50	901.7	73.3	886.6	153.0
100	901.7	73.3	1062.0	183.3

Table 9Transposing or Transferring method result

Peak discharge output of Transfer Method and HEC-HMS results are compiled in the below table and plotted on line of fit on excel.

Return period	Q_{peak} of Study Area using HEC HMS	Q_{peak} of Study Area using transfer from Akaki Basin
2	67.0	39.3
5	97.6	68.6
10	117.0	91.8
25	136.1	125.2
50	154.5	153.0
100	174.9	183.3

Table 10 Peak discharge output of Transfer Method and HEC-HMS results

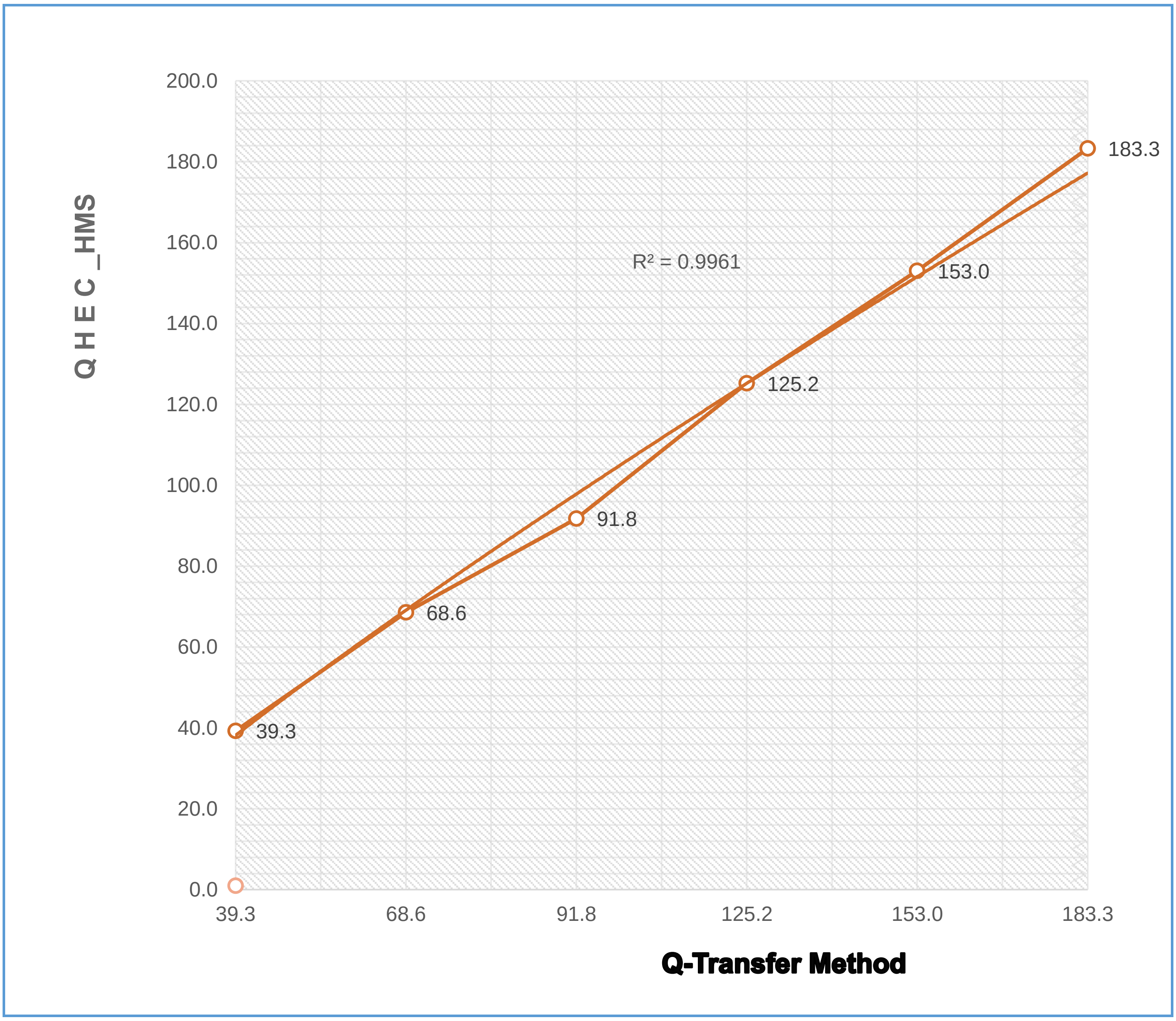


Figure 33 Line of fit with excel

From the above figure it has good relationship between the simulated output of HEC-HMS model and the

Transfer method results. **Therefore the HEC-HMS model is ideal for the study area** so based on the result of hydrological analysis it is possible to continue the hydraulic performance, sediment aggradations and trash accumulation analysis of the Arsema Bridge.

4.3.HEC-RAS Water Surface Profile without the Bridge

A HEC-RAS model was generated based on the geometry of the river channel and dimensions of Arsema 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 without bridge, with bridge and furthermore the sediment transport simulation and the hydraulic performance of the bridge analyzed with help of HEC-RAS.

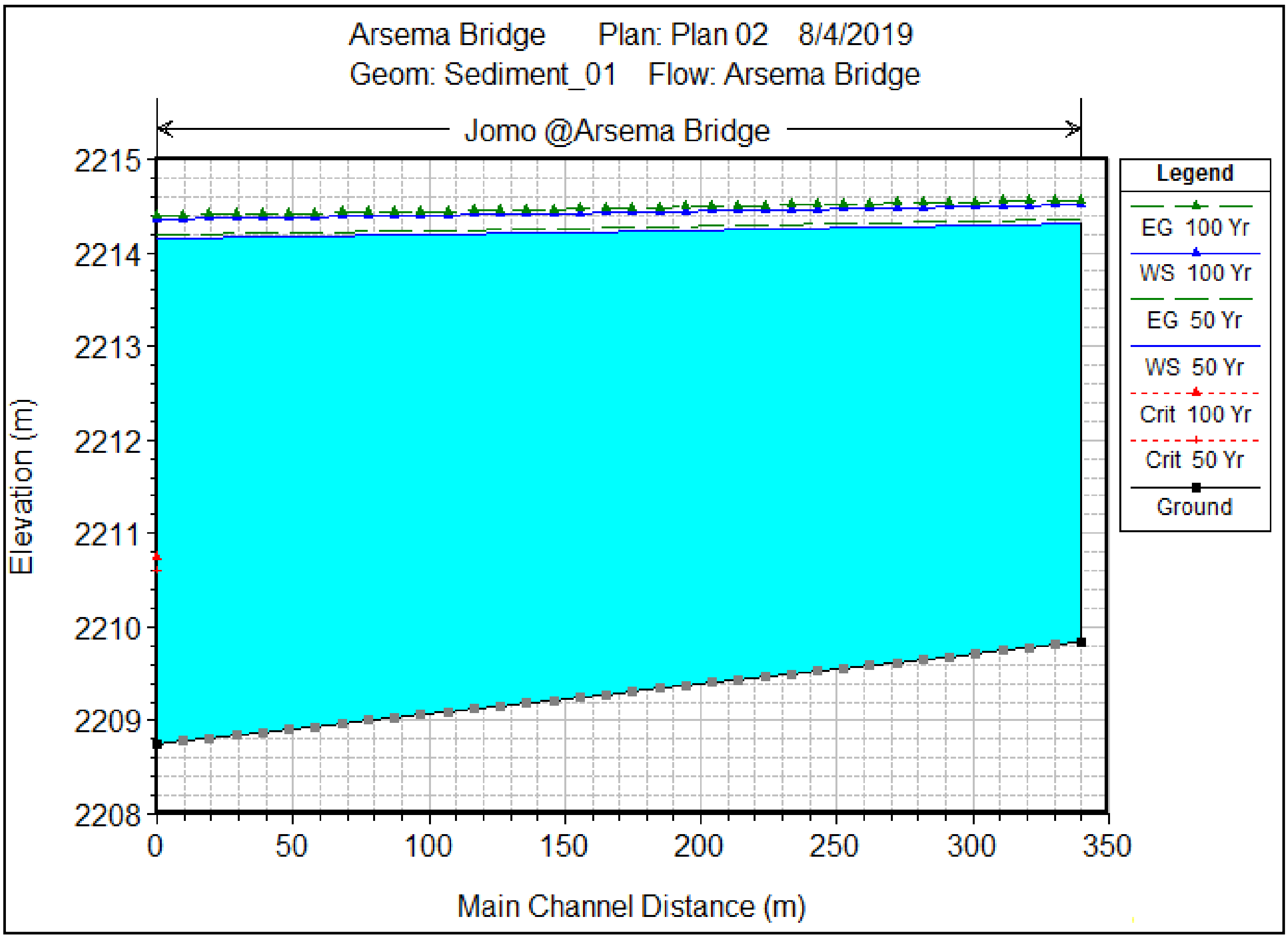


Figure 34 Water Surface Profile Output Of HEC –RAS.

From the above Figure it is likely to observe that the water surface profile for the 50year design discharge at the upstream side is **2214.36m** amsl without considering the bridge in the model. The simulated water surface elevation is much greater than even the low chord level, **2212.856m** amsl which is used to determine the bridge height during the design phase. This can be considered as one indication that existence of the bed elevation change in the vicinity of the bridge. Having this in mind the next step is to insert the bridge and see if there exists further rise in the water surface elevation.

4.4.HEC-RAS Water Surface Profile with the Bridge

After inserting all geometric data from the surveying detail into the model the next step was to determine whether the bridge is overtopped or not with the given design discharge inputs.

Water surface profile both at the upstream and downstream of the bridge were computed and presented in the below Figure 35, 36 and 37.

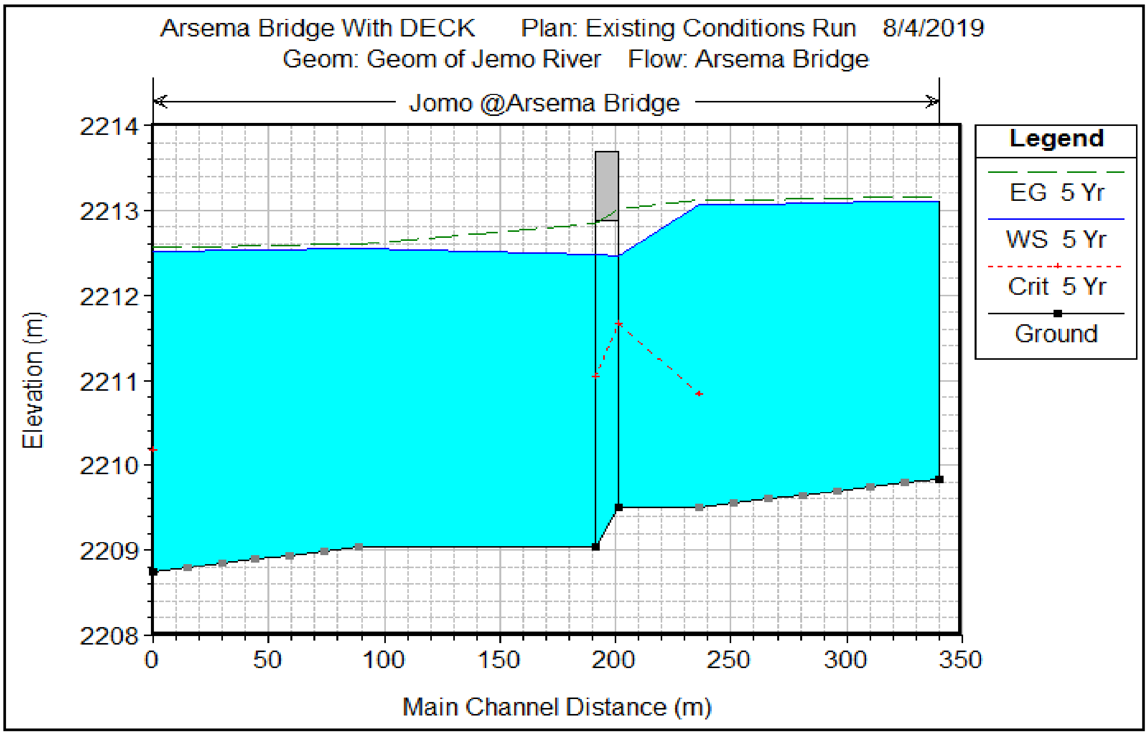


Figure 35 HEC-RAS water surface profile including the bridge at 5 yr. return period.

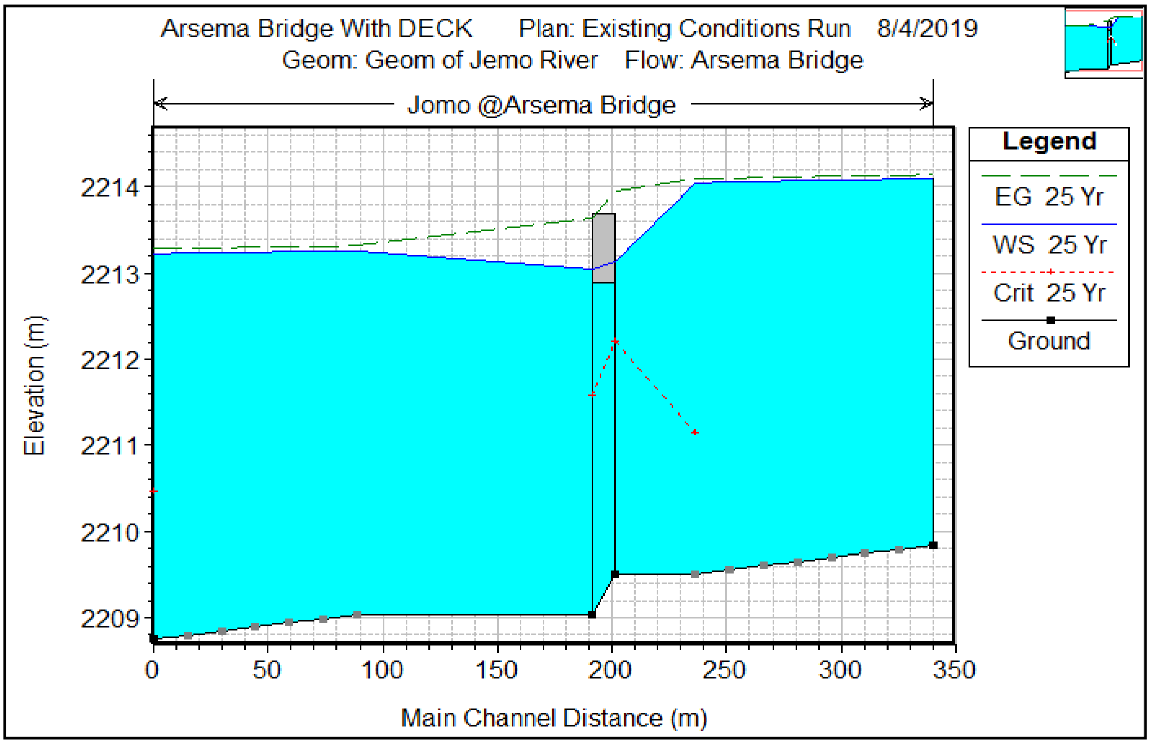


Figure 36 HEC-RAS water surface profile including the Bridge at 25 yr. Return Period.

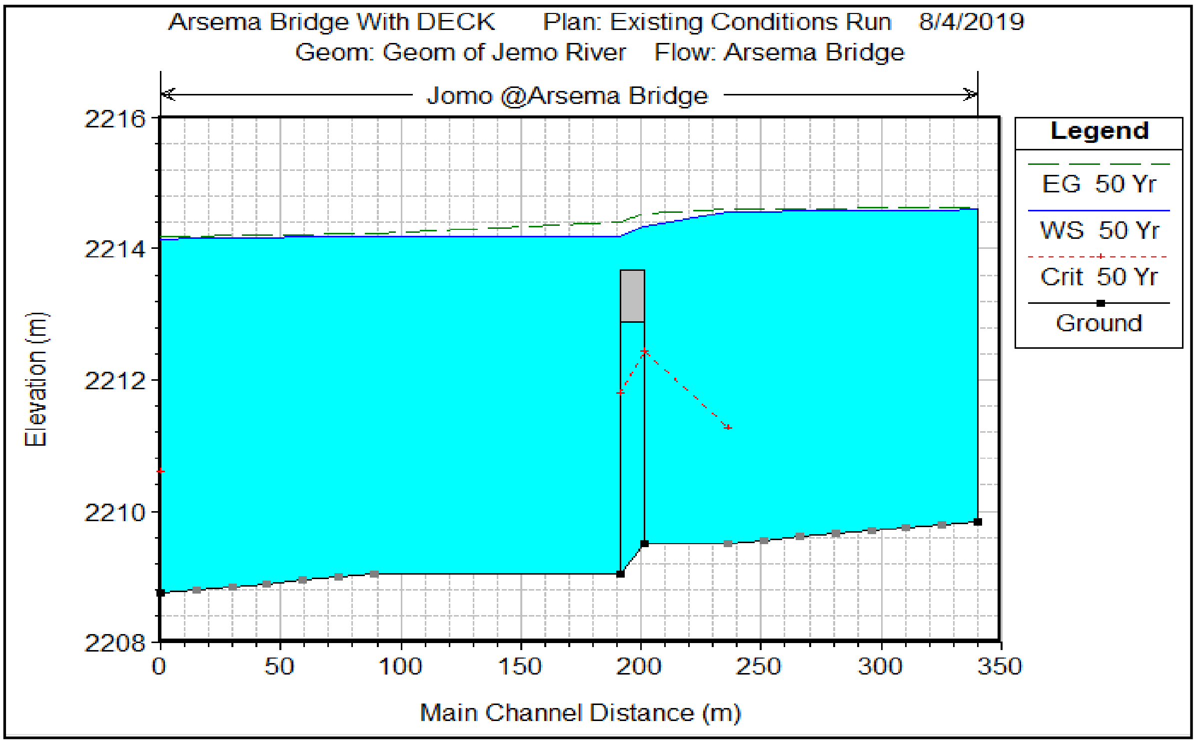


Figure 37 HEC-RAS water surface profile with the Bridge at Q_{50}.

The water surface profile at Figure 35 for the 5 year return period design flood is below the Bridge low chord. Therefore the water can pass through the bridge opening even though it is a pressurized flow type.

However the water surface profile at Figure 36 and 37 for the 25 and 50 year return period is totally above the Bridge high chord, the water surface profile for the Q_{25} and Q_{50} is **2214.05m** and **2214.57m** amsl respectively. The highest elevation (High chord) of the Bridge is **2213.684m** amsl which shows an elevation difference of **0.366m** and **0.886m** below the water surface level of Q_{25} and Q_{50} respectively.

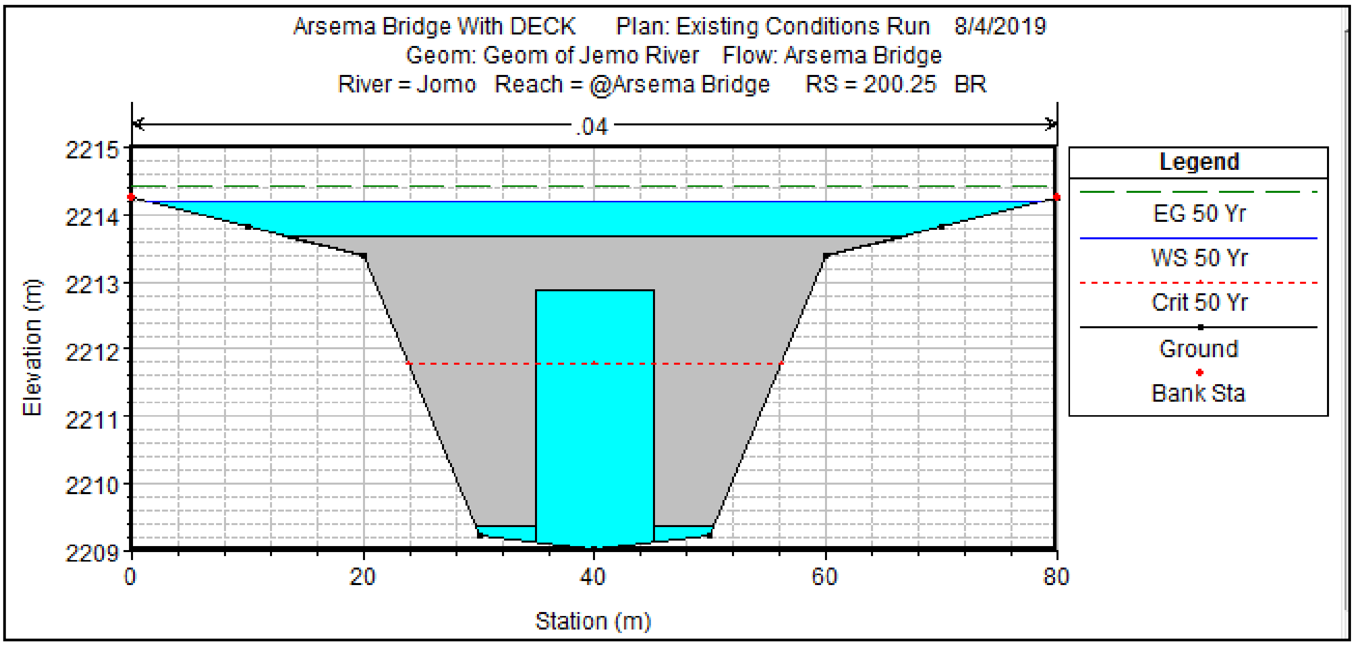


Figure 38 Maximum Water Surface Profile.

There is also a **4.901m** difference between the maximum water surface elevation and invert level of bridge. In addition there is **0.21m** difference in maximum water surface elevation before and after the bridge inserted into the model which implies the Bridge create back water influence and then contribute its part for flooding of upstream residential area. **This clearly shows that the current Bridge opening capacity is not capable enough to convey even the 25 year design flood.**

4.5. Sediment Transport Analysis Results in HEC-RAS

Based on the current cross section and bed material of the river, HEC-RAS computed the bed elevation change along the river bed. For the purpose of comparison, the less frequent 50 yr. peak flood and the more frequent 2 yr. peak flood were used as an input.

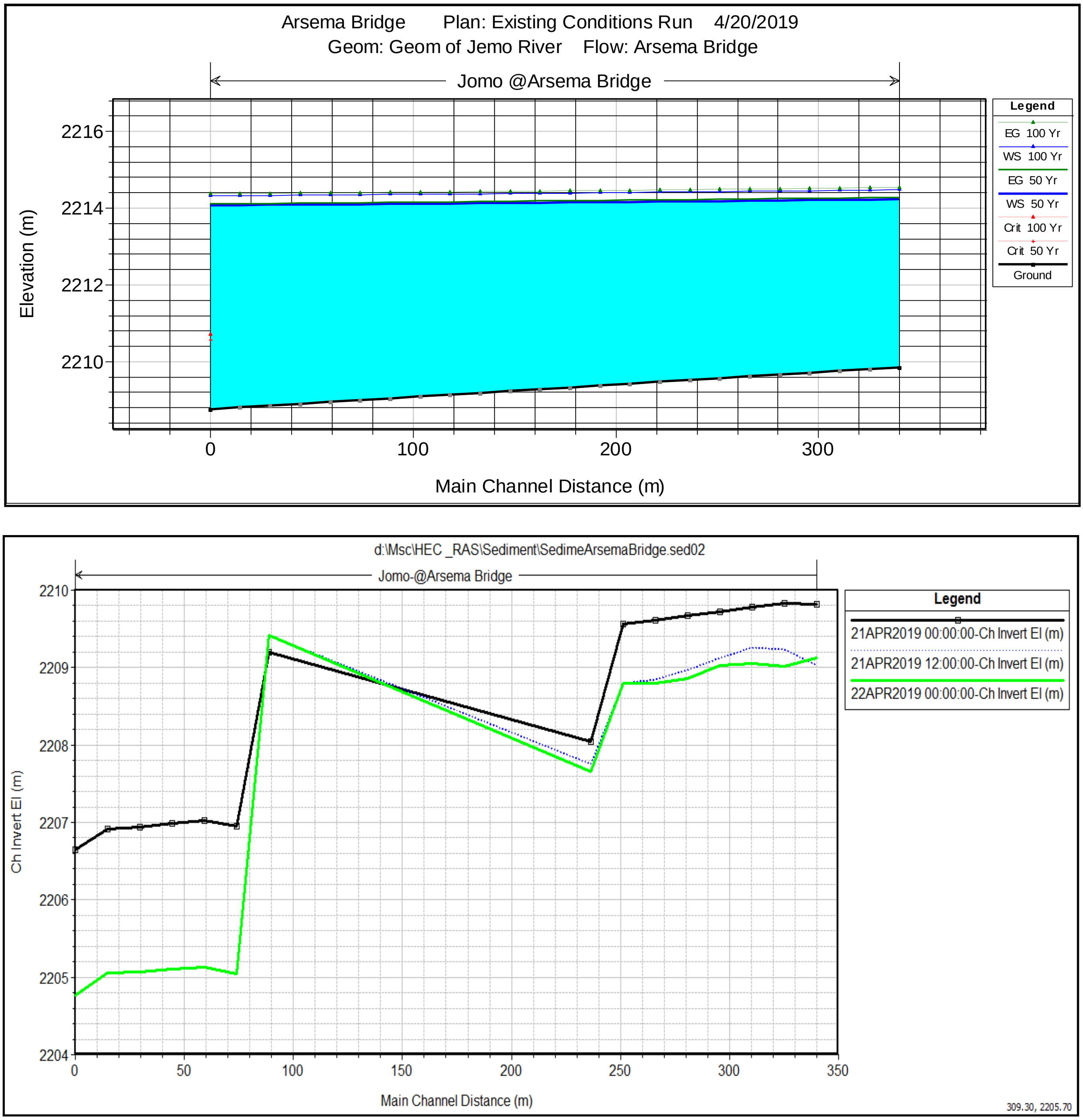


Figure 39 River Bed Profile Plot along the River (Q_{50})

From Figure.39 we can conclude that there exists a series of sediment deposition and scouring both at the upstream and downstream side of the Bridge. Deposition of bed observed at the immediate upstream and downstream side of the Bridge whereas Scouring observed at the downstream side of the Bridge. The bed elevation change outputs are compiled in below Table.

River	Reach	RS	Ch Dist	(21APR2019 00:00:00)-Ch Invert El (m)	(21APR2019 00:00:00)-Invert Change (m)	(21APR2019 00:00:00)-Eff Depth (m)	**(21APR2019 12:00:00)-Ch Invert El (m)**	(21APR2019 12:00:00)-Invert Change (m)	**(21APR2019 12:00:00)-Eff Depth (m)**	**(22APR2019 00:00:00)-Ch Invert El (m)**	(22APR2019 00:00:00)-Invert Change (m)	**(22APR2019 00:00:00)-Eff Depth (m)**
Jemo	@Arsema Bridge	340	14.7831	2209.82	0	3.872521	2209.026	-0.7945919	4.300218	2209.129	-0.6915548	4.215694
Jemo	@Arsema Bridge	325.26*	14.7831	2209.83	0	3.853912	2209.237	-0.5930775	4.075884	2209.012	-0.8179687	4.225622
Jemo	@Arsema Bridge	310.52*	14.7831	2209.777	0	3.889018	2209.263	-0.5139605	4.040319	2209.049	-0.7286176	4.183902
Jemo	@Arsema Bridge	295.78*	14.7831	2209.723	0	3.926558	2209.129	-0.5937201	4.136177	2209.026	-0.6968209	4.201078
Jemo	@Arsema Bridge	281.04*	14.7831	2209.666	0	3.966187	2208.972	-0.6938788	4.269448	2208.861	-0.8054447	4.340458
Jemo	@Arsema Bridge	266.30*	14.7831	2209.611	0	4.003565	2208.85	-0.7612941	4.373072	2208.801	-0.8104576	4.400335
Jemo	@Arsema Bridge	251.57*	14.7831	2209.56	0	4.038761	2208.793	-0.7674186	4.430418	2208.793	-0.7674186	4.422664
Jemo	@Arsema Bridge	236.83*	147.83	2208.04	0	5.066377	2207.757	-0.2835757	5.017371	2207.655	-0.3849019	5.0783
Jemo	@Arsema Bridge	89.43*	14.7831	2209.204	0	1.602082	2209.413	0.2085534	1.5726	2209.413	0.2085647	1.572602
Jemo	@Arsema Bridge	74.70*	14.7831	2206.957	0	2.442171	2205.045	-1.911816	2.557162	2205.046	-1.911207	2.557165
Jemo	@Arsema Bridge	59.96*	14.7831	2207.024	0	2.260084	2205.127	-1.896948	2.358415	2205.127	-1.896948	2.35842
Jemo	@Arsema Bridge	45.22*	14.7831	2206.994	0	2.17893	2205.101	-1.892466	2.278028	2205.102	-1.891577	2.278033
Jemo	@Arsema Bridge	30.48*	14.7831	2206.937	0	2.119331	2205.07	-1.866726	2.178486	2205.075	-1.862317	2.178494
Jemo	@Arsema Bridge	15.74*	14.7831	2206.913	0	1.679415	2205.054	-1.858495	1.707625	2205.056	-1.856645	1.707598
Jemo	@Arsema Bridge	1	0	2206.642	0	1.68943	2204.766	-1.876025	1.7204	2204.765	-1.87688	1.71993

Table 11 sediment deposition and scouring the bridge 50 yr return period peak flood.

In order to investigate the short term effects of the river flow on the river bed profile, more frequent flood (Q_2) was used in the model bed elevation change result was shown in the below figure and table.

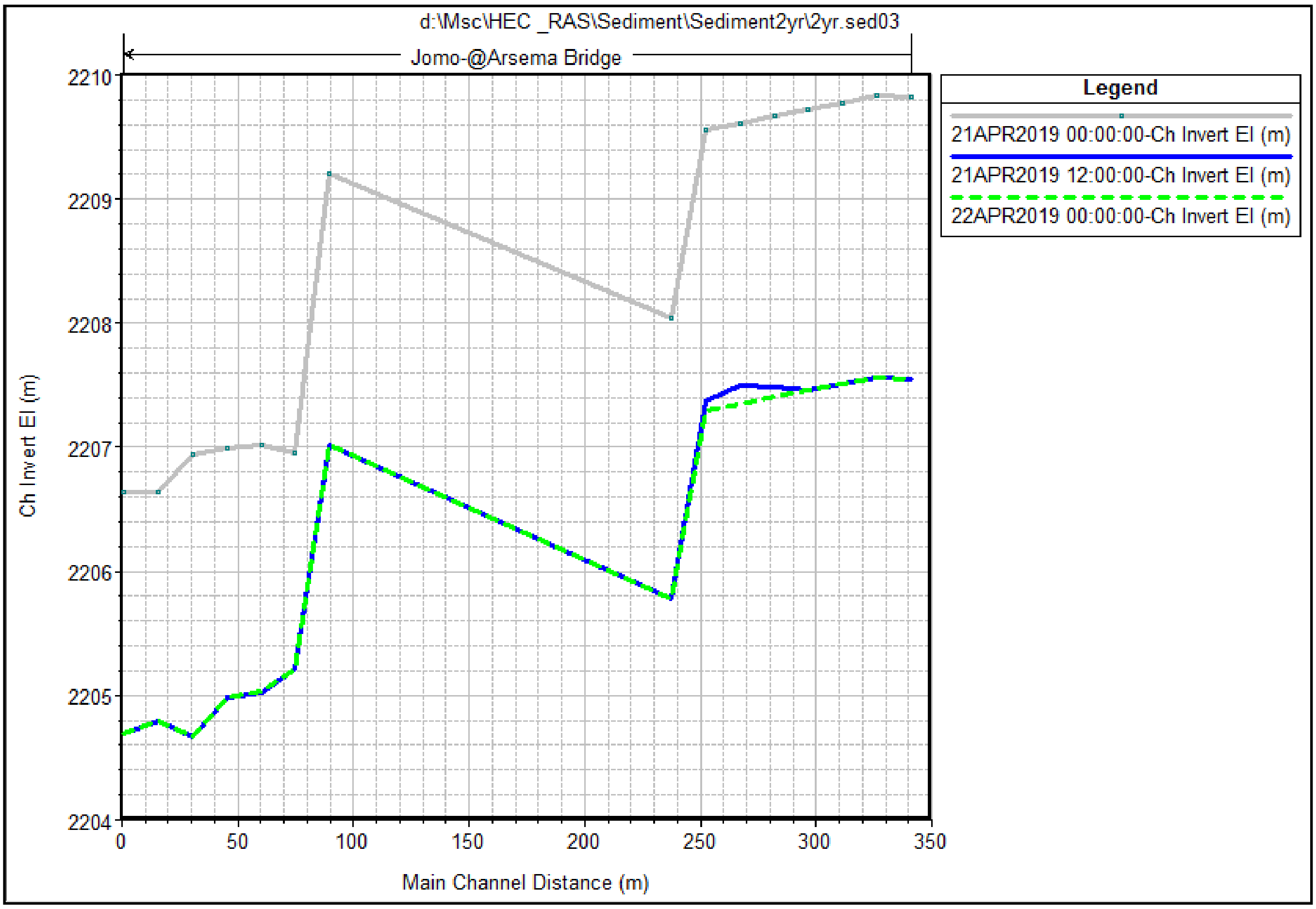


Figure 40 River bed profile plot along the river (Q2)

Figure.40 also showed the same pattern of sediment aggradations and degradation as in Figure 39. The major difference here is the magnitude of bed elevation change both due to aggradations and degradation reduced, which is directly connected to the reduction in flow input. The outputs of bed elevation are summarized in the below Table.

River	Reach	RS	Ch Dist	(21APR2019 00:00:00)-Ch Invert El (m)	(21APR2019 00:00:00)-Invert Change (m)	(21APR2019 00:00:00)-Eff Depth (m)	(21APR2019 12:00:00)-Ch Invert El (m)	(21APR2019 12:00:00)-Invert Change (m)	(21APR2019 12:00:00)-Eff Depth (m)	(22APR2019 00:00:00)-Ch Invert El (m)	(22APR2019 00:00:00)-Invert Change (m)	(22APR2019 00:00:00)-Eff Depth (m)
Jemo	@Arsema Bridge	340	14.7831	2209.84	0	1.505079	2207.944	-1.896013	1.943891	2207.731	-2.108807	1.804432
Jemo	@Arsema Bridge	325.26*	14.7831	2209.793	0	1.505992	2208.158	-1.635222	1.679234	2207.733	-2.059537	1.768236
Jemo	@Arsema Bridge	310.52*	14.7831	2209.745	0	1.507378	2208.178	-1.566847	1.612007	2207.716	-2.029471	1.751458
Jemo	@Arsema Bridge	295.78*	14.7831	2209.698	0	1.5082	2208.169	-1.52894	1.570803	2207.686	-2.012234	1.749051
Jemo	@Arsema Bridge	281.04*	14.7831	2209.65	0	1.509453	2208.151	-1.498286	1.54137	2207.663	-1.986514	1.734594
Jemo	@Arsema Bridge	266.30*	14.7831	2209.603	0	1.508719	2208.122	-1.481044	1.513771	2207.681	-1.922551	1.667117
Jemo	@Arsema Bridge	251.57*	14.7831	2209.556	0	1.508274	2208.084	-1.471907	1.492559	2207.668	-1.887638	1.641795
Jemo	@Arsema Bridge	236.83*	14.7831	2209.508	0	1.507478	2208.043	-1.464871	1.470369	2207.641	-1.866932	1.623448
Jemo	@Arsema Bridge	222.09*	14.7831	2209.461	0	1.505787	2207.994	-1.467063	1.454927	2207.607	-1.853988	1.611712
Jemo	@Arsema Bridge	207.35*	14.7831	2209.414	0	1.50351	2207.94	-1.474185	1.442467	2207.573	-1.841104	1.598389
Jemo	@Arsema Bridge	192.61*	14.7831	2209.366	0	1.501102	2207.883	-1.483154	1.430819	2207.532	-1.833602	1.59071
Jemo	@Arsema Bridge	177.87*	14.7831	2209.319	0	1.497852	2207.825	-1.493769	1.418379	2207.493	-1.8256	1.580916
Jemo	@Arsema Bridge	163.13*	14.7831	2209.271	0	1.493684	2207.759	-1.511841	1.412895	2207.451	-1.820109	1.572779
Jemo	@Arsema Bridge	148.39*	14.7831	2209.224	0	1.487612	2207.694	-1.530594	1.406869	2207.405	-1.818975	1.569374
Jemo	@Arsema Bridge	133.65*	14.7831	2209.177	0	1.480628	2207.63	-1.54666	1.395224	2207.361	-1.816177	1.562127
Jemo	@Arsema Bridge	118.91*	14.7831	2209.129	0	1.472345	2207.554	-1.57505	1.390273	2207.316	-1.813268	1.553677
Jemo	@Arsema Bridge	104.17*	14.7831	2209.082	0	1.461043	2207.482	-1.599721	1.394715	2207.266	-1.816263	1.551077
Jemo	@Arsema Bridge	89.43*	14.7831	2209.034	0	1.448014	2207.406	-1.628474	1.384492	2207.216	-1.817767	1.54678
Jemo	@Arsema Bridge	74.70*	14.7831	2208.987	0	1.430627	2207.333	-1.653556	1.390061	2207.167	-1.819812	1.541106
Jemo	@Arsema Bridge	59.96*	14.7831	2208.94	0	1.409225	2207.259	-1.680496	1.388516	2207.141	-1.798656	1.505769
Jemo	@Arsema Bridge	45.22*	14.7831	2208.892	0	1.380691	2207.182	-1.709598	1.388973	2207.13	-1.762142	1.434022
Jemo	@Arsema Bridge	30.48*	14.7831	2208.845	0	1.340696	2207.101	-1.74363	1.395605	2207.101	-1.744021	1.392904
Jemo	@Arsema Bridge	15.74*	14.7831	2208.797	0	1.279972	2207.098	-1.698898	0.9734208	2207.116	-1.681255	0.9750781
Jemo	@Arsema Bridge	1	0	2208.75	0	0.9624332	2206.908	-1.841876	0.9758446	2206.802	-1.947643	0.9759675

Table 12 Sediment deposition and scouring both at the US and DS side of the bridge at Q_2

On both cases the HEC-RAS model results shown that river bed erosion exists on the place where the Bridge is located. On the other hand is no significant bed change were observed on the over bank side of the river as shown in the below Figure.

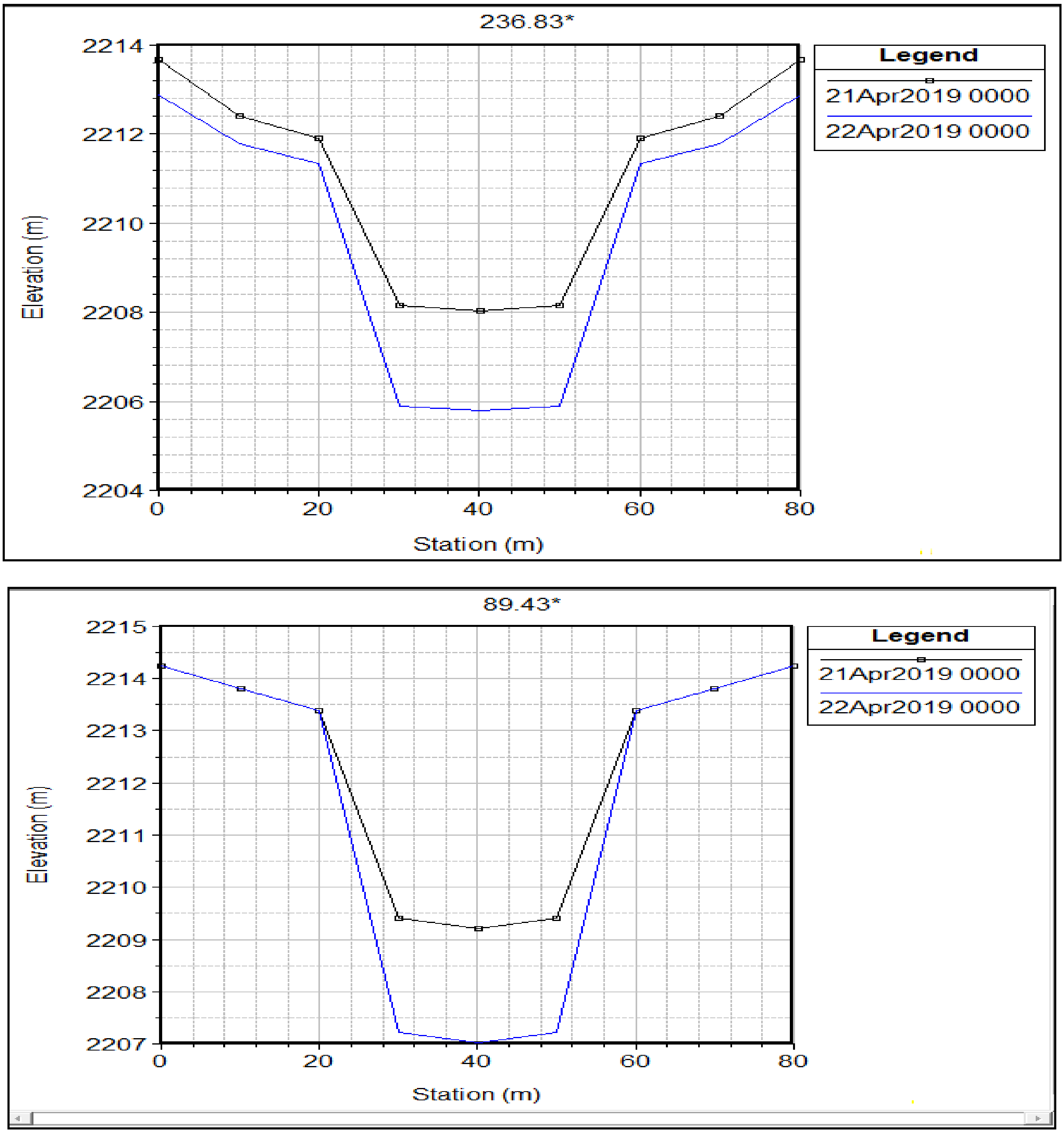
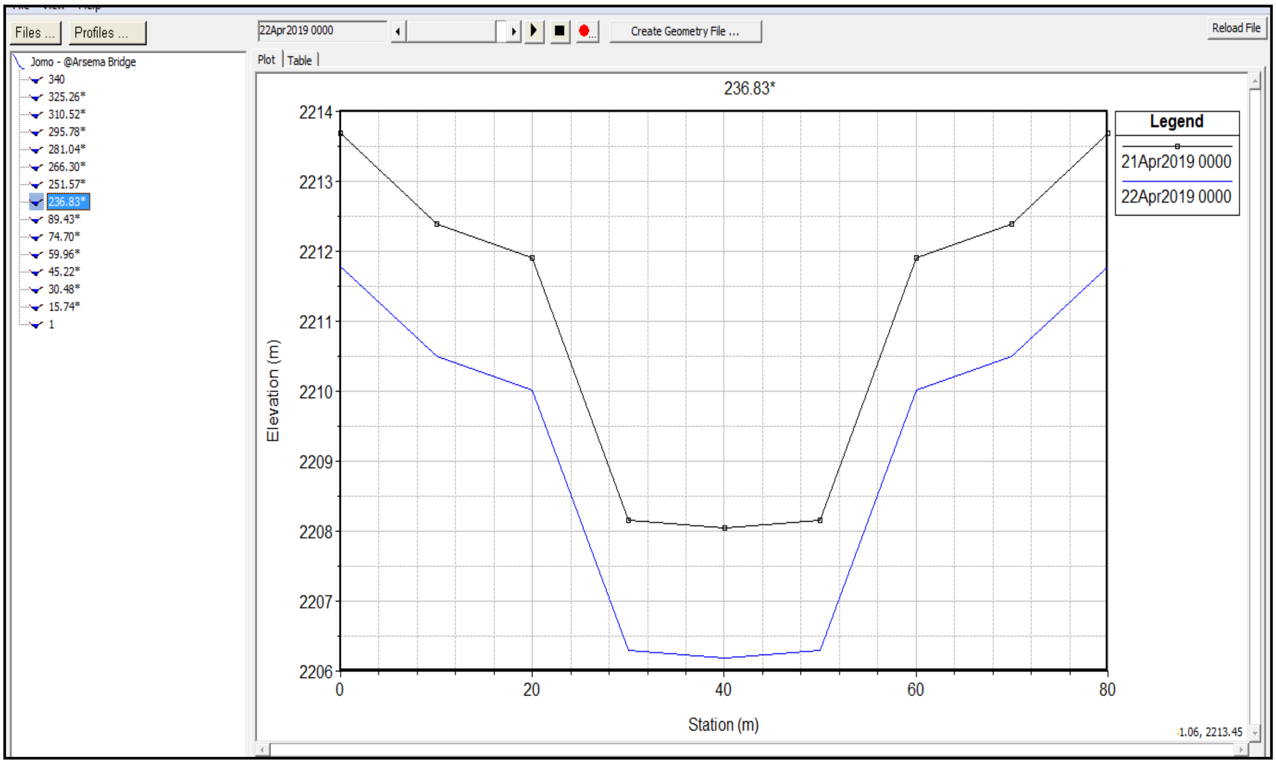


Figure 41 HEC_RAS sediment profile at 89.43 (DS) and 236.83 (US) of the Bridge simulation at (Q_2)



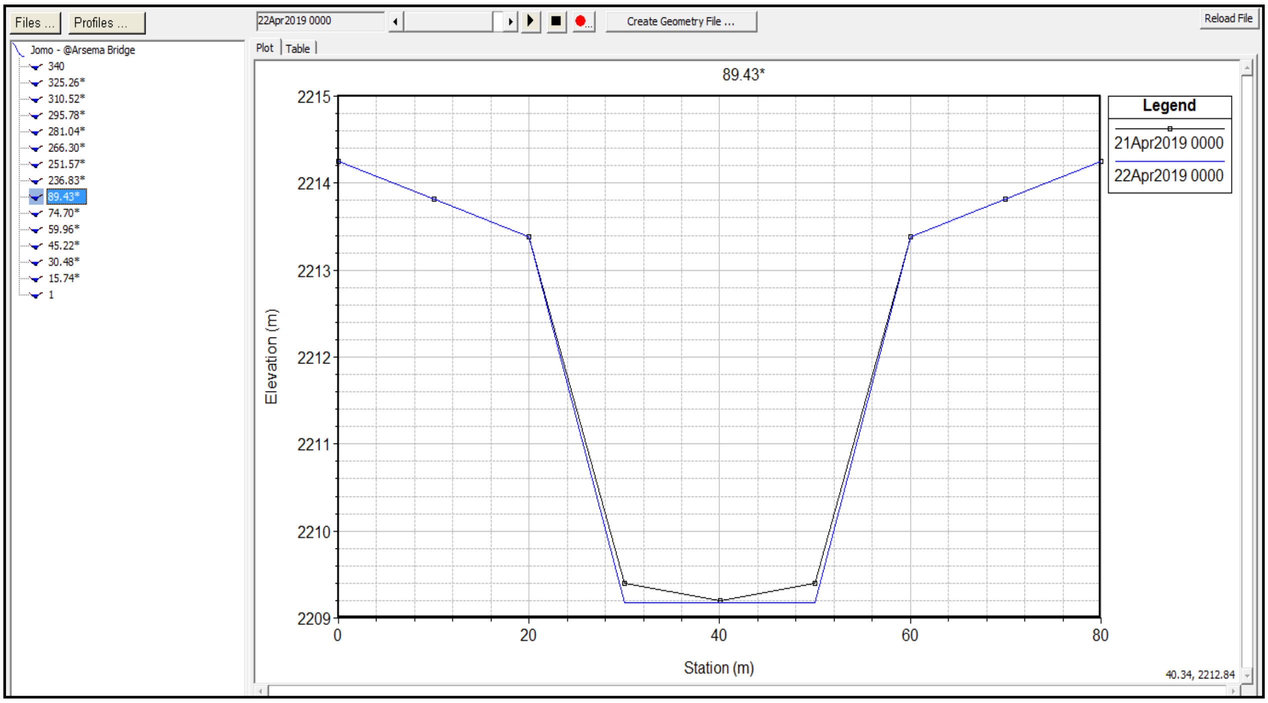


Figure 42 HEC_RAS sediment profile at 89.43* DS and 236.83* US of the Bridge simulation at (Q50)

The possible explanation for the bed and bank scour of the river channel would be one the cause to increase in velocity of the flow and consequently the shear force due to the reduction in the opening size of the bridge. From field observation bed and bank erosion is not a problem only at the bridge location.

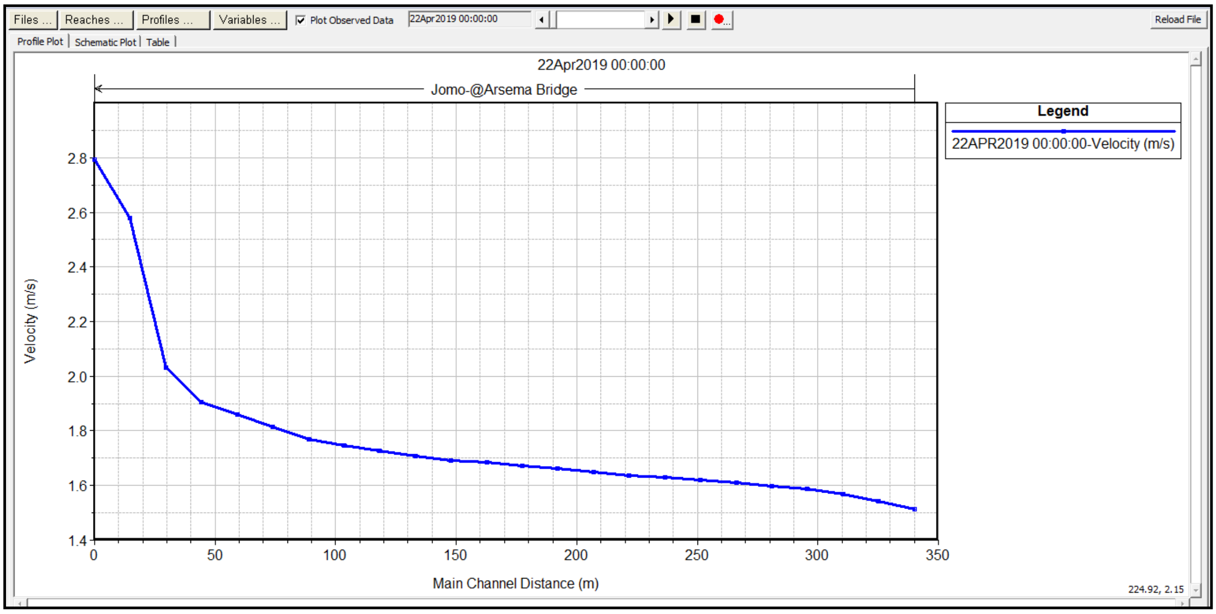


Figure 43 HEC_RAS Velocity output of the flow along the river at the end of simulation (Q_{50}).

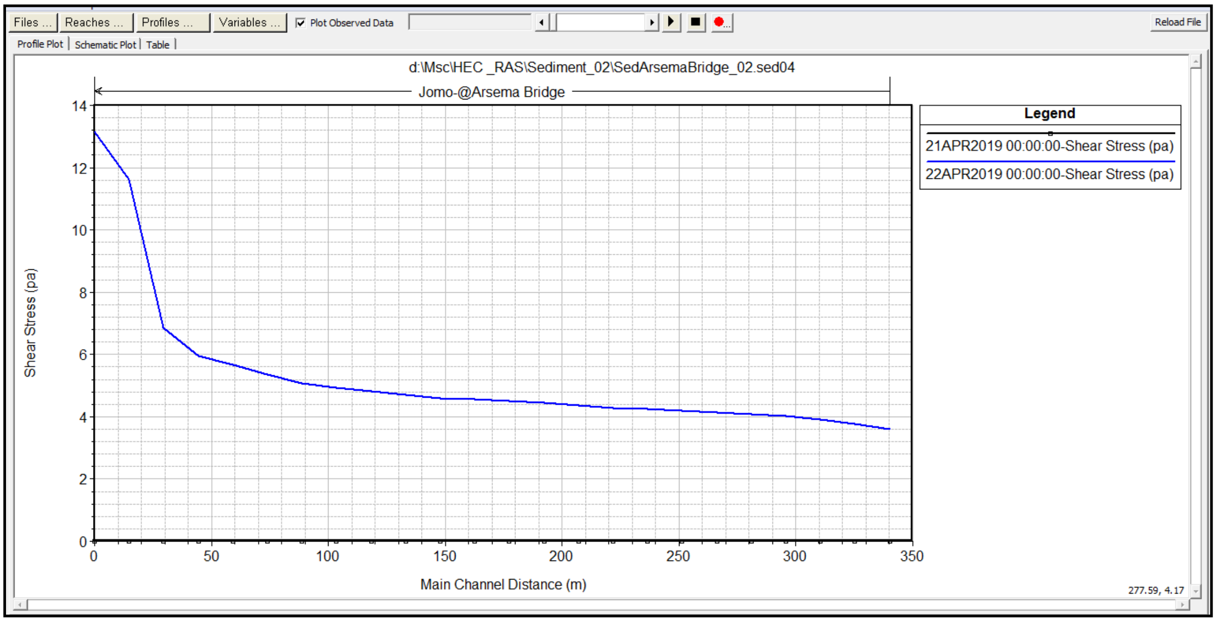


Figure 44 HEC_RAS shear force output of the flow along the river at the end of simulation (Q_{50})

Based on field visit and figure materials taken at the site, sedimentation occurs both at upstream and downstream side of the Bridge.

The bridge was located on the river banks making about 30^{O} skew angle with the river flow direction and this facilitates trash and sediment accumulation on the upstream side of the abutment of the Bridge.

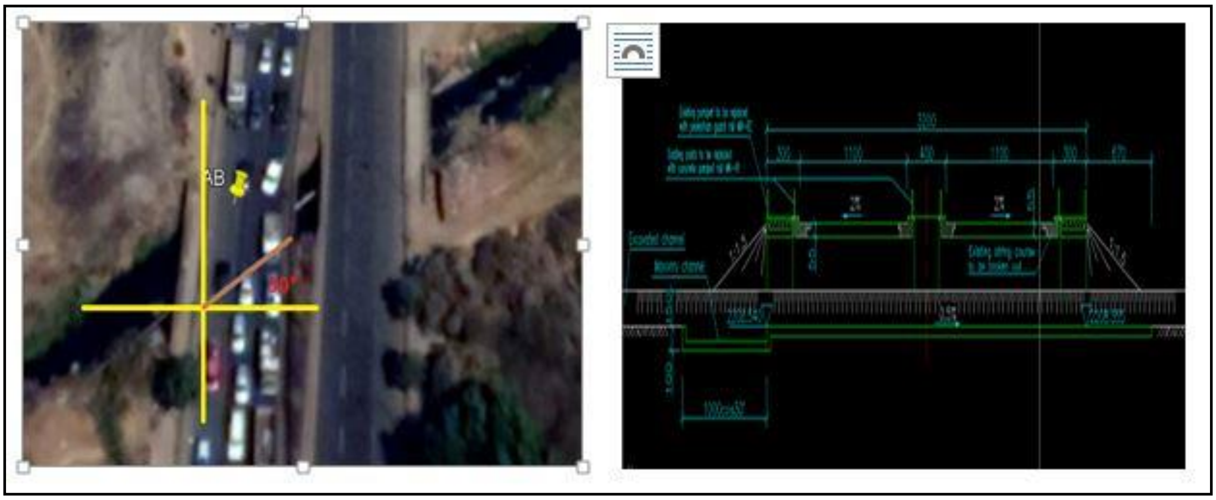


Figure 45 Skew angle and Bridge alignment with respect to the flow direction.

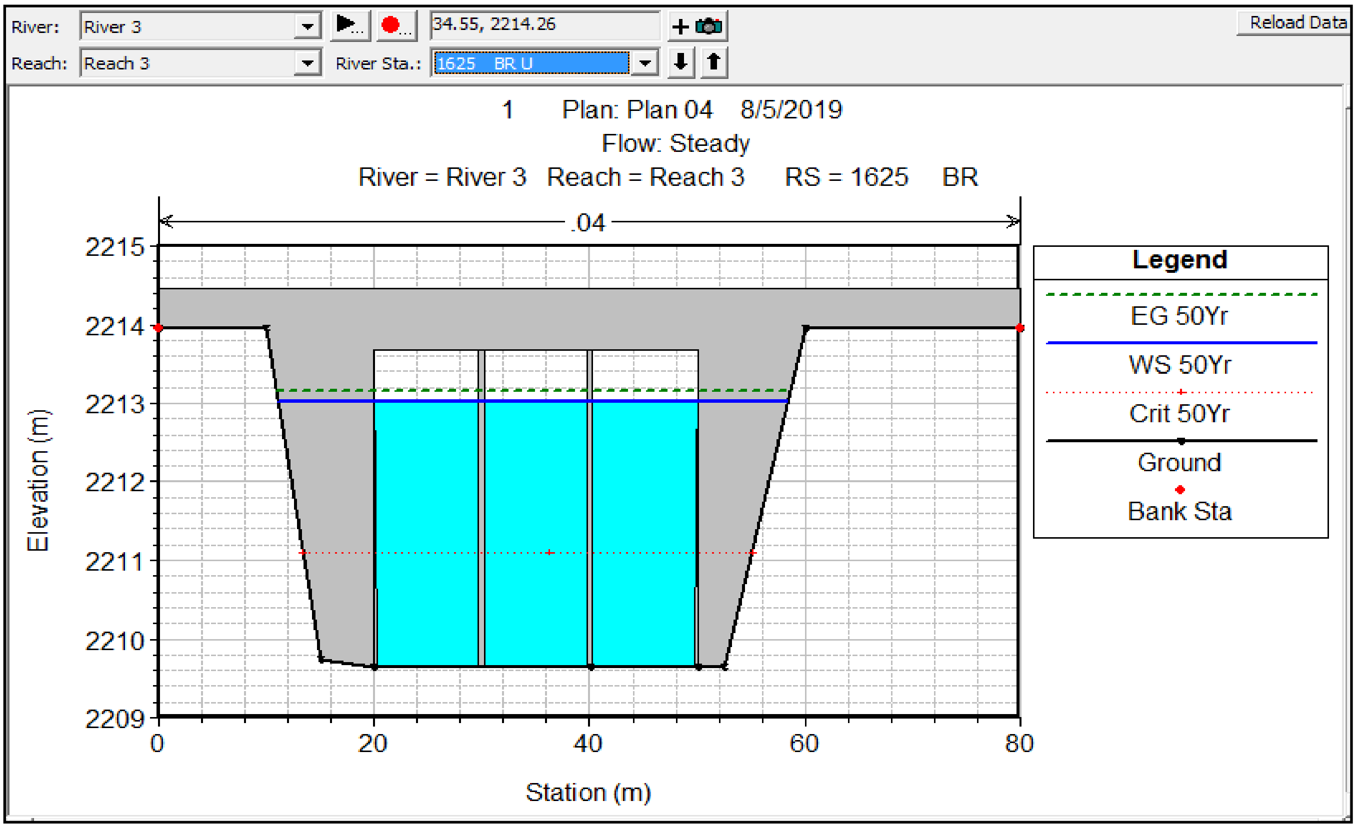
As it is clearly shown that the bridge alignment against the flow of the water at skews angle of 30^0 and further more river create meandering along the bridge cross-section where it meet the road. In order to compensate the head lose the river needs to rise up and create back water effects and flood inundation just upstream of the Bridge. This creates excess time for sediment particles to settle down on the bed and overbank of the river. When this process repeated through time, sedimentation and trash accumulation will be develop on the upstream side of Bridge, this is what actually happened on Arsema Bridge.

4.6. Mitigation Measure

4.6.1. Redesign of the Bridge and Relocate Sewer Line

Redesign of bridge shall be taken as one of the remedial measure by increasing the opening width of the existing bridge from 10.1 m to 30 m with two piers which has 10 meter length of center to center interval of between them, then HEC _RAS was used to analysis the hydraulic computation of the Water Surface, Energy Grade Line, low chord and high chord Elevation as it's result is attached in the appendix E of this thesis work, W.S.E. of bridge on the US and DS are **2213.02 m** and **2212.96 m** amsl respectively.

Whereas high chord of the bridge is **2214.45 m** amsl at both US and DS of the bridge, which is a bit higher than the existing road surface on the bridge (high chord of bridge) i.e. **2213.684 m amsl** which has a variation of **0.766 m** which leads us to refill road surface to accommodate this in both left and right side of the bridge a length of 150m, detail profile attached on Appendix F of this thesis work.



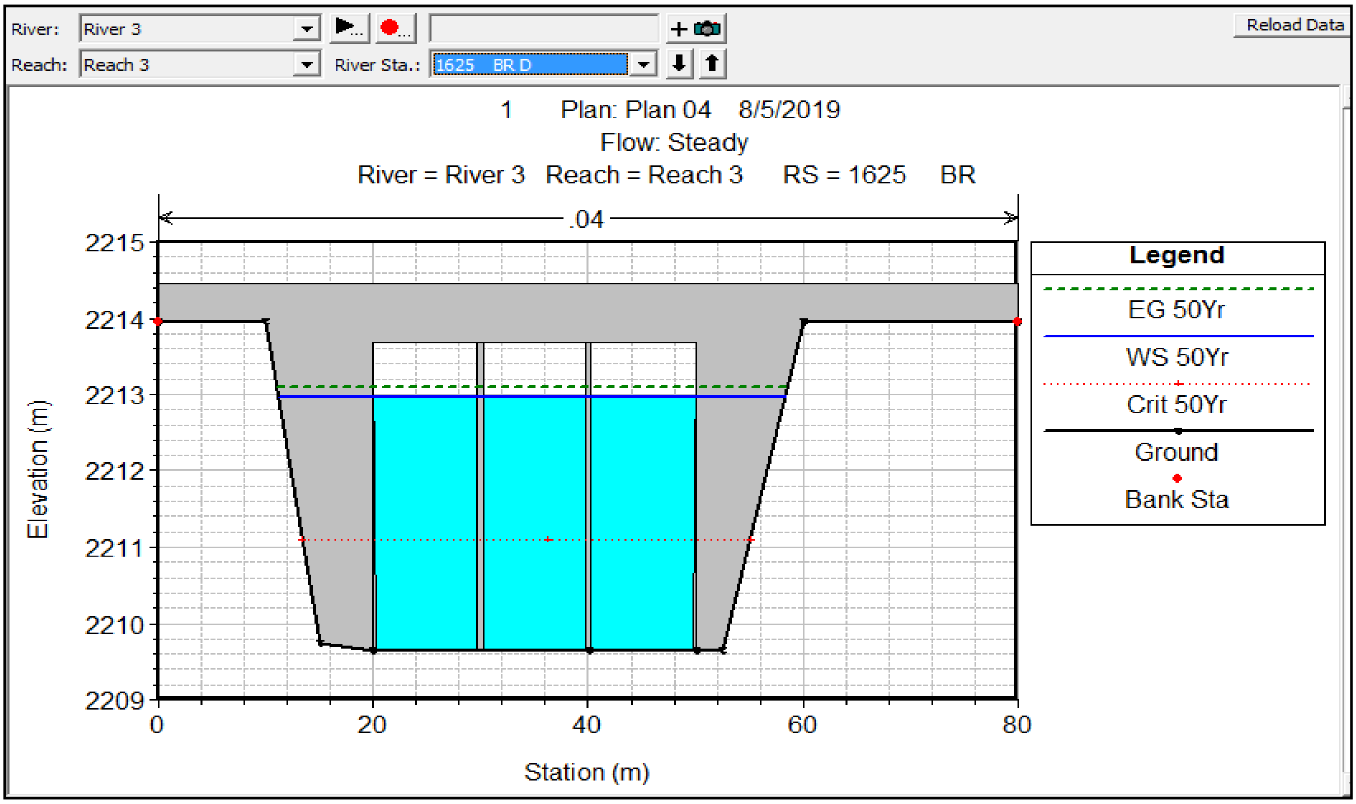


Figure 46 HEC-RAS water surface profile of newly recommended Bridge at Q_{50} of US and DS.

Relocation of Sewer main line which passes through to embankment of the road is one key remedial measure has to be considered.

4.6.2. Design of Trash Trap Screen

Inlet of the bridge blocked by trash due to existence of land fill site of Addis Ababa City, installation of trash screen on the entrance face of the bridge is the main remedial measure. So based on the design guide line of TSSG, 2009 which was prepared and published by Environmental agency of UK, then design of trash trap screen determined as per the below main assumption and principle.

4.6.3. Design of trash trap screen area

If the culvert has detail debris and trash recorded data, the design of screen area is determine by using the evidence based method. The result of the area shall be between three and thirty times of minimum cross-section area of the culvert being protected.

For my case, I have the bridge however I have no enough or detail data source of trash load and size, so I adopted three times area of the bridge cross section to be considered as trash trap screen area.

$$A_{MB} = W * H$$

A_{MB} = minimum bridge cross-section area, W= bridge opening width, H= height of bridge opening.

H= h_{WSE} + Free Board. h_{WSE}= design flood water surface level.

$$A_{MB} = \lfloor 30 - 2[40.40 - 39.60 + 29.60 - 30.40] \rfloor \text{ m} * 4.03 \text{ m}$$

$$= 28.4*4.03 \text{ m}^2$$

$$= 114.452 \text{ m}^2$$

Trash Trap Screen Area = 3* A_{MB} =343.4 m^2. It's detail drawing attached in the appendix G.

4.6.4. Selection of optimum bar Spacing

It is essential to select the most appropriate bar spacing to maximize the effectiveness of the screen to meet its design objective. The spacing should prevent the passage of material of the type and size likely to pose a significant risk at the site, trash screens placed upstream of culverts and inverted siphons should have a minimum clear spacing of 150 mm between bars in general however project located in urban location a clear space of 300 mm between bars appropriate where there is a need to exclude oil drums or sofas, but allow smaller debris to pass it.

So 300 mm bar spacing was taken for my design, the detail layout and plan is attached on appendix G, then I highly recommend redesign of the bridge with trash screen at entrance of flood to the bridge.

4.6.5. Develop Flood Hazard Map

Upstream of the bridge land use is residential area, and highly affected by flood, so the **flood hazard map** is one of the remedial measure to protect life and the property of people then flood mapping of the area analyzed and mapped by **HEC-RAS Mapping** as it's result attached in the below figure, each residence house located under the swapping area of the flood, has to consider at least two meter high flood protection work**.**

Flood Mapping

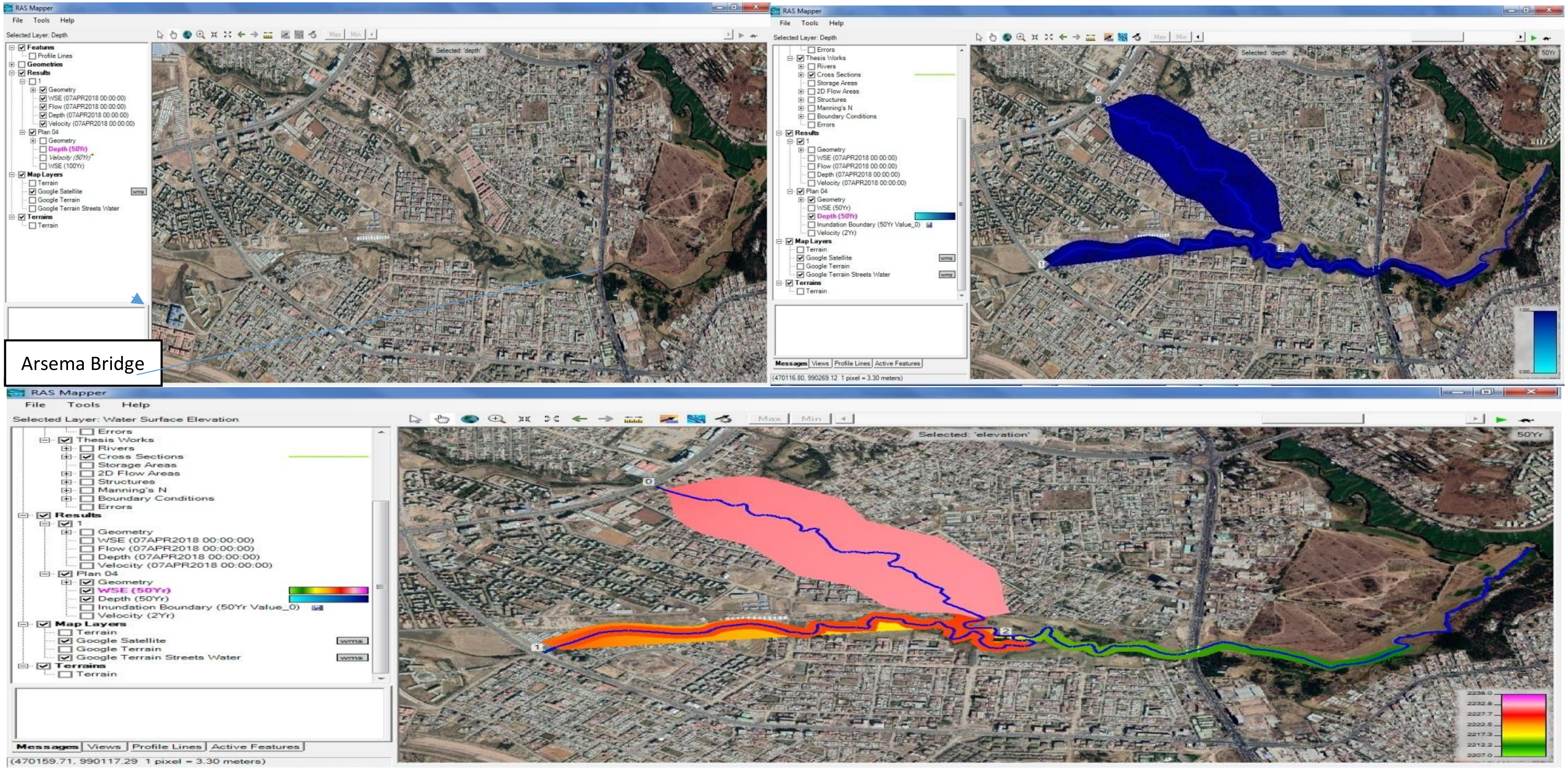


Figure 47 Flood Hazard Map of upstream side of the bridge

5. Conclusion and Recommendation

5.1.Conclusion

This thesis work tries to address the basic hydrologic and hydraulic aspects of bridge design in order to evaluate the current hydraulic performance of the bridge. As described in detail in analysis part, Sediment aggradations, trash accumulation and under size of the bridge opening 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 over flooding and bank over topping which create pressure on the residence building around the bridge especially on the upstream side.

From surveying detail both from field and as built drawing design document, and analysis result clearly showed that the bridge location is totally on the flood plain which is submerged.

Therefore during the hydrological analysis phase this should have been considered.

Even if I couldn't find the hydrological report of bridge as urban river crossing structure, the hydrologic analysis result of Q_{50} and Q_{100} year return period are **154.5 m^3/sec** and **174.9 m^3/sec** respectively, based on the actual existing structure, as built drawing and hydrologic analysis result, There is a significant difference to cause such a problem, then I can conclude that error in the hydrologic computation is the direct problem.

The hydraulic modeling results of Q_{50}, **water surface elevation** is **2214.36** m and **2214.06** m amsl at the river station 340 and 1 respectively before bridge inserted into the model, and **2214.57** m and **2214.06** m amsl at the river station 340 and 1 respectively just after bridge is inserted in to the model. From the results I can conclude that the bridge play, Its role for over flooding of the surrounding resident due to its back water effect, to reduce suffering of human, **flood hazards map** is developed with help of HEC-RAS Mapping.

Based on the above results and field observation that sewer line which pass through bridge is one of cause trash accumulation on the entrance face of the bridge then it aggravate silt deposition at inlet of the bridge then it leads reduction of the bridge opening for proper hydraulic performance, in addition to its under sizing problem.

The Bridge skewness, layout, agricultural activity along the river reach, the soil type, land use and land cover is play important role in the sediment deposition.

5.2. Recommendation

Until now the problem and their end results were discussed and addressed in detail. From the analysis result the major causes for the over topping and flooding the surrounding resident area of the bridge were under size of bridge opening, trash accumulation and river bed elevation change in the vicinity of the bridge.

Therefore the main concern should be how to reduce trash and sediment accumulation on the inlet of the bridge channel as well as how to protect the life of people and the property from flooding.

As recommendation the following mitigation measure should be taken seriously to prolong the life of the road and to reduce the negative impact of the economic.

5.2.1. Redesign the Bridge

Redesign and construct a new bridge with **Trash Trap Screen** is as major recommendation. Even existing bridge needs installation of **Trash Trap Screen** on entrance of bridge until to replace with new redesign bridge.

5.2.2. Flood Hazard Map.

The result of RAS- Mapping analysis show that many number of residence house would be flooded even by Q_2 year return period. So to minimize the flooding of their residence it is highly advisable to protect their life and property by constructing 2.0m high flood protection or guide wall depending of their OGL against flood hazard map.

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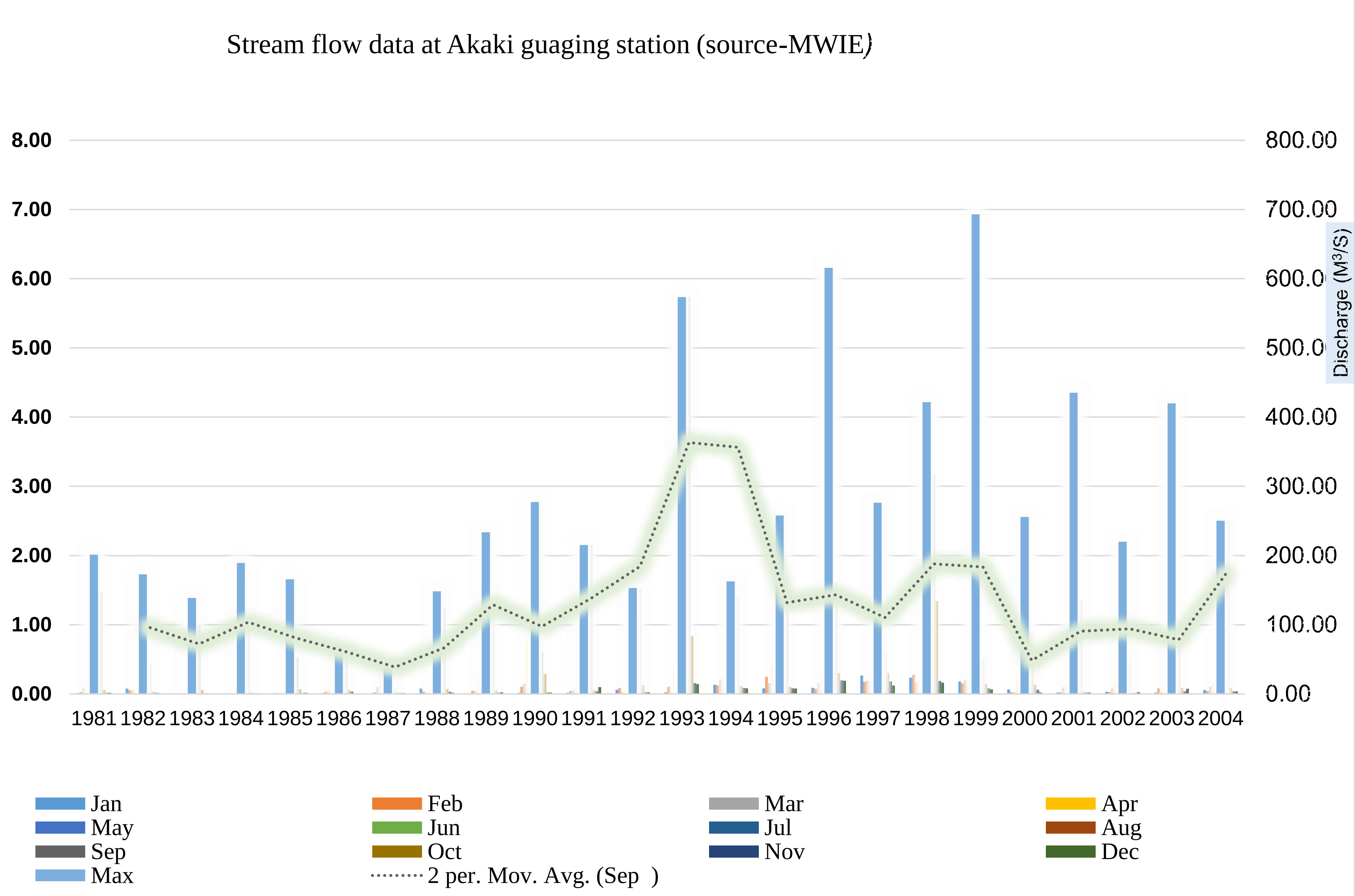
Appendix

Appendix A: Akaki River Flow Data

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Max
1981	1.13	2.49	9.03	6.46	1.46	2.93	176.17	201.31	146.51	5.86	1.70	1.58	201.31
1982	7.92	5.29	4.92	7.28	9.03	4.05	76.36	172.77	44.85	3.76	1.74	1.11	172.77
1983	0.84	1.02	0.76	0.68	14.20	3.91	38.07	138.72	99.41	5.96	0.93	0.76	138.72
1984	0.84	1.02	0.76	0.68	3.91	26.98	189.38	162.94	107.89	1.86	1.02	0.84	189.38
1985	1.02	1.11	0.68	2.10	27.31	2.35	101.09	165.65	53.19	6.71	1.41	1.63	165.65
1986	1.41	2.88	5.06	7.91	5.96	11.47	67.82	59.42	68.78	5.59	3.16	0.68	68.78
1987	0.87	2.35	9.85	23.41	36.55	7.91	24.36	35.06	8.75	2.10	1.21	1.21	36.55
1988	7.91	3.16	1.63	7.10	2.74	3.02	20.64	148.35	124.28	7.08	3.08	1.67	148.35
1989	1.10	4.62	4.62	22.76	1.95	10.91	80.49	233.77	133.43	5.00	1.68	2.39	233.77
1990	1.55	10.40	15.00	76.39	3.88	4.62	94.89	277.22	61.21	29.01	1.95	1.95	277.22
1991	1.68	4.43	5.60	9.90	4.43	18.89	102.68	124.28	215.22	5.80	3.53	9.90	215.22
1992	6.01	8.69	2.70	3.19	3.88	5.00	52.06	112.42	153.07	12.76	2.39	2.24	153.07
1993	2.24	10.15	1.68	34.23	21.11	37.35	189.48	403.47	573.57	83.73	15.48	14.11	573.57
1994	13.07	12.56	20.69	24.88	32.86	31.58	121.28	162.58	139.12	11.11	8.92	8.12	162.58
1995	8.12	24.88	15.76	40.17	40.17	33.29	53.84	257.98	124.92	10.20	7.93	7.73	257.98
1996	8.92	7.36	15.76	23.43	31.16	130.49	343.78	615.76	161.52	30.34	19.71	19.07	615.76
1997	26.38	17.52	19.38	22.04	25.25	36.41	83.73	276.32	59.10	30.34	17.82	12.07	276.32
1998	23.43	27.53	16.63	19.07	71.81	45.16	277.76	421.52	316.73	134.29	18.75	16.34	421.52
1999	18.13	15.20	20.36	18.13	16.92	43.12	138.14	693.10	50.43	14.63	7.94	6.76	693.10
2000	6.40	2.29	2.77	17.97	20.20	9.71	51.14	255.78	46.29	13.42	6.04	2.03	255.78
2001	1.95	2.38	9.48	14.94	20.98	31.98	383.74	435.34	135.31	3.09	2.12	1.87	435.34
2002	2.99	2.57	8.15	15.58	4.20	10.44	219.87	105.00	52.59	2.12	1.43	2.57	219.87
2003	2.04	8.20	2.19	5.44	3.27	8.02	110.95	420.06	104.22	8.76	3.69	7.48	420.06
2004	5.59	4.39	10.81	15.24	4.39	10.38	157.18	169.75	250.47	8.76	3.80	3.69	250.47


Stream flow data at Akaki guaging station (source-MWIE)
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
800.00
700.00
600.00
500.00
400.00
300.00
200.00
100.00
0.00
Discharge (M3/S)
1981 1982 1983 1984 1985 1986 1987 1988 1989 1990 1991 1992 1993 1994 1995 1996 1997 1998 1999 2000 2001 2002 2003 2004
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Max
2 per. Mov. Avg. (Sep)

Appendix B: Jemo River Profile Surveying Data

River Station	Northern (Y)	Eastern (X)	Elevation (Z)
340	989552.937	470341.543	2209.841
320	989540.842	470356.425	2209.669
300	989528.451	470369.848	2209.450
280	989514.331	470382.779	2209.347
260	989503.220	470398.131	2209.270
240	989495.842	470422.179	2209.501
220	989496.408	470442.216	2209.383
200	989504.524	470462.483	2209.644
180	989511.995	470471.842	2209.648
160	989518.422	470487.577	2213.656
140	989528.335	470507.347	2209.534
120	989537.300	470531.008	2209.091
100	989550.946	470547.003	2209.403
80	989562.478	470562.604	2209.390
60	989569.549	470580.079	2209.119
40	989566.444	470599.659	2209.003
20	989562.750	470611.387	2208.800
10	989554.278	470627.012	2208.665
5	989545.517	470643.582	2208.360
1	989531.089	470660.349	2208.849

Appendix C: Soil Sample Gradation Data

	Sieve Weight (gm)	**Sample**	**Σ% Retain Sample**	**%Pass Sample**
9.5	446	4.6	0.66	**99.34**
4.75	567.4	12.9	4.5	**95.5**
2	378	20.1	12.12	**87.88**
1	526.1	37.5	15.1	**84.9**
0.5	483.7	139	42.3	**57.7**
0.25	461.7	143.7	73.3	**26.7**
0.15	271.7	73.5	89.25	**10.75**
0.075	273.2	18.9	98.23	**1.77**
pan	396.8	12.4	100	**0**

Appendix D: SCS Curve Numbers (CN) (Source: ERADDM, 2002.)

Table 5-8 Runoff Curve Numbers- Urban Areas[1]

Cover description	Curve numbers for hydrologic soil groups				
Cover type and Hydrologic condition	Average % impervious area[2]	A	B	C	D
Open space (lawns, parks, cemeteries, etc.)[3]					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50 % to 75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Desert urban areas:					
Natural desert cover		63	77	85	88
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average plot size:					
0.05 hectare or less	65	77	85	90	92
0.1 hectare	38	61	75	83	87
0.135 hectare	30	57	72	81	86
0.2 hectare	25	54	70	80	85
0.4 hectare	20	51	68	79	84
0.8 hectare	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

[1] Average runoff condition, and $I_a = 0.2S$

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

Table 5-9 Cultivated Agricultural Land[1]

Cover description			Curve numbers for Hydrologic soil group			
Cover Type	Treatment[2]	Hydrologic condition[3]	A	B	C	D
Fallow	Bare soil	-	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row Crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C & T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
	Small grain SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
	Close-seeded SR or broadcast	Poor	66	77	85	89
		Good	58	72	81	85
	Legumes or C Rotation	Poor	64	75	83	85
		Good	55	69	78	83
	Meadow C&T	Poor	63	73	80	83
		Good	51	67	76	80

[1] Average runoff condition, and $I_a = 0.2S$.
[2] Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
[3] Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or closed-seeded legumes in rotations, (d) percent of residue cover on the land surface (good > 20%), and (e) degree of roughness.

Poor : Factors impair infiltration and tend to increase runoff.
Good : Factors encourage average and better than average infiltration and tend to decrease runoff.

Table 5-10 Other Agricultural Lands[1]

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range-continuous forage for grazing[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow-continuous grass, protected from grazing	--	35	59	72	79
Brush-weed-grass mixture with brush the major element[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[4]	48	65	73
Woods-grass combination[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[4]	55	70	77
Farms—buildings, lanes, driveways, and surrounding lots	--	59	74	82	86

[1] Average runoff condition, and $I_a = 0.2S$

[2] Poor: < 50% ground cover or heavily grazed with no mulch
Fair: 50 to 75% ground cover and not heavily grazed
Good: > 75% ground cover and lightly or only occasionally grazed

[3] Poor: < 50% ground cover
Fair: 50 to 75% ground cover
Good: > 75% ground cover

[4] Actual curve number is less than 30; use CN = 30 for runoff computations.

[5] CNs shown were computed for areas with 50% grass (pasture) cover. Other combinations of conditions may be computed from CNs for woods and pasture.

[6] Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair : Woods grazed but not burned, and some forest litter covers the soil.
Good : Woods protected from grazing, litter and brush adequately cover soil.

Appendix E: Output of HEC- RAS

Bridge Output

File Type Options Help

River: River 3 Profile: 50Yr

Reach Reach 3 RS: 1625 Plan: Plan 04

Plan: Plan 04 River 3 Reach 3 RS: 1625 Profile: 50Yr

E.G. US. (m)	2213.75	Element	Inside BR US	Inside BR DS
W.S. US. (m)	2213.66	E.G. Elev (m)	2213.15	2213.09
Q Total (m3/s)	154.50	W.S. Elev (m)	2213.02	2212.96
Q Bridge (m3/s)	154.50	Crit W.S. (m)	2211.10	2211.10
Q Weir (m3/s)		Max Chl Dpth (m)	3.37	3.31
Weir Sta Lft (m)		Vel Total (m/s)	1.60	1.64
Weir Sta Rgt (m)		Flow Area (m2)	96.28	94.42
Weir Submerg		Froude # Chl	0.28	0.29
Weir Max Depth (m)		Specif Force (m3)	187.27	181.61
Min El Weir Flow (m)	2214.45	Hydr Depth (m)	3.35	3.29
Min El Prs (m)	2213.68	W.P. Total (m)	48.63	48.24
Delta EG (m)	0.27	Conv. Total (m3/s)	3794.7	3693.3
Delta WS (m)	0.27	Top Width (m)	28.72	28.71
BR Open Area (m2)	115.39	Frctn Loss (m)		
BR Open Vel (m/s)	1.64	C & E Loss (m)		
BR Sluice Coef	0.27	Shear Total (N/m2)	32.18	33.59
BR Sel Method	Momentum	Power Total (N/m s)	51.64	54.96

Bridge Output

File Type Options Help

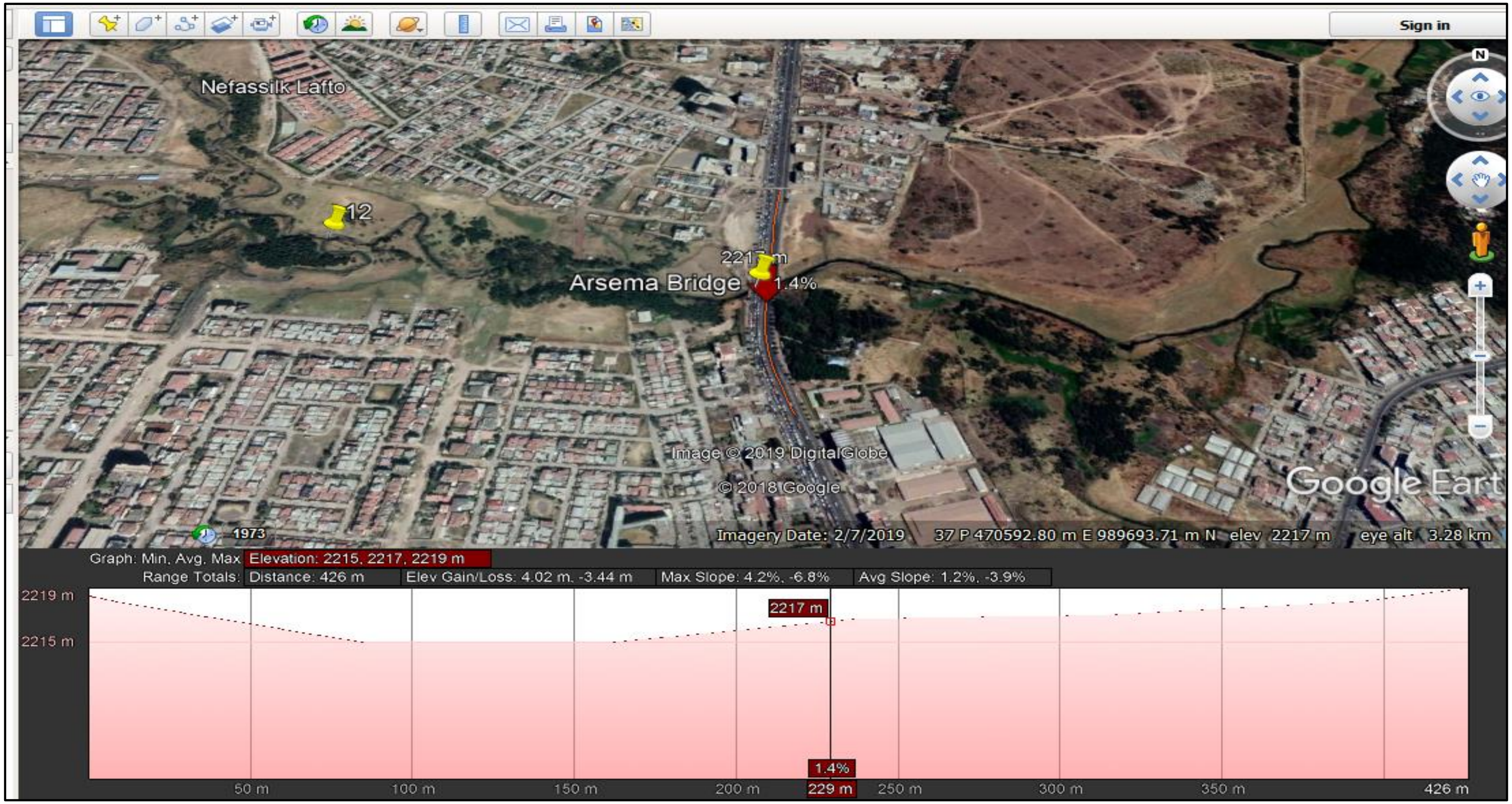
River: River 3 Profile: 100Yr

Reach Reach 3 RS: 1625 Plan: Plan 04

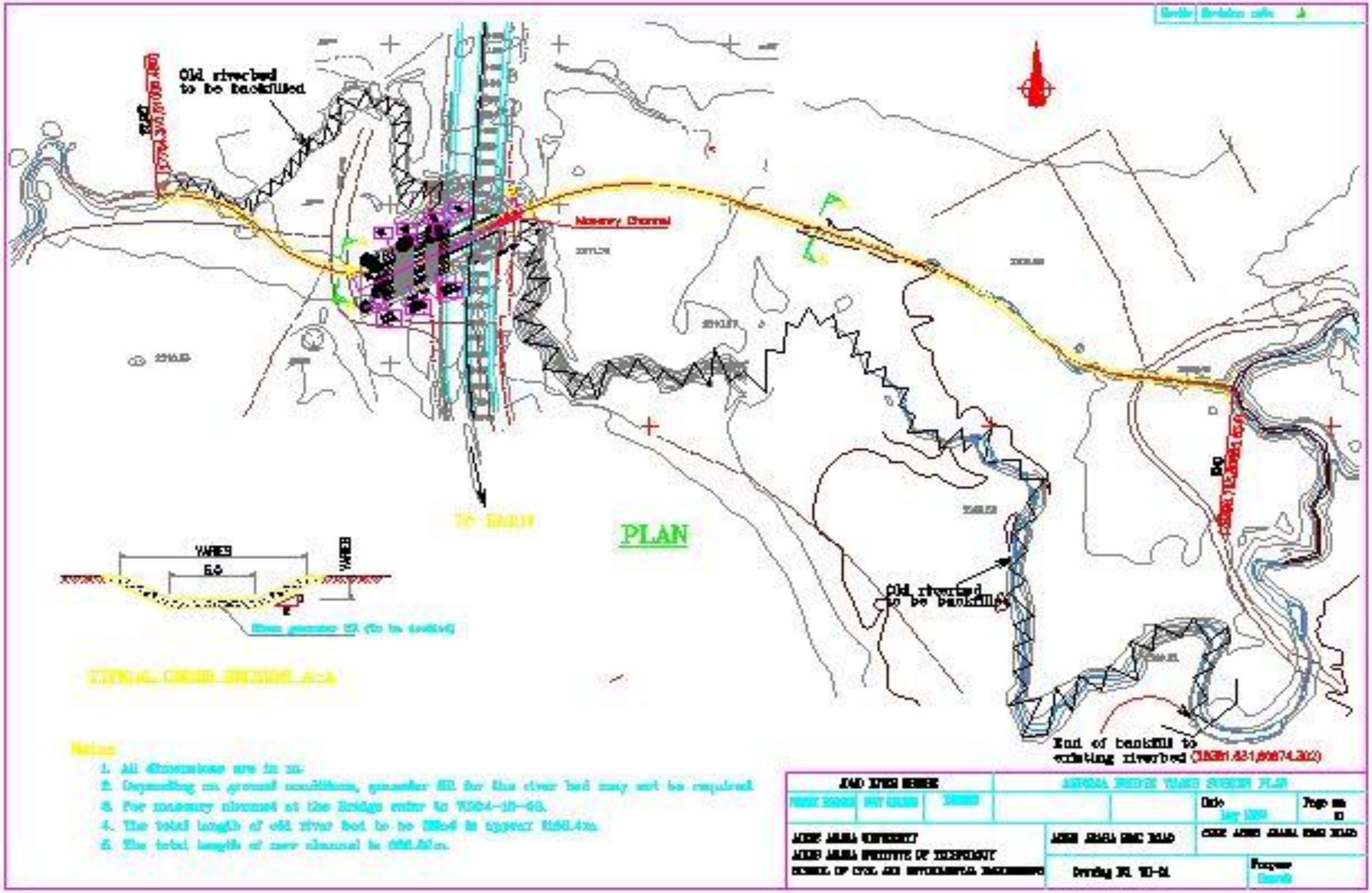
Plan: Plan 04 River 3 Reach 3 RS: 1625 Profile: 100Yr

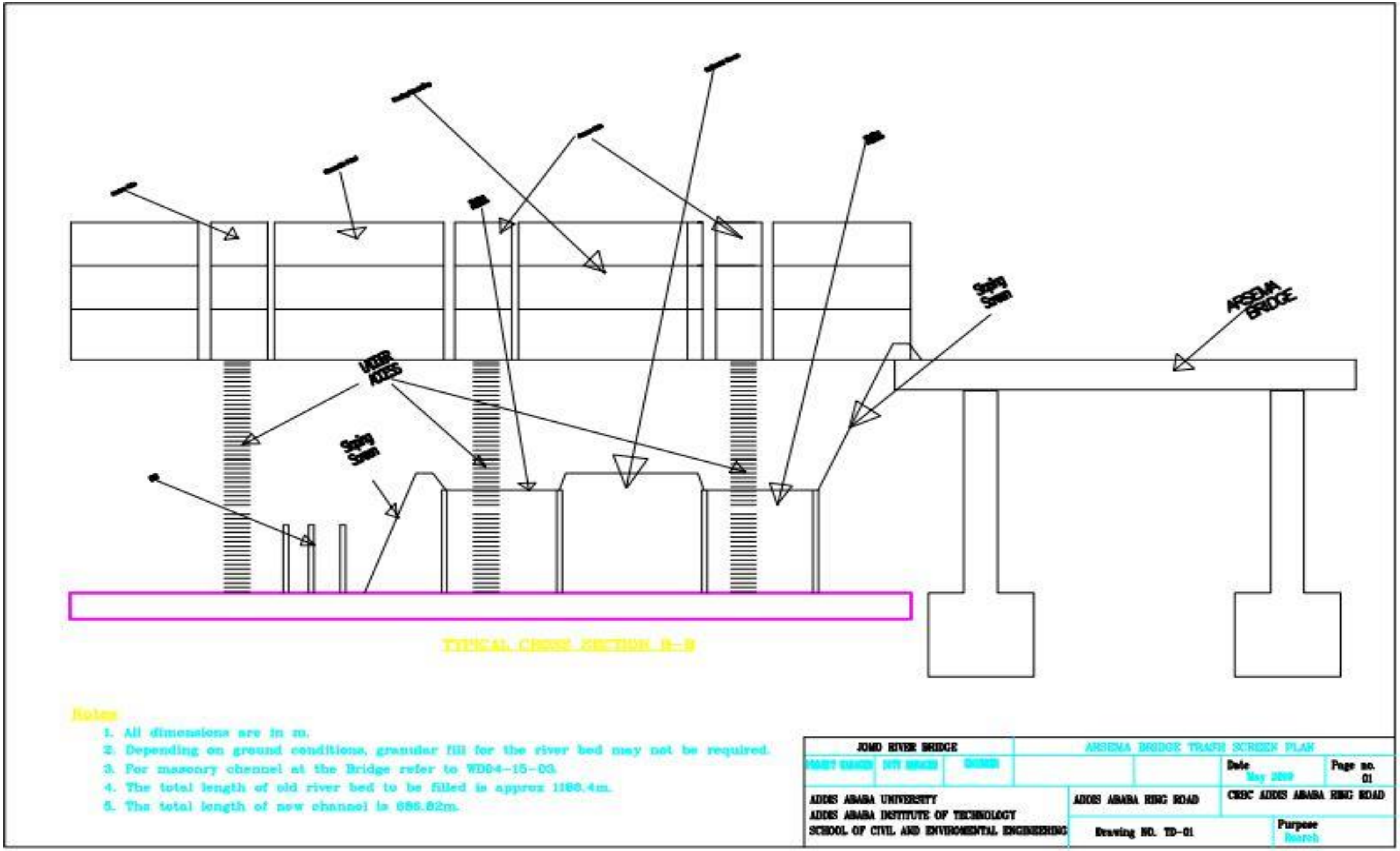
E.G. US. (m)	2213.92	Element	Inside BR US	Inside BR DS
W.S. US. (m)	2213.82	E.G. Elev (m)	2213.34	2213.28
Q Total (m3/s)	174.90	W.S. Elev (m)	2213.19	2213.12
Q Bridge (m3/s)	174.90	Crit W.S. (m)	2211.22	2211.22
Q Weir (m3/s)		Max Chl Dpth (m)	3.54	3.47
Weir Sta Lft (m)		Vel Total (m/s)	1.73	1.76
Weir Sta Rgt (m)		Flow Area (m2)	101.23	99.13
Weir Submerg		Froude # Chl	0.29	0.30
Weir Max Depth (m)		Specif Force (m3)	209.82	203.17
Min El Weir Flow (m)	2214.45	Hydr Depth (m)	3.52	3.45
Min El Prs (m)	2213.68	W.P. Total (m)	49.67	49.23
Delta EG (m)	0.29	Conv. Total (m3/s)	4068.0	3951.9
Delta WS (m)	0.29	Top Width (m)	28.73	28.73
BR Open Area (m2)	115.39	Frctn Loss (m)		
BR Open Vel (m/s)	1.76	C & E Loss (m)		
BR Sluice Coef	0.27	Shear Total (N/m2)	36.94	38.68
BR Sel Method	Momentum	Power Total (N/m s)	63.83	68.24

Appendix F : Road Profile



Appendix G: Trash Trap Screen Drawing


Old riverbed
to be backfilled
Masonry Channel
PLAN
VARIES
6.0
Old riverbed
to be backfilled
End of backfill to
existing riverbed


ARSEMA BRIDGE
Notes
1. All dimensions are in m.
2. Depending on ground conditions, granular fill for the river bed may not be required.
3. For masonry channel at the Bridge refer to WD04-15-03.
4. The total length of old river bed to be filled is approx 1188.4m.
5. The total length of new channel is 686.82m.
JOMO RIVER BRIDGE
ARSEMA BRIDGE TRASH SCREEN PLAN
Date
Page no.
01
ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
ADDIS ABABA RING ROAD
CRBC ADDIS ABABA RING ROAD
Drawing NO. TD-01
Purpose