ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

ASSESSMENT OF FAILURE ON DRAINAGE STRUCTURES ALONG THE ETHIOPIAN NATIONAL RAILWAY LINE OF SEBETA-MIESO (CASE STUDY OF AKAKI RIVER CROSSING DRAINAGE STRUCTURE)

A Thesis submitted to the school of graduates of Addis Ababa University in partial fulfillment of the requirements for Master of Science degree in Railway Engineering at Addis Ababa University

By

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November, 2015
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The undersigned have examined the thesis entitled ‘**Assessment of Failure on Drainage Structures along the Ethiopian National Railway line of Sebeta-Mieso (Case study of Akaki River crossing drainage structure)**’ presented by **Yerosan Abera**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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UNDERTAKING

I certify that research work titled "**Assessment of Failure on Drainage Structures along the Ethiopian National Railway line of Sebeta-Mieso (Case study of Akaki River crossing drainage structure)**" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

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Abstract

A railway drainage system gives vital role for effective, efficient operation of rail track. This study worked on an assessment of railway drainage system problem along the Addis Ababa-Mieso railway line, specifically on Akaki rives crossing. It was done to check adequacy of hydraulic structure provided on Akaki River crossing by undertaking hydrologic and hydraulic analysis.

Hydrologic modeling of the Akaki catchment area was developed by HEC-GeoHMS program with the help of Arc-GIS and hydrologic analysis was computed by HEC-HMS program. The catchment land use, soil type, rainfall data, Akaki river stream flow data, etc were used to develop hydrological model. SCS unit hydrograph and flood frequency analysis methods were used to estimate instantaneous peak design discharge for 50 and 100 year return period. Model input parameters were calibrated and verified with observed flow data of the river at Akaki gauging station. Design discharge computed in this case for 50/100 year return period was 743.2m3/s and 834.5m^3/s respectively at bridge location from catchment area of 894km^{2,} where these values were used to check adequacy of the bridge.

Hydraulic models were developed to determine water-surface profiles along the stream reaches for the discharges established in the hydrologic analyses The HEC-RAS step-backwater hydraulic analysis model was used to determine water-surface profiles for the bridge. Cross-sectional elevation data, hydraulic-structure geometries, roughness coefficients along with peak-discharge estimated were used as input for the model. Based on available data's water surface profile for peak discharge was determined by the program.

Finally adequacy of the bridge was evaluated for both peak discharges. Although the site of the bridge was fixed based on geometrical alignment of railway, the output of HEC-RAS analysis confirmed that Akaki Railway Bridge has sufficient size to safely convey both peak design flow during the design period. Thus, it is Akaki Railway Bridge was hydraulically efficient over its design period.

__Key words;__ Hydrologic models- HEC-HMS, HEC-RAS, Akaki New Railway Bridge, Objective function, Calibration, validation, Simulation;

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List of Acronyms

ERC	Ethiopian Railway Corporation
ERADDM	Ethiopian Road Authority Drainage Design Manual
AREMA	American Railway Engineering and Maintenance-of-way Association
AAWSA	Addis Ababa Water and Sewerage Authority
EMSA	Ethiopian Metrological Service Agency
MoWIE	Ministry of Water, Irrigation and Energy
DEM	Digital Elevation Model
HEC	Hydrologic Engineering Centre
SCS	Soil Conservation System
CN	Curve Number
ENS	Nash–Sutcliffe efficiency coefficient
MBE	Mass Balance Error
PEV	percentage Error Volume
IDF	Intesity Duration Frequency
FHWA	Federal Highway Administration
GIS	Geographic Information System
HWM	Highest Water Mark
EGL	Energy Grade Line
WS	Water Surface
AMC	Antecedent Moisture Condition
UH	Unit Hydrograph
MCM	million cubic meters

1. INTRODUCTION

1.1 General

Paying attention to rail mode of transportation was one of the policy of the Federal Democratic Republic of Ethiopia in order to provide effective passenger and freight transport services. Currently about 2000km of standard gauge railway infrastructure is under construction. Among this the Addis Ababa – Mieso-Dire Dawa – Dewale route is actively under construction.

In order to achieve the desired goals and objectives, it is important to deal with the safety, efficiency and effectiveness of each railway components. Adequate drainage system is essential in the design of railways and highway since it affects their serviceability and usable life. The need to effectively remove surface water from all passengers, vehicle and rolling stock environments are essential for the network to operate safely and reliably. For the highway or railway designer, the primary focus of hydrology is the water that moves on the earth's surface and facility that can safely convey quantity of water.

This study is mainly concerned with the assessment of flooding problems associated with the Addis Ababa-Mieso railway drainage systems specifically at Akaki River crossing which might highly affect its functionality. Currently site observation shows that there is some failure on culvert structures, bridges and ditches before it has been opened for traffic. Before the structures failure gets worse and cause loss of human life and resource, it is essential to conduct studies to identify problems, consequence of problems and give future hypothesis. From site observations and different reports, it can be understood that there is many failures of drainage facilities along the railway alignment from Sebeta - Mieso. According to *Manaye Ewnetu (2013),* site visit report, the main causes of this failure is as there is no detail hydrologic and hydraulic analysis was conducted. This study carried out detail hydrologic and hydraulic analysis of Akaki River rail way crossing and evaluated the adequacy of bridge provided.

Based on the findings obtained summary conclusion and recommendation were derived for this study.

1.2 Statement of the problems

Developing hydrologic and hydraulic analysis for railway drainage system is used to build efficient hydraulic structure that will bring many benefits from economic and functional perspectives. Saving rail track from continues maintenance, using natural and human recourses properly, creating the environment friendly and hypothesizes suitable design and construction methods that fit Ethiopian environmental condition are just some of them. Hydrologic and hydraulic analysis for railway drainage system seems to "easy" but plays great role in safety, efficiency and effectiveness of rail road, generally the country's development as the whole.

Different site observations show that, as "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).

Akaki river crossing new railway bridge is designed by design flood of 554m^3/s for 100 year return period (source: Ethiopian Railway Corporation). This study was worked on whether the determined peak flow hydro logically correct and adequacy of the bridge weather it is hydraulically efficient to pass determined design discharge.

Thus, this study analyzes both hydrology and hydraulics of big Akaki River to check adequacy of bridge for new railway line crossing the river.

1.3 Objectives

a. General objective

The main objective of the study is

To check adequacy of Akaki River crossing Bridge of new railway line from Sebeta to Djibouti by under taking hydrological and hydraulics analysis and to ensure that drainage structures are designed to minimize future maintenance requirements and provide its service efficiently through its life time.

b. Specific objective

- Develop hydrological models for the catchment area.
- To conduct hydrological analysis.
- Analyze rainfall runoff characteristics of the catchment
- To determine peak discharge flow at railway crossing.
- Calibrating and validation of hydrological parameters.

- Develop hydraulics models of the reach and bridge.
- To check adequacy of hydraulic structure provided at this crossing with different return period of discharge.

1.4 Significance of the Research

Ethiopia is recently constructing a lot of mega structures where Railway track is one which serves huge passenger and freight transportation. Ethiopian Railway Corporation (ERC) identifies eight route corridors which connect capitals, regions with each other and with ports. Sebeta- Mieso-Dawele rail way line, which is currently under construction, experiences many drainage system problems. Site observation shows that capacity, location and direction of culverts and bridges were not been analyzed correctly. There is also no hydrological, hydraulics and environmental analysis has been under taken before construction has started.

Akaki River crosses Sebeta to Mieso railway line, near to southern Addis Ababa, was used for this study to check adequacy of bridge by undertaking hydrological and hydraulic analysis, taking inconsideration for upstream catchment change due to urbanization (land use) and flooding plain of the catchment area especially around railway crossing.

Thus, this study has been tried to analyze hydrological and hydraulics of the Akaki river depending on available data and relevant information.

1.5 Scope of the study

The scope of the study is limited to identification of problems and directing solutions for future implementation regarding railway drainage structure failure (Addis Ababa-Mieso route) especially Akaki river crossing. This study covers hydrological and hydraulic modelling and analysis of Akaki river catchment and land use characteristics of the area. Even thought, causes of failures of drainage structure come from different factor like material of construction, method of construction, and change of surrounding area and hydrologic, hydraulic analysis, this study mainly worked on hydrologic and hydraulic analysis of the river.

1.6 Description of the study area

a. Location

The Akaki river catchment is located along the North eastern to south eastern of Addis Ababa between 8^{o}46’0’’-9^{0}14’10’’N latitude and 38^{o}34’15’’-39^{o}04’16’’E longitude. It is surrounded to the north by Intotohill,Alaltu, Chancho and Sonkole, to the west by Ejersa and to the east by Lencha, Meta Chene Mt. Yerer, Tulu Dimtu. This catchment covers most of Addis Ababa city which has a total area of about 894km^{2}

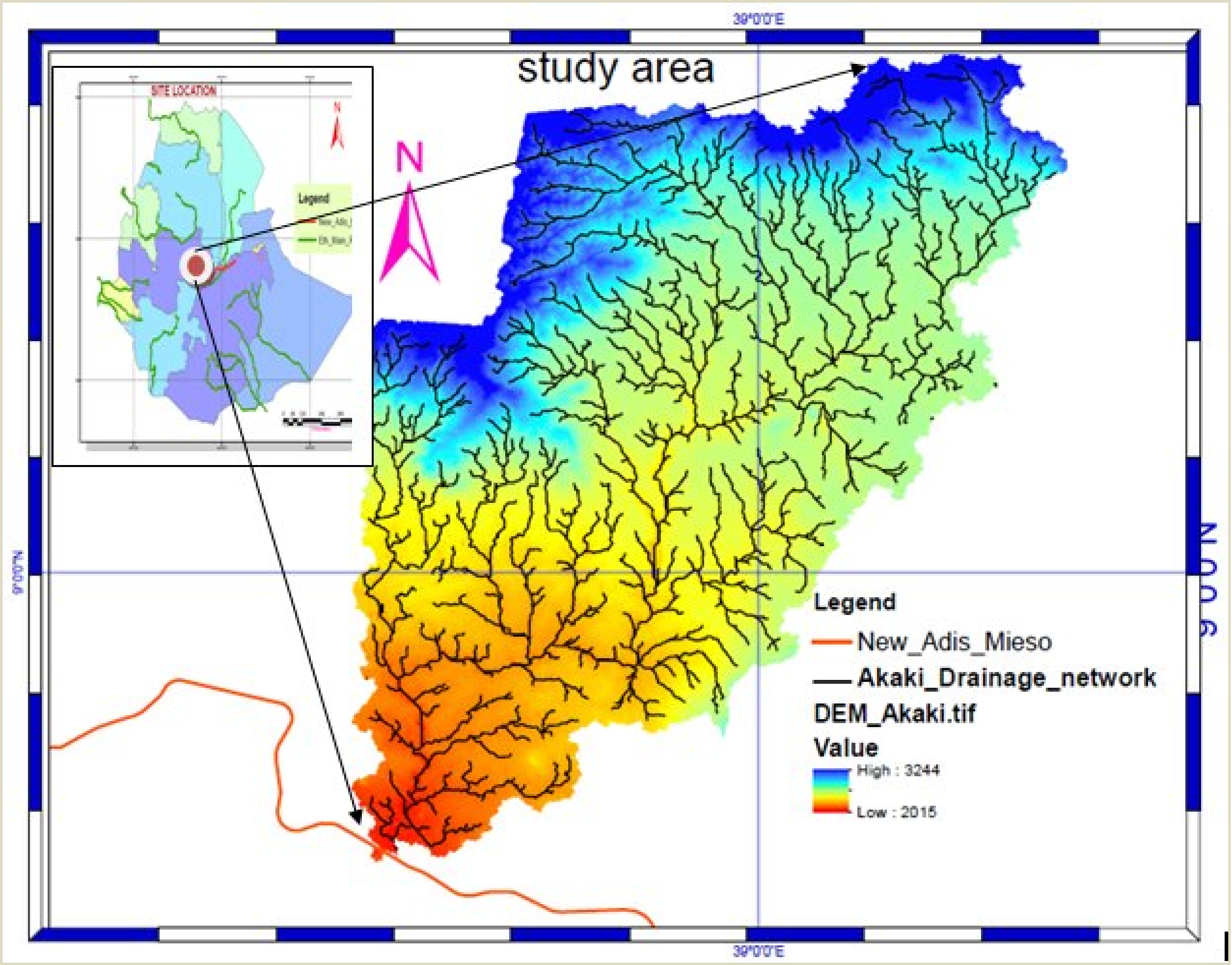


Figure 1-1 : Study area location

b. Land use

The existing land use coverage of the catchment has been broadly divided into six groups. According to geographical information data obtained from ERC, land use of the catchment is covered by Cultivated Land; Rain fed; Cereal Land Cover System; lightly stocked, Grassland;

lightly stocked, Forest, Plantation forest; Closed (>80% crown cover), Forest; Montane broadleaf; Open (20-50% crown cover), Urban and Woodland; Open (20-50% tree cover). However, currently this land use has been changed to paved and built up areas due to the expansion of the Addis Ababa City. This minimises the infiltration of precipitation into the ground and most of the rainfall is directly converted to runoff which determines the drainage capacity of the structure. Thus, for this study the land use of the catchment area was considered as built up urban development for the computation of hydrologic parameters in later section.

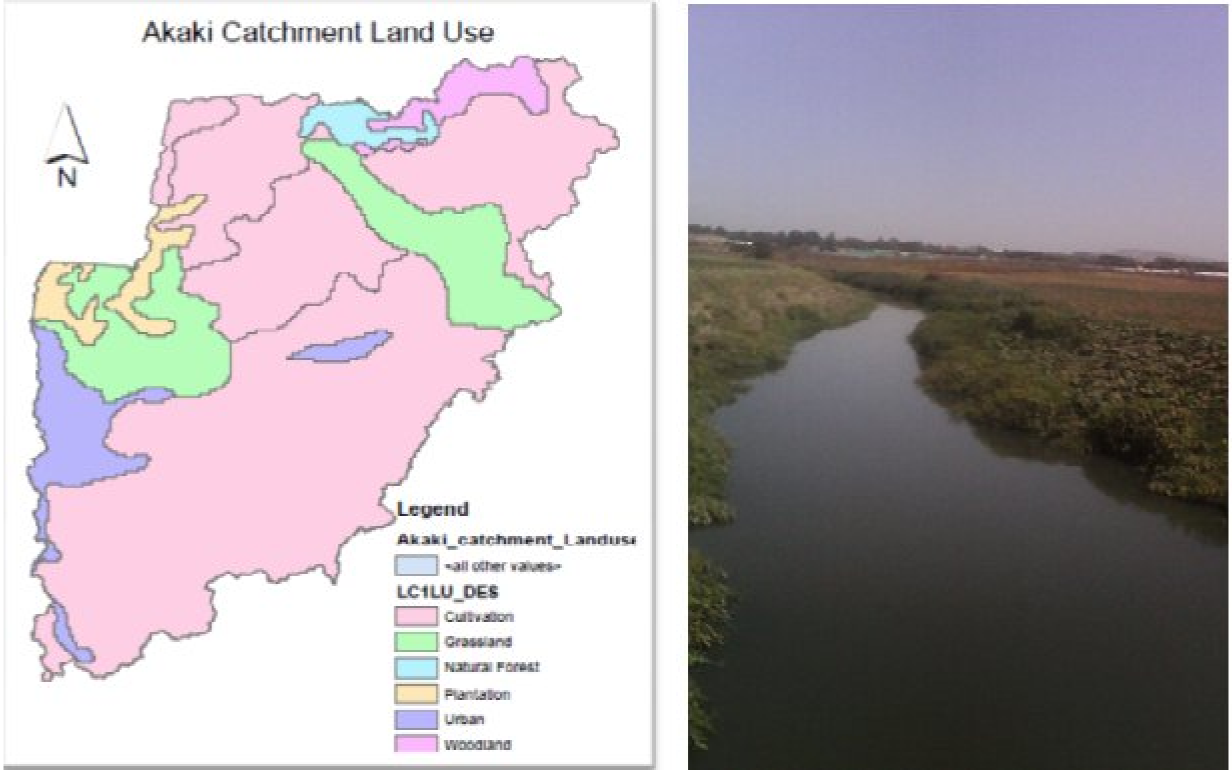


Figure 1-2: Land use map of Akaki river catchment

c. Soil Type

The Akaki catchment is an integral part of the evolution and development of Ethiopian showa plateau and rift valley system. According to *AAWSA (2000*) sited in academic study of *Tesfaye Alema (2009),* the catchment is covered by volcanic rock over laying by fluvial and residual soil in which black cotton soil is the most dominant one.

According to soil information data obtained from ERC, the soil types that dominantly cover the catchment have been divided into nine classes as shown in figure below. It includes; calcic xerosols, chromic cambisols, chromic luvisols, chromic vertisols, eutricnitisols, leptosols, orthicsolonchaks, pellicvertisols and verticcambisols.

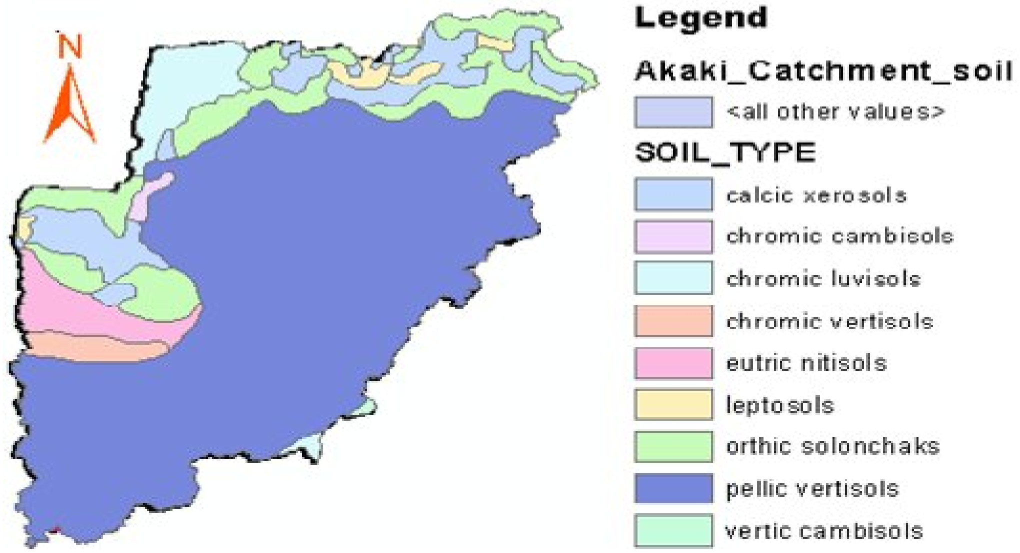


Figure 1-3: Soil map of catchment area

d. Hydro metrology

The Akaki River is one of the tributary of Awash, has many drainage networks. It originates from the Intoto ridge, north of Addis Ababa, and drops AkakiAbasamuel artificial lake, southeast of Addis Ababa. From the catchment area small drainage networks are drained to longest flow path which cross the railway alignment.

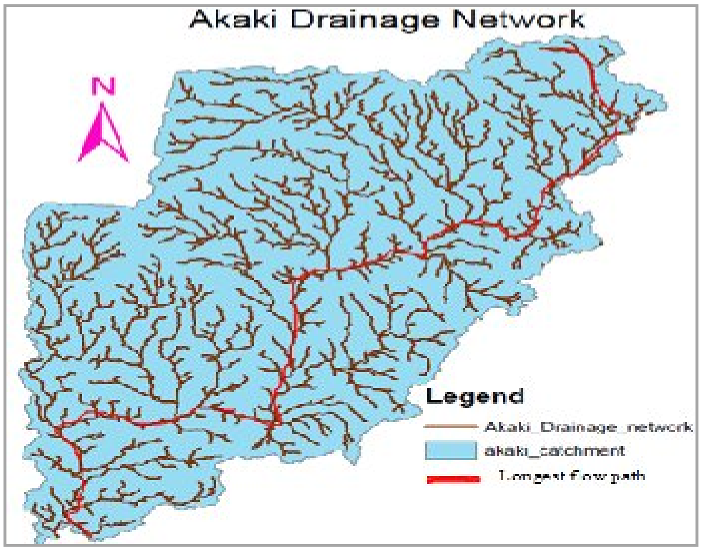


Figure 1-4: Akaki catchment drainage network

Based on information collected from Ministry of Water, Irrigation and Energy, the Akaki River is gauged since 1981 to 2004 at the Addis Ababa - DebreZeit Road Bridge, with a catchment area of 878km$^{2.}$ The mean annual discharge of Akaki River at this location is computed from the record for the period of 1981 to 2004, and is found to be 9517 l/s or 10.84 l/s/km2 which are equivalent to mean annual yield of 339 mm. The monthly distribution of runoff volume of Akaki River is shown below.

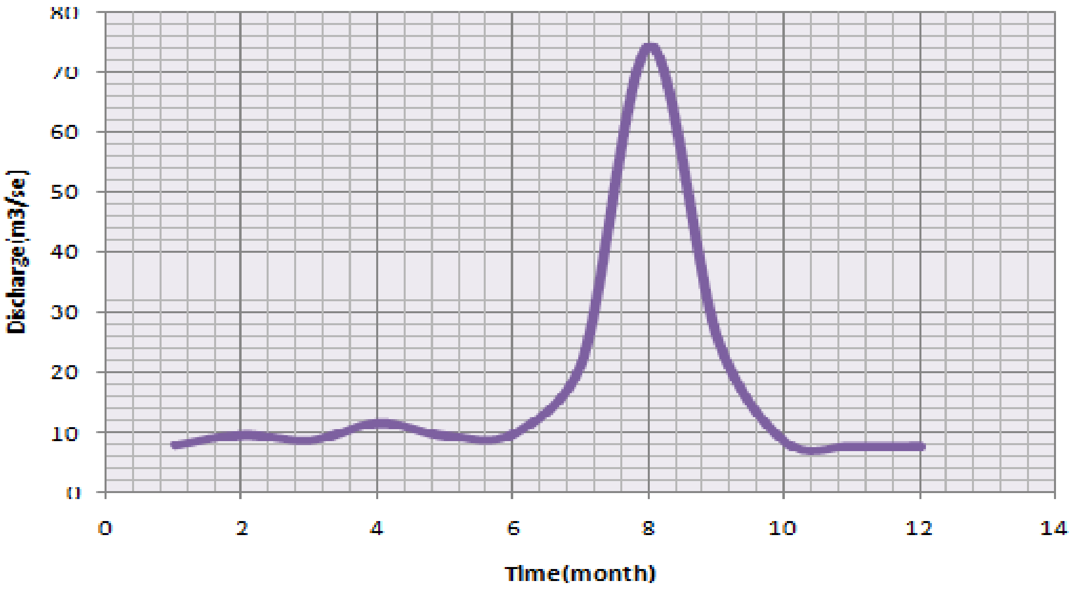


Figure 1-5: Monthly variation of Akaki River at Akaki Bridge (1981-2004)

i. Temperature and Climate

In order to understand the environment and the possible impact of human activity, basic knowledge of weather and climate is required. The former is the physical condition of the atmosphere at a specific time and place with regard to wind, temperature, cloud cover, fog, and precipitation. Weather is highly variable and somewhat unpredictable. As a result, a longer-term view of the weather pattern of a particular locality is frequently more useful as an environmental tool.

National Atlas of Ethiopia (1981) cited in (*AAWSA,2003*) defined five traditional climatic zones: "Kur" (Alpine), 3000m and above; "Dega" (temperate), 2300m to about 3000m; "WeinaDega" (Sub tropical), 1500 to about 2300m; "Kolla" (Tropical), 800m to about 1500m and "Bereha" (Desert), less than 800m.

Despite its proximity to the equator, the study area experiences a temperate Afro-Alpine climate. Daily average temperatures is range from 9.9 to 24.6 0C. The climate of the Addis Ababa is characterized by two distinct seasonal weather patterns. The main wet season, locally known as Kiremt, extends from June to September, contributing about 70% of the total annual rainfall. A minor rainy season, locally known as Belg, contributes moisture to the region from mid- February to mid - April (*Daniel, 1977*).

Maximum and minimum monthly temperature in the catchment

Temp.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Ann. Ave.
Max.	23.3	24.1	24.6	23.9	24.6	22.9	20.3	20.1	21.1	22.4	22.6	22.8	22.7
Min.	8.2	9.5	10.9	11.5	11.7	10.8	10.8	10.8	10.5	9.2	7.9	7.5	9.9

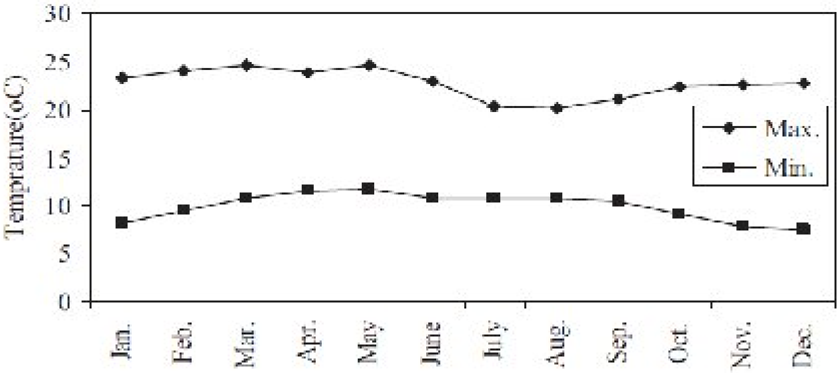


Figure 1-6: Temperature variations in Akaki catchment (source: Tesfaye, March 2009)

ii. Rainfall Distribution

According to Daniel (*1977*) classification of Ethiopia's rainfall region, Addis Ababa is located in the region where the rainy months are contiguously distributed. In this region there are seven rainy months from March to September/and the small rains occur from March to May. The big rains are from June to September. High concentration of rainfall occurs in July and very high concentration in August.

Rainfall distributions of Akaki catchment were collected from Ethiopia Metrological Agency at three gauging station, (Bole, Akaki and observatory). The precipitation at these gauging stations covers most of the catchment area with mean annual precipitation of about 300mm.

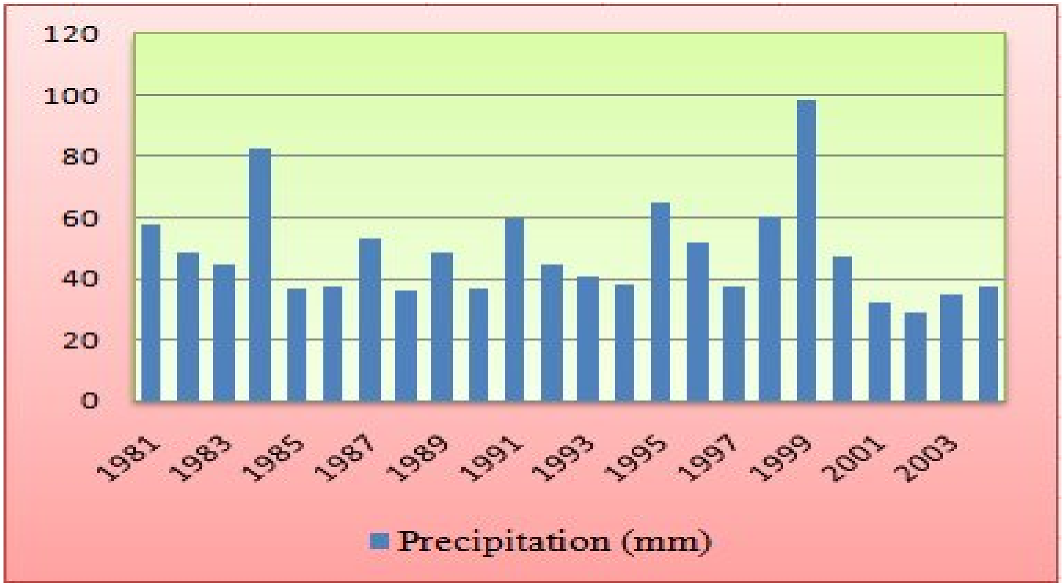


Figure 1-7: Daily maximum Rainfall distribution (hyetograph) of the catchment area

e. Hydraulics

The Railway alignment, from Sebeta- Mieso, transverses different topographical and hydro geological condition in which it crosses swampy and low land areas at Akaki Beseka.At this location quantity of surface water come from the catchment is critical issues for the provision of hydraulic structures that can safely convey it.

Akaki river railway bridge, which has clear span length of 163.83m, was provoded to cross railway line over. Adequacy of openning space provided was evaluated at diffirent design peroid.

Site observation shows that there is water level mark during summer which indicates water level increases to half depth of the bridge as shown on photo 1.3 below. As it is stated before, at this crossing there is flood plain and swampy area where embankment is constructed to reduce span length of the bridge. This leads to increase water level especially during summer (Ethiopian kiremt) seasons.

(Jan 2014) Sep. 2015

Photo 1-1-1: Flood level mark on Akaki River crossing Railway Bridge

When embankment is constructed at this location, the natural water course of the catchment is changed in which it concentrates to one direction and rising upstream water profile(back water) to an ever reach. As shown in figure below, which is taken during winter (Ethiopian Bega) season, the flooding area (seen to North) and embankment constructed on both side of river crossing hinders the flow of runoff especially during summer which leads to large area flooding. Local communities are complaining that water level after this embankment was constructed takes some of their farming land that they could use for cropping during summer.

Photo 1-1-2: Flood plain and Embankment at Akaki new railway crossing (Jan 2014)

2. LITERATURE REVIEW

2.1. General

The expansion of infrastructures is vital for a country development. From most infrastructures, Railway lines are the basic ones that used for huge transportation purpose. In our country there is a good endeavor of expansion of rail road to its modern and safest mode of transportation. The attempt to elevate this mode of transportation is supported by providing stable and efficient railway components like track system, drainage system, controlling system, operation system, management system, etc. Therefore, Proper drainage system is essential for a railway to function properly. The primary purposes of railroad drainage systems are to minimize water depths occurring on track surfaces during heavy storms. However, many of drainage structures are not functioning well to the desired life time and quality. The attempt to alleviate the failures on the drainage structures is very little, even though the problem is so much large. Many times side ditches, culverts and bridges are found to be clogged, collapsed and washed away by the flood. Consequently, the quality of railroads is much deteriorated and their life time is shortened which leads to greater loss of human life, loss of property and negatively impacts the country's development as the whole. To address these problems investigations on hydrology, hydraulic, soil condition, topography of the area etc, are necessary. Thus giving attention for investigation of failures with the cross drainage structures is far most important duty of the academicians as well as professionals. In this case the problem on the Akaki River Bridge will be considered. Special attention shall be given to the failures in bridge structures since any malfunction on these structures creates a wide-ranging problem.

Properly designed openings, control of flood flows, and protection of roadway and structures are of vast importance from the standpoints of safety, economy, and continuity of operation during flood periods. With the ever-present menace of floods and then disastrous consequences, every related problem is deserving of accurate and exhaustive survey and careful planning. This concerns the determination of the location, size, and shape of drainage structures. It also includes consideration of flood flows and water-borne materials in surface waters, the protection of the roadway in contact with surface water, and the protection of structures carrying tracks over waterway openings (AREMA 2010).

According to AREMA, survey requirements of cross drainage structure depend in some degree upon whether the waterways are to be crossed by a newline, or whether the replacement of an existing waterway structure is involved. For the crossing of a new line the survey requirements are extensive and general in nature, involving the determination of the drainage area and its shape; the stream and slope profile; soil, vegetation and climatic characteristics; as well as topographical details in the vicinity of the most probable point of crossing.

A significance of the cost of the most highway projects is attributable to drainage facilities, such as bridges, culverts, and storm drains. Design of these facilities involves a hydrologic analysis to determine the design discharge, and a hydraulic analysis of the conveyance capacity of the facility (Bisrat Temesgen, et.al. 2015).

Along the Sebeta- Mieso rail line segment there are different culverts, bridges, cut and embankment. According to Site Visit Investigation Report and Recommended Rehabilitation Work*s* (Manaye Ewnetu*, 2013*), the Sebeta to Adama railway project is not near to the standard that is expected for a project of national importance due to different problems existed. "Appreciation of soil conditions, catchment design flows and flow velocities has not been taken into consideration. This is critical for the longevity and future operation and maintenance of the railway line". Problems associated with the drainage system of rail track are, orientation, location, alignment, number and size of culverts used to convey the runoff.

According to academic study of Beza Negussie (*2010),* the main causes for failure of the drainage structures are due to the following factors:

1) Basin Characteristics: Size, shape, land use, geology, soil type, surface infiltration and storage etc.

2) Stream channel Characteristics: geometry and configuration, natural and artificial controls, channel modifications, aggradations, degradation and debris.

3) Flood plain characteristics.

4. 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 problems of the drainage structures in the following main groups:

a) Hydrological failure

b) Hydraulic failure

c) Failure due to aggradations or degradation

d) Orientation or location of structure.

It is known that the construction of rail road disturbs the natural drainage pattern of streams; and the design and provision of crossing structures is the basic measures which involve two sequential activities, the estimation of peak flood flow and the design of a suitable structure to accommodate it. These two activities are developed under field of Hydrology and Hydraulics respectively. In relation with this the hydrological study is conducted in order to determine the peak discharge to facilitate the provision of waterway for the respective watershed; interrupted due to the construction of the railroad. That is to assist the determination of the hydraulic opening sizes, it is quite necessary to conduct hydrologic study.

2.2. Hydrology and hydrological models

Hydrology is the science, which deals with the occurrence, distribution and disposal of water on the planet earth; it is the science which deals with the various phases of the hydrologic cycle. Hydrologic cycle is the water transfer cycle, which occurs continuously in nature; the three important phases of the hydrologic cycle are: (*a*) Evaporation and evapotranspiration (*b*) precipitation and (*c*) runoff (Ven Te Chow, 1982). The hydrology of a region is determined by its weather patterns and by physical factors such as topography, geology and vegetation. Also, as civilization progresses, human activities gradually encroach on the natural water environment, altering the dynamic equilibrium of the hydrologic cycle and initiating new processes and events. For example, it has been theorized that because of the burning of fossil fuels, the amount of carbon dioxide in the atmosphere is increasing. This could result in a warming of the earth and have far-reaching effects on global hydrology (H.M. Raghunath, 2006)

The hydrological study is undertaken in order to compute and evaluate peak discharges for watercourses crossing structures. Underestimation of peak discharge cause undersized structures and results more drainage problems and on the other hand overestimation of peak discharge results a structure that is oversized and costs more than necessary. Therefore the aim of the hydrologic analysis should be to derive the maximum reliable discharge for a given waterway for a specific design period.

Many drainage structures are destroyed or fail completely during severe floods. If the cause of the damage is not a structural deficiency, it may due to insufficient hydrologic and hydraulic evaluations that result uncertainties in bridge design. The optimum return period for a flood hydrograph may be determined through a hydro-economic analysis for bridge design. Regardless of the size or cost of the drainage feature the most important step prior to hydraulic design is estimating the discharge (rate of runoff) or volume of runoff that the drainage facility will be required to convey or control. Lack of relevant hydrologic data, such as precipitation and runoff may lead to difficulties in obtaining the design parameters (A. Melih and Feridun, 2014). Hydrologic analysis is necessary in establishing the quantity of surface water that must be considered in the design of all drainage facilities.

To make an informed decision, it is necessary to determine what level of hydrologic analysis is justified, what data are available or must be collected and what methods of analysis are available including the relative strengths and weaknesses in terms of cost and accuracy.

2.2.1. The Hydrological Modelling Process

Modelling is the use of mathematics as a tool to explain and make predictions of natural phenomena (Vijay P.,Donald K. 2002). Mathematical modelling may involve words, diagrams, mathematical notation and physical structure. Mathematical modelling of watershed can address a wide range of environmental and water resources problems. Planning, designing and managing water resources systems involve impact prediction which requires modelling. A mathematical / computational procedure for performing operations on the model is used for getting outputs from inputs of a watershed. A model is a representation of reality in simple form based on hypotheses and equations.

As Chow (1982), Hydrologic models may be divided into two categories: physical models and abstract models'. Physical models include scale models which represent the system on a reduced scale, such as a hydraulic model of a dam spillway; and analog models, which use another physical system having properties similar to those of the prototype

Abstract models represent the system in mathematical form. The system operation is described by a set of equations linking the input and the output variables. These variables may be functions of space and time, and they may also be probabilistic or random variables which do not have a fixed value at a particular point in space and time but instead are described by probability distribution.

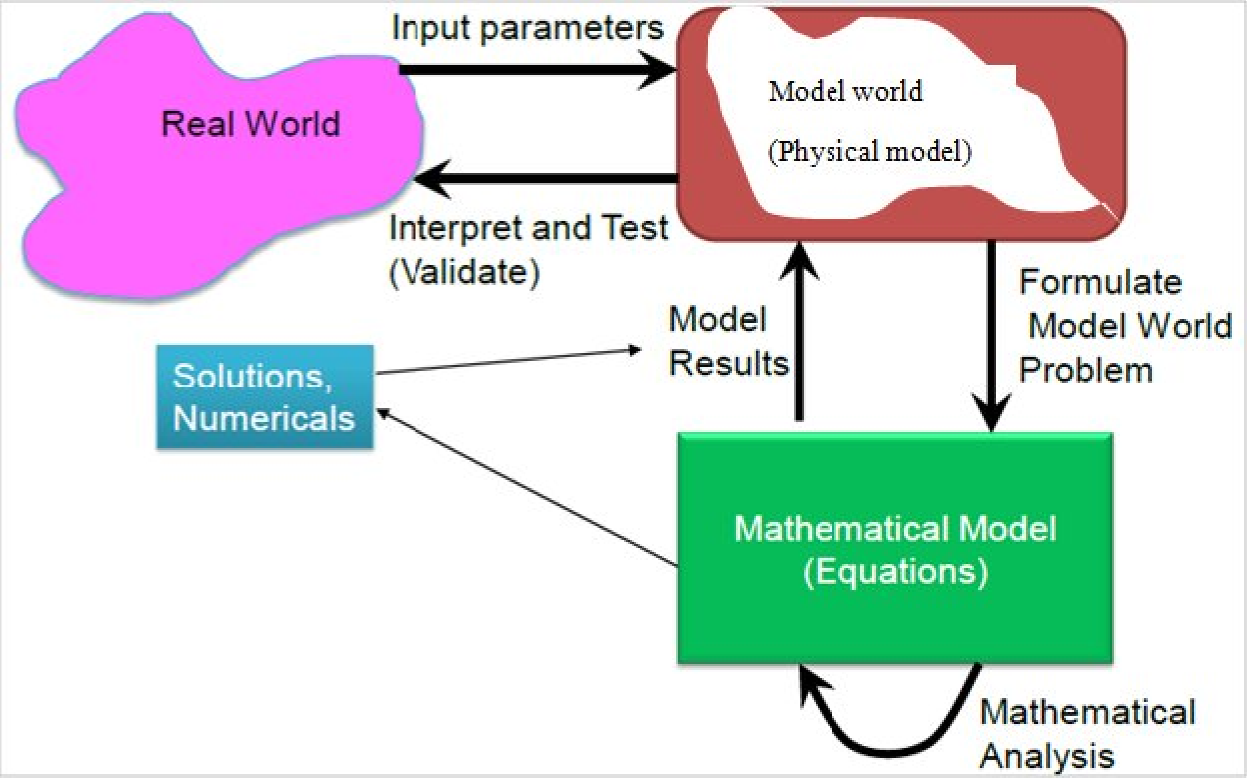


Figure 2-1: Modelling process

2.2.2. Classification of Watershed Models

Hydrological model can be classified depending different criteria. It could be classified based on nature of the algorithms (Empirical, Conceptual, Physically based), based on nature of input and uncertainty (Deterministic, Stochastic), based on nature of spatial representation (Lumped, Distributed, Black-box), based on type of storm event (Single event, Continuous event), etc.

According to Bisrat Temesgen,et.al. (2015), there are numerous criteria which can be used for choosing the "right" hydrologic model. These criteria are always project-dependent, since every project has its own specific requirements and needs. Further, some criteria are also user-depended (and therefore subjective). Among the various project-depended selection criteria, there are four common, fundamental ones that must be always answered : based on these criteria the Hec-HMS model was selected for the following criteria's: 1) Required model outputs important to the project and therefore to be estimated by the model 2) Hydrologic processes that need to be modeled to estimate the desired outputs adequately 3) Availability of input data 4) Price (Does the investment appear to be worthwhile for the objectives of the project?). For an event based modeling runoff volume and direct runoff are the most principal components of the water cycle.

HEC-HMS is a physically based, semi-distributed hydrologic model developed by the US Army Corps of Engineers to simulate the hydrologic response of a watershed subject to a given hydro meteorological input(Scharffenber et al.,2010).The model uses underlying DEM information to partition the basin into sub-watersheds. The model can simulate individual storm events as well as continuous precipitation input at minute, hourly, or daily time steps.

The HEC-HMS requires a variety of model parameter to simulate runoff production. These include SCS curve number, SCS unit hydrograph, and baseflow estimation methods which are necessary to calculate water losses, runoff transformation, and base flow rates. The values of the model parameters have the potential to change along with changing sub-basin sizes (H. L. Zhang, et.al.2013). Hydrologic mechanisms on watershed include losses due to ponding, infiltration, and base flow production. The SCS loss model for basin loss is given by

$Pe = (P - Ia)^2/(P - Ia + S)$

where Pe is excess precipitation, P is accumulated precipitation, Ia is initial abstraction and can be initialized as 0.2S, and S is the potential maximum retention and is a function of curve number (CN): S = (25400-254CN)/CN (SI system). The initial abstraction and CN are required parameters.

The SCS unit hydrograph (UH) rainfall–runoff transformation model is a dimensionless unit hydrograph Ut expressed as a ratio to peak (RP) discharge Up for any fraction of time t/Tp, where Tp is the time to peak. The peak discharge is given by UP = CA/TP, where C is the conversion constant (2.08 in SI) and Ais the sub-watershed area. The time of peak Tp is calculated as Tp = 1t/2+tp , where 1t is the time step in HEC-HMS and tp is the time lag defined as the time difference between the center of excess precipitation and the center of UH where tp is a required input parameter.

The exponential recession model for baseflow is given by

$Qt = Q_0k^t$,

Where Q0 is initial baseflow and k is an exponential decay constant. During the recession period of a flood event, a RP is specified to derive the threshold flow at which the baseflow is calculated as a fraction of peak flow. Q0, k and the RP are required parameters. Hydrologic mechanisms in the transport in the channel contain Muskingum parameters and constant channel loss also required in HEC-HMS.

HEC-HMS model require calibration (optimization of parameters) and validation to give accurate hydrologic results. According to Oleyible and Li, (2010) mentioned in H. L. Zhang, et.al.(2013) the initial step in model calibration is a manual adjustment of model parameters using the trial-and-error method, which enables the modeler to make a subjective adjustment of parameters that gives an appropriate fit between observed and simulated hydrographs. HEC-HMS also provides objective functions that used for automated parameter estimation. These include the sum squared residuals (SSR), which emphasize water balance; peak error (PE); mass balance error (MBE); the Nash–Sutcliffe efficiency coefficient (ENS), and the relation coefficient (R^2)

The objective function is defined as follow.

$$\mathrm{SSR} = \quad (\quad - \quad)$$

Where Qo and Qs are observed and simulated flow, respectively, while the other measures of PE, MBE, NSE, and $R^2$are used to assess the hydrological modeling performance. The performance measures are defined as

$$\mathrm{PE} = \frac{-}{\qquad}$$

$$\mathrm{MBE} = \frac{-}{\qquad}$$

$$= 1\frac{\sum \quad (\quad - \quad)}{\sum \quad (\quad - \quad)}$$

$$= \frac{\sum \quad (\quad - \quad)(\quad - \quad)}{\overline{(\quad - \quad)}\;\overline{(\quad - \quad)}}$$

where Qo,m, and Qo, are observed peak discharge, averaged discharge, respectively, and Qs,m and Qs are corresponding calculated discharge values.

2.3. Hydraulic design of cross drainage structures

The hydraulic design of a bridge over a waterway involves establishing a location bridge length, orientation and roadway and bridge profiles such that the risks associated with backwater and increased velocities are not excessive.

The hydraulic analysis of a channel determines the flow patterns, flood levels and velocity at which a given discharge will flow in a channel of known geometry, roughness and slope. The depth and velocity of flow are necessary for the design or analysis of channel linings and drainage structures. The most commonly used hydraulic analysis procedure used for routine bridge design is HEC-RAS, developed by the United States Army Corps of Engineers. This software is well suited to the hydraulic analysis of bridges and culverts and is widely used around the world for this type of work. Hec-RAS is capable of performing one-dimensional water surface profile calculations for steady gradually varied flow in natural or constructed channels. 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 (Gray W. Burnner, 2010).

According to Gray W. Burner, (2010) there are four methods used to compute losses through bridge.

- Energy Equation (standard step method)
- Momentum balance
- Yarnell Equation
- FHWA WSPRO method

This method allows comparing the answers from several techniques all in a single execution of the program. If more than one method is selected the program select the method that computes the largest energy loss through the bridge as the final solution. Adetail discussion of each method as follows;

a. Energy Equation (standard step method)

The energy based method treats a bridge in the same manner as a natural river cross section, except the area of the bridge below the water surface is subtracted from the total area and wetted perimeter is increased where the water is in contact with the bridge structure.

The energy equation is written as follows

$$+ \quad + \frac{\quad}{2} = \quad + \quad + \frac{\quad}{2} + h$$

Where: Y1, Y2: Depth of water at cross-sections, Z1, Z2: Elevation of the main channel inverts and α1, α2 are Velocity weighing coefficients for V1, V2 (also called as Average velocities and equal to (Total discharge/Total flow area), g is the gravitational acceleration, and he is energy head loss.

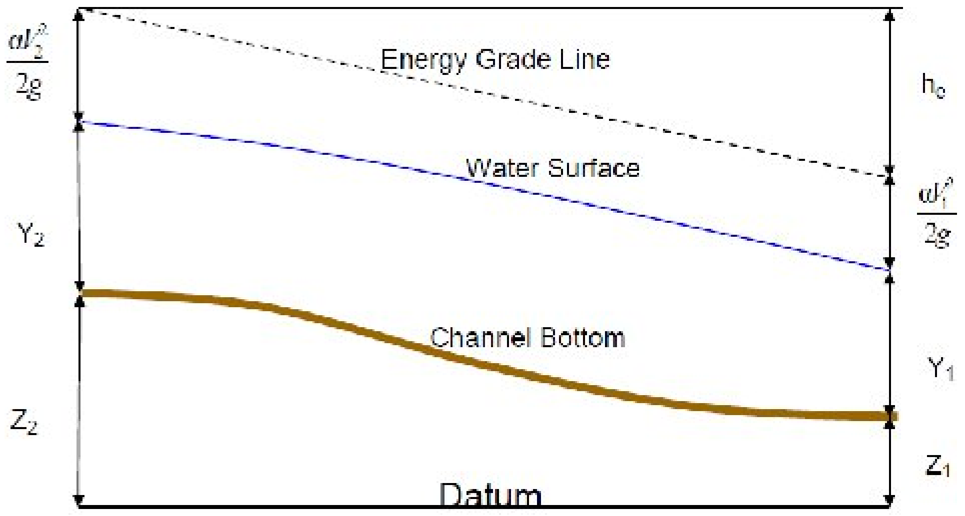


Figure 2-2: Representation of terms in energy equation (source: Gary W 2010)

The energy head loss (he) between two cross sections is comprised of friction losses and contraction or expansion losses. The equation for the energy head loss is as follows (Gary W 2010);

$$h\ =\ \bar{\ }\ +\ \ \frac{\qquad}{2}-\frac{\qquad}{2}$$

Where: Sf= Representative friction slope between two section, L=Discharge weighted reach length, C = Expansion or contraction loss coefficient,

b. Momentum balance Method

The momentum method is based on performing momentum balance from cross section 2 to cross section 3. The momentum balance is performed in three steps, from section 2 to BD, from BD to BU and from BU to 3 (Gary W 2010) as shown on figure below. Equation for momentum balance is as follow,

$$+\frac{\qquad}{\ }=\qquad+\frac{\qquad}{\ }-\qquad\qquad+\quad-$$

Where, A_2,A_{BD} =Active flow area at cross section 2 and BD

A = obstructed area of the pier on downstream side

Y , Y = vertical distance from water surface to center of gravity of low area

A and A respectively

Y = vertical distance from water surface to center of gravity of wetted pier area

on downstream side

β , β = velocity weighting coef icient for momentum equation

$Q \quad , Q \quad = \text{Discharge}$

g= Gravitational acceleration

$= \qquad h$

$= \qquad h \qquad h$

The second step of momentum equation has the same with the fist except the location where as the third step has the following equation;

$$A \; Y \; + \frac{\beta \; Q}{gA} = A \quad Y \quad + \frac{\beta \quad Q}{gA} - A \quad Y \quad + \frac{1}{2} C \; \frac{A \quad Q}{gA} + F \; - W$$

Where C $=$ coef icient of low going around the piers

Momentum balance Method equation requires roughness coefficient and drag coefficient which depends on shape of piers.

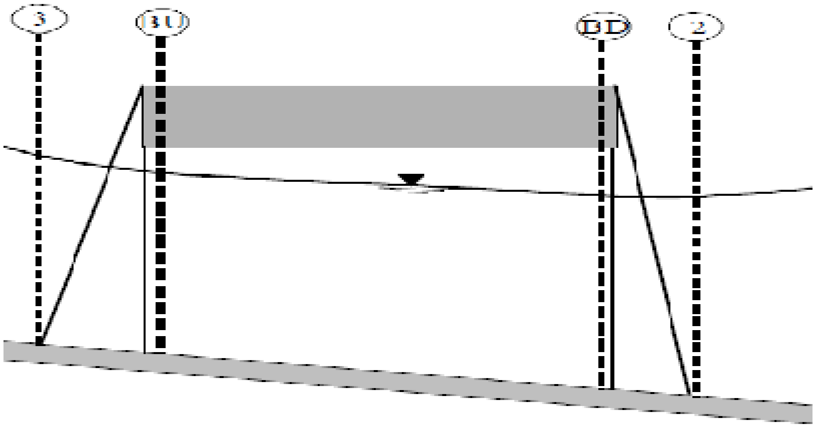


Figure 2-3: Scheme of near and inside of bridge cross section (source: Gary W 2010)

c. Yarnell Equation

The Yarnell equation is an empirical equation that is used to predict the change in water surface from just downstream of the bridge (section 2) to just upstream of the bridge (section 3) as shown on figure above. Yarnell Equation is given as follow;

$$H \quad = 2K(K + 10\omega - 0.6)(\alpha + 15\alpha \;) \frac{V}{2g}$$

Where $H_{3\text{-}2}$=drop in water surface elevation from section 3 to 2

K = Yarnell's pier shape coefficients

ω = Ratio of velocity head to depth at section 2

α = obstructed area of the piers divided by the total un obstructed area at section 2

V_2 = Velocity downstream at section 2

The computed water surface elevation at section 3 is simply downstream water surface plus H3-2

Yarnell's pier coefficient value is required in HEC-RAS water surface elevation computation.

d. FHWA WSPRO method

The low flow hydraulic computation of the Federal Highway Administration (FHWA) WSPRO computer program has been adapted as an option for low flow hydraulic in HEC-RAS. It computes water surface profile through energy equation as an iterative solution.

Generally, water surface profile computation begins at a control section or with a known depth in the channel and proceeds upstream or downstream. A successive computational procedure used to compute the water surface elevation at the next section (upstream or downstream) from the control section. The distance between sections is critical because the water surface will be represented by straight line. Thus, if the depth of flow is changing quickly over short distances, neighboring section should be spaced closed to represent accurately the water surface profile. The step procedure is carried out in the downstream direction for supercritical flows and in the upstream direction for subcritical flows.

The reach should extend sufficiently far downstream that the tail water level for the bridge is represented correctly and should extend upstream sufficiently to represent the afflux caused by the bridge.

The HEC-RAS model usually works upstream from a downstream boundary (for subcritical flow, which is the most common in natural channels), and this downstream level needs to be carefully assessed. The downstream boundary will usually be based on a downstream slope or the level of the receiving water. The downstream boundary should be sufficiently far downstream that the level does not affect the tail water of the bridge, but in some cases, careful analysis and sensitivity testing may be needed. The main parameter used for HEC-RAS analysis is Manning's n, a measure of channel roughness. Calibrated hydraulic models for other projects in the region can also be a useful starting point for estimation of Manning's n. It is often useful to consider a sensitivity analysis using Manning's n where there is uncertainty in selection of an appropriate value. Any observed flood data comes in useful with the hydraulic modeling. The flood levels may be available for specific historical floods and these levels can be used directly in model calibration. If there is only general information, this can also be used to help in ensuring that the model results are at least reasonable. The observed flood levels are used to estimate Manning's n roughness values to make sure that the calculated flood levels match the observations. The observed flood levels can also help in testing for the effects of backwater (Afflux) and for the downstream starting level for the HEC-RAS model. Afflux is usually a concern in urban areas or at locations where there is a building or other infrastructure upstream of the bridge and in the region of influence. Afflux is at a maximum for the flood that just overtops the bridge and associated approach embankments and immediately upstream. Afflux is less for smaller floods that flow relatively unconstrained under the bridge and for floods where the bridge is overtopped and water can spread over an extent of floodway. Afflux reduces with distance upstream, with the extent of influence depending on the channel slope and other conditions. Afflux must be calculated for the appropriate distance upstream from the bridge and for a range of flood sizes. It may be impossible to reduce the afflux at a building to zero, but the individual risk should be assessed carefully.

The hydraulic modeling should consider the flood levels along the reach of the water course containing the bridge. The analysis should also consider the possibility of backwater from a downstream stream, dam or the ocean. If these conditions seem likely, careful analysis of the downstream conditions is needed to check for the possibility of backwater.

The maximum backwater effect at a drainage structure shall be 0.5 meters when the floor elevations of buildings or dwellings are lower than 1.5 meters above the natural design flood elevation. Otherwise, the maximum backwater shall be 1.0 meters below the floor elevation of upstream buildings or dwellings and the 100-year flood elevation shall be 0.3 meters below the floor elevation (ERADDM- 2002)

The hydraulic analysis should be carried out for a range of flood probabilities. Usually these probabilities include the standard AEPs up to AEP 1% and the AEP 0.05% floods. As well the flood that just overtops the bridge should also be considered, since this will be the most critical event considering velocities through the bridge and the afflux. However in some cases, a range of large floods may need to be analyzed if there are significant impact issues, especially if floods significantly larger than the design flood do not overtop the bridge

3. METHODOLOGY

3.1. Data collection

- Rainfall records of different stations were collected from the Ethiopian Meteorological Services Agency, (Bole, Akaki and Observatory gauged rainfall data).
- Observed stream flow data of Akaki River were collected from Water, Irrigation and Energy Minister.
- DEM, Land use and soil characteristics of the catchment area were collected from Ethiopian Railway Corporation (ERC)
- Akaki river cross section survey data and bridge structure detailing data are also collected from Ethiopian Railway Corporation (ERC)
- Site observation and interviewing with local people about flooding problems and water level marks during summer is conducted.

3.2. Hydrological Model

The hydrological model of study area was undertaken in order to compute and evaluate peak discharges for Akaki River crossing of the new railway project. Calculation of these peak discharge values then enabled to check the hydraulic opening sizes. There are different methods used to determine peak flow. U.S. Soil Conservation Services (SCS) Unit hydrograph method was employed to estimate the design discharge depending on the size of catchments area. Frequency analysis was also used to compute the design flood by using gauged stream flow data. The quantities of design rainfall were estimated by using the applicable software where as design flood was estimated separately for comparison purpose. Hydrologic modeling and analysis of this study was accomplished by software's like Global Mapper, Arc Hydro Tools, and HEC- Geo HMS with the help of Arc GIS and HEC-HMS programs.

3.2.1. Catchment Area

The catchment area of study area which extends from Intoto Mountains to Akaki Abasamuel artificial reservoir with 30mx30m resolution topographical maps was by Global Mapper and transformed to Digital elevation Model (Akaki tiff). Akaki river catchment has different land use and hydro geological settings. These land use and geological settings of the area has direct effect on rainfall runoff coefficient. Determining the size of the drainage area that contributes to flow at the site of the drainage structure is a basic step in a hydrologic analysis regardless of the method used to evaluate flood flows.

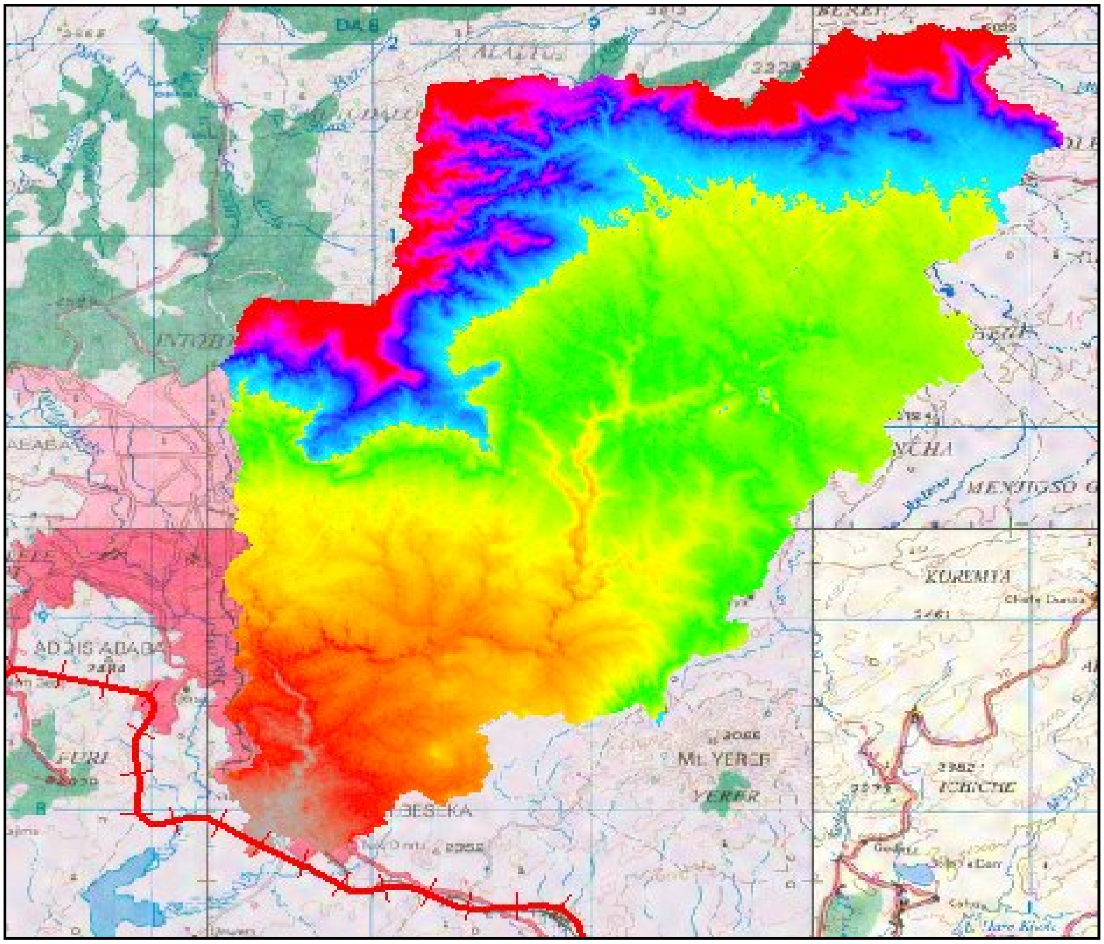


Figure 3-1: Akaki River catchment (tiff. format)

3.2.2. Hydrologic Process

3.2.2.1. Watershed and Stream Network Delineation using Arc Hydro Tools

The first step in doing any kind of hydrologic modeling involves delineating streams and watersheds, and getting some basic watershed properties such as area, slope, flow length, stream network density, etc. The processing of DEM to delineate watersheds is referred to as terrain pre-processing.

There are several tools for terrain pre-processing. However for this study Arc Hydro tools were used to process a DEM to delineate watershed, sub-watersheds, stream network and some other watershed characteristics that collectively describe the drainage patterns of a basin. Arc Hydro Terrain Preprocessing should be performed in sequential order as follow.

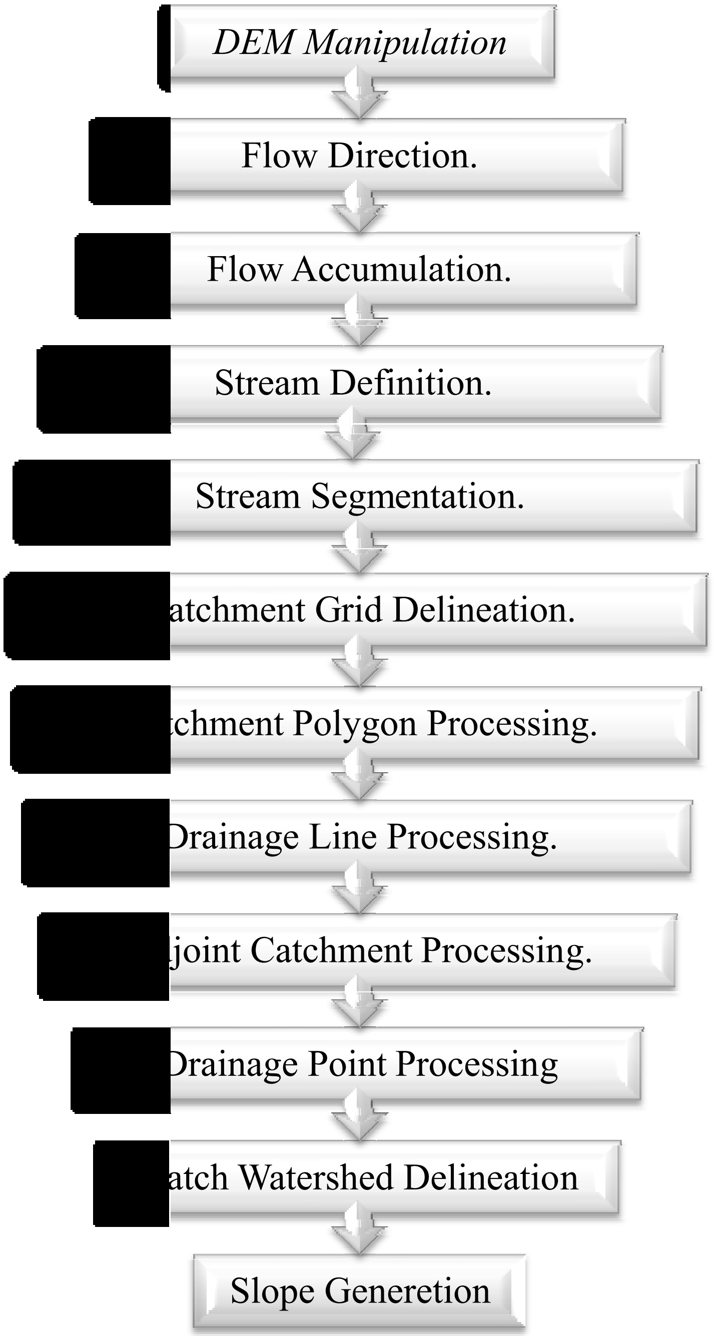


Figure 3-2: Flowchart of terrain pre-processing

3.2.2.2. HMS-Model Development using Geo-HMS

HEC-GeoHMS is a program that works with Arc GIS (for this case, version10) used to create input files for hydrologic modeling HEC-HMS. It transforms spatial information to model files for HEC-HMS. Arc GIS has a capability of data processing and coordinate transformation which results DEM. HEC-GeoHMS operates on the DEM to delineates sub basins and hydrologic inputs like longest flow path, centriods, river and basin slopes, etc. To accomplish this, HEC-GeoHMS has different components in which each steps should followed sequentially. Terrain processing involves using the DEM to create a stream network and catchments. This processing of HEC-Geo HMS is done by Arc Hydro tools using the previous procedure for this case.

3.2.2.3. HEC-GeoHMS Project Setup

The HEC-GoeHMS project setup menu has tools for defining the outlet for the watershed (Project Point) and delineating the watershed (Project Area) feature classes for the HEC-HMS project.

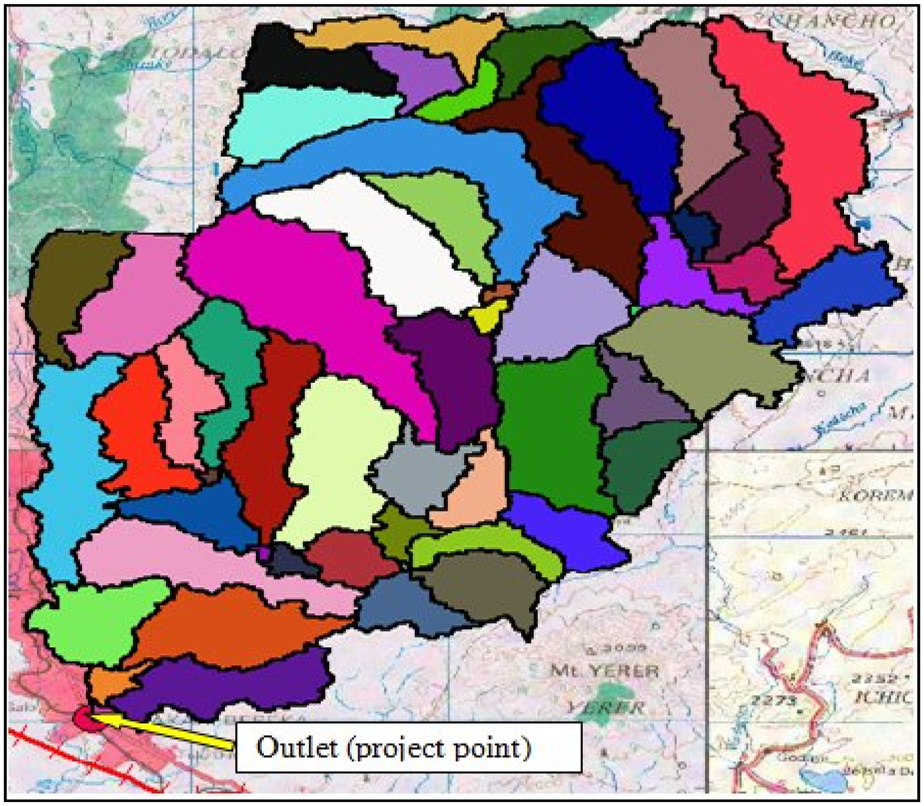


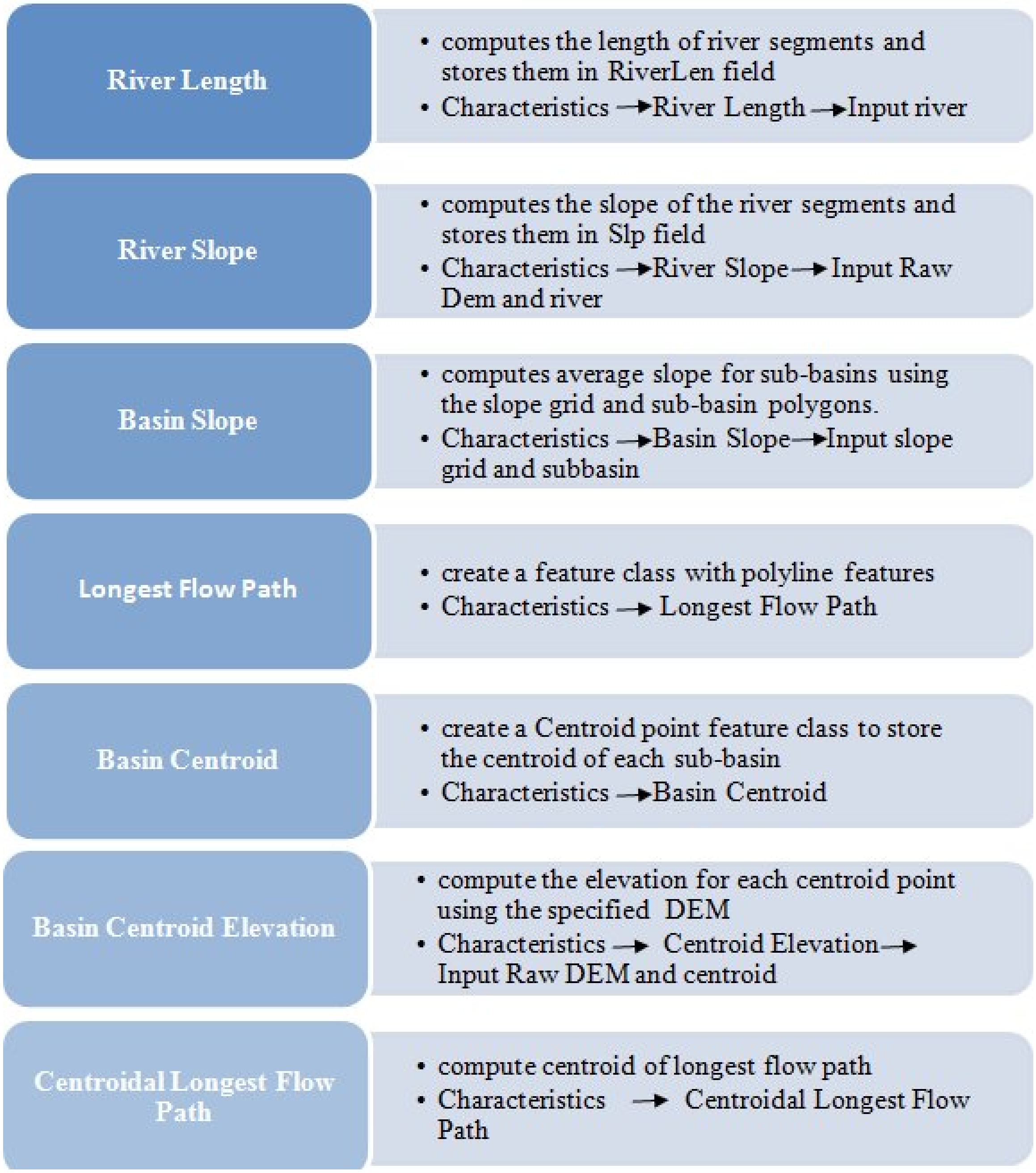
Figure 3-3: Project Point and project area with sub-basins

Project point of this study was selected at existing Bridge of Road from Addis Ababa to Adama at where river gauging device exists. The purpose of choosing this location is that at this location there is a gage station with flow gage data, to calibrate hydrologic parameters for observed flow data with determined flow by hydrologic modeling. Peak flow was determined at new railway crossing by adding additional flow from downstream area of project point to railway crossing.

3.2.2.4. Extracting Basin Characteristics

In the basin characteristics, the HEC-GeoHMS provide tools for extracting physical characteristics of streams and sub-basins into attribute tables.

These topographic characteristics extracted from Hec- Geo HMS are listed below.



River Length	• computes the length of river segments and stores them in RiverLen field • Characteristics →River Length →Input river
River Slope	• computes the slope of the river segments and stores them in Slp field • Characteristics →River Slope→ Input Raw Dem and river
Basin Slope	• computes average slope for sub-basins using the slope grid and sub-basin polygons. • Characteristics →Basin Slope→Input slope grid and subbasin
Longest Flow Path	• create a feature class with polyline features • Characteristics → Longest Flow Path
Basin Centroid	• create a Centroid point feature class to store the centroid of each sub-basin • Characteristics →Basin Centroid
Basin Centroid Elevation	• compute the elevation for each centroid point using the specified DEM • Characteristics → Centroid Elevation→ Input Raw DEM and centroid
Centroidal Longest Flow Path	• compute centroid of longest flow path • Characteristics → Centroidal Longest Flow Path

Figure 3-4: Process of extracting physical characteristics of sub basin and rivers

Some of these physical characteristics are stored in attribute table of the features where others like centroid and longest flow path are displayed with the help of points and polyline respectively.

Figure 3-5: Centroid and longest flow path of the watershed

3.2.2.5. HMS Inputs/Parameters Estimation

The HEC-GeoHMS provides tools to estimate hydrologic parameters and methods used in HMS. These parameters include SCS curve number, time of concentration, channel routing coefficients, etc. In HEC-GeoHMS transform (rainfall to runoff) method, channel routing, Loss method and base flow methods can be specified, even though in HEC-HMS can also be assigned.

a. Transform method

Transform method is the process of converting excess rainfall to direct runoff and determine peak flow. There are many methods developed for calculation of the design flood like; rational formula and U.S. Soil Conservation Services (SCS) Unit hydrograph rainfall-runoff models. But their applicability depends mainly on the availability of hydrological data and the size of catchments area. Depending on size of catchments area and availability of hydrological data, this study was used SCS Unit hydrograph rainfall-runoff model.

b. Loss method

The loss rate module in a subbasin is responsible for simulating how much precipitation infiltrates into the ground and how much remains on the ground surface. The deficit constant method, Gridded deficit constant, green ampt method, Initial constant, SCS curve number etc are some method to determine quantity of precipitation in filtered to the ground. All methods require different hydrologic parameters. Depending on available data and easily accessibility of parameters, SCS curve number was used for this study to determine amount of loss.

c. Routing method

The routing method handles movement of the water in the reach. The muskingum method is popular and relatively simple to use is used for this case. The muskingum cunge method is similar to the plain muskingum method; however it uses a measured cross section and channel properties to determine the routing coefficients. The lag method includes no attenuation and simply delays the water travelling through the reach by a certain amount of time.

d. Base flow methods

Base flow is the sustained runoff prior to rainfall that was temporally stored in the watershed. There are different method to determine base flow quantities and parameters. Constant monthly, bounded recession, linear recession are the methods. Recession method was selected for this study depending on available information.

3.2.2.6. Creating HEC- HMS project

HEC-GeoHMS provides hydrologic input for HEC- HMS like background shape file, basin model file and metrological model file. The process is as follow.

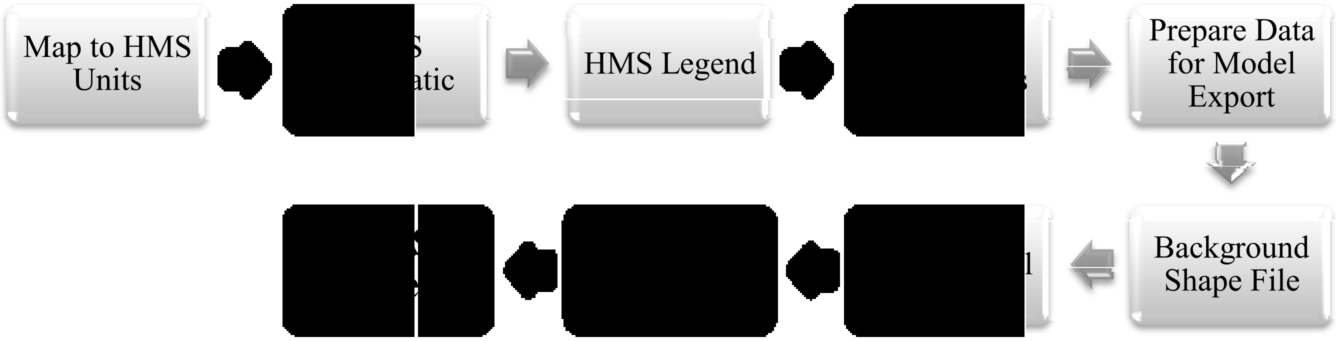


Figure 3-6: Process of creating HEC HMS project using HEC- Geo HMS

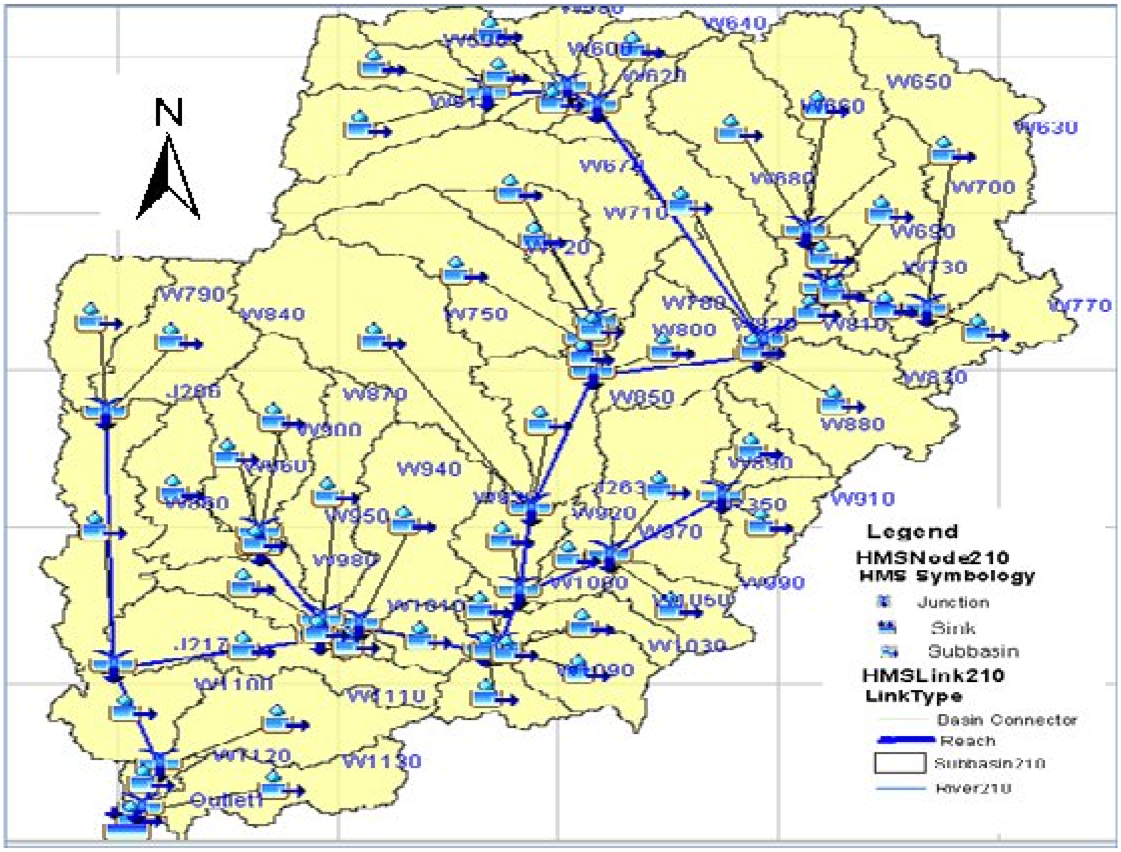


Figure 3-7: Basin model created.

HEC-HMS has different components like basin model, metrological model, control specification etc, to determine hydrologic parameters that determined later in hydrologic analysis. In hydrologic analysis the following methodology was followed.

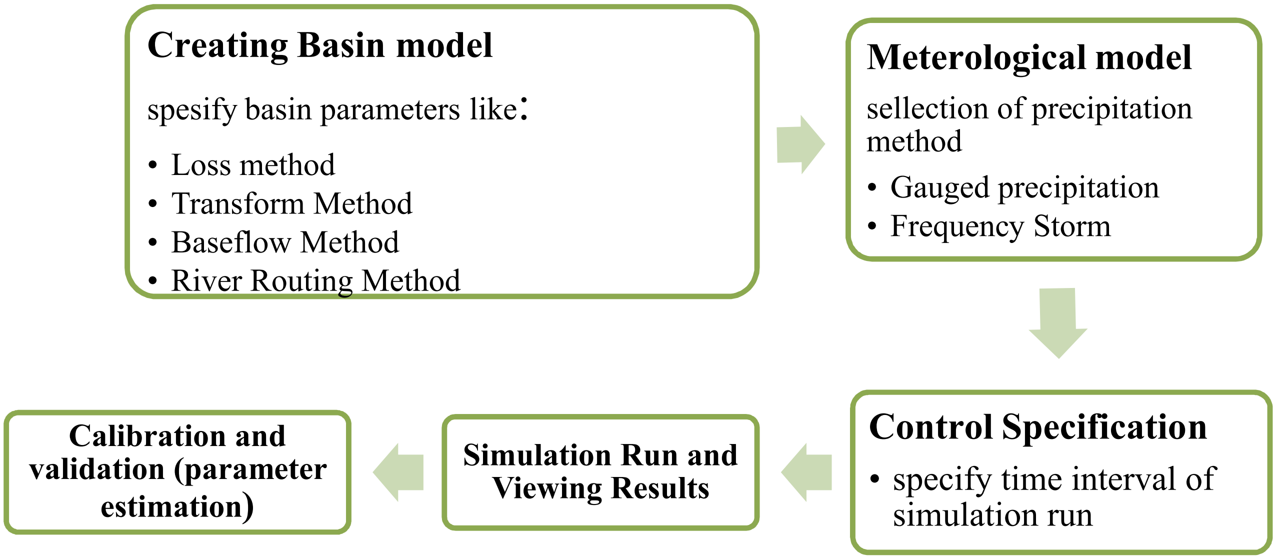


Figure 3-8: HEC- HMS process

3.2.3. Rain fall (Precipitation) methods

Rainfall is the most common factor used to predict design discharge. Precipitation gauge were collected from the Ethiopian Meteorological Services Agency (EMSA) of Bole station, Observatory and Akaki station. Monthly total and daily maximum rainfall data is available at this station.

The data from these gauging stations have been used to develop meteorological model. Meteorological model calculates precipitation inputs for subbasins. It uses both Historical methods: like Gage weights, Inverse distance, User-specified hyetograph, Gridded precipitation and Hypothetical methods like Frequency storm, SCS storm, Standard project storm calculation. The gage weights method uses multiple gages and any weights specified. Normal sources of weights are thiessen polygons or arithmetic mean methods. The inverse distance method also uses multiple gages but the weights are always inverse distance squared. This was originally designed for real-time forecasting use. The user specified method, selects a time-series gage for each subbasin and optionally a total storm depth override. The gridded method uses radar rainfall or other sources of gridded data.

Thus for this study gage weight method and frequency storm was used from both historical method and hypothetical method respectively. Thiessen polygons method was used to determine gage weight factor for mean areal precipitation computation. It is based on the assumption that the precipitation depth at any point within the watershed is the same as the precipitation depth at the nearest gage in or near the watershed. The method of constructing the polygons implies the following steps.

Adjacent stations are connected with lines. Perpendicular bisectors of each line are constructed (perpendicular line at the midpoint of each line connecting two stations). The bisectors are extended and used to form the polygon around each gauge station. Rainfall value for each gauge station is multiplied by the area of each polygon.

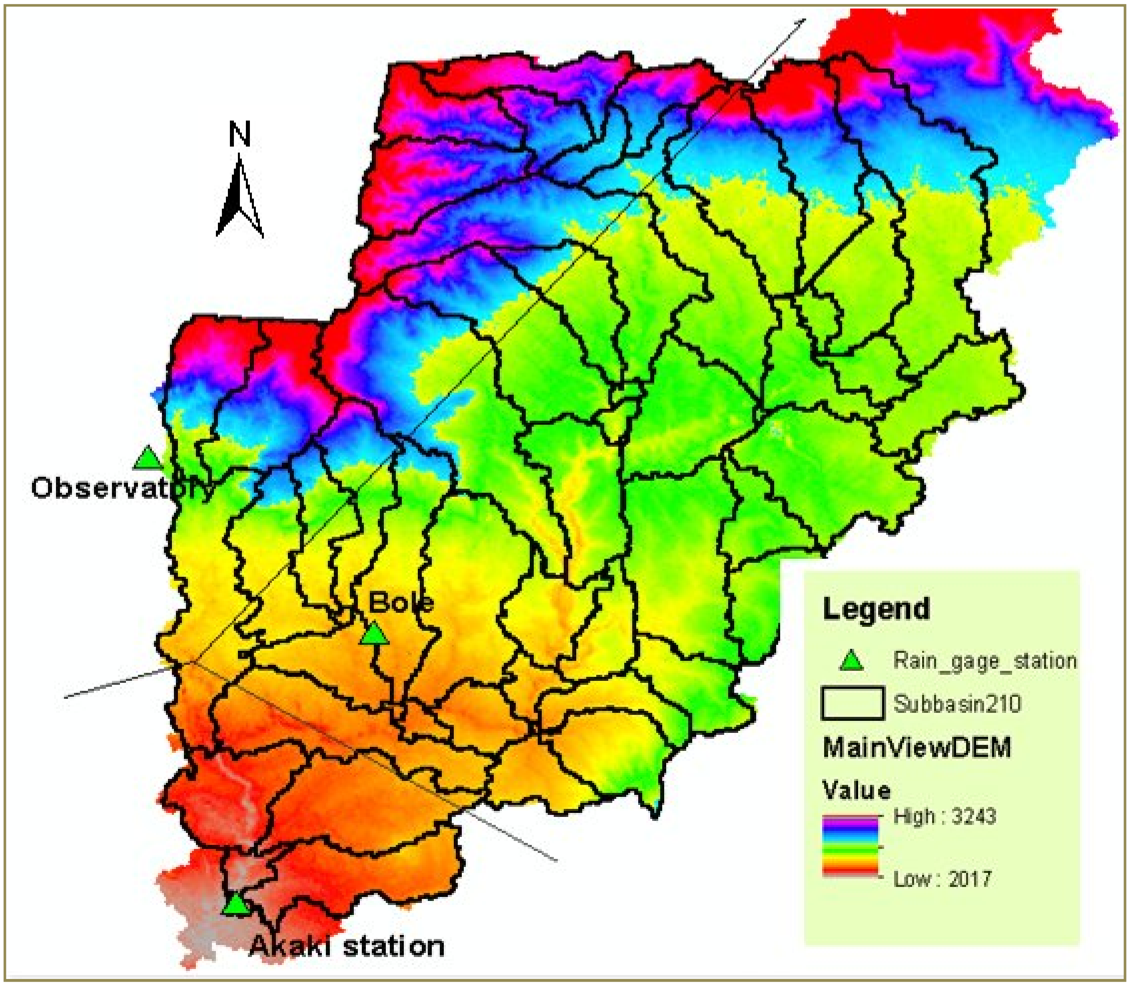


Figure 3-9: Thiessen polygon of the catchment

3.2.4. Flood Frequency methods

The stream flow data of Akaki River was acquired from Ministry of Water, Irrigation and Energy (MoWIE) to estimate design flood at new railway crossing site. Gumbel (Extreme Value I) and Log-Pearson III methods were used to estimate the design flood.

I) Gumbel (Extreme Value Type I)

According to Chow (1982), the following expression is used for the computation of design flood using Gumbel method.

$$= \quad +$$

Where, **K_{T}=** frequency factor for each return period,

S = standard deviation of stream flow

X= mean of stream flow

X_T= design flood flow for a given return period.

But, K_T which is the frequency factor for return period, was computed for corresponding return periods using Gumbel's distribution as given by the expression;

$$= -\frac{\sqrt{6}}{\quad} \; 0.57721 + \ln \quad \ln \frac{T}{T-1} \qquad \text{Where T=Return period}$$

II) Log-Pearson III methods

The Log Pearson III method is also used for the computation of design flood frequency in addition Gumbel (extreme value) Method. It is the most reliable method for stream gage data of at least 25 years. However, for this case stream gauge data is 24 year which is near to the minimum requirement of the method that was used for comparison purpose. The method defined by three standard statistical parameters: the mean, standard deviation and coefficient of skew. Formulas for the computation of these parameters given below:

Log Q =Avg(log Q) +KS

Where k=frequency factor,

s= Standard deviation

The frequency factor k depends return period and the skew coefficient where its value was determined from table Bulletin#17 (1982), see Appendix A

3.3. Hydraulic Modelling

The main objective of hydraulic modeling is to check hydraulic structure size on stream cross section, whether it can safely pass peak discharge estimated during hydrologic analysis of a given watershed. Bridges are one of hydraulic structure that has to be designed hydraulically to accommodate the peak flood without excessive restricting the flow of the stream or incurring damage either to the structure or the surround land. Manning's Formula, HEC RAS, ISIS and Hy8 programs are used to design or check cross hydraulic structures. Hydraulic modeling and analysis of this study was simulated by HEC RAS software application.

3.3.1. Hydraulic modelling using HEC-RAS

HEC- RAS is an integrated package of hydraulic analysis program that performs steady and unsteady flow of water surface profile calculation.

3.3.1.1. Basic data required for HEC-RAs

a. Geometric Data

Geometric data usually shows physical characteristic like; cross section (profile) and slope of the channel

b. Cross-Section Data (Channel shape)

Cross sections are defined by Station(x) and elevation (y) in the plane of the cross section perpendicular to the flow. It is specified in terms of ground surface profiles and the measured distance between them (reach lengths) for the analysis of flow in natural stream. Cross sections are located at intervals along a stream to characterize the flow carrying capability of the stream and its adjacent floodplain. Channel cross section data of Akaki River were collected from ERC, is presented on appendix E

c. Stream slope

The channel slope can be determined from elevation difference between successive cross sections data for further generation of cross section data at different station. Two cross section data at 100m reach lengths is given. The reach lengths for the left overbank, right over bank and channel are specified on the cross section data.

d. Structure specifications

Hydraulic structure data's (bridge deck, abutment, piers etc) are also considered geometric data's. A new railway bridge on Akaki River has clear span length of 163.84m (32.81m+32.74m+32.74m+32.74m+32.81m), slopped by 1.2% to one side, was used for analysis. Structure specification like width of the road deck, deck elevations (high chord), number of piers and pier width are used to model the bridge by the program. The high chord defines the deck and the low chord defines the waterway opening (or underneath part of bridge).

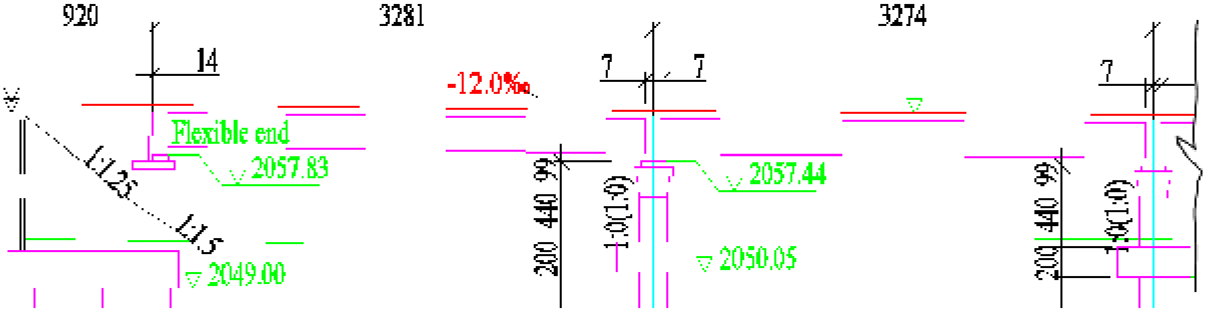


Figure 3-10: Railway Bridge Structural information

e. Flow data (Hydrologic Studies)

Steady flow data are required in order to perform a steady water surface profile calculation steady flow data consists of flow regime, boundary conditions and peak discharge information.

3.3.1.2. Basic Steps in Developing Hydraulic Modeling using HEC-RAS

There are steps that can be used to analyze simple culvert or bridge using geometric and flow data's.

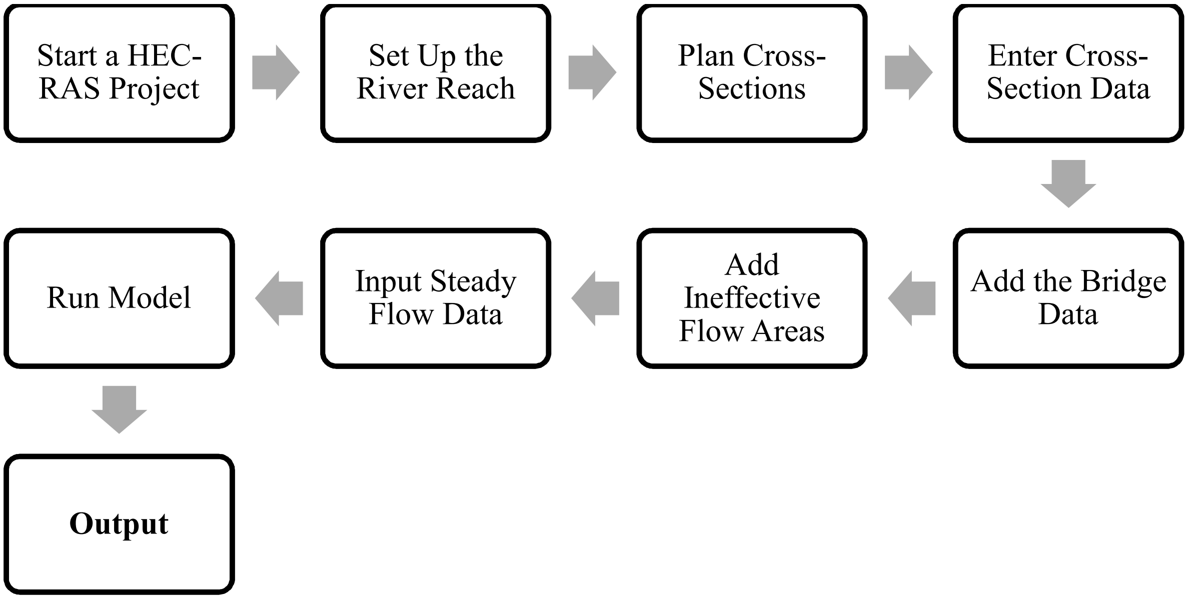


Figure 3-11: Hydraulic modelling process using HEC RAS

4. ANALYSIS

4.1. Hydrologic Analysis

The analysis of the rate and volume of runoff and time distribution of flow is fundamental to the design and check capacity of drainage structures. Underestimation of peak discharge cause under-sized structures and results more drainage problems and on the other hand overestimation of peak discharge results a structure that is oversized and costs more than necessary. Therefore the aim of the hydrologic analysis should be to derive the maximum reliable discharge for a given waterway for a specific design period. The catchments area, slope, soil type, and vegetation, intensity of rainfall and duration of storm are those factors which affect the maximum discharge.

There are many methods developed for calculation of the design flood like; rational formula and U.S. Soil Conservation Services (SCS) Unit hydrograph rainfall-runoff models. But their applicability depends mainly on the availability of hydrological data and the size of catchments area. Depending on size of catchments area and availability of hydrological data, this study was used SCS Unit hydrograph rainfall-runoff model. Frequency analysis method was also used to compute the design flood of gauged Akaki stream. Values of the two methods were computed and compared with design peak flow.

4.1.1. Rainfall-Runoff Analysis

4.1.1.1. Soil Conservation Services (SCS) Unit hydrograph Method

SCS unit hydrograph is the most commonly used rainfall-runoff model for larger catchment area that gives reliable design discharges. SCS model has parameters which depend on climate and morphological condition. These parameters can be determined by different software applications like HEC-Geo HMs and Hec Hms. SCS unit hydrograph analysis of this study was accomplished by HEC-HMS software.

HEC-HMS Models the rainfall-runoff process in a watershed based on watershed physiographic data

- Offers a variety of modeling options in order to compute UH for basin areas.
- Offers a variety of options for flood routing along streams.
- Capable of estimating parameters for calibration of each basin based on comparison of computed data to observed data

Thus, calibrated and validated HEC-HMS model was used to compute peak discharge.

4.1.1.2. Hydrologic Elements

There are different hydrologic elements that can influences the amount of runoff produced from a given watershed. The following hydrologic elements were used to represent basin model of the catchment.

Sub basin – Used for rainfall-runoff computation on a watershed.

Reach – Used to convey (route) stream flow downstream in the basin model.

Junction – Used to combine flows from upstream reaches and sub-basins.

Sink – Used to represent the outlet of the physical watershed

The model of Akaki Catchment contains 114 hydrologic elements. These elements are made up of 57 sub basins, 28 river reaches, 28 junctions, and 1 outlet at the project point.

4.1.1.3. Rainfall - Runoff methods using HEC-HMS program

As specified before, the sub-basin element is used to convert rainfall to runoff. So the information on methods used to compute loss rates, hydrograph transformation and base flow is required for each sub-basin element. All methods have different parameters that determined as follow, but, their actual values were determined after calibration with stream flow.

a. Loss Rate Methods

The loss method allows choosing the process which calculates the rainfall losses absorbed by the ground. There are many methods developed for calculation of the precipitation loss method like deficit constant method, exponential method, green ampt method, initial constant method, SCS curve number method, etc. Their applicability depends on physical characteristics of the watershed and available data. Thus SCS curve number method was used for this study. Soil Conservation Services (SCS) Curve Number (CN) model estimates precipitation excess as a function of cumulative precipitation. It depends on land use; soil cover and antecedent moisture content of the soil. For SCS Curve Number method, each sub-basin requires a value for the Curve Number and percent imperviousness .The following equation is used to estimate SCS parameters.

$$pe = \frac{(P - Ia)^2}{P - Ia + S}$$

Where P_e=cumulative excess precipitation or direct runoff at time t,

P =cumulative precipitation depth at time t,

I_a= Initial abstraction (initial loss)
S= potential maximum retention

The relationship between Ia and S was developed from experimental catchment area data (Ven Te Chow). It removes the necessity for estimating I for common usage. The empirical relationship used in the SCS runoff equation is:

$$I_a = 0.2S$$

This implies,

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

S is related to the soil and cover conditions of the catchment area through the CN.As Technical reference Manual of HEC-HMS, CN has a range of 30 for permissible with high infiltration rate to 100 for water bodies, and S is related to CN by:

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

i. Estimation of Curve Number

CN can be determined as a function of land use, soil type and anticipated moisture of the catchment. The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C, and D). Hydrologic soil groups of the catchment area are dominantly categorized under group B and D as shown below. Land use of the catchment area as known covered mostly by developing urban area. Thus CN of sub basins were obtained from the tables developed by SCS referred to as TR-55.

Ethiopian Road Authority Drainage Design Manual (ERADDM, 2013) recommend that for antecedent moisture conditions (AMC) in Ethiopia, use dry for Region D1, wet for Region B1, and average AMC for all other regions. For this case the catchment area is located under rainfall region A2 (see figure 4.4). Values in tables are based on an average antecedent moisture condition, i.e., soils that are neither very wet nor very dry when the design storm begins. For a watershed with several soil type and land uses representative average curve number, CN was calculated. However, as Arlen D.Feldman, (2000), “for urban district, residential district and newly graded area, table of CN used include composite CN. That is CN shown are composite values of directly connected impervious area and open space. If CN of these land uses are selected no further accounting of directly connected impervious area is required”.

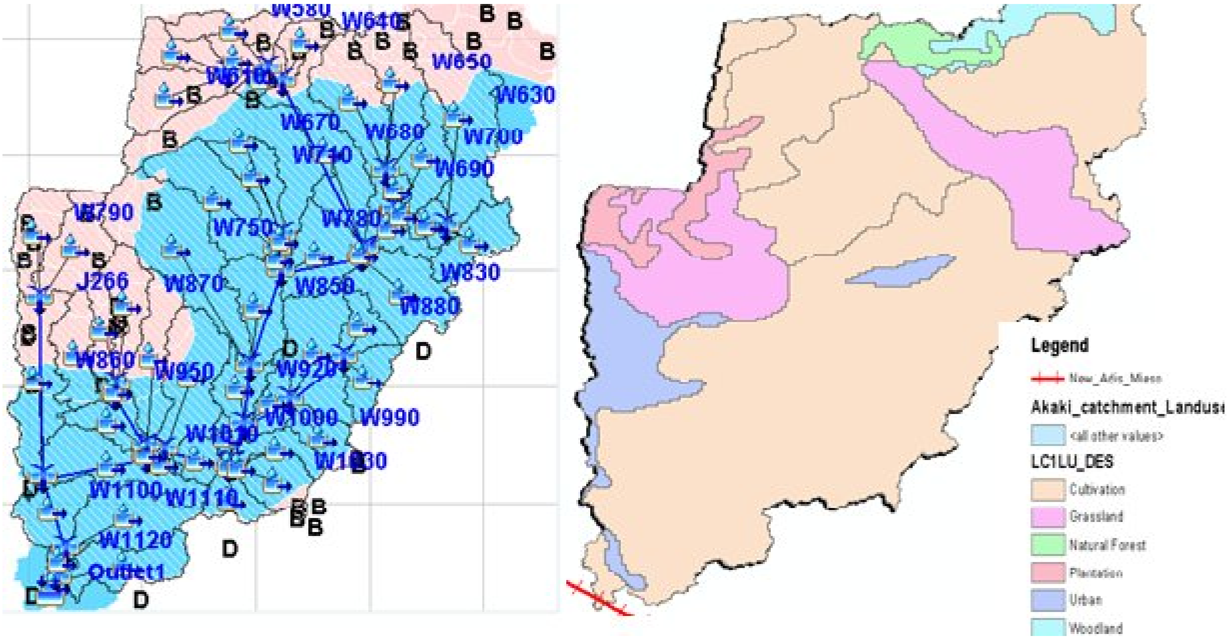


Figure 4-1: Hydrologic soil Group and land use of the catchment

Depending on the above equations loss rate parameters like initial abstraction and Curve number were determined

Table 4-1: CN and Initial abstraction for sample of sub basins

Watershed Name	Watershed ID	soil group	Land use	CN	Weighted CN	S	Ia
W580	**58**	B	DUA	86	86	41.349	8.270
W590	**59**	B	cultivated	72	72	98.778	19.756
W600	**60**	B	DUA	86	86	41.349	8.270
W610	**61**	B	DUA	86	86	41.349	8.270
W620	**62**	BD	DUA	86	86	41.349	8.270
W630	**63**	BD	DUA	94	94	16.213	3.243
W640	**64**	B	Forest	60	60	169.333	33.867
W650	**65**	BD	DUA	86	86	41.349	8.270
W660	**66**	BD	DUA	94	94	16.213	3.243
W670	**67**	D	cultivated	86	86	41.349	8.270
W680	**68**	BD	cultivated	74,86	84.8	45.528	9.106
W690	**69**	D	cultivated	86	86	41.349	8.270
W700	**70**	D	cultivated	86	86	41.349	8.270

Note: I_a = *initial abstraction mm*
S = potential maximum retention, mm
DUA=Developing urban areas
U=Urban districts:
W = subbasin nomenclature

b. Transform Method

The transform method in the subbasin converts excess precipitation into runoff at the subbasin outlet. The clark, scs, and snyder unit hydrographs are some methods used to convert excess rainfall to direct runoff. Depending on available data and catchment area, this study used SCS unit hydrograph method. The SCS unit hydrograph method requires only one parameter for each sub-basin is lag time between rainfall and runoff in the sub-basin

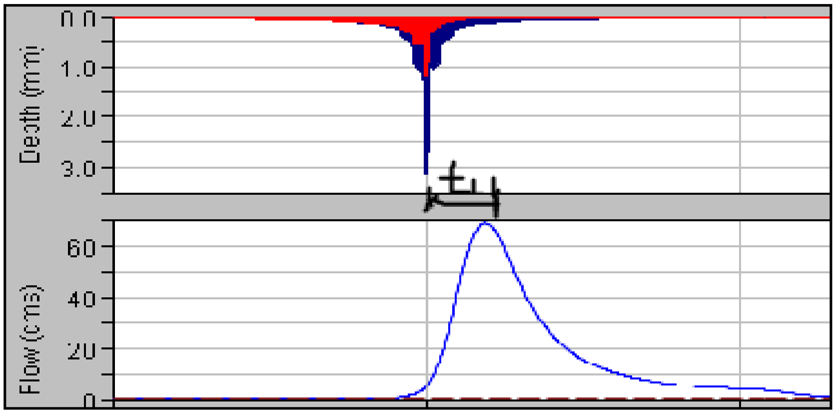


Figure 4-2: Transform method parameter

Lag time is equal to the time between the center of the excess rain and peak of the unit hydrograph. The parameter that is specified here is t_L, is related to *Tc* (time of concentration).

Where, T_{LAG} = 0.6 x (Time of Concentration).

i. Time of Concentration

Time of concentration is estimated according to the Natural Resource Conservation Service (NRCS) TR-55 methodology. To estimate time of concentration information like, hydraulic length of catchment, measured along flow path from the catchment boundary to the outlets of subbasins, Slope of the catchment and roughness coefficient of the channel are required. These parameters were determined previously in hydrologic modeling using HEC-Geo HMS as longest flow path, river length, river slope, basin slope, basin centroid, under extraction of basin characteristics. Time of concentration has Overland Flow, shallow flow and channel flow components which can be estimated mostly from physical characteristic of the catchment. Overland flow travel time and channel flow time of each subbasins were computed as follow.

Time of Concentration for Overland Flow

"Overland flow is the type of flow that occurs in small, flat or in upper reaches of catchments, where there is no clearly defined watercourse. Run-off, then, is in the form of thin layers of water flowing slowly over the fairly uneven ground surface. The kerby formula is recommended for the calculation of Tc in this case. It is only applicable to parts where the slope is fairly even" (*ERA DDM 2013*)

$$c= 0.604\,\frac{rL}{.}\;\;^{.}$$

Where Tc= time of concentration (hours)

r = roughness coefficient, which depends on land use of subbasin was obtained from ERA DDM 2013

L=hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km)

S=Slope of the catchment

Time of Concentration for channel flow

The recommended empirical formula for calculating the time of concentration in natural channels was developed by the US Soil Conservation Service as follow.

$$=\frac{.}{\qquad\qquad}\;\;^{.}$$

Where:

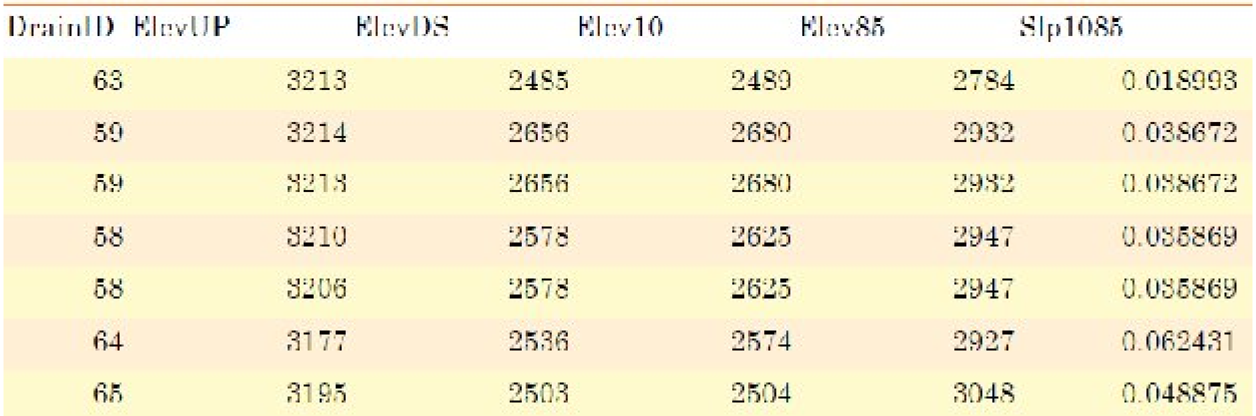
Tc = time of concentration (hours).

L = hydraulic length of catchments measured along flow path from the catchment boundary to the point where the flood needs to be determined (km).

Sav= average slope (m/m).

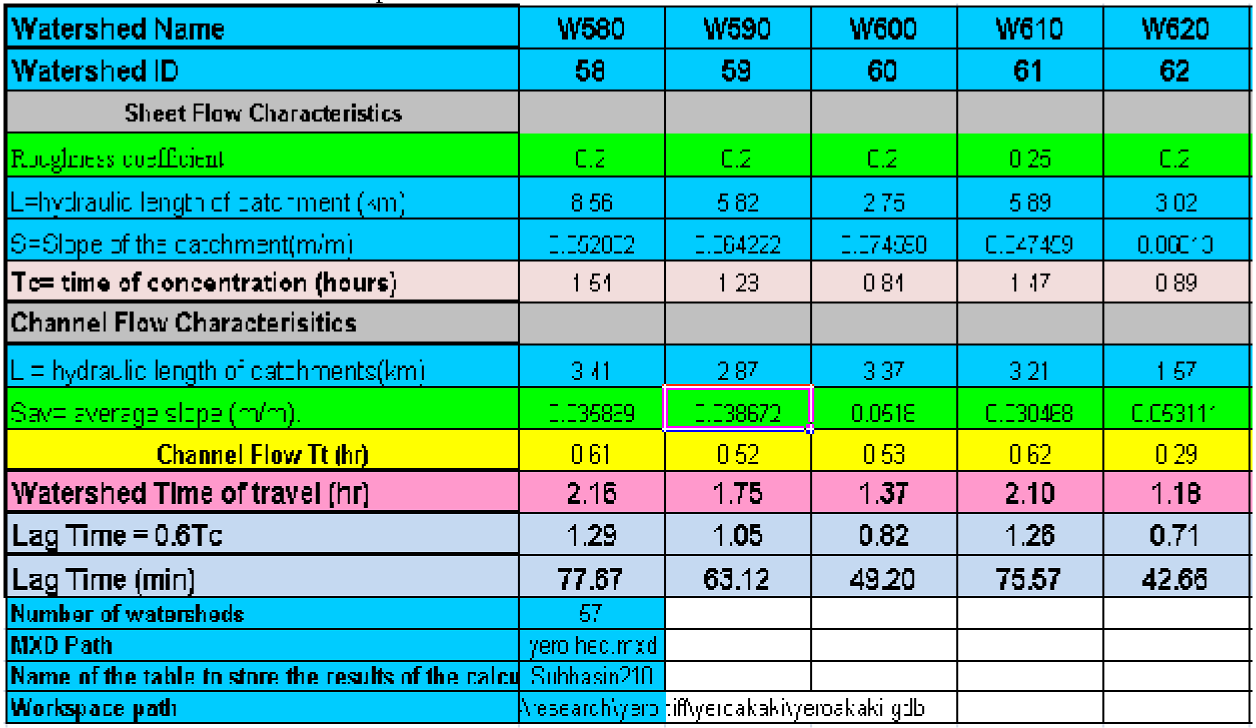
The average slope was determined according to the US Geological Survey referred to as the 1085-slope using HEC-Geo HMS in longest flow path attribute table

Table 4-2: Average slope of flow path using HEC-Geo HMS



DrainID	ElevUP	ElevDS	Elev10	Elev85	Slp1085
63	3213	2485	2489	2784	0.018993
59	3214	2656	2680	2932	0.038672
59	3213	2656	2680	2932	0.038672
58	3210	2578	2625	2947	0.035869
58	3206	2578	2625	2947	0.035869
64	3177	2536	2574	2927	0.062431
65	3195	2503	2504	3048	0.048875

Table 4-3: Time of concentration and lag time according to TR-55 methodology for sample of sub basins



Watershed Name	W580	W590	W600	W610	W620
Watershed ID	58	59	60	61	62
Sheet Flow Characteristics					
Roughness coefficient	0.2	0.2	0.2	0.25	0.2
L=hydraulic length of catchment (km)	8.56	5.82	2.75	5.89	3.02
S=Slope of the catchment(m/m)	0.052022	0.064222	0.074050	0.047409	0.00010
Tc= time of concentration (hours)	1.54	1.23	0.84	1.47	0.89
Channel Flow Characterisitics					
L = hydraulic length of catchments(km)	3.41	2.87	3.37	3.21	1.57
Sav= average slope (m/m).	0.035829	0.038672	0.0516	0.030468	0.05311
Channel Flow Tt (hr)	0.61	0.52	0.53	0.62	0.29
Watershed Time of travel (hr)	2.16	1.75	1.37	2.10	1.18
Lag Time = 0.6Tc	1.29	1.05	0.82	1.26	0.71
Lag Time (min)	77.67	63.12	49.20	75.57	42.66
Number of watersheds	57				
MXD Path	yero hec.mxd				
Name of the table to store the results of the calcu	Subbasin210				
Workspace path	\esearch\yero	iff\yer\cakaki\yeroakaki.gdb			

c. Baseflow Method

Base flow is existing flow runoff prior to rainfall that was temporally stored in the watershed. As observed flow data of Akaki river shows, there is a flow quantity throughout the year. “Base flow of big Akaki River comes from groundwater discharge”, (*Tesfay Alema, march2009).* There are different models to determine base flow quantities. Constant monthly, bounded recession, linear recession, nonlinear boussinesq are models developed in HEC-HMS. The monthly constant is simply an average flow rate to use for each month of the year. The nonlinear boussinesq method starts from an initial value and then recedes exponentially; it resets after each storm event. The recession method also starts from an initial value and then recedes exponentially which resets after each storm event. The difference between nonlinear boussinesq and recession is that in the recession the controlling parameter is simply a ratio entered where as nonlinear boussinesq method uses the drainable porosity, conductivity, and characteristic flow length to compute the required ratios. Exponential Recession Model was selected for this study as the ground water recharged during rainy season. Recession Model requires initial base flow and recession constant. Base flow at any time t, can be determined as, (Chow)

$$=$$

Where, Q_t = Base flow at any time t

Q_0 =Initial base flow

K=an exponential decay constant which defines the ratio of base flow at time t to base flow one day before.

Typical value of recession constant proposed by Pilgrim and Cordery (1992) specified on Technical Reference Manual of HEC-HMS depends on flow condition.

Table 4-4: Recession constant

Flow condition	Recession constant, Daily
Groundwater	0.95
Interflow	0.8-0.9
Surface runoff	0.3-0.8

Source: (Arlen D.Feldman, 2000)

Initial base flow which can be specified in m^3/s or $m^3/s/km$ was determined from mean stream flow hydrograph of observed flow data of Akaki River. From different base flow separation method straight line method was used to estimate initial base flow which is equal to dry season mean flow. According to *Tesfaye Alema groundwater flow analysis,* base flow to the river from groundwater during dry season is about 252912m3/day. That is similar with average observed flow hydrograph as shown below.

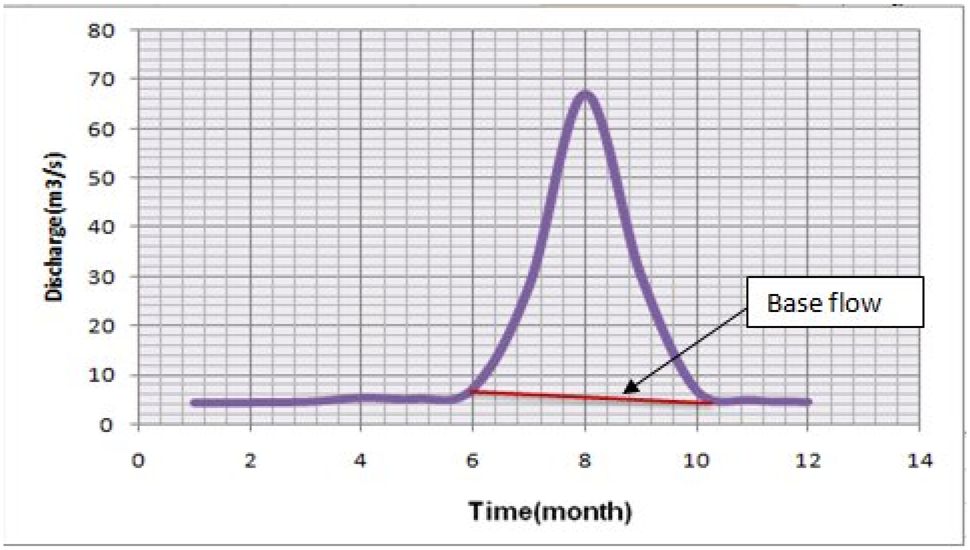


Figure 4-3: Mean stream flow hydrograph of Akaki River (1981-2004)

d. River Routing

Routing is the process of predicting temporal and spatial variation of a flood wave as it travels through a river (or channel) reach or reservoir

Runoff movement through river reaches is simulated by flood routing. Most of the flood-routing methods available in HEC-HMS are based on the continuity equation and some relationship between flows and storage. From different models available in HEC-HMS, Muskingum method was used for this study as it is more popular and simple river routing mechanism. Muskingum method is based on continuity equation;

$$S = K\,[XI + (1-X)\,O]$$

Where S= storage in the river reach

K = storage time constant (hr)

X =weighting factor which varies between 0-0.5. In natural streams, X is between 0 and 0.3 with a mean value near 0.2

I= inflow

O= outflow

The model requires storage time constant (k) and weighting factor (x) parameters to rout flow through reaches.

i. Estimation of K, X

Actually Muskingum routing parameters, k and x can be determined if there is observed inflow and outflow data for each reach are available. For in case of such data are not available, feasible range of parameter value was used and finally actual value calibrated with observed outflow by the model. According to Ven Te *chow*, feasible value of X in natural streams ranges between 0 and 0.3 with a mean value near 0.2 is used for this case.

Initial value of k can be determined as,

$= 0.6—$ Where L= length of river reach (km)

V_{av} = average flow velocity in reach (m/s)

As constraints $K < t_p/5$ where, t_p = is time to peak, is time interval between excess rain fall and peak flow. $t_p = 0.5D + 0.6t_c$, where D=Excess rainfall duration (hr), t_c= time of concentration in reaches (hr)

e. Precipitation

There are different methods used to determine precipitation in puts to each subbasin in watershed. As specified under hydrological model, there are Historical methods like Gage weights, Inverse distance, User-specified, Gridded and Hypothetical methods like; Frequency storm, SCS storm, Standard project storm. For this case gauge weight precipitation was used to calibrate and validate the HEC-HMS model, whereas, frequency storm precipitation was used to determine peak discharge.

i. Gauged weight precipitation

Historical rainfall data of the catchment were collected from EMSA of three stations (Bole, Observatory and Akaki). HEC-HMS model has the capability to assign this gauge data with each individual sub-basin or all sub-basins. Gage weights of subbasin were determined by using Thiessen polygon method as specified previously.

ii. Frequency Storm

The frequency storm uses statistical precipitation data to produce peak discharge for different return periods (exceedance probability). The program is equipped to define an event for each precipitation depth for different duration within the storm. The total precipitation depths are specified for selected exceedance probability.

According to ERA Drainage Design Manual, following return periods were considered for the design of:

Side ditch	10 years return period
Culvert, pipe (span < 2m)	10 year return period
Culvert, 2m<span<6m	25 year return period
Short Span bridges 6<span<15m	50 year return period
Medium span bridges 15<span<50m	50 year return period
Long span bridges span >50m	100 year return period

Thus, for this case return period of 100 year was used to check hydraulic structure where as 50 year return period was also used for comparison purpose.

The most source of statistical data used for frequency storm is Intesity Duration Frequency (IDF) curve developed to design any hydraulic structure for highway drainage in Ethiopia. For this case IDF curve developed by *ERA DDM 2013* was used to specify precipitation depth for 1% &2% exceedance probablity i.e 100&50 year return peroid respectively. The ERADDM divided the country into 8 different hydrological (rainfall) regions. The catchment area of this study is located under A2 rainfall region as shown below. Hence Intensity- Frequency – Duration (IDF) Curve of rainfall region A2 is used.

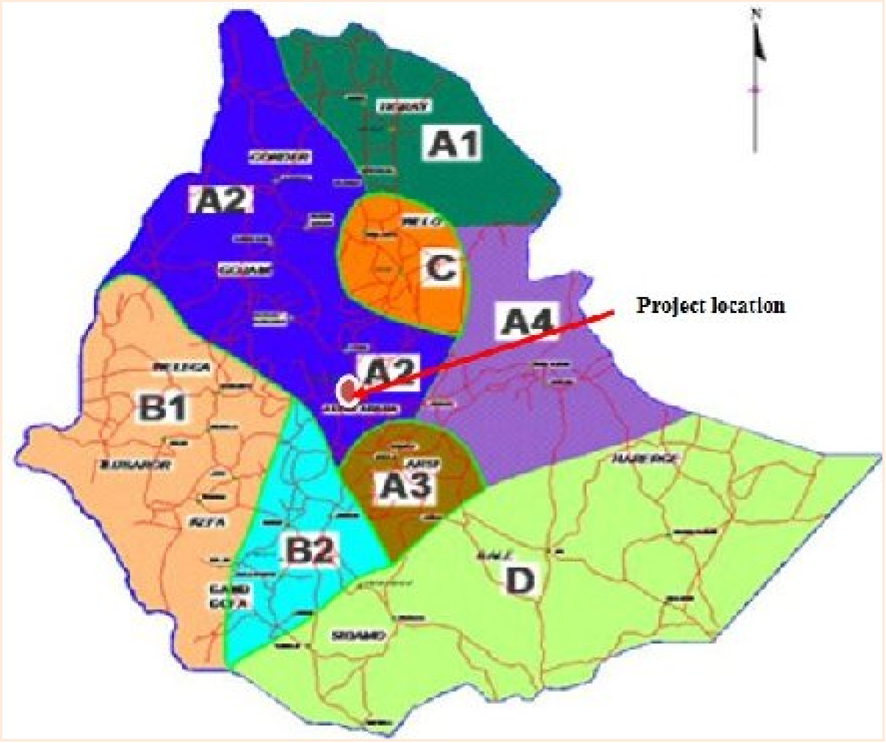


Figure 4-4: Rainfall region of project location (Source: ERADDM- 2013)

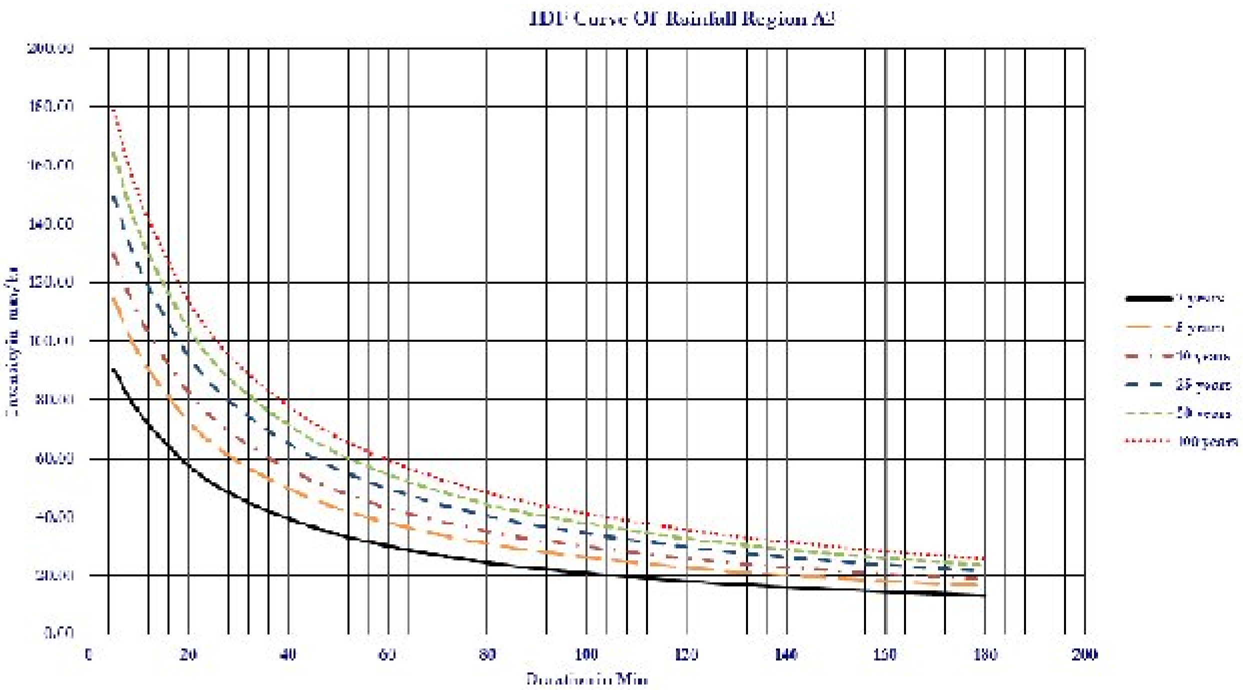


Figure 4-5: IDF Curves of Rainfall Region of A2 (Source: ERADDM- 2013)

Table 4-5: 24hr Rainfall depth (mm) vs. Frequency (yr), (Source: ERADDM- 2013)

Return Period Years	24 hr Rainfall Depth (mm) vs Frequency (yr)							
	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

In order to check capacity of hydraulic structure it is important to know 50 year and100 year return period peak flow. Thus, the 24hr rainfall depth given in table above was used to determine peak flow at two percent or one percent exceedance probability. It was disaggregated to different hours less than 24hr by the following equation. *(ERADDM2013)*

$$R_t / R = (t / 24)\ [(b + 24)^n / (b + t)^n]$$

Where:

Rt/R: Rainfall ratio Rt: R24

Rt: Rainfall in a given duration,'t' (hr)

R24: daily maximum rainfall.

n: constant

b: constant

t: time (hr)

Based on studies of a large number of rainfall gauges in East Africa, the average values of b and n are found to be 0.3 and 0.9 respectively (range of n is 0.78 to 1.09). These values have been adopted for this case.

4.1.1.4. HEC-HMS Model Calibration

The model evaluation procedure includes sensitivity analysis, calibration and validation. The sensitivity analysis of the model was performed to determine the important parameters which needed to be precisely estimated to make accurate prediction of basin yield (D. Roy, et. al. 2013). HEC-HMS has many parameters associated with stream flow calibration. These are sub-basin parameters used in loss method, transform method, base flow method and river routing method as shown in table below. The values of each parameter were initially specified from various watershed and channel characteristics to estimate runoff and routing hydrograph. Their actual values were obtained by trial–and-error method and automatic optimization algorithm built in HEC-HMS with observed flow data.

Table 4-6: Sub-basin parameters

Model	Method used	Parameters
Losses	SCS Curve Number	Initial Abstraction Curve Number % impervious
Transform	SCS Unit Hydrograph	Lag time
Base flow	Recession	Initial Discharge Recession constant Threshold Ratio
Channel Routing	Muskingum	Muskingum k

		Muskingum x

The above listed parameters were used for this study in addition to other parameters like time of concentration (t_c), potential maximum retention of the soil, roughness coefficient, hydraulic length of the channel, average slope of the channel, etc. Most of the parameters, as specified earlier, were estimated from physical characteristics of watershed where as other parameters have no direct physical meaning. These parameters were determined by using available empirical formula and feasible ranges had been taken from existing literatures.

The question is how can the appropriate parameters value be selected? And what types of data be required to get relatively consistence value?

Calibration (optimization) techniques can answer the first question, how can the appropriate parameter value be selected? And stream flow data was required.

Calibration of a model refers to the process that involves adjustment and refinement of parameter values to provide the best match between observed flow data and simulated values of hydrographs for each hydrologic element.

For this case observed stream flow data of Akaki River was collected from Ministry of Water, Irrigation and Energy (MoWIE) for a period of 1/01/1981-31/12/2004 GC. Daily rainfall records of Bole, Observatory and Akaki stations are available for the same. The observations in the time-period (1981 to 1995) were used for calibrating the model and the data from the time-period (1995 to 2004) were used to validate the model.

The automated calibration procedure in HEC-HMS was used an iterative method to minimize an objective function. HEC-HMS program is equipped with the feature that optimizes the parameters following the following process.

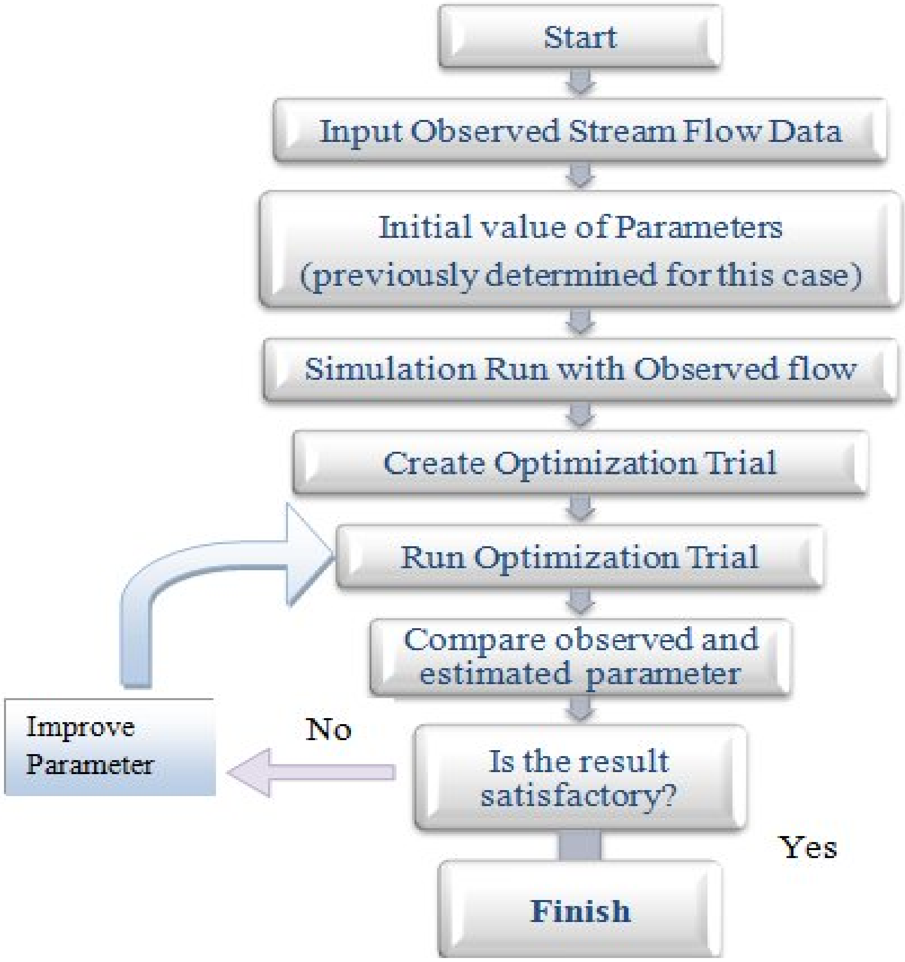


Figure 4-6: General Calibration Procedure (Arlen D.Feldman, 2000)

"The quantitative measure of the goodness-of-fit between computed result from the model and the observed flow is called the **objective function**. An Objective function measures the degree of variation between computed and observed hydrograph," William A.Scharffenberg and Matthew J.Fleming, Aug, (2010). The program uses iterative procedure to minimize objective function by comparing observed flow hydrograph with simulated. Parameter values are adjusted by **search method.** Search method uses the objective function as input to an algorithm that determines how to adjust parameter values to find the optimum match. According to Scharffenberg et al. (2010), "there are two search method used to minimize objective function and find optimum parameter value. The univariate gradient method and Nelder and Mead methods are used. The Univariate gradient method evaluates and adjusts one parameter at a time while other parameters remain constant. It is used if only one parameter is selected. A Nelder and Mead method relies on direct search in which parameters are selected prior to iteration to identify good estimates or to reject bad estimates and to generate batter estimate from the pattern established by the good. It uses a downhill simplex to evaluate all parameters simultaneously and find which parameter to adjust". There are many different parameters were used in this study that Nelder and Mead method is more applicable to adjust all parameters at the same time.

a. Calibration Method

There are seven functions used in the program to measure overall average difference (objective function) between simulated and observed flow *(Arlen D.Feldman, 2000*). These are peak-weighted root square error, sum of squared residual, sum of absolute residual, percent error in volume, percent error in peak, root-mean-square log error and time weighted error. Form these methods this study uses Peak-weighted Root-Square-Error method that compares all ordinates, squaring differences and weight the squared differences. "The weight assigned to each ordinate is proportional to the magnitude of ordinate. Ordinates greater than the mean of the observed hydrograph are assigned a weight greater than one and the smaller, a weight less than one" *(Arlen D.Feldman, 2000*).

The following Peak-weighted Root-Square-Error equation is used in the program to determine goodness- of- fit between observed and simulated hydrograph.(USACE 1998)

$$= \frac{1}{\ } \qquad (\ ) - \ \ (\ ) \qquad \frac{(\ ) - \ \ (\qquad)}{2 \ \ (\qquad)} \qquad ^{-}$$

Where, Z= objective function

NQ= Number of computed hydrograph ordinates

( )= observed flows

( )= computed flow, calculated with selected set of model parameter

(   )= mean of observed flow.

Parameter values are adjusted by search method and objective function for the target element are recomputed until small objective function value attained or maximum number of iteration is exceeds. HEC-HMS program has an algorithm that uses the above search method to adjust parameter values and to minimize objective function value. In order to compute parameters value by the program the following procedure was followed.

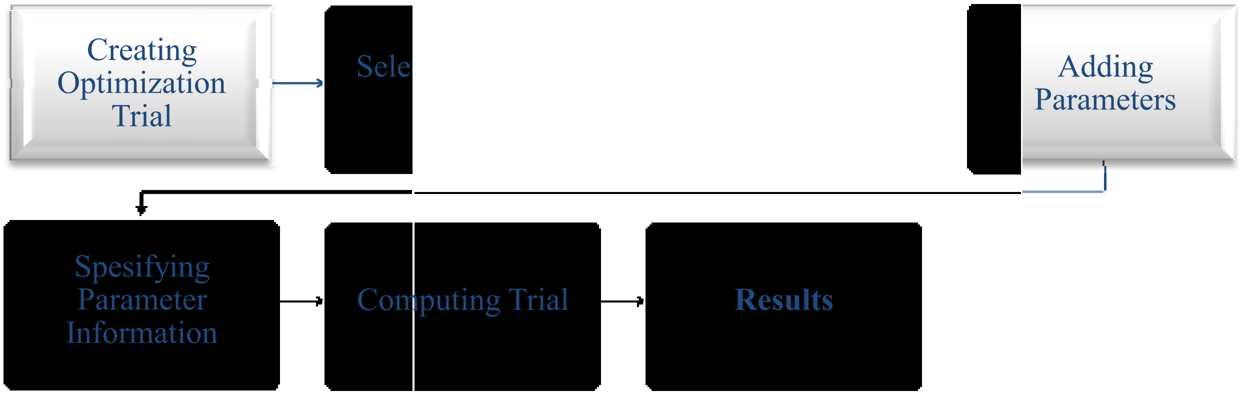
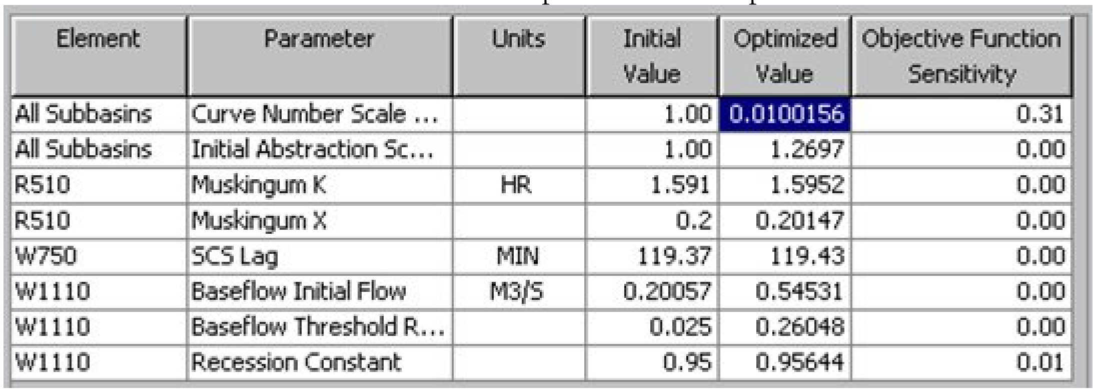


Figure 4-7: Calibration process by HEC-HMS

a. Calibration Results

The effectiveness of calibration was evaluated by comparing simulated peak flow and total volume with measured stream flows. After many trials the following Calibration parameters and calibration results were obtained.

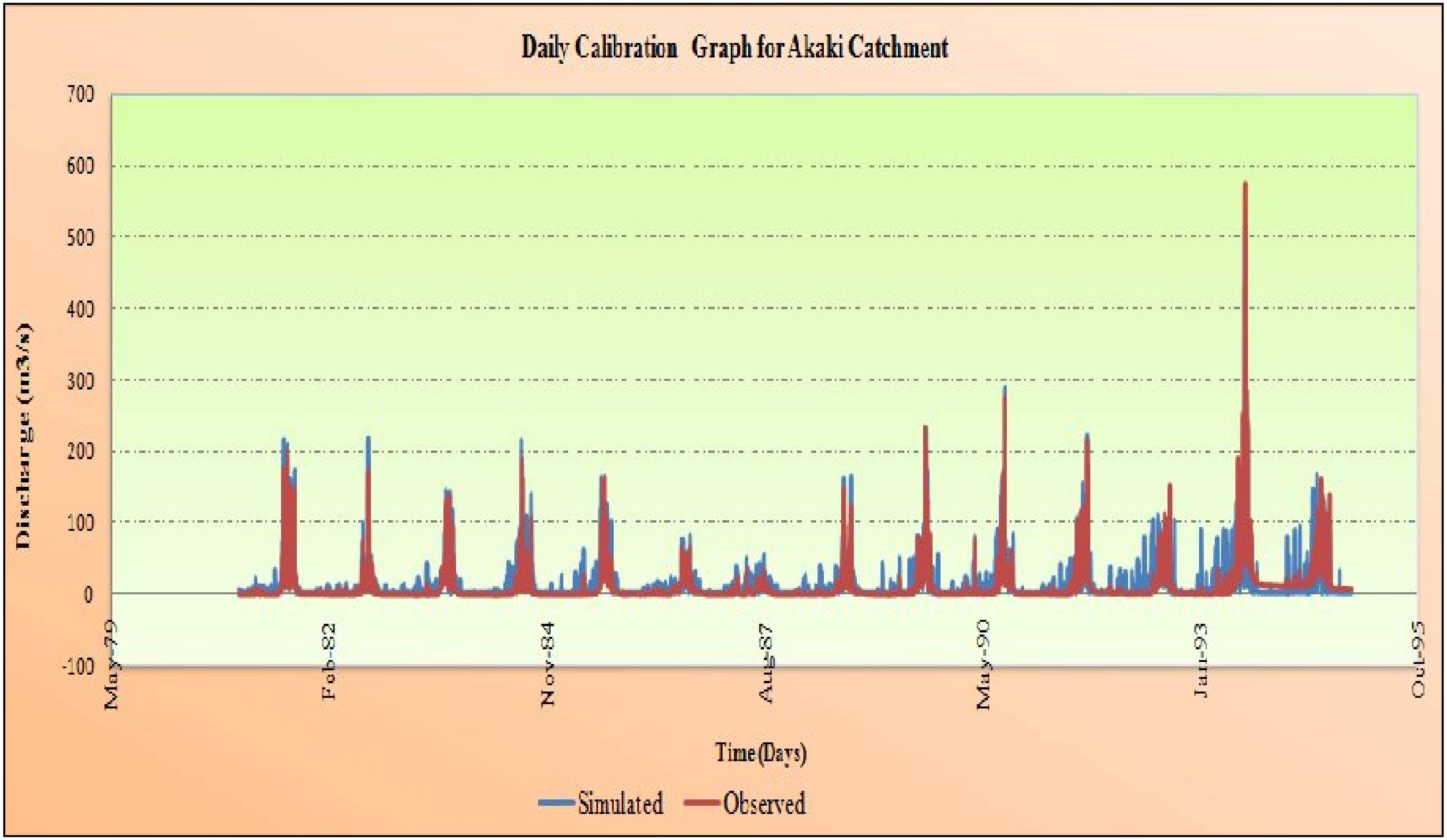
Table 4-7: sample of calibrated parameters



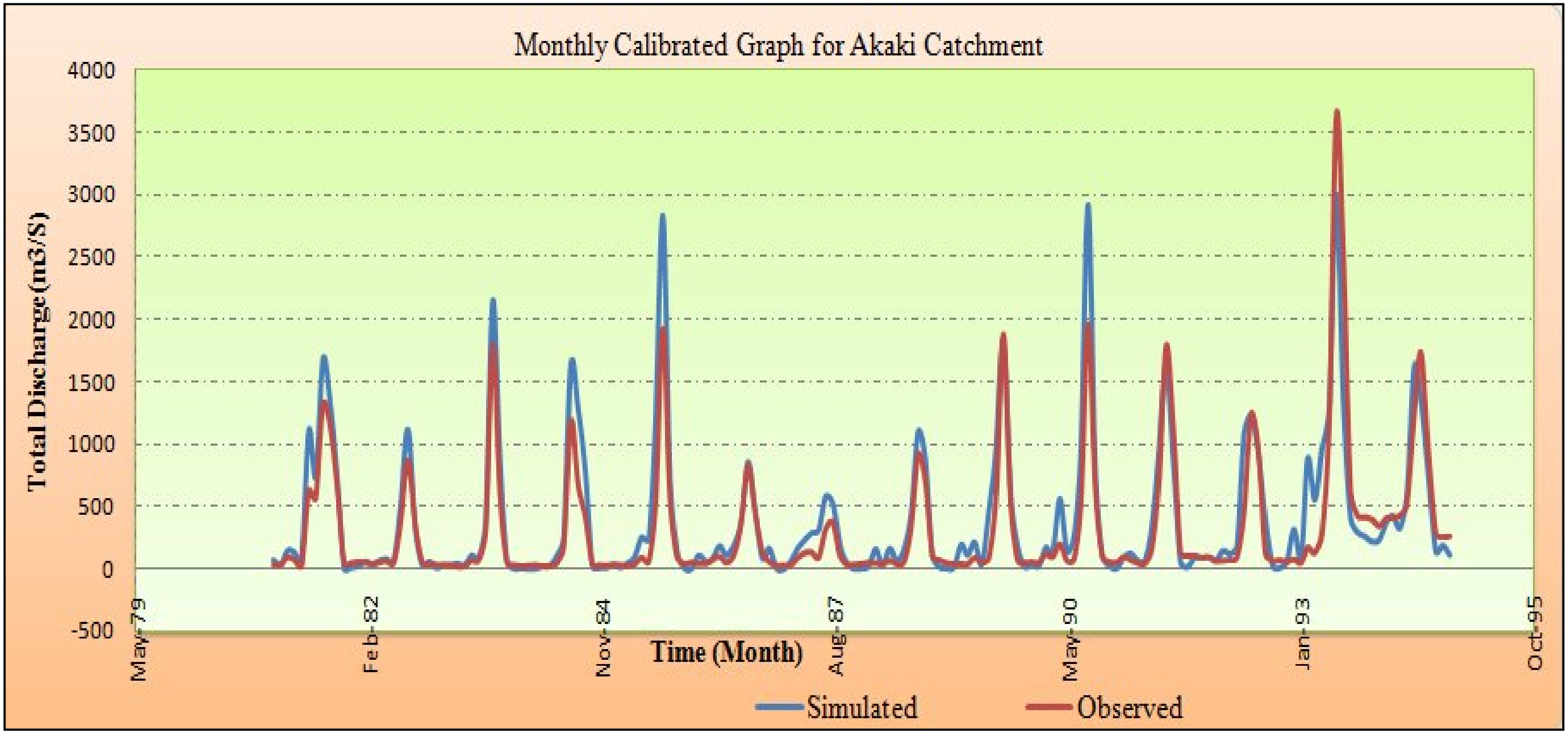
Element	Parameter	Units	Initial Value	Optimized Value	Objective Function Sensitivity
All Subbasins	Curve Number Scale ...		1.00	0.0100156	0.31
All Subbasins	Initial Abstraction Sc...		1.00	1.2697	0.00
R510	Muskingum K	HR	1.591	1.5952	0.00
R510	Muskingum X		0.2	0.20147	0.00
W750	SCS Lag	MIN	119.37	119.43	0.00
W1110	Baseflow Initial Flow	M3/S	0.20057	0.54531	0.00
W1110	Baseflow Threshold R...		0.025	0.26048	0.00
W1110	Recession Constant		0.95	0.95644	0.01

Table 4-8: Calibration result

Measure	Simulated	Observed	Percent difference
Total volume (MCM)	65118.7	52266.3	24.6
Peak Flow (m3/s)	512.1	573.6	-10.7
Time of peak	31Aug1993	31Aug1993	



a. Daily calibration Hydrograph of simulated and observed flow (1981-1995)



b. Monthly calibration Hydrograph of simulated and observed flow (1981-1995).

Figure 4-8: Calibration Hydrograph comparison of simulated and observed flow at outlet

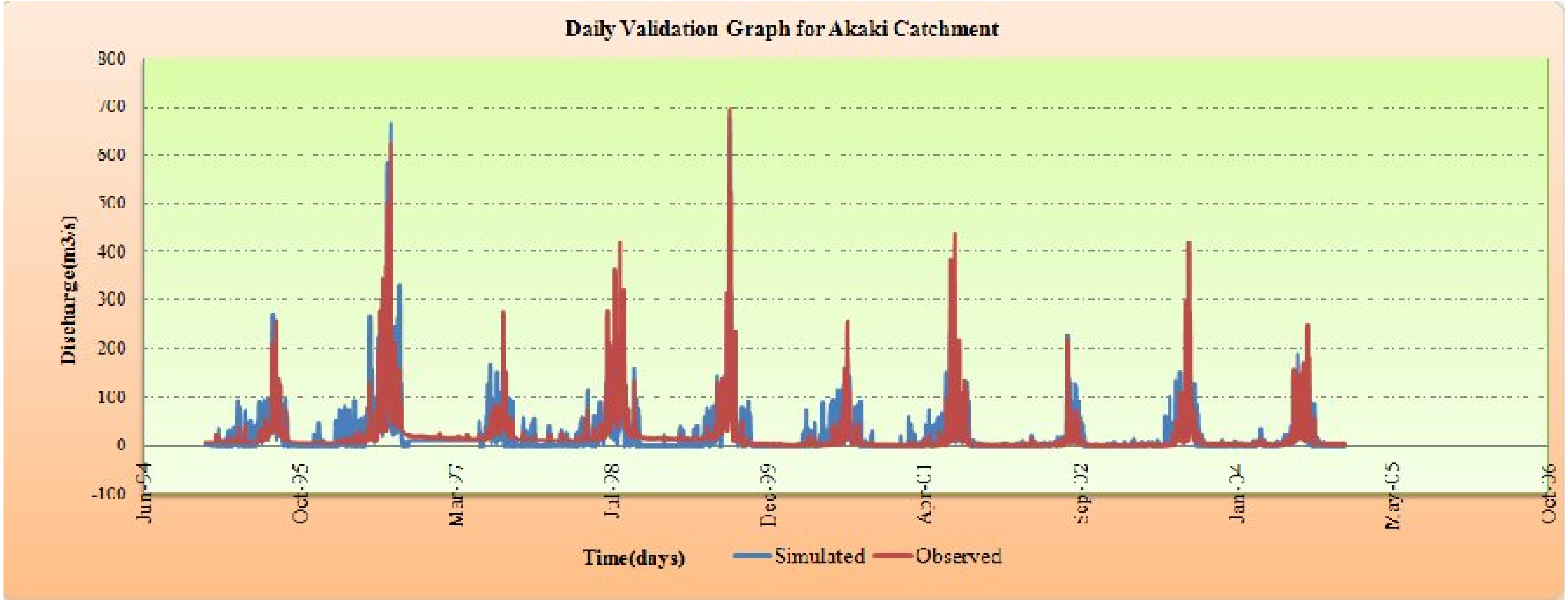
Figures 4.8(a), 3(b) show a time series comparison of simulated and observed stream flows for the watershed for the calibration period of 1981-1995. The peak values of the simulated flow slightly match with the peak values of measured flow

4.1.1.5. Model Validation

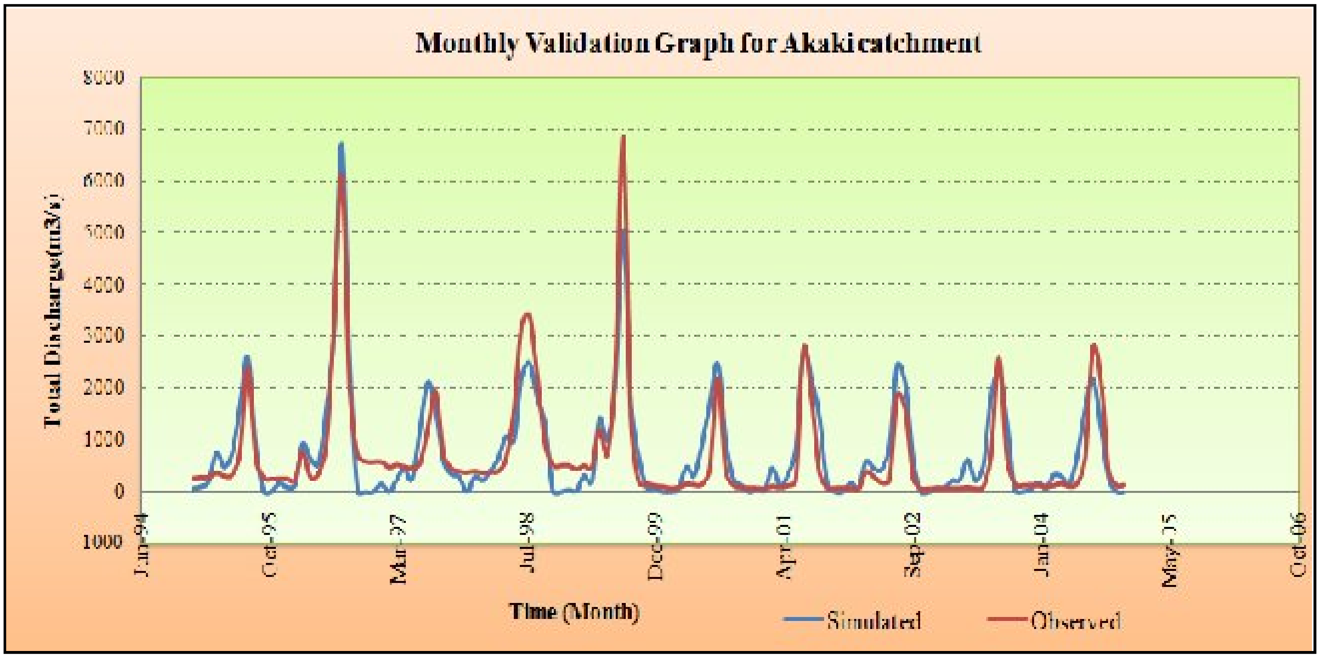
Model validation was used to determine the effectiveness of the calibrated parameters in sub basins to predict the flow discharges. For this study, the calibrated HEC-HMS model was then used to estimate daily stream flow from the sub basins for the period 1/01/1995 - 31/12/2004. The observed and simulated hydrographs are presented on figure 4.9 below.

Table 4-9: Validation result

Measure	Simulated	Observed	Percent difference
Total volume (MCM)	88571.78	74426.2	19.0
Peak Flow (m3/s)	678.1	693.6	-2.2
Time of peak	11Aug1999	11Aug1999	



a. Daily validation Hydrograph of simulated and observed flow (1995-2004)



a. Monthly validation Hydrograph of simulated and observed flow (1995-2004)

Figure 4-9: Validation Hydrograph of simulated and observed flow (1995-2004)

Figures 4.9 (a), (b), shows a time series comparison of simulated and observed stream flows for the watershed for the validation period of 1995-2004. The peak values of the simulated flow also match well with the peak values of measured flow.

4.1.1.6. Model Performance Evaluation

The performance evaluation of hydrologic model is commonly used to know efficiency of the model to provide accurate result through comparisons of simulated and observed variables. According to P. Krause, et.al (2005) the reasons why hydrologists need to evaluate model performance is that, to provide a quantitative estimate of the model's ability to reproduce historic and future watershed behavior and to compare current modeling efforts with previous study results. The process of assessing the performance of a hydrologic model requires both visual inspections and mathematical estimate of the error between the simulated and observed hydrologic variable.

Different efficiency criteria are used to evaluate performance of hydrologic models; such as the Nash-Sutcliffe efficiency, coefficient of determination and volume difference are frequently used in hydrologic modeling. This study adopts all the efficiency criteria for model evaluation were presented below.

a. Nash-Sutcliffe efficiency (NSE)

The efficiency NSE is defined as one minus the sum of the absolute squared differences between the predicted and observed values normalized by the variance of the observed values during the period under investigation. It is given as;

$$\mathrm{NS} \quad = 1 - \frac{\sum_{=1} \; - \;^{2}}{\sum_{=1} \; - \;^{2}}$$

Where, with O observed and P predicted values

The range of NSE lies between 1.0 (perfect fit) and negative infinity.

b. Coefficient of determination R^2

The coefficient of determination r2 is defined as the squared value of the coefficient of correlation according to Bravais-Pearson (P. Krause, et.al, 2005). It is calculated as:

$$= \frac{\sum \; (\; - \;)(\; - \;)}{\overline{(\; - \;)} \;\; \overline{(\; - \;)}}$$

The range of r^2 lies between 0 and 1 which describes how much of the observed dispersion is explained by the prediction. A value of zero means no correlation at all whereas a value of 1 means that the dispersion of the prediction is equal to that of the observation.

c. Percentage error Volume (PEV)

The PEV value measures the deviation between the simulated and the observed volume of stream flow. It is calculated as;

$$= \frac{-}{} \times 100$$

Where *Vo* and *Vc* are the observed and computed runoff volume, respectively.

Using the above efficiency measures, the performance of this study was determined as shown in table below for both calibration and validation.

Table 4-10: Performance measures of the model for the calibration and validation **years**

Model performance for Akaki Catchment			Nash-Sutcliffe Efficiency (NSE) %	Coefficient of Determination (R^2) %	Relative Volume error %
Akaki Catchment	Daily	Calibration	52.6	57.2	24.6
		Validation	48.2	68.7	19.0
	Monthly	Calibration	77.8	83.02	19.5
		Validation	73.5	87.33	21.5

According to Motovilov *et al*., 1999 sited in D. Roy, S. *et al. (2013)*, simulation results are considered to be good for values of NSE greater than or equal to 0.75, while for values of NSE between 0.75 and 0.36 the simulation results are considered to be satisfactory.

As it is shown in table 4.9 above, the performance measures, Nash-Sutcliffe model efficiency values for monthly flow were slightly well for both the calibration and validation years whereas daily flow is reasonably satisfactory. This indicates that there is close agreement between observed and simulated runoff.

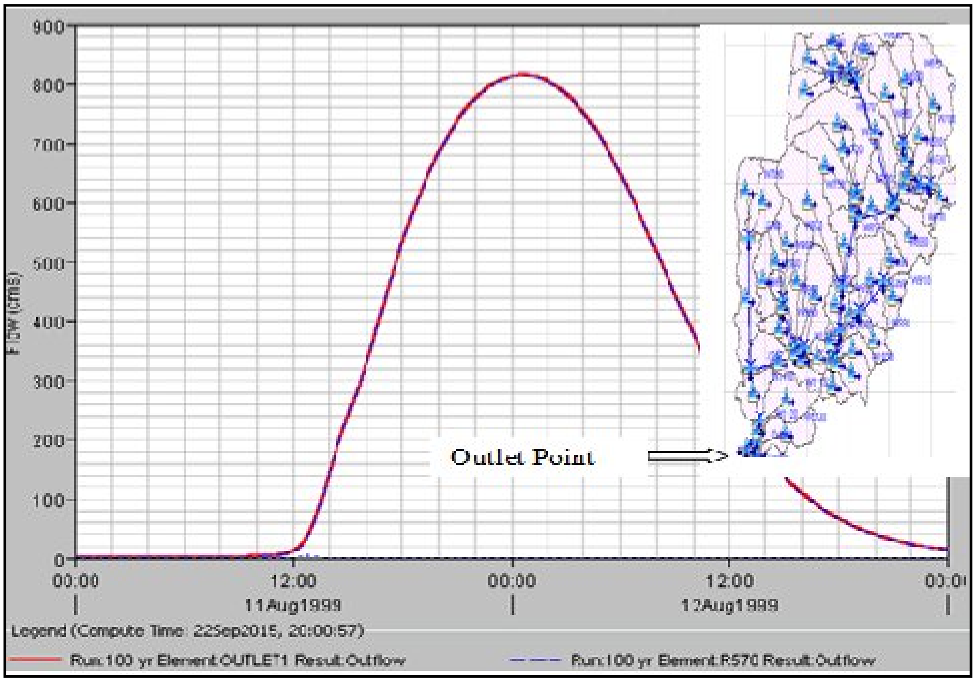
The value of the coefficient of determination r^2 ranges between 57.2-87.33%, which is greater than zero less than one. As described before the value of coefficient of determination r^2 ranges between 1(best fit) and 0 (no correlation between observed and simulated flow). Thus, as shown in table above coefficient of determination indicate that there is good correlation between observed and simulated flows for both the calibration and validation year.

The PEV values for the catchment were found to lie between 19-24.6%. The acceptable level of PEV for hydrologic simulations is ±20 %.(D. Roy, S. *et al. 2013).* Thus, the PEV values of the Akaki catchment are close to acceptable levels of accuracy (±20 %) for simulations models for validation and calibration years.

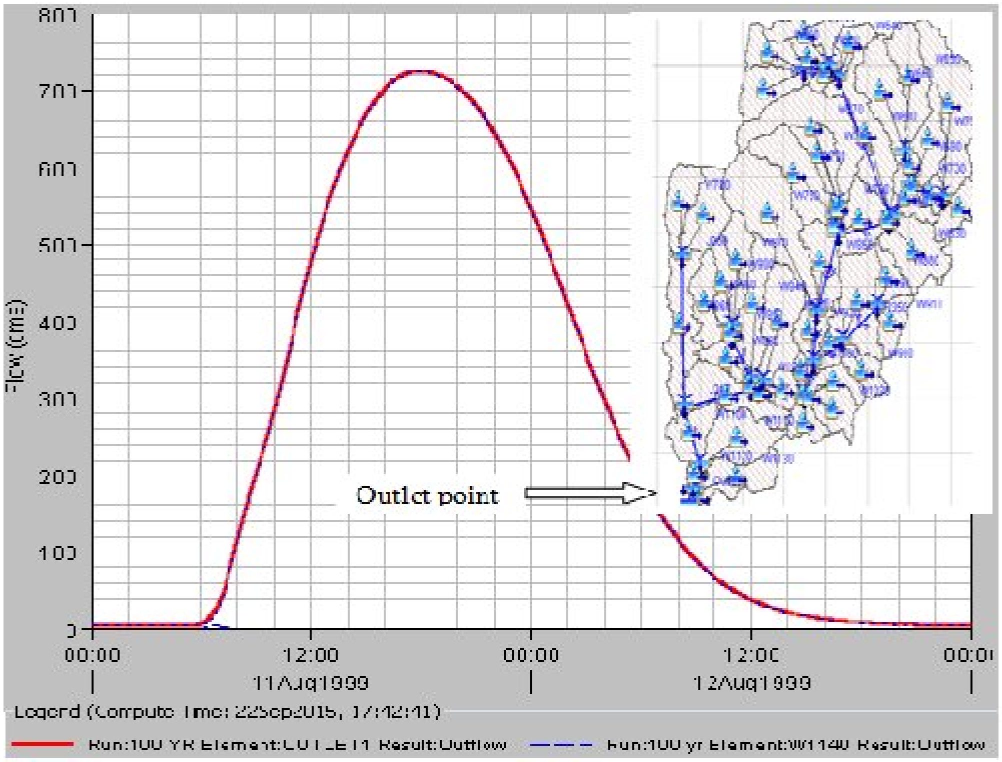
Generally, the performance evaluation shows that, HEC-HMS model developed for Akaki catchment was acceptable and reasonably satisfactory that can now be used for hydrologic analysis.

4.1.1.7. Executing peak discharges using Calibrated and Validated HMS Model.

In previous section calibrated and validated HEC-HMS model was developed for Akaki catchment. Now it could be used to determine peak design discharge using frequency storm precipitation. The frequency storm, which uses statistical precipitation data, was used to produce peak discharge for 50 and 100 return periods (exceedance probability). As specified in section 4.1.1.3, the statistical precipitation data for frequency storm precipitation were adopted from ERADDM 2013 after disaggregated to smaller rainfall duration of 24hr. The result for both 50 and 100 year return periods were found 727.1m3/s and 815.6m3/s respectively at outlet point as show on the following hydrograph figures.



a. Outlet output hydrograph for 1% exceedance probability



b. Outlet output hydrograph for 0.2% exceedance probability

Figure 4-10: a,b Outlet output hydrograph for 1% and 0.2% exceedance probability

4.1.2. Flood Frequency Analysis

The method and procedure which are described in section 2.2.4 are adopted for the design flood analysis i.e. Gumbel (Extreme Value Type I) and Log-Pearson III methods were used to estimate the design flood of Akaki River and compared with other method.

The result of stream flow data analysis by using Log-Pearson III and Gumbel methods are presented in table below. The full analysis procedure is presented on the appendix B and C for both methods.

Table 4-11: Flood Frequency Analysis Result

METHOD OF COMPUATATION	RP	Mean	STDEV	Design Flood
	Year	m^3/s		m3/s
Gumbel (Extreme Value Type I)	50	278.386	163.523	711.826
	100	278.386	163.523	803.7093
Log-Pearson -III Distribution	50	278.386	163.523	685.5362
	100	278.386	163.523	753.6776

The results obtained by SCS unit hydrograph and flood frequency analysis methods were discussed and compared in detail under section 5. The hydrologic analysis results at Railway Bridge were 743.2m^3/s and 835.5m^3/s for 50 and 100 years return period respectively that used to check hydraulic capacity of Akaki Railway Bridge with the help of HEC-RAS program.

4.2. Hydraulic Analysis

Hydraulic analysis of this study was carried out to check adequacy of bridge structure constructed, whether it can safely pass peak flow estimated previously. There are different software programs, like Hec-Ras, ISIS, Hy8 etc. are applicable to design or check adequacy of hydraulic structure. The hydraulic analysis for Akaki River consisted of modelling the flow characteristics using the U.S. Army Corps of Engineers Hydrologic Engineering Center water surface profiling computer program HEC-RAS version 4.1.0 for new railway line. Physical characteristics of the river (cross section), manning's coefficient, contraction and expansion coefficients, ineffective flow area and quantity of flow are important inputs of HEC-RAS program which separately discuss below.

a. River Cross section

Cross section survey data of Akaki River and detail of bridge structure data were collected from ERC. There is two cross section survey data's around the bridge upstream and downstream located at 100m distance (see Appendix D). The cross section data's were interpolated through the river cross section at an interval of 10m to get accurate channel profile.

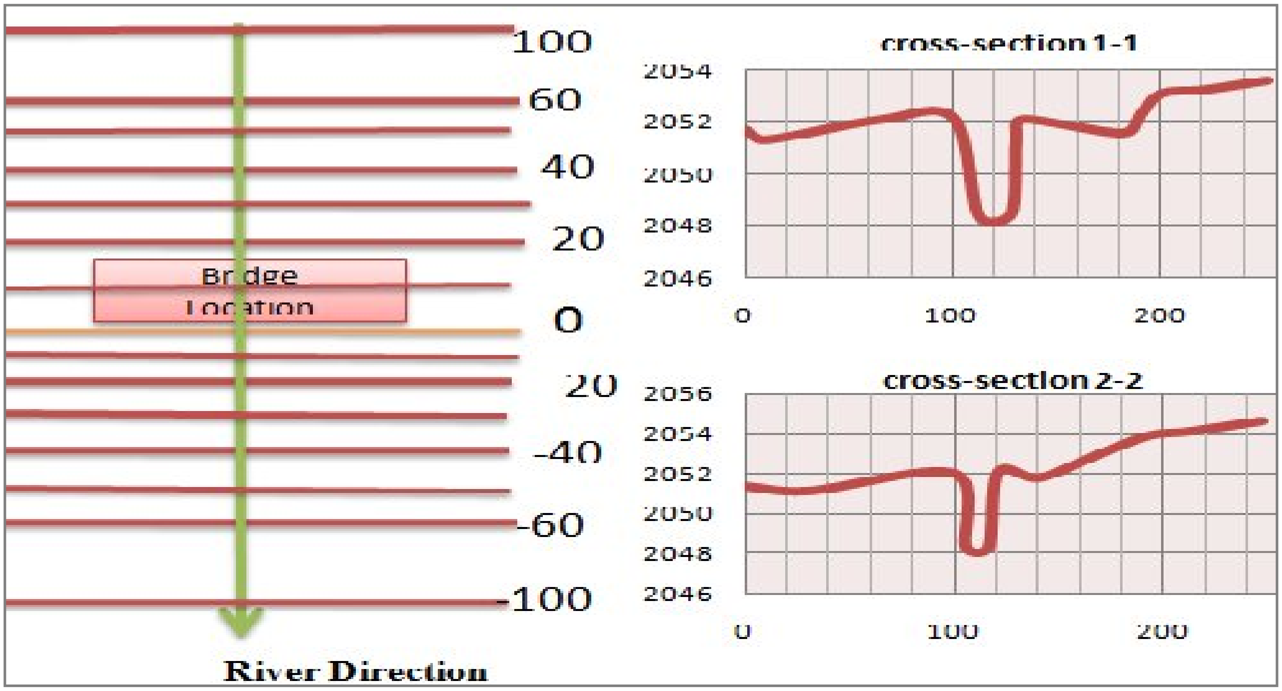


Figure 4-11: Akaki River cross section at railway crossing

b. Manning's Roughness Coefficient

Manning's roughness coefficient 'n' depends on many factors which includes, surface roughness, channel vegetation cover, channel alignment and irregularity etc. Appropriate value of 'n' determines the accuracy of water surface profile within the cross section. According to Gary W. Burner (2010) US Army Corps of Engineers, the manning 'n' value for floodplain of Akaki river bridge cross section was selected from the table of 'n' value developed by Chow (1959) "Hydraulics of open channel" book given on Hec-RAS reference manual. As shown on the photo, the Akaki River channel was described as floodplain in which main channel has short grass and both banks are cultivated area with mature row crops. Therefore manning's roughness coefficient n=0.03 and n=0.035 were adopted for short grass and cultivated area respectively. See appendix E.

. n=0.03 n=0.035

Photo 4-1: Akaki River Channel Roughness Coefficient Values ‘n’ (Jan. 2015)

c. Contraction/ Expansion Coefficient

Water surface profiles are computed from one cross section to next by energy equation with an iterative procedure called standard step method. Contraction and expansion coefficients are used to evaluate energy losses between adjacent cross sections. HEC-RAS program assumes that the contraction is occurring whenever velocity head at downstream is greater than velocity head at upstream and the reverse is for expansion. Contraction and expansion coefficients depend on flow condition and channel degrees of constriction. John C. Warner et.al. (2010) suggests that, where the change in river cross section is small, and the flow is subcritical, the contraction and expansion are typically in the order of 0.1 and 0.3 respectively. When the change in effective cross section area is abrupt, such as at bridge, contraction and expansion coefficients of 0.3 and 0.5 are often used. The above both values were adopted depending on channel cross section physical characteristics as input data for HEC-RAS to calculate energy losses between cross section in this study.

d. Ineffective Flow Areas

Ineffective flow area is also one of bridge geometric data that would happen due to bridge obstruction. If the floodplain width coming into the structure is much wider than the opening, ineffective flow areas should be entered. These are the areas outside the main flow, where water is slow or stagnant just at upstream and downstream of bridge embankment. The ineffective area option is used in HEC-RAS program, to enter station and elevation at upstream and downstream cross sections of bridge, to keep all the flow in the area pass under the bridge. As specified by Steven S. Piper et.al. (2010), the upstream position of ineffective flow stations in the cross-sections is in the 1:1 manner, i.e., for each meter distance from the cross-section to the structure, go one meter over from the edge of the structure and downstream as 2:1 positioning, i.e., 2 meter out and 1 meter over. The elevations specified for ineffective flow should correspond to elevations where significant weir flow passes over the bridge. Using the same principle stated above the ineffective flow area of Akaki floodplain at railway crossing was entered the program.

4.2.1. Bridge Hydraulic Computations

The basic computational procedure for the HEC-RAS program is water surface profiling based on energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion. The momentum equation is utilized in situations where the water surface profile is rapidly varied, such as at bridges (USACE, 1998)

HEC-RAS program has the ability to analyze water profile near and inside bridge for different flow type. These types of flow are; low flow (Class A, B, and C), high flow and combined flow methods.

Low flow occurs when the water flow only through the bridge opening and considered as an open channel flow i.e. when the water surface does not exceeds the highest point of low cord on the upstream of the bridge (Steven S. Piper et.al. 2010). HEC-RAS program uses Momentum equation to identify Class A, B or C. If the momentum downstream of the bridge is greater than the critical depth momentum inside the bridge, the flow is considered subcritical (Class A**)**. If the momentum downstream is less than the momentum at critical depth, then it is Class B and assumed that the flow will pass through critical depth and a hydraulic jump will occur at downstream. Class C- the profile is considered completely supercritical through the bridge.

High flow occurs when the water surface encounters the highest point of the low cord on the upstream of bridge. Combined flow occurs when both low flow and high flow occurs simultaneously which follows the iterative solution for upstream energy.

For this study case combined flow method is more applicable to determine water surface profile, because the aim of this work is to check whether the computed flow is safely pass under bridge (low flow) constructed or overflow (high flow) would occur for specified flood return periods. HEC-RAS program has a capability to switch from low flow condition to high flow depending on types of flow. Each flow types require different methods to determine its profile.

All methods use Standard Step calculations for the transition sections. Four methods are available to compute bridge hydraulic losses between each cross section. Energy Equation (standard step method), Momentum Balance, Yarnell Equation and WSPRO low-flow model are the methods that are used to determine flow profile *(Gray W. Burnner, 2010)*. All these method were selected for this study to determine flow surface profile. When all methods are used, the program identifies the method that yields larger profile value and uses for computation. HEC-RAS program required additional data for momentum balance and Yarnell equation methods which depend on bridge pier shape. Momentum balances drag Coefficients (Cd) and Yarnell's pier shape coefficient (k) obtained from Hec-Ras reference manual (2010) as give in table below.

Table 4-12: Yarnell 'K' Coefficient and Drag coefficient 'Cd' of various pier shapes

Pier Shape	**Yarnell 'K' Coefficient**
Semi-circular nose and tail	0.90
Twin-cylinder piers with connecting diaphragm	0.95
Twin-cylinder piers without diaphragm	1.05
90 degree triangular nose and tail	1.05
Square nose and tail	1.25
Ten pile trestle bent	2.50
Pier Shape	**Drag Coefficient Cd**
Circular pier	1.20
Elongated pier with semi-circular ends	1.33
Elliptical pier with 2:1 length to width	0.60
Elliptical pier with 4:1 length to width	0.32
Elliptical pier with 8:1 length to width	0.29
Square nose piers	2.00
Triangular nose with 30 degree angle	1.00
Triangular nose with 60 degree angle	1.39
Triangular nose with 90 degree angle	1.60

Triangular nose with 120 degree angle	1.72

Source: *(US Army Corps of Engineers, Hec-Ras Reference manual, 2010).*

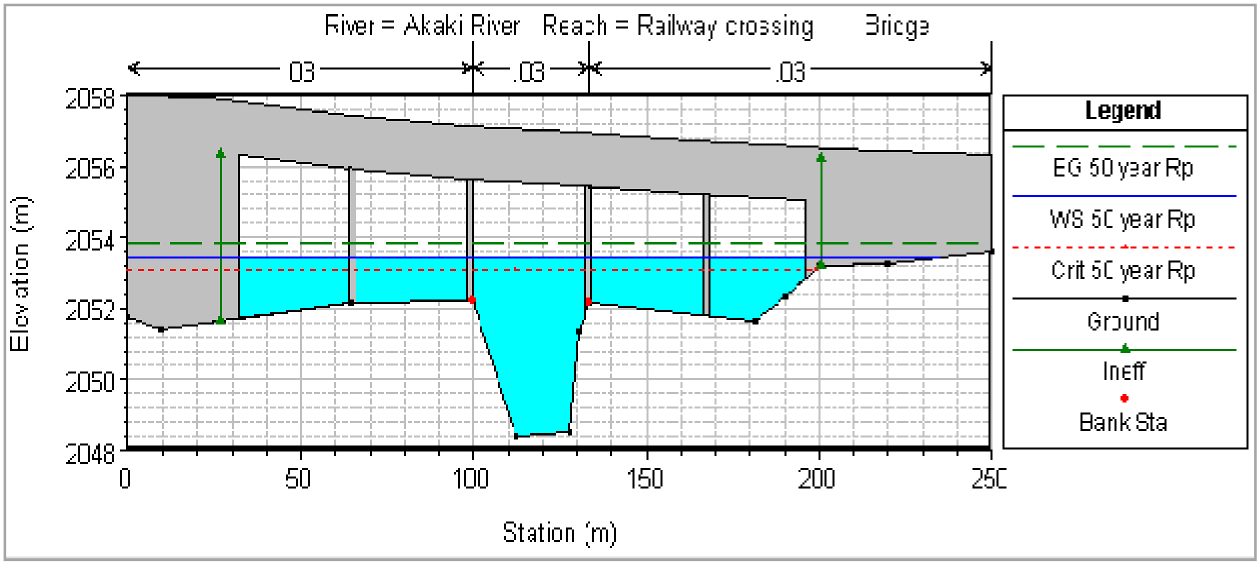
The Akaki river bridge piers have a Semi-circular nose and tail as shown in figure below. Thus, Yarnell's shape Coefficient and drag coefficients are 0.9 and 1.33 respectively. These values were used to compute the complete water surface profile at bridge upstream and downstream.

Photo 4-2: Akaki New railway Bridge Pier shape

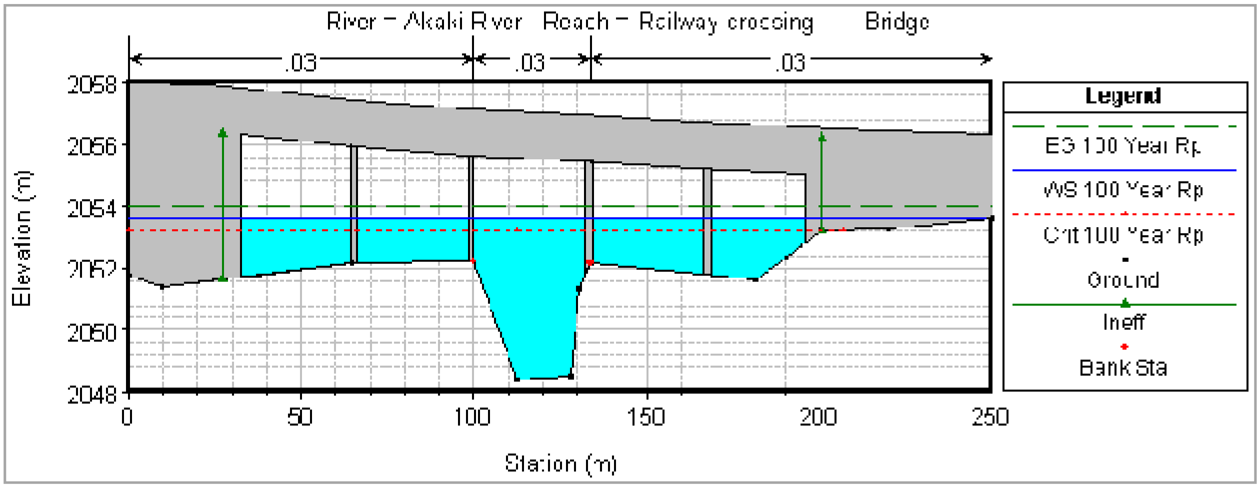
HEC-RAS model require also boundary conditions to establish the starting water surface at the ends of the river system. The program provides three options to define boundary condition at which the program starts computation depending on flow regimes. In subcritical flow regime only downstream boundary condition is required, where as for supercritical flow regime boundary condition at upstream of river system is required. In a mixed flow regime both upstream and downstream end boundary conditions are necessary, Gray, W. Brunner. (2006). From different boundary conditions available in HEC-RAS, normal depth boundary condition was assigned for this case depending on available data. The normal depth boundary condition require water surface slope, in which 0.0082 was used, that is the same with channel bed slope, for this study.

4.2.2. Bridge Hydraulic Computations

The hydraulic calculation of Akaki River Bridge was done by HEC RAS program for the 50 years and 100 years return period peak discharge obtained under section4.1.2. The normal water level which corresponds to flood with 50/100 years design period has been taken as High Water Mark (HWM) was obtained. The analysis results were presented here as follow.

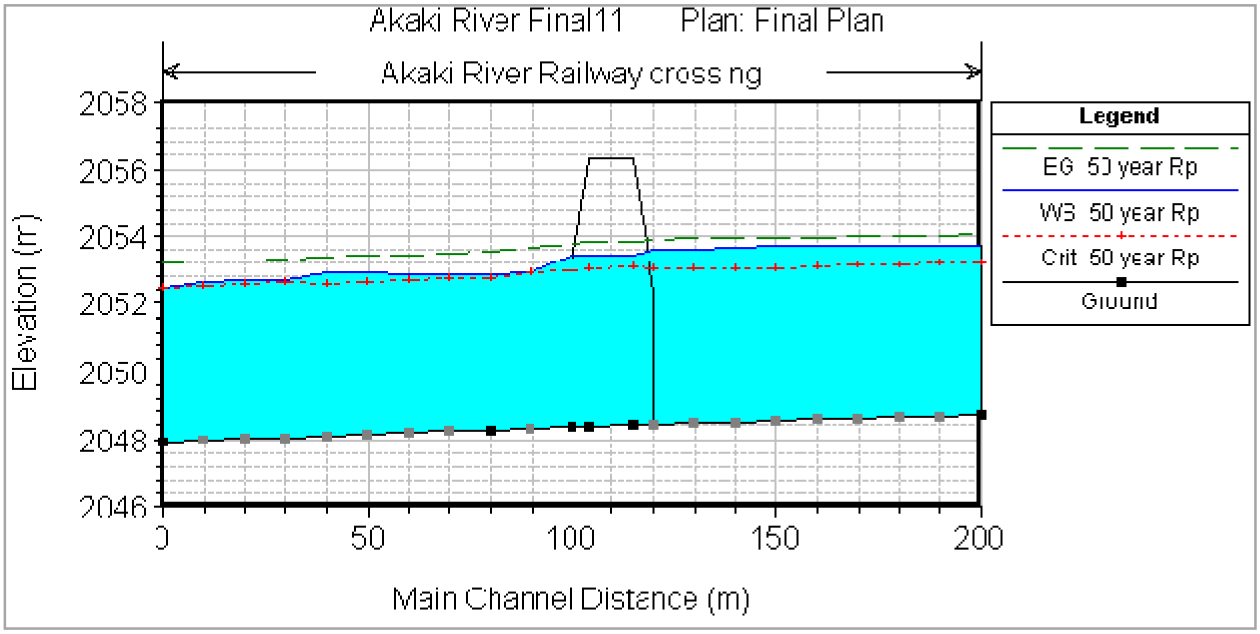


a) 50 years design period peak discharge level.

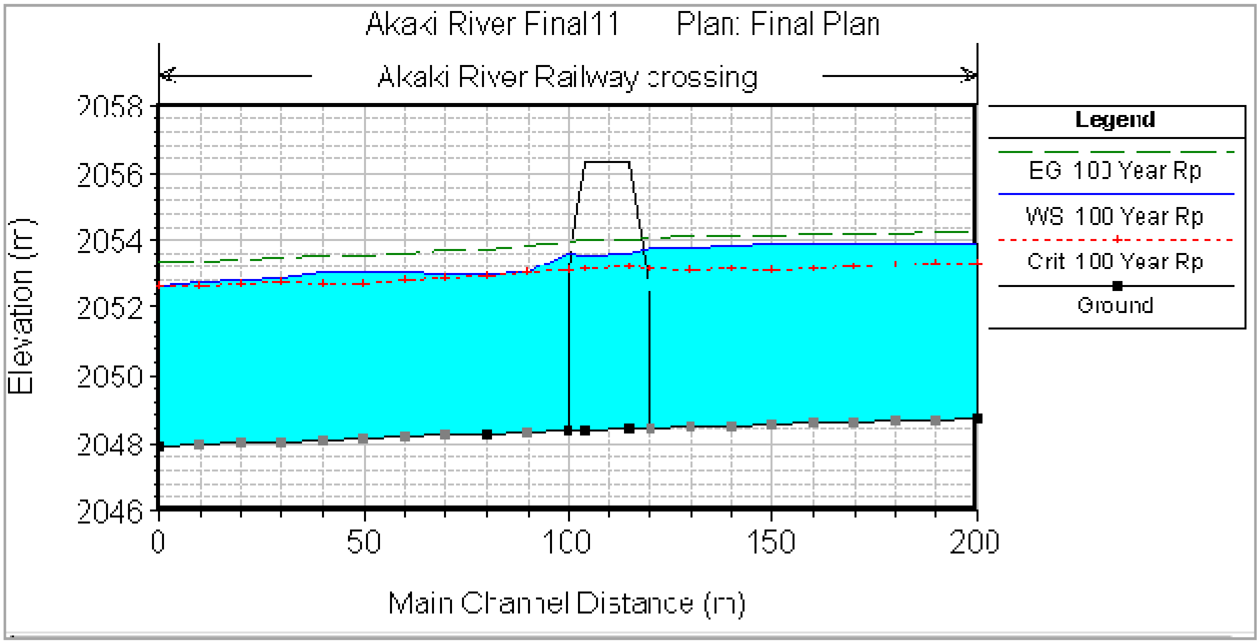


b) 100 years design period peak discharge level

Figure 4-12: HEC – RAS 50/100 years design flood level at Bridge cross section



a) 50 years design flood level profile



b) 100 years design flood level profile

Figure 4-13: HEC – RAS 50/100 years design flood level profile

Table 4-13: HEC – RAS Bridge summary results

Reach	River Sta	Profile	E.G. Elev (m)	W.S. Elev (m)	Crit W.S. (m)	Frctn Loss (m)	C & E Loss (m)	Vel Chnl (m/s)
Railway crossing	30.*	50 Year	2053.92	2053.63	2053.04	0.02	0.00	2.98
Railway crossing	30.*	100Year	2054.1	2053.8	2053.12	0.02	0.00	3.06
Railway crossing	20.*	50 Year	2053.9	2053.6	2053.04	0.01	0.04	2.93
Railway crossing	20.*	100Year	2054.08	2053.77	2053.16	0.01	0.04	3.01
Railway crossing	BR U	50 Year	2053.85	2053.43	2053.08	0.02	0.02	3.45
Railway crossing	BR U	100Year	2054.03	2053.58	2053.2	0.02	0.01	3.57
Railway crossing	BR D	50 Year	2053.81	2053.42	2053.02	0.01	0.02	3.33
Railway crossing	BR D	100Year	2054	2053.57	2053.13	0.01	0.02	3.47
Railway crossing	-10	50 Year	2053.77	2053.43	2052.98	0.02	0.00	3.13
Railway crossing	-10	100Year	2053.95	2053.58	2053.08	0.02	0.00	3.25
Railway crossing	-20	50 Year	2053.64	2052.93	2052.93	0.03	0.01	4.3
Railway crossing	-20	100Year	2053.81	2053.06	2053.06	0.04	0.00	4.46

Hydraulic calculation results given above shows, Bridge cross section, water surface profile and bridge summary results. The detail discussion on hydraulic analysis result was presented in later section.

5. RESULTS AND DISCUSSION

5.1. Discussion on Hydrologic Analysis results

Hydrologic analysis of Akaki River Railway Bridge was carried out by two methods, i.e. SCS unit hydrograph with the help of HEC-HMS program and flood frequency analysis method for stream flow data's. In SCS unit hydrograph methods two types of precipitation methods were used to develop the catchment model and determine peak discharge. These are weight gauged precipitation method and frequency storm method that requires historical (gauged precipitation) data and hypothetical data respectively. Gauged precipitation data of the catchment area were collected from gauging station near and in the catchment which was used for calibration and validation of HEC-HMS model. Frequency storm method requires statistical data (precipitation-depth). Intesity Duration Frequency (IDF) developed by Ethiopian Raod Authority (ERADDM, 2013) for different rigons Ethiopia was used as a source of data to determine peak flow.

The accuracy of peak discharges estimated were depends on accuracy of parameter value used during hydrological modelling. Thus, hydrological model developed require calibration (parameter estimation) and validation with observed stream flow. For this study HEC-HMS model calibration and validation were done and its performance was evaluated as described in section 4.1.1.6.

The peak discharges were computed using SCS method with the help of calibrated and validated HEC-HMS model by storm frequency precipitation methods which results 727.1m3s and 815.6m3/s for 50 and 100 year design period respectively.

Flood frequency analysis method was also used to estimate peak flow at Akaki river gauging station using stream flow data. Two methods of frequency analysis methods, i.e., Gumbel (Extreme Value Type I) and Log-Pearson III methods were used to estimate the design flood of Akaki River and results were presented in table 4.10.

5.1.1. Comparison of Hydrologic Analysis Results

The design peak discharge which was computed using SCS method with the help of HEC-HMS program was compared with design flood of gauged stream data which was computed by using flood frequency analysis methods as shown in table below.

Table 5-1: Comparison of SCS Unit hydrograph with Flood Frequency Analysis Method

RAINFALL-RUNOFF METHOD	METHOD OF COMPUATATION	RP	Design Flood
		Year	m3/s
SCS Unit hydrograph	Calibrated HEC-HMS model	50	727.10
		100	815.60
Flood Frequency Analysis for Stream flow	Gumbel (Extreme Value Type I)	50	711.83
		100	803.71
	Log-Pearson -III Distribution	50	685.54
		100	753.68

From the above comparison, it can be shown that, both flood frequency analysis methods and SCS unit hydrograph have nearly similar values. However, the SCS Unit hydrograph method, which estimated by calibrated and validated HEC-HMS model, shows greater value than flood frequency analysis methods i.e. Gumbel (Extreme Value Type I) and Log-Pearson III results for both the selected return periods. In order to check adequacy of hydraulic structure, it is preferable to use greater value of peak discharge to reduce hesitation come from larger value. Thus, depending on available data and level of accuracy of hydrologic analysis, obtained result by the SCS Unit hydrograph is more preferable i.e. performance evaluation of SCS Unit hydrograph model shows that, the model is slightly good model that can be used to determine reasonable peak flow. From the method used in the SCS Unit hydrograph, Frequency Storm Method yields. 727.10m3/s and 815.60m3/s for 50 and 100 years return period respectively which is more important to check hydraulic structure. Thus, Frequency Storm Method of SCS Unit hydrograph rainfall-runoff model was used for this study.

The three methods of hydrologic analysis results obtained above for Akaki catchment are at the Akaki gauging station near old Addis Ababa- Adama road with the catchment area of 877.7km^2, whereas at the railway crossing it has an area of 893.7km^2 i.e. the gauging station is upstream of the railway crossing site. The peak discharges at the railway crossing was also determined by calibrated and validated HEC-HMS model with additional catchment area downstream of gauging station. The results were obtained by routing the hydrograph computed previously at river gauging station to Railway Bridge and give results of 743.2m^3/s and 835.5m^3/s for 50 and 100 years return period respectively. These values of hydrologic analysis were used to check adequacy of hydraulic analysis of Akaki Railway Bridge with the help of HEC-RAS program.

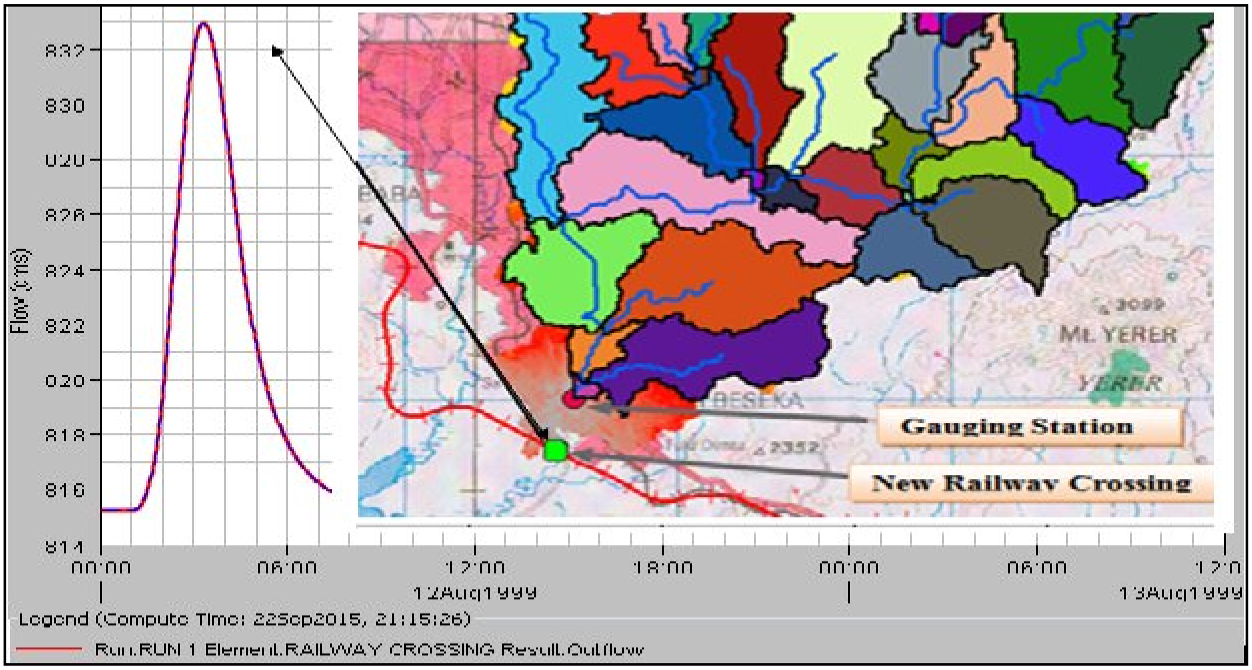


Figure 5-1: Railway crossing hydrograph and location of river gauging station

The design peak discharge of Akaki Railway Bridge is also collected from ERC. The 100 year peak discharge obtained by the designer was about 554m3/s. When these values compared with computed it is smaller than peak discharge computed for the same period. It is even smaller than maximum stream gauge (693.6m3/s). This supports the idea "No hydrological and hydraulics analysis including sediment transport analysis has been undertaken to ensure that drainage structures are designed to minimise future maintenance requirements" Manaye (2013); Sebeta to Adama site visit report.

5.2. Discussion on Hydraulic Analysis Result

With the help of HEC-RAS program, Hydraulic calculation of Akaki Railway Bridge was done for 50 years and 100 years return period peak discharge and water surface profiles were obtained for both design period. Water surface profile calculated were based on hydraulic parameters (manning's roughness, contraction and expansion, pier shape coefficients etc) used. However, actual values of these parameters need to be calibrated and verified with historical water mark to get consistent water surface profile. For this case no historical high water marks data's are available for calibration and validation, as it is newly constructed bridge. Thus, parameter Values are adopted based on recent field inspection which are more close to the reality. Therefore, the hydraulic profiles calculated around the bridge are considered as that the computation is consistent and acceptable.

Figure4.12 a,b &4.13 a,b clearly shows water surface level at bridge cross section and water surface profile along the river reach for both 50 years and 100 years return period peak discharge. Calculated water profile for both return periods were depends on methods specified in section 4.2.1 where the method that gives greater values is used by the program to fix water surface level. The comparisons of each method are given in table below.

Table 5-2: Water surface profile comparison, (at bridge location)

Reach	River Sta	Profile	E.G. Elev	W.S. Elev	Br Sel Method	Energy EG	Moment EG	Yarnell EG	WSPRO
Railway crossing	BR	50 year	2053.9	2053.6	WSPRO	2053.89	2053.88	2053.81	2053.9
Railway crossing	BR	100 year	2054.08	2053.77	WSPRO	2054.07	2054.06	2053.98	2054.08

As shown in table above the WSPRO method has been used to provide a water surface profile for the bridge cross section when compared with other methods. WSPRO method yields larger water surface profile that used to check whether bridge opening has sufficient space to pass peak discharge.

A HEC-RAS program has a capability to calculate both subcritical and supercritical flow profiles. Thus, as shown on figure 4.12and table 4.12.., the water surface elevation determined was greater than critical depth that indicates the computed flow profile was subcritical flow for both return periods. This shows low flow type (class A) was occurred as the water flows only through the bridge opening and considered as an open channel flow. Water surface level is about 0.61m (2053.77m water surface and 2053.16m critical depth elevation) greater than critical depth at upstream of the bridge, such that a backwater occurred at upstream of the bridge due to the construction of embankment and bridge piers. Backwater should be converged to critical depth at upstream and downstream of river system. However, for this case there is no water surface profile was converged to critical depth at upstream of the bridge. The distance over which upstream backwater occurs depends on the channel conditions which require additional channel survey data and left for further investigation. But a water surface profile was converged toward the desired depth at the downstream of the bridge within channel length provided.

Table 4-12 also shows that variations in flow velocity in the river channel. In some segments of the flow (e.g., near the center of the stream), the velocity may be considerably higher than the allowable velocity (0.5-4 m/s). This higher value of velocity leads to channel erosion during summer as shown on figure below. Higher value of flow velocity was occurred downstream of the bride site.

a. River channel during winter

b. River channel during summer

Photo 5-1: Akaki River channel at new Railway crossing

In order to protect erosion of the bank it is important to provide structures like masonry retaining wall, launching aprons at the bridge location and also provide energy dissipation mechanisms such as cascading of the channel**.**

5.2.1. Adequacy of the Akaki Railway Bridge

Adequacy of hydraulic structure highly subjected detail hydrological and hydraulic analysis. This study was also under gone both hydrological and hydraulic analysis of Akaki River Bridge. A newly constructed Akaki Railway Bridge, which has clear span length of 163.84m (32.81m+32.74m+32.74m+32.74m+32.81m), was checked by using the 50 years and 100 years return period peak discharge.

Based on the HEC- RAS result shown on figure4.12.a and b, this bridge has sufficient opening size that can safely passes computed peak discharge for 50 and 100 years return period respectively. The minimum lower cord elevation of the bridge is 2056.33m, where as the maximum water surface elevation at bridge location are 2053.9m and2054.08m for 50/100 year peak flow respectively i.e. greater than 2m clear space above water surface level. As information from contractors and supervisors of the project, the opening size of Akaki Railway Bridge was dominated by geometrical alignment of railway line not by hydrological analysis where 100 year design flow was 554m3/s as described previously. That is why the opening space is adequate to pass peak discharge determined during this study even though it is larger than design flow.

The hydraulic calculation confirmed that Akaki River Bridge can give its full service efficiently for the return period specified. Adequacy of this bridge was evaluated depending on currently available data’s.

6. SUMMARY, CONCLUSION AND RECOMMENDATION

6.1. Summary

The need to effectively remove surface water from railway track environments is essential for the rail track to operate safely and reliably. New railway line from Sebeta to Mieso crosses Akaki river at south-eastern of Addis Ababa where the reach characterized as flat and floodplain. Site visit reports of experts indicate that there is a drainage system problem along this line like inappropriate location, size and alignment of drainage structure which leads to inlet and outlet scouring of culverts, back water problems (rising upstream water level), etc. The main causes of this problems starts from insufficient hydrological and hydraulic analysis of drainage structures.

This study identified one drainage structure along new railway line from Sebeta- Mieso where Akaki River crossing Railway Bridge was selected for detail hydrologic and hydraulic investigation to check its adequacy for the design period.

Big Akaki river catchment, which covers most part of Addis Ababa, has a total area of 894km^2 at new railway crossing. The catchment area dominantly covered by urban and urban development's that require intensive analysis for the dynamics hydrologic parameters. The catchment land use, soil type, rainfall data, Akaki river stream flow data, provided bridge structural information and channel cross section data's were collected from different concerned bodies as mention previously.

Hydrological model of the catchment area was developed by software application like Arc GIS, HEC-Geo HMS and HEC-HMS programs. HEC-HMS program has a capability to develop hydrograph of the catchment at outlet. HEC-HMS model of the catchment was calibrated and validated by using gauged precipitation and stream flow data. Calibrated parameters were adjusted by the trial-and-error method implemented in the HEC-HMS model. Model performance was also evaluated by efficiency evaluation criterias; such as Nash-Sutcliffe efficiency (NSE), coefficient of determination (r^2) and percentage error volume (PEV). SCS method was used to determine peak flow using frequency storm after hydrologic model performance evaluated and results 743.2m3/s and 834.5m3/s for 50 and 100 year return period respectively. Flood frequency analysis (Gumbel Extreme Value Type I and Log-Pearson III) methods also used to determine peak design discharge where 803.71m3/s and 753.68m3/s results were obtained for 100 year return period respectively. Results of both methods were compared with each others, where SCS (HEC-HMS) results are more reliable value that used as design peak discharge to check the capacity of the bridge.

Hydraulic modelling and analysis of the bridge was developed by HEC-RAS program which has a capability to model river channel and determine water surface profile inside and upstream/downstream of the bridge. Cross-sectional elevation data, hydraulic-structure detailing, roughness coefficients and peak-discharge estimated were used as input for the models. Starting water-surface elevations computation for the stream was established using normal depth boundary condition used in the HEC-RAS models. Based on available data's water surface profile for 50/100 year peak discharge was determined by the program.

Finally adequacy of the bridge was evaluated for both peak discharge determined during hydrologic analysis. Based on HEC-RAS program output, Railway Bridge provided on Akaki River has sufficient size to safely convey both peak design flow during the design period. However, there is water level rise above the main channel (backwater) at upstream of the bridge due to flow constriction by the embankment construction at bridge location. Water level rises about 0.6m from normal water depth that leads backwater to occur. Backwater occurs around upstream of the bridge covered by agricultural land which displaces cultivated crops during summer. Due to insufficient survey data, backwater length and area had not been included in this study which needs further investigation.

6.2. Conclusion

This study investigates the hydrological and hydraulic analysis of Akaki Railway Bridge. In order to develop hydrologic and hydraulic model of the catchment, different programs; such as HEC-Geo HMS with Arc GIS, HEC-HMS, HEC-RAS were used. Detailed hydrological assessment had been undertaken with different method such as SCS unit hydrograph with the help of calibrated and validated HEC-HMS model, and Flood frequency analysis methods. A comparison of computed peak discharge by HEC-HMS model and flood frequency analysis shows little difference where SCS method was used to check capacity of the bridge. However, peak discharge determined during this study is slightly different from design discharge used in hydraulic design of Akaki River Bridge used by the designer. Design discharge computed by designer for 100 year return period was about 554m^3/s where as the computed peak flow in this case for the same period was 843.5m^3/s at bridge location from catchment area of 894km^2. Even though the bridge span length is adequate to convey currently determined peak discharge, the peak discharge obtained during bridge design is too small when compared with currently determined peak discharge. Even the computed design discharge is less than maximum daily stream flow (e.g. observed stream flow 693.60m^3/s on August 1999) for a catchment area of 877.7km^2. This indicates that as stated in Sebeta to Adama Double Track Railways Project Site Visit Investigation Report and Recommended Rehabilitation Works by Manaye(2013), there is no detailed hydrological and hydraulics assessment has been undertaken which will inform the design process in terms of the forces the various structures will have to deal with during the design life of the project. The recent failures of the various structures as a result of the flood waters only go to confirm the findings of this investigation works.

According to Akaki railway bridge detailing data collected from ERC, the bridge span and opening space is designed according to the landform (alignment) of the location i.e. geometrical alignment railroad determine span length and opening size of the bridge rather than its hydrology. Based on the available data's and software results it is concluded that there is a significant space to pass peak design flow efficiently under the bridge.

6.3. Recommendation

Even thought, the bridge is safe to convey peak discharge determined, data inadequacy was encountered critically. Consequently, further works need to be undertaken in data collection and field investigations, among which the following recommendations were critical for the life of the bridge that drawn from this study.

- ✓ To get accurate estimation of dynamic peak flood flow in the study area there is a need to have continues flow data around the bridge location. This study uses only a 24 year stream gauge data which stopped since 2004. Additional stream gauging flow data would provide the understanding of the river hydrology.
- ✓ Data for calibration and verification of hydraulic parameters for computation of hydraulic profile of the reach around the bridge should be clearly collected. The parameters should be calibrated with historical high water marks and/or gauged stream flow data to ensure that they accurately represent local channel conditions.
- ✓ Detailed survey data should be collected for floodplain upstream of the bridge to evaluate upstream length and area covered by backwater due to bridge construction. In

subcritical flow conditions, the backwater tails off upstream until it reaches the normal water surface. The distance upstream over which backwater occurs depends on the channel conditions and flow conditions. For design frequency conditions, the allowable backwater should be considered based on the risk of incurring flood-related damage to the railway and adjacent property.

- ✓ Flow velocity is higher at bridge location than specified flow speed for earthen material, thus, it is important to provide structures like masonry retaining wall, launching aprons at the bridge location and also provide energy dissipation mechanisms such as cascading of the channel**.**

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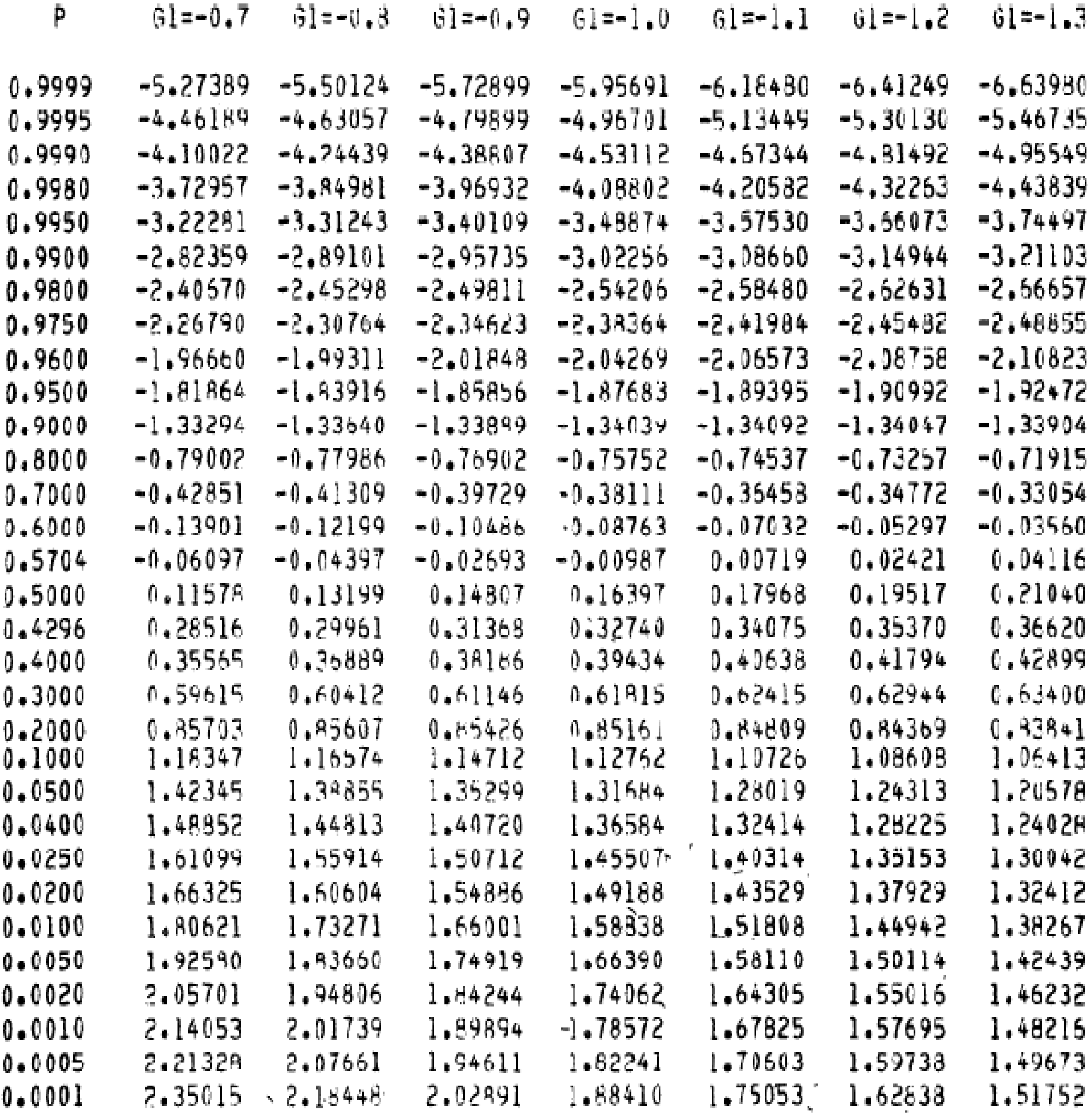
APPENDICES

Appendix A: Flood frequency Table Value 'k'



P	G1=-0.7	G1=-0.8	G1=-0.9	G1=-1.0	G1=-1.1	G1=-1.2	G1=-1.3
0.9999	-5.27389	-5.50124	-5.72899	-5.95691	-6.18480	-6.41249	-6.63980
0.9995	-4.46189	-4.63057	-4.79899	-4.96701	-5.13449	-5.30130	-5.46735
0.9990	-4.10022	-4.24439	-4.38807	-4.53112	-4.67344	-4.81492	-4.95549
0.9980	-3.72957	-3.84981	-3.96932	-4.08802	-4.20582	-4.32263	-4.43839
0.9950	-3.22281	-3.31243	-3.40109	-3.48874	-3.57530	-3.66073	-3.74497
0.9900	-2.82359	-2.89101	-2.95735	-3.02256	-3.08660	-3.14944	-3.21103
0.9800	-2.40570	-2.45298	-2.49811	-2.54206	-2.58480	-2.62631	-2.66657
0.9750	-2.26790	-2.30764	-2.34623	-2.38364	-2.41984	-2.45482	-2.48855
0.9600	-1.96660	-1.99311	-2.01848	-2.04269	-2.06573	-2.08758	-2.10823
0.9500	-1.81864	-1.83916	-1.85856	-1.87683	-1.89395	-1.90992	-1.92472
0.9000	-1.33294	-1.33640	-1.33899	-1.34039	-1.34092	-1.34047	-1.33904
0.8000	-0.79002	-0.77986	-0.76902	-0.75752	-0.74537	-0.73257	-0.71915
0.7000	-0.42851	-0.41309	-0.39729	-0.38111	-0.36458	-0.34772	-0.33054
0.6000	-0.13901	-0.12199	-0.10486	-0.08763	-0.07032	-0.05297	-0.03560
0.5704	-0.06097	-0.04397	-0.02693	-0.00987	0.00719	0.02421	0.04116
0.5000	0.11578	0.13199	0.14807	0.16397	0.17968	0.19517	0.21040
0.4296	0.28516	0.29961	0.31368	0.32740	0.34075	0.35370	0.36620
0.4000	0.35565	0.36889	0.38166	0.39434	0.40638	0.41794	0.42899
0.3000	0.59615	0.60412	0.61146	0.61815	0.62415	0.62944	0.63400
0.2000	0.85703	0.85607	0.85426	0.85161	0.84809	0.84369	0.83841
0.1000	1.18347	1.16574	1.14712	1.12762	1.10726	1.08608	1.06413
0.0500	1.42345	1.38855	1.35299	1.31684	1.28019	1.24313	1.20578
0.0400	1.48852	1.44813	1.40720	1.36584	1.32414	1.28225	1.24028
0.0250	1.61099	1.55914	1.50712	1.45507	1.40314	1.35153	1.30042
0.0200	1.66325	1.60604	1.54886	1.49188	1.43529	1.37929	1.32412
0.0100	1.80621	1.73271	1.66001	1.58838	1.51808	1.44942	1.38267
0.0050	1.92590	1.83660	1.74919	1.66390	1.58110	1.50114	1.42439
0.0020	2.05701	1.94806	1.84244	1.74062	1.64305	1.55016	1.46232
0.0010	2.14053	2.01739	1.89894	-1.78572	1.67825	1.57695	1.48216
0.0005	2.21328	2.07661	1.94611	1.82241	1.70603	1.59738	1.49673
0.0001	2.35015	2.18448	2.02891	1.88410	1.75053	1.62638	1.51752

Source (Bulletin#17, 1982 Flood flow Frequency)

Appendix B: Gumbel (Extreme Value) method, Flood flow Frequency

	year	xi	(xi-mean)^2
1	1981	201.312	5941.01
2	1982	172.772	11155.1
3	1983	138.717	19508.5
4	1984	189.383	7922.24
5	1985	165.65	12710.3
6	1986	68.777	43937.6
7	1987	36.554	58484.6
8	1988	148.353	16909.6
9	1989	233.769	1991.03
10	1990	277.219	1.37
11	1991	215.224	3989.9
12	1992	153.074	15704.0
13	1993	573.569	87130.6
14	1994	162.58	13411.9
15	1995	257.976	416.7
16	1996	615.761	113819.1
17	1997	276.323	4.2
18	1998	421.518	20485.6
19	1999	693.102	171986.04
20	2000	255.776	511.39
21	2001	435.338	24632.67
22	2002	219.869	3424.70
23	2003	420.059	20070.1
24	2004	250.468	779.63
	mean	274.30	27288.69
	stdev	168.69	41985.27
	sum	6583.143	654928.59
	No. of data=24		
xi= annual maximum or peak discharge			

Return Period	KT	Q
2	-0.16436	246.5703
5	0.719822	395.7273
10	1.30523	494.4822
25	2.044895	619.2592
50	2.593621	711.826
100	3.138295	803.7093

$$X_T = X + K_T s$$

$$Kt = \frac{-\sqrt{6}}{\Pi}\left[0.57721 + \ln\left\{\ln\frac{T}{T-1}\right\}\right]$$

Appendix C: Log Pearson III flood frequency Analysis

Rank	year	Ranked Max.Discharge	log Q	(logQ-avg(logQ))^2	(logQ-avg(logQ))^3
1	1999	693.102	2.841	0.2341	0.1132
2	1996	615.761	2.789	0.1870	0.0809
3	1993	573.569	2.759	0.1613	0.0648
4	2001	435.338	2.639	0.0794	0.0224
5	1998	421.518	2.625	0.0717	0.0192
6	2003	420.059	2.623	0.0709	0.0189
7	1990	277.219	2.443	0.0074	0.0006
8	1997	276.323	2.441	0.0071	0.0006
9	1995	257.976	2.412	0.0030	0.0002
10	2000	255.776	2.408	0.0026	0.0001
11	2004	250.468	2.399	0.0017	0.0001
12	1989	233.769	2.369	0.0001	0.0000
13	2002	219.869	2.342	0.0002	0.0000
14	1991	215.224	2.333	0.0006	0.0000
15	1981	201.312	2.304	0.0028	-0.0001
16	1984	189.383	2.277	0.0063	-0.0005
17	1982	172.772	2.237	0.0143	-0.0017
18	1985	165.65	2.219	0.0190	-0.0026
19	1994	162.58	2.211	0.0213	-0.0031
20	1992	153.074	2.185	0.0296	-0.0051
21	1988	148.353	2.171	0.0345	-0.0064
22	1983	138.717	2.142	0.0462	-0.0099
23	1986	68.777	1.837	0.2699	-0.1402
24	1987	36.554	1.563	0.6305	-0.5007
		Average	Average	Sum	Sum
		274.298	2.357	1.9016	-0.3495

Variance	0.083
standard deviation	0.288
Skew coefficient	-0.697

Log Q =Avg(log Q) +KS

Return Peroid	K(-0.697)	LogQ	Discharge
5	0.857	2.603816	401.621
10	1.1835	2.697848	498.710
25	1.4885	2.785688	610.503
50	1.6633	2.8360304	685.536
100	1.8062	2.8771856	753.678

Appendix D: Values of Roughness Coefficient n (Uniform Flow)

Type of Channel and Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense Weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Backhoe-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS			
1 Minor streams (top width at flood stage < 30 m)			
a. Streams on Plain			
1. Clean, straight, full stage, no rims or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8 Very weedy reaches, deep pools, or floodways			

with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravel, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
2 Flood Plains			
a. Pasture, no brush			
1. Short grass			
2. High grass	0.025	0.030	0.035
	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of spouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
3 Major Streams (top width at flood stage > 30 m). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025		0.060
b. Irregular and rough section	0.035		0.100
4 Various Open Channel Surfaces			
a. Concrete	0.012		0.020
b. Gravel bottom with:			
Concrete	0.020		
Mortared stone	0.023		
Riprap	0.033		

c. Natural Stream Channels			
Clean, straight stream	0.030		
Clean, winding stream	0.040		
Winding with weeds and pools	0.050		
With heavy brush and timber	0.100		
d. Flood Plains			
Pasture	0.035		
Field Crops	0.040		
Light Brush and Weeds	0.050		
Dense Brush	0.070		
Dense Trees	0.100		

Appendix E: Channel cross section survey data (source: ERC)

Cross section 1-1	
0	2051.698
9.532	2051.323
64.61	2052.096
100	2052.16
112.338	2048.36
127.842	2048.469
130.32	2051.308
133.616	2052.107
181.235	2051.581
189.996	2052.285
200	2053.112
219.584	2053.245
251.23	2053.61

Cross section 2-2	
0	2051.362
29.65	2051.13
100	2052.01
105.29	2048.287
116.185	2048.273
120.795	2052.085
140.65	2051.796
169.45	2053.019
193.05	2053.941
212.584	2053.245
247.23	2053.61