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Investigating Highway Drainage Problems in the Sile River Bridge, South, Ethiopia

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***Abstract*—Roads are among the basic infrastructures. Structures provided on the road shall be kept healthy to serve intended purposes. Investigating the cause of failure of cross drainage structures will help to have some understanding on the problem and to make future designs better. The sile River Bridge is the one among those encountered failure. Now the bridge is not giving service since it is scoured to a depth of 3m. Sile River is ungauged and parameters are obtained from a neighboring catchment, Kulfo, Watershed. Hydrologic analysis has been conducted by estimating the volume of runoff by initial and constant rate method and the direct runoff by unit hydrograph method. The channel flow is modeled by Muskingum method and the values obtained have been transferred to Sile River by factorizing on areal basis. The high flood estimation resulted is 208.4 m3/s at the Sile River Bridge by taking the bridge crossing as an outlet. The inputs for the hydraulic model have been prepared by taking a river bed material sample and identifying the gradation in the laboratory test. The hydraulic evaluation by the Hec-RAS shows that the opening of the bridge is not sufficient to pass the 100 year return period flood. The hydraulic model shows that river course is constricted at the bridge increasing the flow velocity, so that the bed and bank materials have been scoured in a rapid manner. The Hec-RAS hydraulic model indicated that there will not be further scour on the river bed than the 2.21 m maximum depth and 3.31wide scour on the left bank. The bridge location at the bend of the river is the main reason for the increased the instability.**

Key words: Design Discharge, parameter transfer, Hec-HMS, Hec-RAS, river instability, scour, Kulfo, Sile, Bridge.

1. Introduction

Proper drainage is essential for a highway to function properly. As discussed in (Jones et al., 2004) the primary purposes of road drainage systems are to minimize water depths occurring on road surfaces during heavy storms and to prevent seepage causing damage to the pavement construction.

When the provided structures fail to accommodate the discharge the road is said to have drainage problem. The problem on highway drainage structures is world-wide. Even international organizations are established to mitigate the problems in this regard. The one is the RODEX Project which is a technical cooperation between road organizations across northern Europe. The document prepared by the organization describes the problem as follows. The fact, known for centuries, is that as long as road structures and sub grade soil do not have excess water the road will work well. But increased water content reduces the bearing capacity of a soil, which will increase the rate of deterioration and shorten the lifetime of the road. In such cases, the road will need rehabilitation more often than a well-drained road structure. Mainly the problem was observed on poorly working structures, such as, culverts, ditches, grass verges, poor cross fall and cracks (Saara and Saarenketo, 2006).

The case in US is given a great attention so that research canters concerning highway drainage problems are acting on it. Among the publications are the following two those are describing the level of consideration that should be given: 1) In road design it has been given more attention to the drainage systems for the highway designer, the primary focus is with the water that moves on the earth's surface and in particular that part which ultimately crosses transportation arterials, i.e., highway stream crossings (FHWA, 1996). 2) Therefore, a significant part of the cost of the most highway projects is attributable to drainage facilities, such as bridges, highway 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 (Olivera and Maidment, 1999).

In our country, the attempt to alleviate the failures on the drainage structures is very little, even though the problem is so much large. Many times side ditches, culverts and bridges are found to be clogged, collapsed and washed away by the flood. Consequently, the quality of roads is much deteriorated and their life time is shortened. To address these problems investigations are necessary. Special attention shall be given to the failures in bridge structures since any malfunction on these structures creates a wide-ranging problem.

The expansion of infrastructures is vital for a country. Even of most infrastructures, roads are the basic ones. In our country there is a good endeavor of expansion of roads, but many of them are not functioning well to the desired life time and quality. Out of the reasons, failure on cross drainage structures comes first. Therefore, addressing the problems related to road drainage and making the way of analysis and design to the state of the art is the current duty of the professionals with in the field Failure on the cross drainage structure is creating too many problems not only on the transport service but also on the economic development of the country. 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 our case the failure on the Sile River Bridge will be considered.

1.2. Objective of the Study

The general objective of this study is to investigate the cause of the Sile River bridge failures from hydrologic and hydraulic perspective with the following specific objectives to:1) To evaluate the bridge opening capacity, 2) to evaluate the stability of the river , and 3) to identify the dominant cause of failure.

2. Description of the sstudy area

The Sile River Bridge was constructed on the River Sile as cross drainage structure for the road that joins Arbaminch and Jinka towns on the Addis Ababa-Jinka main road. The River is located in the SNNP Regional State, Ethiopia. The river originates from highlands having about a 3350m height *masl* and finally at the bridge crossing the elevation becomes 1112 *masl*. The Sile River catchment, especially around the bridge crossing, has a semi-arid climate with an average annual minimum and maximum temperature of 17°C and 32°C, respectively and with bimodal pattern an average annual rainfall of 729.6 mm (Engdawork et al., 2002). There is a farming practice around the vicinity of the bridge. The dominant crops are Mango and Banana.

Regionally the watershed is located in part of the south western highland of Ethiopia and is characterized by highly to moderately rugged topography with a general decreasing trend of elevation from north- west towards south- east direction. Regionally the geology of the area is quaternary and tertiary volcanics with associated sediments, and structurally the area is situated in the south western section of the Main Ethiopian Rift Valley (AMU, 2009). From the 12 major river basins of Ethiopia, the Sile river is cited in the Rift valley basin, the Abaya-Chamo Sub-basin.

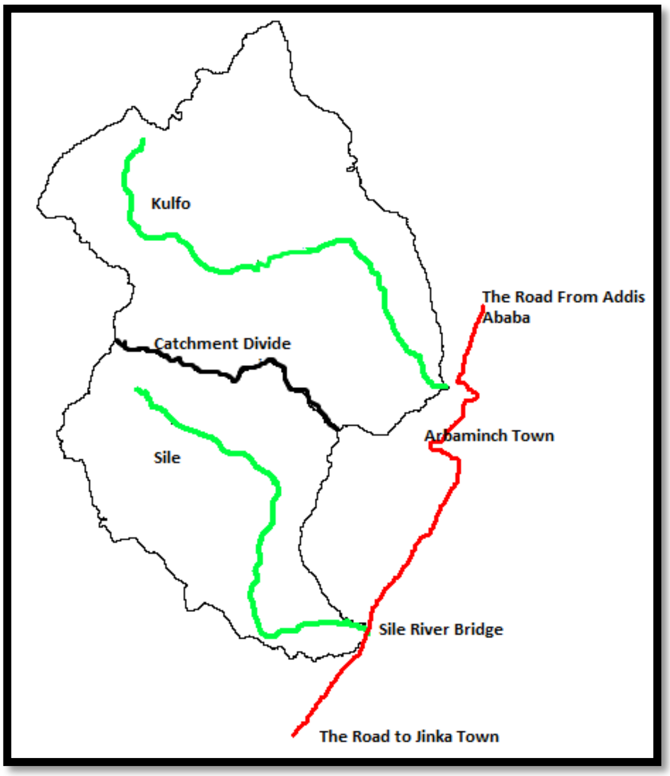


Figure.2.1. The Sile and Kulfo River Catchment Sharing a catchment divide.

Figure.2.2. Scour at Sile River Bridge

The Sile River Bridge is a 20m single span bridge with a clear height of about 3mas shown in figure 2.1. The bridge is located at the bend of the river. This bridge is seriously attacked by scour problem.

Figure 2.3. Bank with embankment protection work in May 2012.

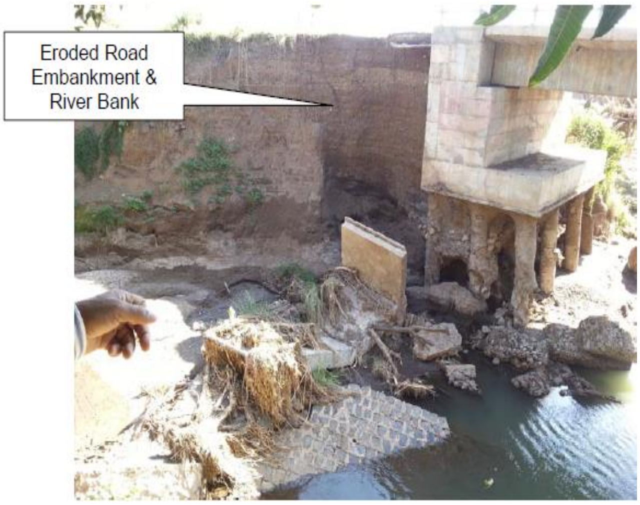


Figure 2.4. Damage on the left bank by the October 2012flooding.

The above figures, Figure 2.2 and Figure 2.3, show the scour on the left river bank is high and as well as on the road embankment is severe. All the protection work is washed away and the part of the road is eroded (Esmael, 2012).The protection works have been taken away by the flood. The change/ the failure rate is rapid as it is observed between May and December-2012. The rate could be illustrated using the picture taken before and after the rainy season of 2012 (Esmael, 2012). The supporting structures of the Sile River Bridge are exposed and are liable to collapse in the future. Currently, the bridge is closed for traffic due to severe erosion. And the service is being provided through temporary bialy bridge which is located just downstream of the existing bridge. The bialy bridge was used as service road during the construction and before the road was upgraded. The failures have to be investigated so that in the future such a problem should not be encountered i.e., the assessed failure causes should be addressed in future designs.

3. Materials and method

The conceptual frame work does explain where the main inputs are to be inserted; the tools to be used and help to understand the procedural flow of the duties with in the case study. The general workflow of this thesis work is presented as follows.

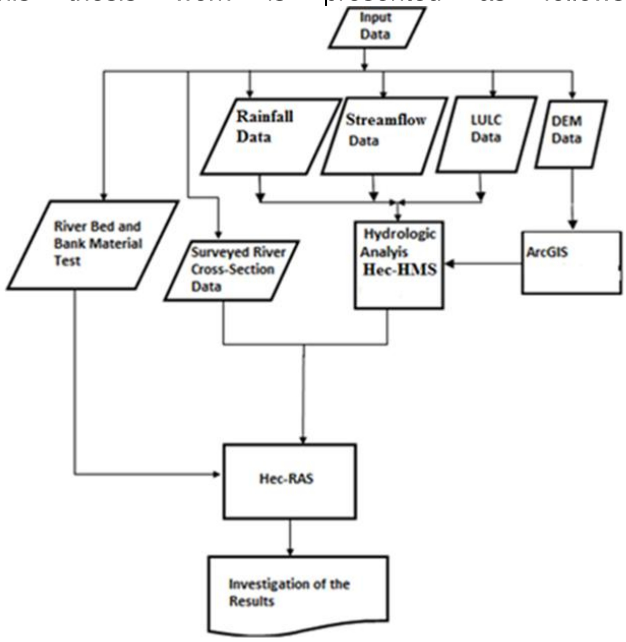


Figure 3.1. Conceptual Frame Work

Data Collection: Both the primary and secondary data were collected to accomplish this study. The primary data is the bed material and the bank material taken from the Sile River where the bridge is located. The secondary data collected are DEM, LULC and Soil map, Meteorological and river cross-section data.

3.1. Hydrologic Modeling

Hydrologic Modeling in the HEC-HMS:

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 (Does the model predict the variables required by the project such as peak flow, event volume and hydrograph, long-term sequence of flows?), 2) Hydrologic processes that need to be modelled to estimate the desired outputs adequately (Is the model capable of simulating regulated reservoir operation?),

3) Availability of input data (Can all the inputs required by the model be provided within the time and cost constraints of the project?), and 4) Price (Does the investment appear to be worthwhile for the objectives of the project?). For an event based modelling runoff volume and direct runoff are the most principal components of the water cycle. Both of these are given special attention in the Hec-HMS model.

Computing Runoff Volume: Hec-HMS Computes runoff volume by computing the volume of water that is intercepted, infiltrated, stored, evaporated, or transpired and subtracting it from the precipitation. Interception and surface storage are intended to represent the surface storage of water by trees or grass, local depressions in the ground surface, cracks and crevices in parking lots or roofs, or a surface area where water is not free to move as overland flow. Infiltration represents the movement of water to areas beneath the land surface. Interception, infiltration, storage, evaporation, and transpiration collectively are referred to in the program and documentation as losses. There are four types of loss models in the Hec-HMS model: The initial and constant-rate loss, deficit and constant-rate, the SCS curve number (CN) loss model, and the Green and Ampt loss model. With each model, precipitation loss is found for each computation time interval, and is subtracted from the MAP depth for that interval. The remaining depth is referred to as precipitation excess. This depth is considered uniformly distributed over a watershed area, so it represents a volume of runoff.

Initial and Constant Loss Model: The underlying concept of the initial and constant-rate loss model is that the maximum potential rate of precipitation loss is constant throughout an event. An initial loss is added to the model to represent interception and depression storage. Interception storage is a consequence of absorption of precipitation by surface cover, including plants in the watershed. Depression storage is a consequence of depressions in the watershed topography; water is stored in these and eventually infiltrates or evaporates. This loss occurs prior to the onset of runoff. Until the accumulated precipitation on the pervious area exceeds the initial loss volume, no runoff occurs.

Estimating Initial Loss and Constant Rate: the initial and constant-rate model, in fact, includes one parameter (the constant rate) and one initial condition (the initial loss). Respectively, these represent physical properties of the watershed soils and land use and the antecedent condition.

Modeling Direct Runoff: **t**here are models that simulate the process of direct runoff of excess precipitation on a watershed. This process refers to the “transformation’’ of precipitation excess into point runoff. There are different available models like: Snyder Unit Hydrograph, SCS Unit Hydrograph, Clark Unit Hydrograph, and Kinematic Wave Model. Among the above listed models, the SCS Unit Hydrograph Model is chosen. The SCS UH Model is a dimensionless, single-peaked UH. The UH peak and the time of peak is related to the duration of the unit of excess precipitation are related by (Feldman, 2000) equation as:

$$Up = C\frac{A}{Tp}. \qquad 3.1$$

$$\mathrm{T_p} = \frac{\Delta t}{2} + t_{lag}. \qquad 3.2$$

In which A = watershed area; and C = conversion constant (2.08 In SI and 484 in foot-pound system). Δt = the excess precipitation duration (which is also the computational interval in the run); and

t_{lag} = the basin lag, defined as the time difference between the centre of mass of rainfall excess and the peak of the UH.

Modelling Channel Flow: The models of channel flow available for the study are:Lag, Muskingum, Modified Puls, Kinematic-wave, and Muskingum Cunge. Each of these models computes a downstream hydrograph, given an upstream hydrograph as a boundary condition. The brief review on the selected routing method, Muskingum, is presented below.

Steady Flow Water Surface Profiles: 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. The energy equation is written as follows (Gary, 2010):

$$Z_2 + Y_2 + \frac{\alpha_2 V_2{}^2}{2g} = Z_1 + Y_{1+}Z1 + \frac{\alpha_1 V_1{}^2}{2g} + h_e \qquad 3.3$$

Where: Y_1, Y_2 : Depth of water at cross-sectioins, Z_1, Z_2 : Elevation of the main channel inverts and α_1, α_2 are Velocity weighing coefficients for V_1, V_2 (also called as Average velocities and equal to (Total discharge/Total flow area),g is the gravitational acceleration, and h_e: Energy head loss.

The energy head loss (h_e) 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, 2010):

Where: $\overline{S_f}$= Representative friction slope between two section, L=Discharge weighted reach length, C = Expansion or contraction loss coefficient, the distance weighted reach length, L, is calculated as (Gary, 2010):

$$\mathrm{L} = \frac{\mathrm{L_{lob}}\,\overline{\mathrm{Q}}_{\mathrm{lob}} + \mathrm{L_{ch}}\,\overline{\mathrm{Q}}_{\mathrm{ch}} + \mathrm{L_{rob}}\,\overline{\mathrm{Q}}_{\mathrm{rob}}}{\overline{\mathrm{Q}}_{\mathrm{lob}} + \overline{\mathrm{Q}}_{\mathrm{ch}} + \overline{\mathrm{Q}}_{\mathrm{rob}}} \qquad 3.4$$

Where: $\mathrm{L_{lob}}, \mathrm{L_{ch}}, \mathrm{L_{rob}}$ = Cross section reach lengths specified for flow in the left overbank, main channel, and right over bank, respectively, and $\overline{\mathrm{Q}}_{\mathrm{lob}} + \overline{\mathrm{Q}}_{\mathrm{ch}} + \overline{\mathrm{Q}}_{\mathrm{rob}}$ are arithmetic average of the flows between sections for the left overbank, main channel, and right over bank, respectively.

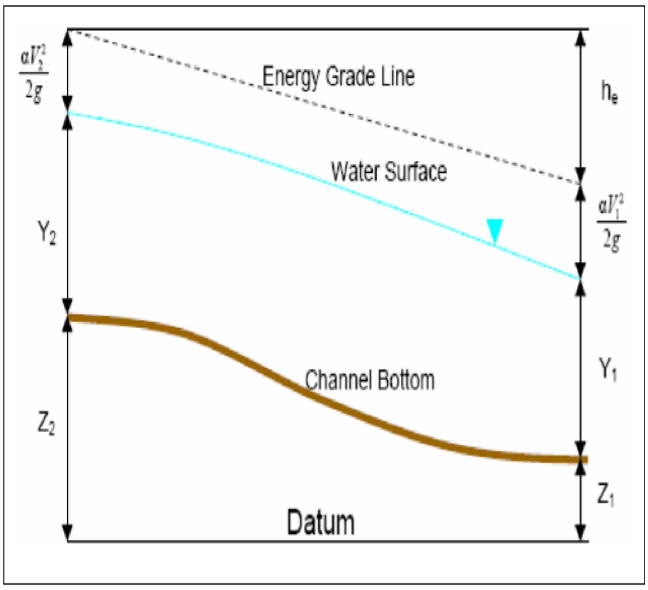


Figure.3.2. Representation of Terms in energy Equation (adopted from Gary, 2010).

Modeling Scour: In the hydraulic model it is optional to calibrate the model. Hec-RAS recommends calibration if observed data is available. The design event for a scour analysis is usually the 100 year event. In addition to this event, it is recommended that a 500 year event also be used to evaluate the bridge foundation under a super-flood condition. After performing the water surface profile calculations for the design events, the bridge scour can then be evaluated. The total scour at a highway crossing is comprised of three components: long-term aggradation or degradation; contraction scour; and local scour at pier and abutments. The scour computations in the HEC-RAS software allow the user to compute contraction sour and local scour at piers and abutments.

Live-Bed or Clear-Water Contraction Scour: To determine if the flow upstream is transporting bed material, the program calculates the critical velocity for beginning of motion V_c (for the D_{50}size of bed material) and compares it with the mean velocity V of the flow in the main channel or overbank area upstream of the bridge at the approach section. If the critical velocity of the bed material is greater than the mean velocity at the approach section (Vc > V), then clear-water contraction scour is assumed. If the critical velocity of the bed material is less than the mean velocity at the approach section (Vc < V), then live-bed contraction scour is assumed (Gary, 2010).

$$V_c = K_u y_1^{1/6} D_{50}^{1/3}. \qquad 3.5$$

Where:V_c = Critical velocity above which material of size D_{50} and smaller will be transported, (m/s), y_1 = Average depth of flow in the main channel or overbank area at the approach section (m), D_{50} = Bed material particle size in mixture of which 50% are smaller (m), and K_u = 6.19.

Live Bed Contraction Scour: The modified version of Laursen's (1960) live-bed scour equation is used:

$$Y_2 = Y_1 \left[\frac{Q_2}{Q_1}\right]^{6/7} \left[\frac{W_1}{W_2}\right]^{K_1} \qquad 3.6$$

$$ys = y2 - yo. \qquad 3.7$$

Where: ys = Average depth of contraction scour.

y2 = Average depth after scour in the contracted section. This is taken as the section inside the bridge at the upstream end in Hec-RAS.

y_1 = Average depth in the main channel or floodplain at the approach section.

y_o = Average depth in the main channel or floodplain at the contracted section before scour.

Q_1 = Flow in the main channel or floodplain at the approach section, which is transporting sediment.

Q2 = Flow in the main channel or floodplain at the contracted section, which is transporting sediment.

W_1 = Bottom width in the main channel or floodplain at the approach section. This is approximated as the top width of the active flow area in Hec-RAS.

W_2 = Bottom width of the main channel or floodplain at the contracted section less pier widths. This is approximated as the top width of the active flow area.

K_1 = Exponent for mode of bed material transport.

Table 3.1. Exponent for mode of bed material transport (Adopted from Gary, 2010).

V* /ω	k_1	Mode of Bed Material Transport
< 0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
> 2.0	0.69	Mostly suspended bed material discharge

$V^* = (gy_1S_1)^{1/2}$, shear velocity in the main channel or floodplain at the approach section.

ω = fall velocity of bed material based on D_{50} (m/s), g = Acceleration of gravity, (m/s^2), S_1 = Slope of the energy grade line at the approach section, (m/m).

Clear-Water Contraction Scour: The recommended clear-water contraction scour equation by the model is an equation based on research from Laursen (1963):

$$Y_2 = \left[\frac{Q_2^2}{C D_m^{2/3} W_2^2}\right]^{3/7} \qquad 3.8$$

Where: D_m = Diameter of the smallest non-transportable particle in the bed material ($1.25D_{50}$) in the contracted section, D_{50} = Median diameter of the bed material, C = 40. Note: If the bridge opening has overbank area, then a separate contraction scour computation is made for the main channel and each of the banks.

Local Scour at Abutments: When the wetted embankment length (L) divided by the approach flow depth (y_1) is greater than 25, the suggested equation is the HIRE equation (Richardson, 1990). When the wetted embankment length divided by the approach depth is less than or equal to 25, the equation by *Froehlich* is better.

The HIRE Equation: The HIRE equation is (Gary, 2010):

$$Y_s = 4Y_1\left(\frac{K_1}{0.55}\right)K_2Fr_1^{0.33} \qquad 3.9$$

Where: y_s = Scour depth

y_1 = Depth of flow at the toe of the abutment on the overbank or in the main channel, taken at the cross section just upstream of the bridge.

K_1 = Correction factor for abutment shape, Table 3.2.

K_2 = Correction factor for angle of attack (θ) of flow with abutment. θ = 90 when abutments are perpendicular to the flow, θ < 90 if embankment points downstream, and θ> 90 if embankment points upstream. $K_2 = (\theta/90)^{0.13}$.

Fr1 = Froude number based on velocity and depth adjacent and just upstream of the

Abutment toe.

Table 3.2. Correction Factor for Abutment

Description	K1
Vertical-wall Abutment	1.00
Vertical-wall Abutment with wing walls	0.82
Spill through Abutment	0.55

Shape, K_1 (Adopted from (Gary, 2010)

The correction factor, K_2, for angle of attack can be taken from Figure 4.4 (Gary, 2010).

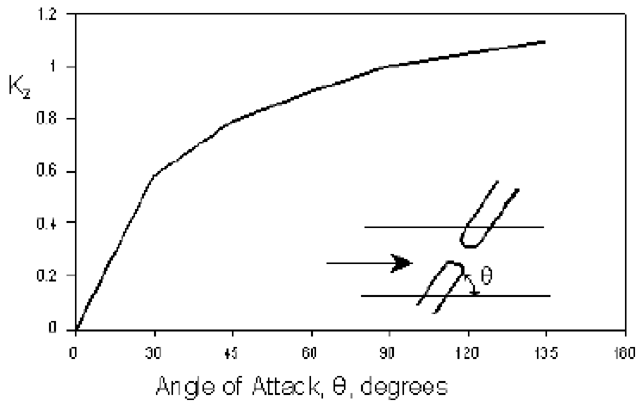


Figure 3.3. Correction Factor for Abutment Skew, K2(Adopted from Gary, 2010)

3.2. Calibration and Validation of the HEC-HMS model

Calibration: Whatever the model form is chosen, there are some unknown constants used to represent the physical process. These so called parameters of the model must be assigned fixed numerical values before the model may be used to predict the runoff, in other words one needs to estimate these parameters such that the best agreement between modelled and observed runoff can be obtained.

Validation: Testing or verification or validation of a model after the parameter values are estimated is the third level of model analysis. As no model is perfect, verification requires both subjective and objective judgments on many aspects to determine whether the results provide adequate information for answering the question facing the decision-makers, and all models can be expected to fail at least on some occasions.

The model is calibrated for 3 events. Then after, the model is validated by 2 events by using the average of calibrated parameters to see performance of the model. In order to evaluate the models, hydrographs were compared visually before using mathematical measure. Since the goal of the hydrologic model in this thesis work is to estimate the peak discharge to compare the observed and computed hydrograph it has been used the percentage error in peak method.

Percent error in peak: measures only the goodness-of-fit of the computed-hydrograph peak to the observed peak. It quantifies the fit as the absolute value of difference, expressed as a percentage, thus treating overestimates and underestimates as equally undesirable. It does not reflect errors in volume or peak timing. This method is a logical choice if the information needed for designing or planning is limited to peak flow or peak stages. This might be the case for studies such as flow and stage uniquely related (Feldman, 2000).

$$Z = 100\frac{q_s(peak) - q_o(peak)}{q_o(peak)}. \qquad 3.10$$

Where Z = objective function; q_o (peak) = observed flows; q_s (peak) = calculated flows ***River Bed and Bank Material Gradation:*** In order to determine the percentage of different grain sizes contained within the bed and bank material it is necessary to perform a sieve analysis test. The test needs the following equipments; Balance, Set of sieves, cleaning brush, sieve shaker.

Test Procedure: The test procedure is adopted from University of Illinois laboratory procedure. (Krishna, 2002), including the sieve analysis with the following procedures; 1) Write down the weight of each sieve as well as the bottom pan to be used in the analysis, 2) Record the weight of the given dry soil sample, 3) Make sure that all the sieves are clean, and assemble them in the ascending order of sieve numbers. Place the pan below last sieve. Carefully pour the soil sample into the top sieve and place the cap over it, 4) Place the sieve stack in the mechanical shaker and shake for 10 minutes, 5) Remove the stack from the shaker and carefully weigh and record the weight of each sieve with its retained soil. In addition, remember to weigh and record the weight of the bottom pan with its retained fine soil.

Data Analysis*:* is a procedure to 1) obtain the mass of soil retained on each sieve by subtracting the weight of the empty sieve from the mass of the sieve + retained soil, and record this mass as the weight retained on the data sheet. The sum of these retained masses should be approximately equals the initial mass of the soil sample. A loss of more than two percent is unsatisfactory, 2) calculate the percent retained on each sieve by dividing the weight retained on each sieve by the original sample mass, 3) Calculate the percent passing (or percent finer) by starting with 100 percent and subtracting the percent retained on each sieve as a cumulative procedure, and 4) Make a semi-logarithmic plot of grain size versus percent finer.

4. Result and discussion

Two models are used, a hydrologic model HEC-HMS and a hydraulic model HEC-RAS. Then using Ministry of Water and Energy stream gage and Meteorology Agency weather station data, the Hec-HMS model is calibrated and validated for five storm events. The hydrologic model is used to generate runoff for the SCS 100 year 24-hr design storm on the Sile River. The design discharge from the hydraulic model is then used to evaluate the bridge opening capacity by the hydraulic model. Qualitatively the stability of the bridge is also seen.

Parameter Transfer: The Sile River where the case study conducted is ungauged catchment and it requires rainfall-runoff parameters from a neighboring catchment. The Kulfo River is found sharing a catchment divide with Sile River as shown in figure 2.2 below and parameter transfer is carried out from it.

The HEC-HMS model has been applied for 5 rainfall events of Kulfo watershed. The model has been calibrated for three rainfall events and validated for two rainfall events. The calibration parameters for rainfall events are given in Table 4.1.

The model parameters have been calibrated by changing the parameters like Initial Loss, constantan rate loss, SCS UH lag time, Muskingum K and Muskingum X.

Table 4.1. Calibrated Parameters for rainfall events of Kulfo Watershed

Date	Initial Loss (mm)	Constant Rate (mm/hr)	SCS UH Lagtime	Muskingum K	Muskingum X
9/1/2007-9/6/2007	3.3	0.38	1000	5	0.3
1/14/2007-1/20/2007	3	0.45	1000	7	0.35
8/6/2006-8/12/2006	3	0.6	1000	7	0.35
Average	3.1	0.48	1000	6.33	0.33

From the visual observation of simulated hydrographs, it is seen that there is an improvement of peak runoff and time to peak by changing the above parameters. Validation of three events has been carried out by averaging values of all calibrated events as shown in Table 4.2.

Table 4.2. Model Result for rainfall events of Kulfo Watershed

Date of Rainfall Events	Peak Runoff (m3/s)		Error%
	Observed	Simulated	
Calibrated Events			
8/6/2006-8/12/2006	25.7	27.5	-7.00
9/1/2007-9/6/2007	19.4	21.6	-11.34
1/14/2007-1/20/2007	24.4	25	-2.46
Validation Events			
5/7/2007-5/13/2007	24.4	26.1	-6.97
10/27/2005-10/31/2005	17.7	21.8	-23.16

Design Discharge Estimation at Sile River Birdge*:* After calibrating the model, the parameters are factored based on the areal factors. From the average values which are used for validating the model are used to estimate the design discharge. According to the ERA (Ethiopian Roads Authority) rainfall region classification the region in which both the Kulfo and Sile watersheds located is Region B2. For each region the respective SCS 24Hr rainfall is allocated. The table below, Table 4.3, is adopted from the ERA Drainage Design Manual showing 24Hr rainfall Depth Vs Frequency.RR is the rainfall region.

Table 4.3. 24 Hr rainfall Depth Vs Frequency (Adopted from ERA, 2012)

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

After factoring based on area, the transferred model parameter values are found to be of Initial Loss of 1.86mm, Constant Rate of 0.29 mm/hr, SCS UH Lag Time of 599.53, and Muskingum K of 3.80 and Muskingum X of 0.20. Using these transferred parameters as well as the 100 yr SCS 24-hr storm, which is adopted from ERA Drainage Design Manual, as inputs the event based modeling in Hec-HMS has resulted in the following hydrograph, Figure 4.1. It shall be noted that the simulation days assigned on the hydrograph are arbitrary.

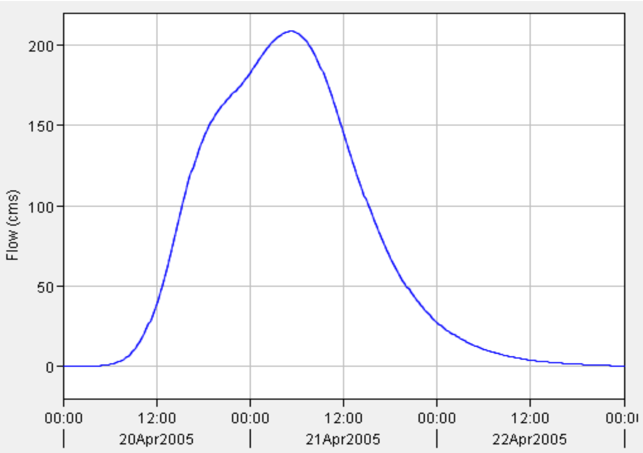


Figure 4.1. The hydrograph at the Sile river bridge

The peak discharge on the River Sile at the bridge outlet is estimated to be 208.4m3/s. It is difficult to find the hydrologic analysis for the Sile river bridge except the as built drawing from the consulting office. Additionally the documents submitted to the ERA from the consulting office show that there is no hydrologic analysis performed to estimate the design discharge. As a result it is not possible to compare the design discharge estimated to the original design on which the bridge opening capacity was determined.

Evaluating the bridge opening capacity: The figure shows whether the bridge can accommodate the design discharge of 100yrs storm under the current condition. The Hydraulic model results (Hec-RAS) show that the bridge is not enough to pass the discharge value. Figure 4.2 reveals this fact.

The result from the model shows that the water surface profile will acquire a height of 7.72 m with the 100yr design discharge. There was a 3m clear height of the bridge and additional 3m scour totaled into 6m height for passage of water, though the discharge still cannot be accommodated with this opening.

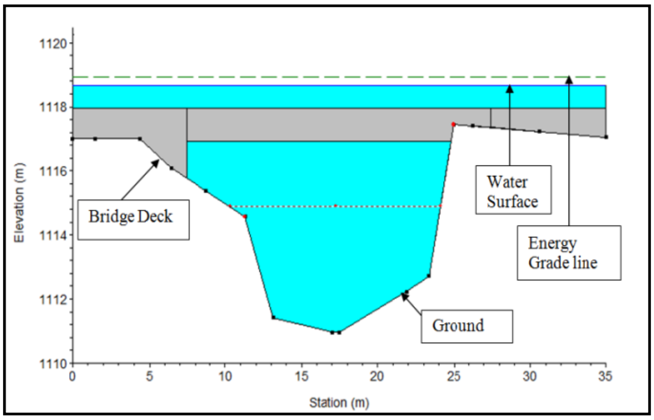


Figure 4.2. Modeled Water Surface profile on the Sile River Bridge

Evaluating the River Stability: The hydraulic structure provided, the bridge, has induced some sort of instability. The opening of the bridge was both understated and constricted. Therefore, the flood is forced to increase velocity when it passes through the bridge. High velocity has an erosive power and the bed material is eroded below the original bed level 3m depth. The river bank is still being scoured heavily every wet season. While modelling the scour at the banks, the equation from Froehlich has been used as per the recommendation from the hydraulic reference manual. The Hec-RAS model output, 4.3., shows that there will not be additional scour below the current bed level but it has been estimated a 2.21 m maximum depth and 3.31wide scour on the left bank of the river at the bridge site.

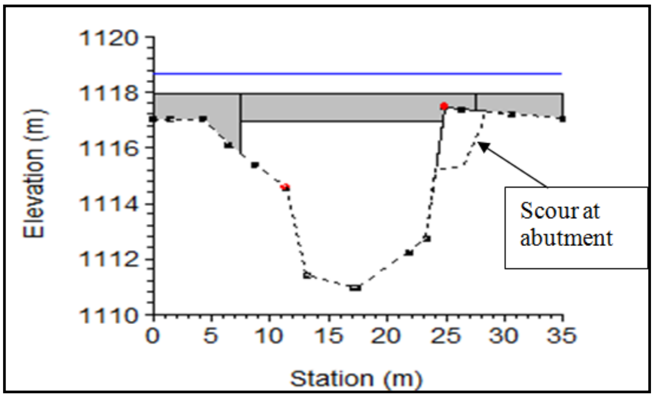


Figure 4.3. Scour depth as modelled by the Hec-RAS

The other important point is that the location of the bridge is just at the river bend as can be seen in Figure 5-5. The bends with alluvial rivers are naturally the sources of instability. Even low flows have the capacity to influence the stability of the river at bends of alluvial rivers (Lagasse et al., 2012). The case on the Sile river bridge can be seen from this perspective. Even if it has been tried to mitigate the problem by provision of river training structures, the gabions and the bank of the river are still in active erosion.

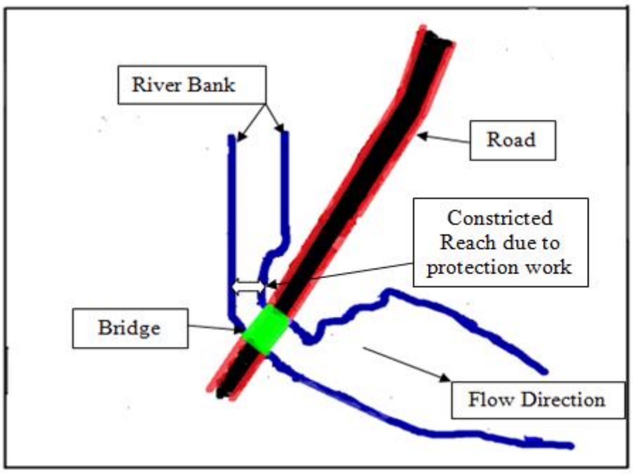


Figure 4.4. The location of the Sile River Bridge

The river starts to constrict just upstream of the bridge abruptly. This constriction causes an increase in velocity of flow. In addition to that the river will not have a chance to self adjustment since the river training work is rigid. As a result of this one of the causes of failure on the bridge is related to the river training work (Li, 2006).

Sieve Analysis Result: By taking the sample mass of 3000gm the sediment under the bridge is graded. The test result is shown in the table below, Table 4.4. The error in the test is 0.5 gm. The value of 50 percent finer, D50, is 16.725 gm.

Table 4.4. Sieve Analysis Result

No.	Sieve size (mm)	Mass of each sieve (gm)	Mass of each sieve + retained soil(gm)	Mass of soil retained-Wn (gm)	Percentage on each sieve, Rn	Cumulative percent retained ∑Rn	% finer, 100- ∑Rn
				Col4-Col3	Col5/Wt *100		
Col1	Col2	Col3	Col4	Col5	Col6	Col7	Col8
1	75mm	454.50	454.50	0.00	0.00	0.00	100.00
2	37.5m	494.00	1261.50	767.50	25.58	25.58	74.42
3	19mm	488.00	1095.00	607.00	20.23	45.82	54.18
4	9.5m	489.00	1084.50	595.50	19.85	65.67	34.33
5	4.75mm	462.50	787.50	325.00	10.83	76.50	23.50
6	2.36mm	439.00	601.00	162.00	5.40	81.90	18.10
7	1.18mm	436.50	548.00	111.50	3.72	85.62	14.38
8	600µ	398.00	515.00	117.00	3.90	89.52	10.48
9	300µ	365.50	548.00	182.50	6.08	95.60	4.40
10	150µ	287.00	383.50	96.50	3.22	98.82	1.18
11	75µ	342.00	364.50	22.50	0.75	99.57	0.43
12	Pan	257.00	269.50	12.50	0.42	99.98	0.02
				Total=2999.5			

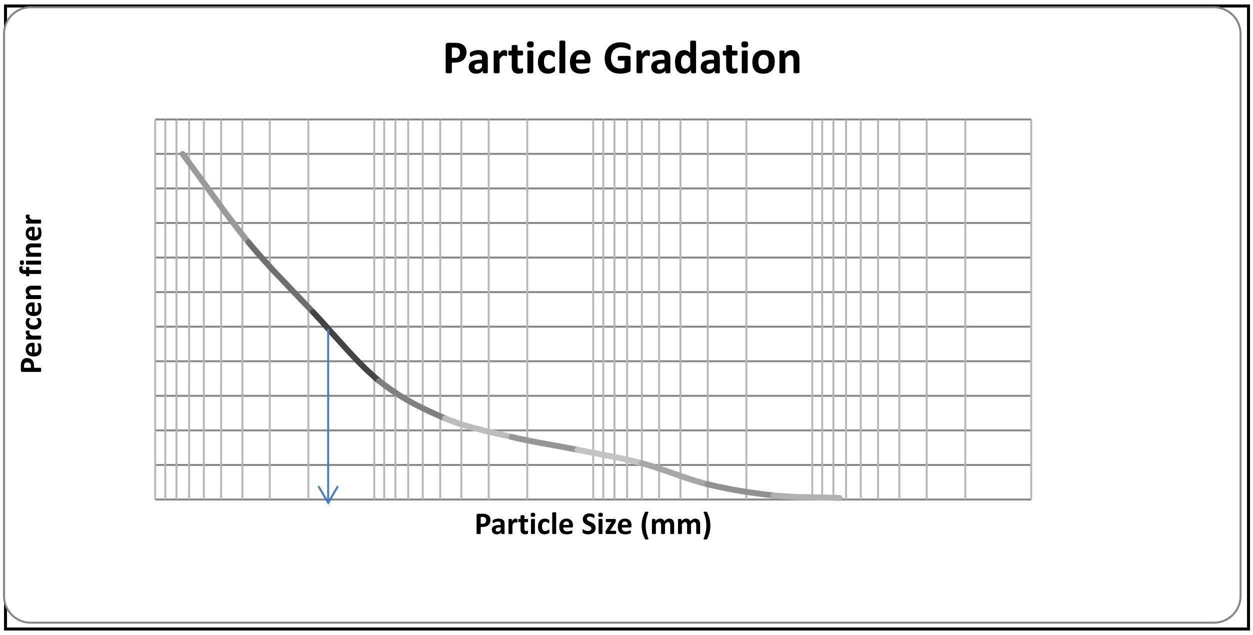
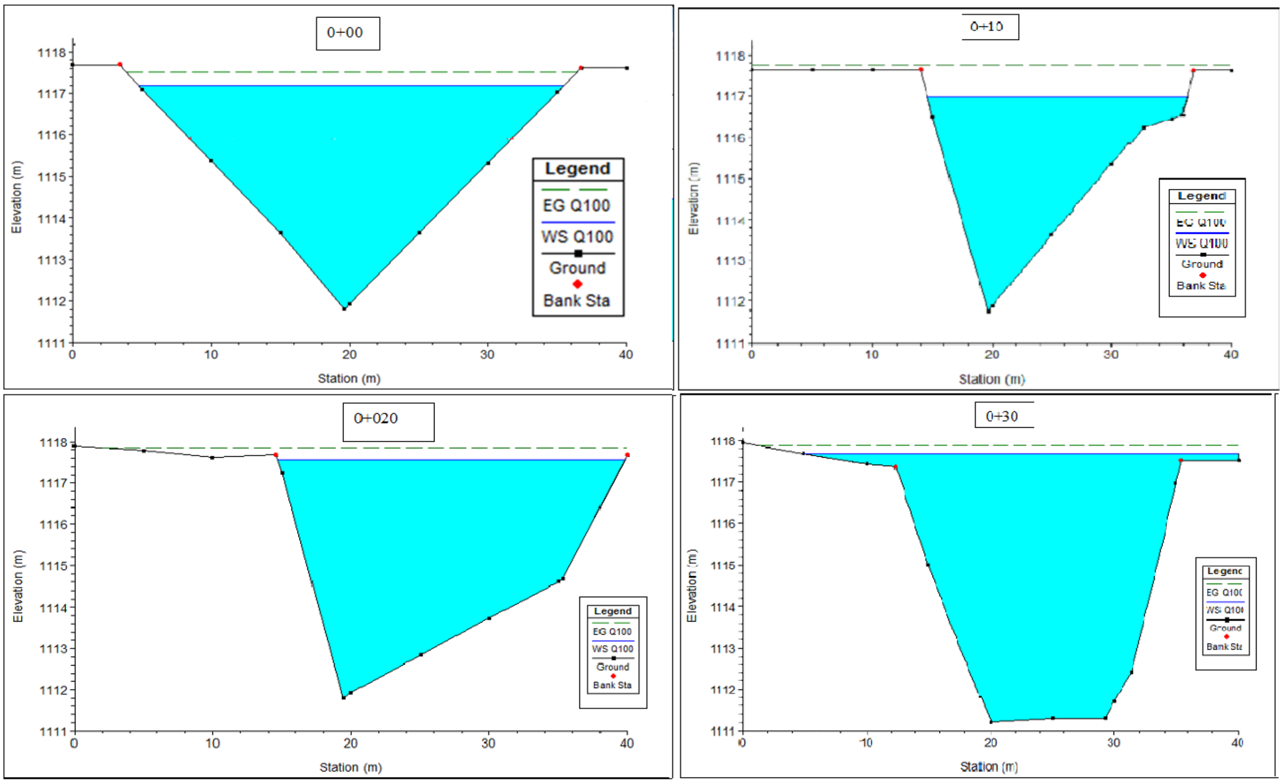
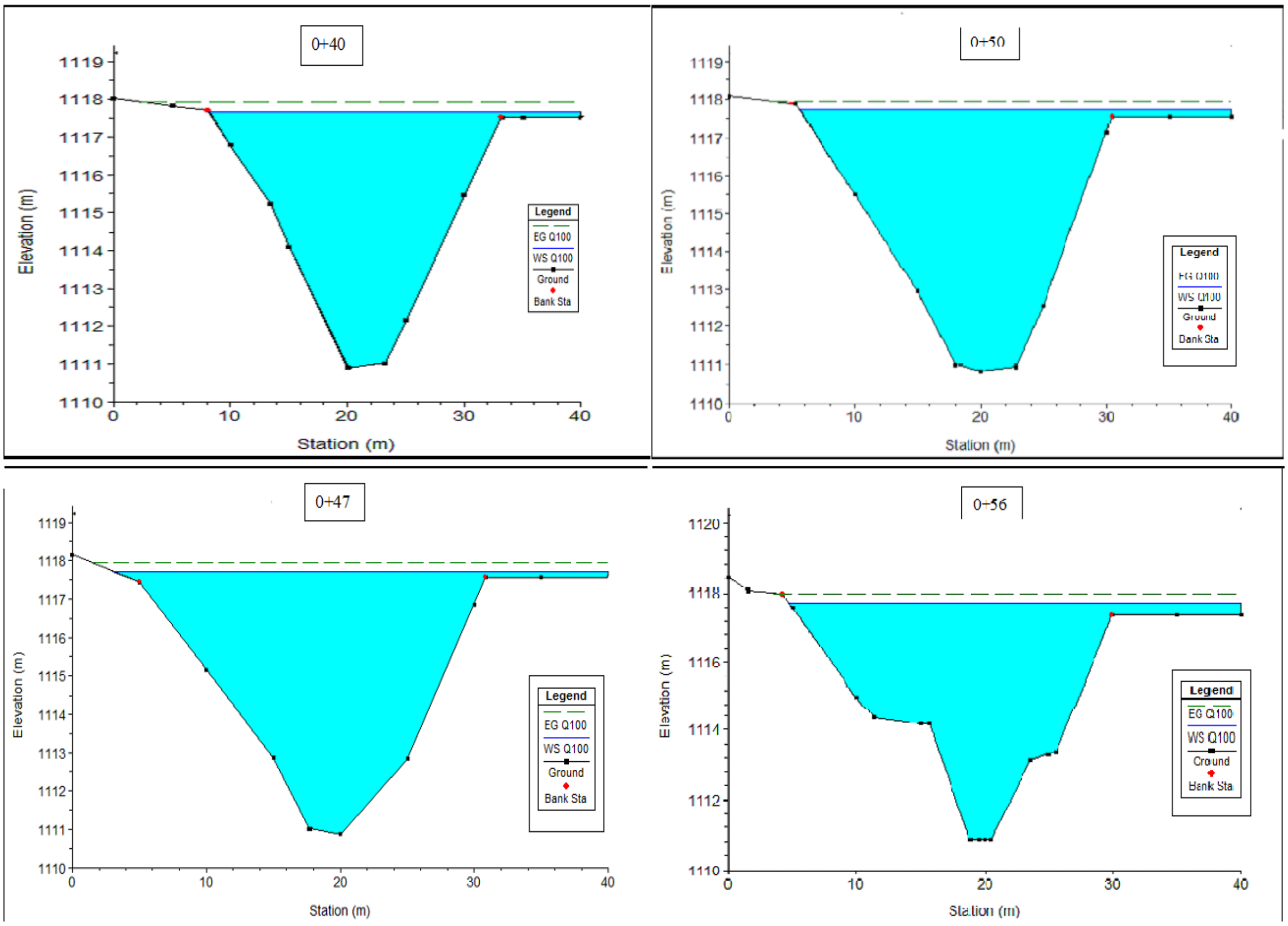


Figure 4.5. Particle size vs. percent fine

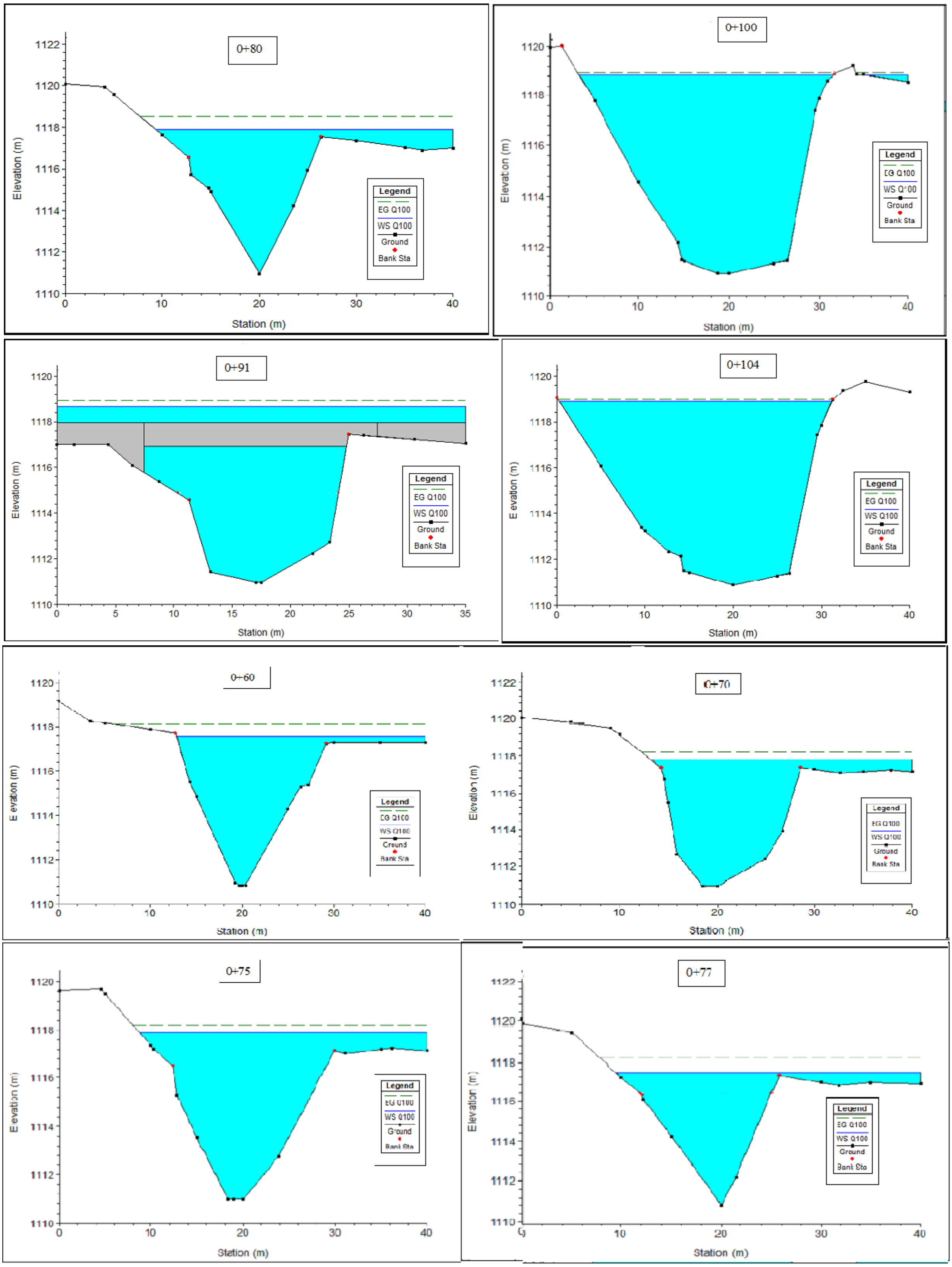
Spatial dynamism of river cross section: Sile river cross section is changing from chainage to chainage and this spatially dynamic varying cross section geometry is described by the following HEC-RAS output figural desplays in figures 4.6-4.9.



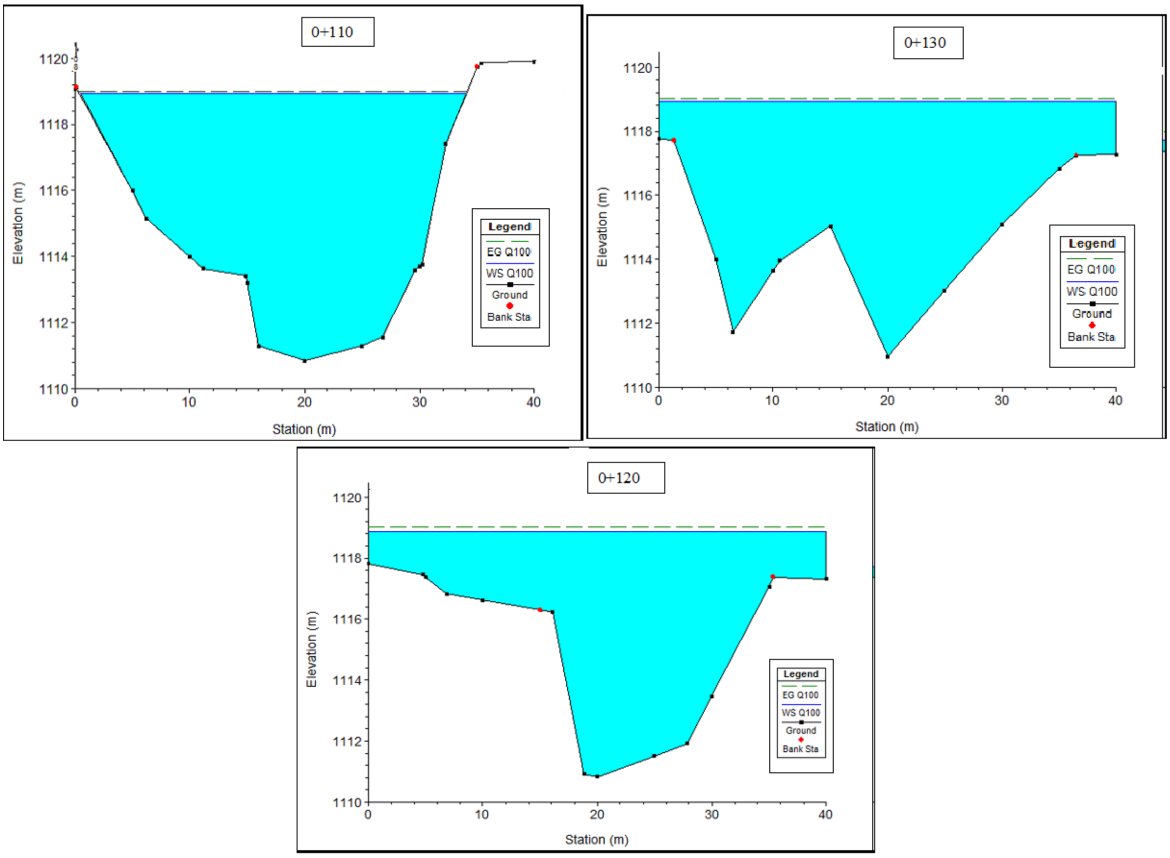
figures 4.6. Spatial dynamism of river cross section for chainage 0+000 to 0+030m.



figures 4.7. Spatial dynamism of river cross section for chainage 0+040 to 0+056m.



figures 4.8. Spatial dynamism of river cross section for chainage 0+060 to 0+077m.



figures 4.9. Spatial dynamism of river cross section for chainage 0+080 to 0+0104m

5. Conclusion and recommendation

The study leads to a conclusion that the bridge crossing is understated. As a result the constricted flow developed a power to Scour the bed and bank of the bridge. It was necessary to design the bridge opening considering the catchment's rainfall potential. Hydraulically the bridge is not found to be placed on the right position.

The bridge should be located to other position where the river reach is straight. Since the area is flat, besides the main bridge some additional culverts should be Provided to accommodate the surplus discharge from the main channel. It is also useful if stream flow gauge is installed on the crossing and ERA should build its capacity on providing data for cross drainage analysis and design

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