DIGITAL MODELLING ON BACKWATER EFFECT DUE TO BRIDGE PIERS AT RASAU RIVER

MUHAMMAD SHABIRIN BIN SHAROM

B. ENG(HONS.) CIVIL ENGINEERING

UNIVERSITI MALAYSIA PAHANG

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MUHAMMAD SHABIRIN BIN SHAROM

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ABSTRAK

Jambatan adalah salah satu struktur penting untuk menghubungkan dua tempat yang biasanya dipisahkan oleh sungai. Pembangunan yang pesat berlaku berhampiran kawasan sungai telah mengakibatkan peningkatan permintaan infrastruktur sungai seperti jambatan. Jambatan baru yang dipanggil Jambatan 3 di Sungai Rasau, Gambang sedang dalam pembinaan untuk menggantikan jambatan yang sedia ada, jesteru, adalah penting untuk memahami ciri hidrologi dan hidraulik sungai untuk memastikan kecekapan reka bentuk jambatan dan keselamatannya. Salah satu fenomena yang boleh berlaku disebabkan oleh kehadiran ceracak jambatan di dalam sungai adalah air berbalik. Air berbalik boleh menyebabkan banyak kerosakan alam sekitar dan oleh itu adalah penting untuk mempertimbangkan kesan air berbalik walaupun tapak jambatan di Sungai Rasau yang dipilih mempunyai tebing yang stabil dan saluran yang lurus. Dalam kajian ini, kaedah yang akan digunakan untuk menentukan kesan air berbalik adalah analisis simulasi profil air menggunakan perisian HEC-RAS. Kes yang terlibat dalam simulasi ini adalah tanpa ceracak jambatan dan dengan kehadiran ceracak jambatan. Hasil simulasi ditunjukkan dalam bentuk profil paras air melawan stesen keratan rentas. Dari kajian ini, dengan adanya jambatan dan aliran puncak 100 tahun, paras air di bawah jambatan telah mematuhi Garis Panduan Pembangunan Sungai kerana mempunyai paras air 11.18m dari paras laut dan ‘freeboard’sepanjang 0.74m di bawah permukaan jambatan. Selain itu, air akan melimpah ke kiri dan kanan tebing sungai tetapi masih tidak melebihi paras jalan raya.

ABSTRACT

Bridge is one of the important structures to connect two places usually separated by a river. Massive development occurs near the river area which resulted in increase in demand of river infrastructure such as bridge. A new bridge called Bridge 3 at Rasau River, Gambang is under construction to replace the existing bridge. Thus it is important to understand about the hydrology and hydraulic characteristic of the river to ensure the efficiency of the bridge's design and its safety. One of the phenomena that can cause by the bridge piers is backwater. Backwater can cause a lot of environmental damages and, hence, it is crucial to consider the backwater effect even the bridge site at the Rasau River selected has stable banks and straight channel. In this study, HEC-RAS software was used to investigate the backwater effect through analysis of water profile. Simulations were carried out without and with the presence of the bridge piers. The result of the simulation showed in the form of water level versus station of the cross section. From this study, with the presence of bridge piers and 100-year ARI, the water level under the bridge had comply with the Guideline for River Development for having a water level of 11.18m from sea level and freeboard of 0.74m under the soffit of the bridge. Apart from that, overflows occurred at both the left and right of the river bank but not exceeded the road levels.

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LIST OF SYMBOLS

Q	Flow rate
i	Rainfall intensity
T_c	Time of concentration
A	Catchment area
α	Porosity
w	Volume of water required
v	Total of the rock
V	Average velocity
y	Flow depth
T	Top width
P	Wetted perimeter
R	Hydraulic radius
d_*	Dimensionless particle diameter
d_s	Particle size
G	Specific gravity of the sediment
g	Gravitational acceleration
v	Kinematic viscosity
Fr_3	Froude number at section 3 downstream of piers
K	Bridge pier backwater coefficient
C_d	Discharge coefficient
H	Total energy different upstream and downstream
β	Stream top width
q	Lateral flow channel per unit length of channel
x	Distance along the channel
t	Time
n	Manning roughness coefficient

LIST OF ABBREVIATIONS

CH	Chainage
cfs	Cubic feet per second
m/s	Meter per second
m^3/s	Cubic meter per second
DID	Department of Irrigation and Drainage
MSMA	Urban Stormwater Management Manual for Malaysia
ARI	Annual recurrence interval

CHAPTER 1

INTRODUCTION

1.1 RESEARCH BACKGROUND

Bridge is one of the important structures to connect two places usually separated by a river. Massive development occurs near the river area which resulted in increase in demand of river infrastructure such as bridge is also increase.

It is crucial to understand the hydrology and hydraulic characteristics of the river in order to ensure the efficiency of the bridge's design and its safety. The piers can act as obstruction to the current flow which can cause a lot of environment disasters as water level increase at the upstream during heavy rain season resulted in flooding to the surrounding area. Furthermore, as water level increase, scour of bridge pier and backwater effect can also occured which in return increase the maintenance cost in the future.

Backwater can cause a lot of environment damages, and thus it is necessary to consider the backwater effect even the bridge site at Rasau River were selected as it has stable banks and straight channel. Selection of site is to avoid postulated hydraulic effects caused by the bridge but it is by no means a simple task to measure (Bradley, 1960).

1.2 PROBLEM STATEMENT

Backwater phenomena occur when there is obstruction of flow which is for this case, the obstruction is caused by the bridge piers and it is such unavoidable hydraulic effect. River water level increase above the normal level is known as the phenomena mentioned. Backwater effect should be in consideration when designing the bridge piers because the number of piers and its arrangement is the major factor affected the problem.

Bridge 3 at Rasau River is one of the bridge construction projects among the three bridges under Project of Upgrading the Federal Road Links Gambang and Segamat. As it is located in Gambang, Pahang the East Coast area where monsoon season keep repeated around the month of November to March every year, hydraulic engineer need to focus in any uncertainty that can affect the bridge construction. The uncertainty mentioned here is the cause of backwater effect due to bridge pier that can leads to occurrences of flood and excessive piers scour.

Backwater effect can increase drastically with the heavy rain season lasted for a long period of time as amount of water flow in the upstream of the bridge increases. Occurrence of flood to area nearby might happen as a result of the backwater effect. The disaster occurrence can leads to the destruction of the locality property also the destruction of Mother Nature’s; wild life habitat and their life, nearby the river. In addition, the water can be over flow up to the above soffit level of the bridge and then on to the road which finally results in damages the bridge surface or road layers.

Furthermore, increase in number of destruction especially the bridge structure component itself can leads to high cost of maintenance. Therefore, it is crucial to investigate how to overcome backwater effect of the Rasau River due to bridge piers and its shape. One of the flooding issues by the river in the East Coast area of Malaysia is in Kelantan River at Tambatan DiRaja, Kelantan which took place in the year 2014. The flood event was the worst recorded in the history of the state which recorded a level of 34.17m compared to that of 29.70m in 2004 and 33.61m in 1967 (Davies, 2015). Figure 1.1 illustrates the Sultan Yahya Petra Bridge located across the Kelantan River where bridge piers act as the structural element caused to backwater in the river.

Figure 1.1 Sultan Yahya Petra Bridge, Kelantan

1.3 RESEARCH OBJECTIVE

This research outlines the following objectives:

i. To determine the backwater effect due to presence of bridge piers.

ii. To determine the water level profile of Rasau River with and without bridge presence.

1.4 SCOPE OF RESEARCH

This research is about the analysis of water profile and hydraulic characteristic which focus on backwater flow at the upstream and extended flow at the downstream of the Bridge 3, Rasau River. The analysis is by using HEC-HMS and HEC-RAS software which is the widely used software to compute water surface profile through system of open channel.

In order to achieve the objectives, the research focused on the analysis of the subcritical and supercritical flow with collected data such as velocity of the river, flow rate (Q), normal depth and increased depth of water level at the upstream. The research focused on the Rasau River only that has rectangular channel, slope embankments and channel with floodplain. Method used to investigate the backwater effect is the simulation analysis of the water profile using HEC-RAS software.

1.5 EXPECTED OUTCOMES

From this research, the simulation data collected would help in verifying the bridge designed. With the knowledge gained from the simulation, the backwater effect at upstream of the bridge can be determined to avoid issues related such as flood in nearby area, excessive scour of bridge piers and overflow of water onto the roadside. From the simulation, the extended flow at the upstream and downstream of the Bridge 3 of the Rasau River can be determined as well as the water profile of the river which important to verify the bridge design in terms of its hydraulic safety .

1.6 SIGNIFICANCE OF RESEARCH

For selected river site, there are difficulties for hydraulic engineer to estimate the extended flow of water due to backwater effect upon bridge designing stage. This is because of insufficient hydraulic information about the river. Therefore, with the simulation result gained using appropriate software the engineer could obtain the water profile of the river that helps in considering backwater effect for bridge design.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

Nowadays, as the construction cost is high, the construction of bridge is not encouraged if it is planned to cross a wide channel of river (Charbeneau & Holley, 2001). Thus, on the both side of the embankments of the river will be filled which creates floodplains that serve as a platform for the piers (Laursen & Toch, 1956). These piers that act as the main support for the bridge will caused obstruction to the water flow. Due to the obstruction, backwater is created. Backwater is the increase in the water surface level relative to the level occurred under unobstructed channel or can be defined as effect of flow produced by a bridge opening that obstructed the free flow of the water in the channel (Yarnell, 1934).

2.2 RAINFALL

In order to assess the spatial variation of Rasau River, volume of rainfall of catchment area needs to be estimated. In order to achieve this, hydrologists need to use additional information from remote sensing by weather radar or satellites or using a network of rain gauges alone. Amount of surface runoff generated in the watershed for a given rainfall pattern is important in order to analyse historical rainfall, evaporation, infiltration, and stream flow data to develop predictive relationships. Simple rainfall-runoff relationship should be used in water resources planning studies to determine the water profile at the river. The simplest rainfall-runoff formula which often used for small catchment area or basins is the Rational Method (equation 2.1) which allows for the prediction of peak flow Q (cfs) from the formula (Bedient, et al., 2013):

$$Q=CiA \qquad 2.1$$

where

C = runoff coefficient, variable with land use,

i = intensity of rainfall of chosen frequency for a duration equal to time of concentration, tc (in/hr),

tc = equilibrium time for rainfall occurring at the most remote portion of the basin to contribute flow at the outlet (min or hr),

A = area of catchment area (acres)

2.3 SOURCES OF WATER

2.3.1 Subsurface water

Subsurface flow processes and the zones in which they occur are shown schematically in Figure 2.1. Three important processes are infiltration of surface water into the soil to become soil moisture, subsurface flow or unsaturated flow through the soil, and groundwater flow or saturated flow through soil or rock strata (Chow, et al., 1988). Soil and rock strata which permit water flow are called porous media. A flow is unsaturated when the porous medium still has some of its voids occupied by air, and saturated when the voids are filled with water. The water table is the surface where the water in saturated porous medium is at atmospheric pressure (Chow, et al., 1988).

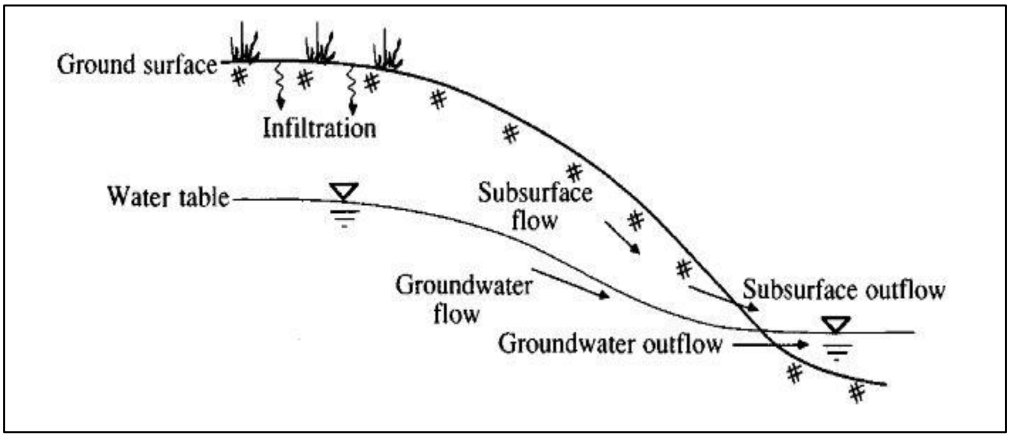


Figure 2.1 Subsurface water zone and processes
Source: Chow (1988).

Infiltration also one of the important process in formation of subsurface flow. Infiltration as describe by Chow et al (1988) is the process of water penetrating from the ground surface into the soil. The term infiltration is used to describe the process of water entry into the soil through the soil surface (Narayana, 1993). Many factors influence the infiltration rate, including the condition of the soil surface and its vegetative cover, the properties of the soil, such as its porosity and hydraulic conductivity, and the current moisture content of the soil. Soil strata with different physical properties may overlay each other forming horizon. For example, a silt soil with relatively high hydraulic conductivity may overlay a clay zone of low conductivity.

The distribution of soil moisture within the soil profile during the downward movement of water is illustrated in Figure 2.2. There are four moisture zones, a saturated zone near the surface, a transmission zone of unsaturated flow and fairly uniform moisture content, a wetting zone in which moisture decreases with depth, and a wetting front where the change of moisture content with depth is so great (Chow, et al., 1988).

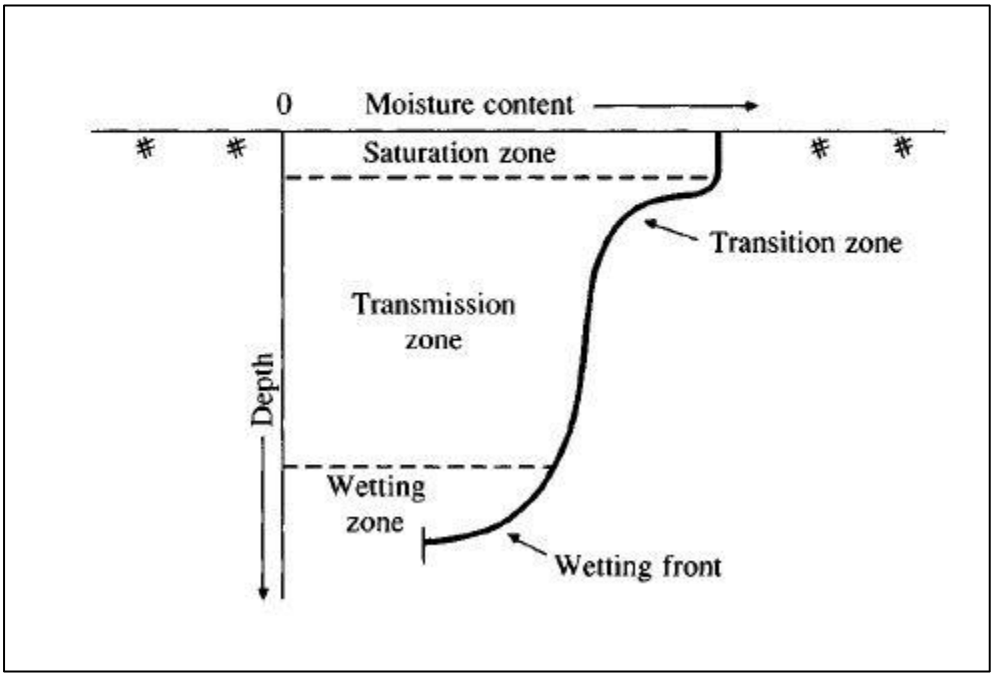


Figure 2.2 Moisture zone during infiltration

Source: Chow (1988).

Subsurface water or groundwater is an important aspect of the environment and it is a source of water supply throughout the world. Groundwater is defined as water that occurs in permeable geology formations known as aquifers, that is, formations having structures that permit appreciable water to move through them under ordinary field conditions (Mohammed & K Huat, 2004). Porosity of the aquifer material is a measure of the contained interstices and it is expressed as a percentage of void space to the total volume of the mass of the aquifer or can be written by Mohammed & K Huat (2004) as in equation 2.2:

$$\alpha = \frac{100w}{v} \qquad 2.2$$

where

α = porosity

w = volume of water required to fill, or saturated, all of the pore spaces

v = total of the rock or a soil forming aquifer

The range for $\propto$ is approximately $0.25 < \alpha < 0.75$ for soils, the value depending on the soil texture as tabulated in Table 2.1:

Table 2.1 Hydraulic conductivity and porosity of unconsolidated porous media

Material	**Hydraulic conductivity K (cm/s)**	**Porosity α (%)**
Gravel	10^{-1} - 10^{2}	25 - 40
Sand	10^{-5} - 1	25 - 50
Silt	10^{-7} – 10^{-3}	35 - 50
Clay	10^{-9} – 10^{-5}	40 - 70

Source: Chow (1988)

A part of the voids is occupied by the water and the remainder by air, the volume occupied by water and the remainder by air, the volume occupied by water being measured by the soil moisture content θ defined by Chow et al (1988) as in equation 2.3:

$$\alpha = \frac{\text{volume of water}}{\text{total volume}} \qquad 2.3$$

Hence $0 \leq \theta \leq \alpha$, the soil moisture content is equal to the porosity when the soil is saturated.

2.3.2 Surface water

Chow et al (1988) define surface water as water stored or flowing on the earth's surface. The surface water system continually interacts with the atmospheric and subsurface water system. It is important to know how exactly sources of river water come from and how actually hydrology characteristics surrounding the river interact with each other. Ward & Robinson (2000) mentioned that most of the precipitation that reaches the ground surface is absorbed by the surface layers of the soil. Once any depression storage has been filled, the remainder precipitation will from over the surface as overland flow, reaching the stream channels quite quickly. The water that penetrates into the soil may percolate under gravity to the groundwater body, be evaporated, or flow laterally close to the surface as throughflow (Ward & Robinson, 2000).

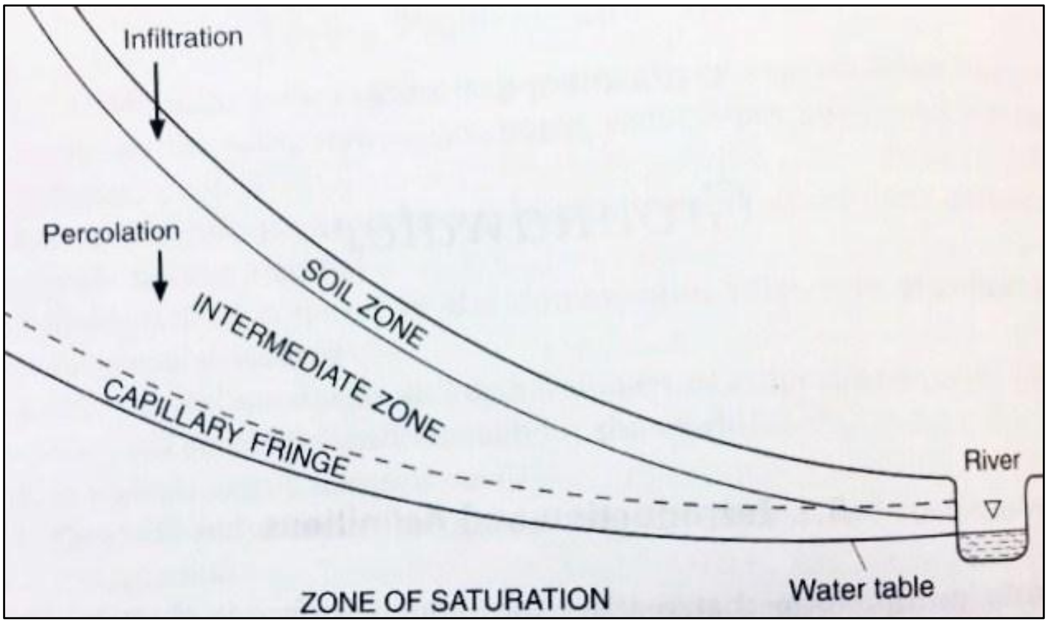


Figure 2.3 The main zone into which subsurface water has been traditionally classified

Source: Bedient (2013).

Figure 2.3 illustrates section of part of a river valley which includes the four main zones into which subsurface water has been traditionally classified. Precipitation enters the soil zone at the ground surface and moves downward to the water table which marks the upper surface of the zone of saturation. Ward & Robinson (2000) added immediately above the water table is the capillary fringe in which almost all the pores are full of water. Water table is defined as the level to which water will rise in a well drilled into the saturated zone (Bedient, et al., 2013). According to Chow et al (1988), the water table is the surface where the water in saturated porous medium is at atmospheric pressure. These water table definition also supported by Asawa (2008) with statement stated that water table is the upper limit of the saturated zone. Between capillary fringe and the soil zone is the intermediate zone, where the movement of water is mainly downwards.

On the valley flanks, water drains may or may not eventually reach the zone of saturation which perhaps several hundred metres below but surely water drains from the soil zone proper into the intermediate zone. In the floodplain areas, however, the capillary fringe often extends into the soil zone or even to the ground surface itself, depending on the depth of the water table and the height of the capillary fringe.

Although convenient as an introduction, this classification tends to obscure the fact that subsurface water is an essentially dynamic system (Ward & Robinson, 2000).

2.4 CATCHMENT AREA

Catchment area or watershed can be define as an important physiographic property that determines the volume of runoff to be expected from a given rainfall event that falls over the area (Bedient, et al., 2013). Basically, for a major river basin watershed areas can be up to square miles and a few acres in an urban area. The loci points (the ridge line) that separates two adjacent watersheds is known as watershed divide. The watershed definition also supported by Raghunath (2006) with the statement stated that the entire area of a river basin whose surface runoff (due to storm) drains into the river in the basin is considered as a hydrologic unit and it is called drainage basin, watershed or catchment area of the river flowing.

Bedient et al (2013) also define watershed with definition saying that a watershed is a contiguous area that drains to an outlet, such that precipitation that falls within the watershed runs off through that single outlet. For example, direct runoff from the surface and stream flow will resulting to zero if the rainfall rate over a watershed area is less than the rate of infiltration into soil and there is ample storage in soil moisture. Figure 2.4 delineate the catchment area as defined based on topographic or elevation data.

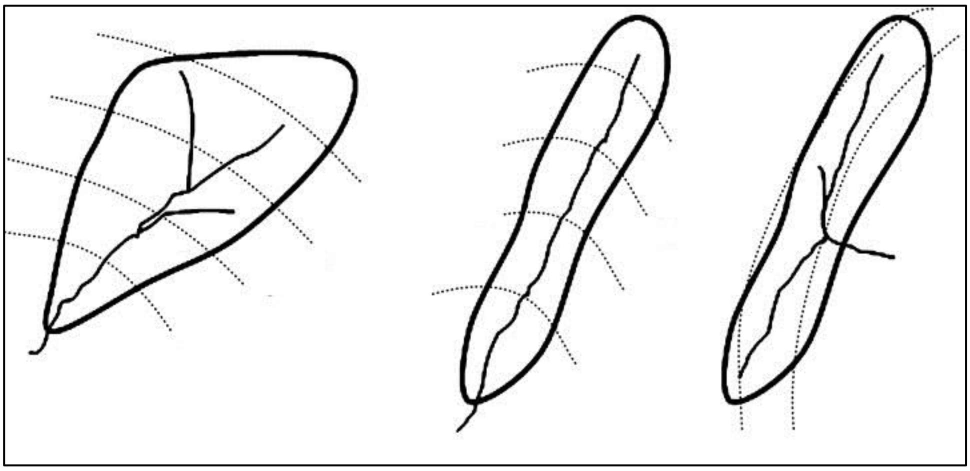


Figure 2.4 Typical watershed area shape

Source: Bedient (2013).

Since catchment area has difference in shapes thus it will affects timing and peak off flow of runoff to the outlet.

2.4.1 Type of soil

Soil types in watershed are critical as they determine infiltration rates that can occur for the area. Soil properties can vary significantly across a watershed area, and the USDA Natural Resources Conservation Service (NRCS) is responsible for developing soils maps to provide information on soil type, soil textures and hydrologic soil groups (Bedient, et al., 2013). Particle diameter in mm, for sand, silt and clay are characteristics to characterize the three main soil classes. Soil texture is important in determining water-holding capacity and infiltration capacity of a soil layer (Mohammed & K Huat, 2004). Thus, sands generally infiltrate water at a greater rate than do silts or clays.

2.5 WATER FLOW IN OPEN CHANNEL

Open channel flow is driven by the component of the gravitational force along the channel slope. Channel slope will appear in all the open-channel flow equation, whereas the pipe flow equations include only the slope of the energy grade line (Houghtalen, et al., 2000). In Figure 2.5, open-channel flow is schematically shown. The free water surface is subjected to only atmospheric pressure, which is commonly referred to as the zero pressure reference in hydraulic engineering practice (Houghtalen, et al., 2000). Chow (1988) also mentioned about open-channel with statement stated that open-channel flow must have a free surface, whereas pipe flow has none, since the water must fill the whole conduit.

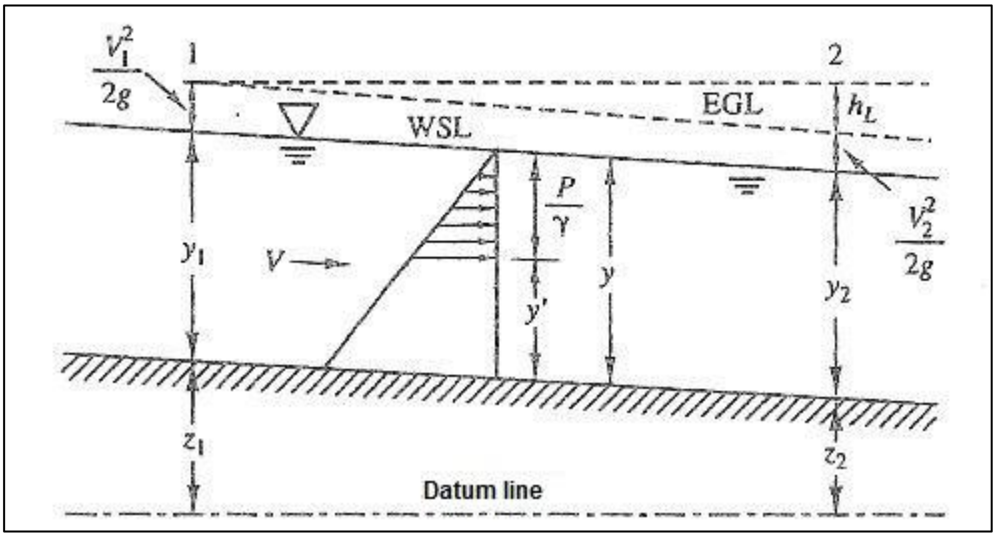


Figure 2.5 Open-channel flow

Source: Bedient (2013).

To solve open-channel flow problems, we must seek the interdependent relationships among the slope of the channel bottom, the discharge, the water depth, and other channel characteristics are needed. The basic geometric and hydraulic definitions used to describe open-channel flow though a channel section based on Houghtalen et al (2000) as shown in Table 2.2:

Table 2.2 Cross-sectional characteristics for various types of channel sections and their geometric and hydraulic relationships

Discharge (Q)	**Volume of water passing through a flow section per unit time**
Flow area (A)	Cross-sectional area of the flow
Average velocity (V)	Discharge divided by the floe area, V = Q/A
Flow depth (y)	Vertical distance from the channel bottom to the free surface
Top width (T)	Width of the channel section at the free surface

Chow (1988) added that problems related to flow in open-channel is more difficult to solve if compare with in pressure pipes. Flow conditions in open channels are complicated by the fact that it is here assumed that the velocity is uniformly distributed across the conduit section; otherwise a correction would have to make.

If the flow were curvilinear or if the slope of the channel were large, the piezometric height would be appreciably different from the depth of flow. As the result a result, the hydraulic grade line would not coincide exactly with the water surface.

2.5.1 Type of flow in open-channel

Open-channel flow can be classified into many types and described in various ways. The following classification is made according to the change in flow depth with respect to time and space. Firstly is classification of flow based on time as the criterion and the flow are steady and unsteady flow. Chow (1988) stated that flow in an open channel is said to be steady if the depth of flow does not change or it can be assumed to be constant during the time interval under consideration. Chow (1988) added, the flow is unsteady if the depth changes with time.

However, if the change in flow condition with respect to time is a major concern, the flow should be treated as unsteady. In floods or surges which are typical examples of unsteady flow, the stage of flow changes instantaneously as the waves pass by and time become important consideration in the design of control structure (Chow, 1988). For any flow, the discharge Q at a channel section is expressed by equation 2.4:

$$Q = VA \tag{2.4}$$

where

V = mean velocity

A = flow cross-sectional area normal to the direction of the flow

Furthermore, when space as the criterion, there are uniform flow and varied flow. Open channel is said to be uniform if the depth of flow is the same at every section of the channel (Chow, 1988). Steady uniform flow is the fundamental type of flow treated in open channel hydraulics. The depth of the flow does not change during the time interval. Unsteady uniform flow would require that the water surface fluctuate from time to time while remaining parallel to the channel bottom (Chow, 1988). Flow is varied if the depth of flow changes along the length of the channel. Varied flow may be further classified as either rapidly or gradually varied. Figure 2.6 illustrates various type of open channel flow.

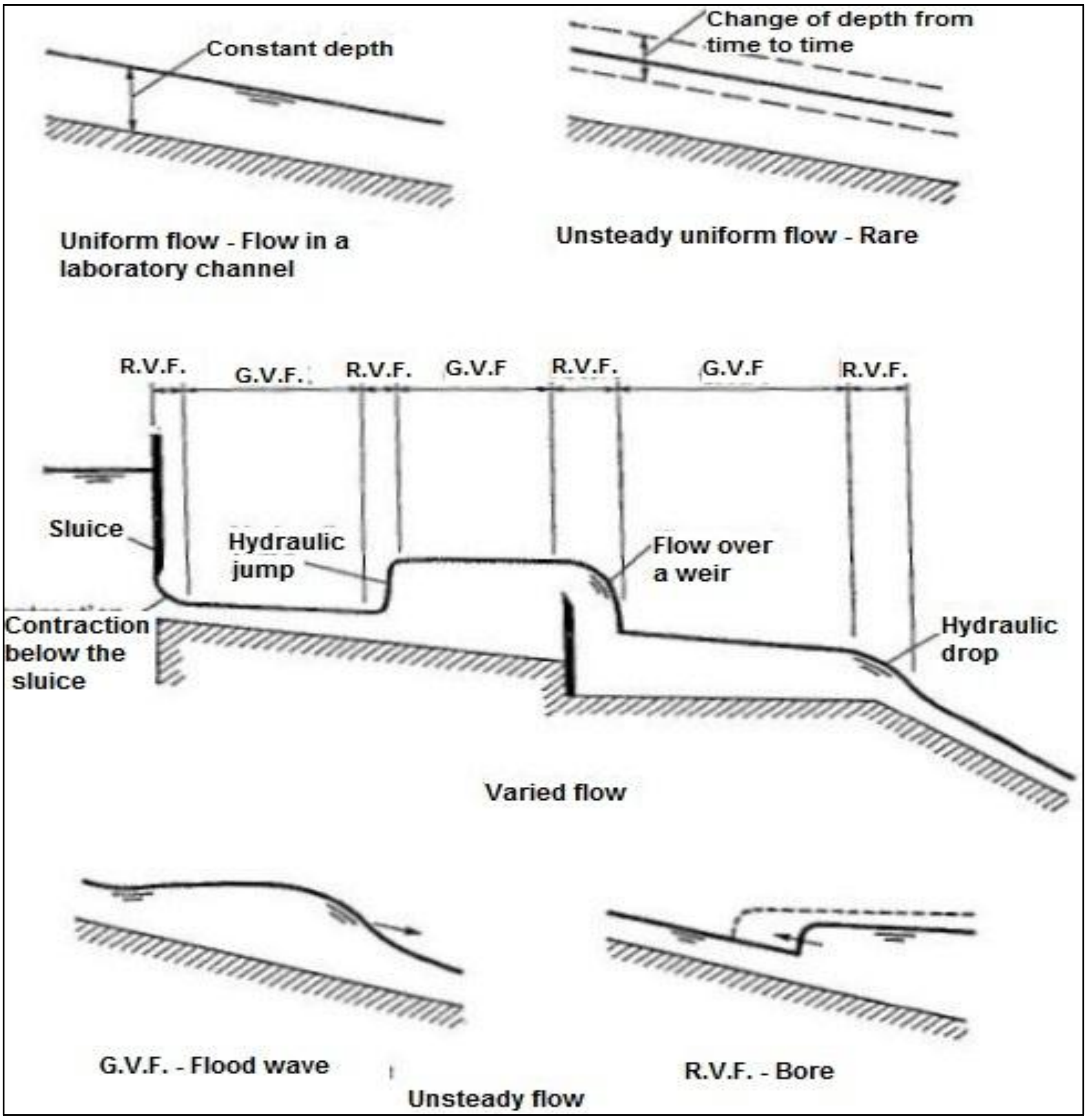


Figure 2.6 Various types of open-channel

Source: Chow (1988).

For clarity, the classification of open-channel flow is summarized as the flow chart shown in Figure 2.7:

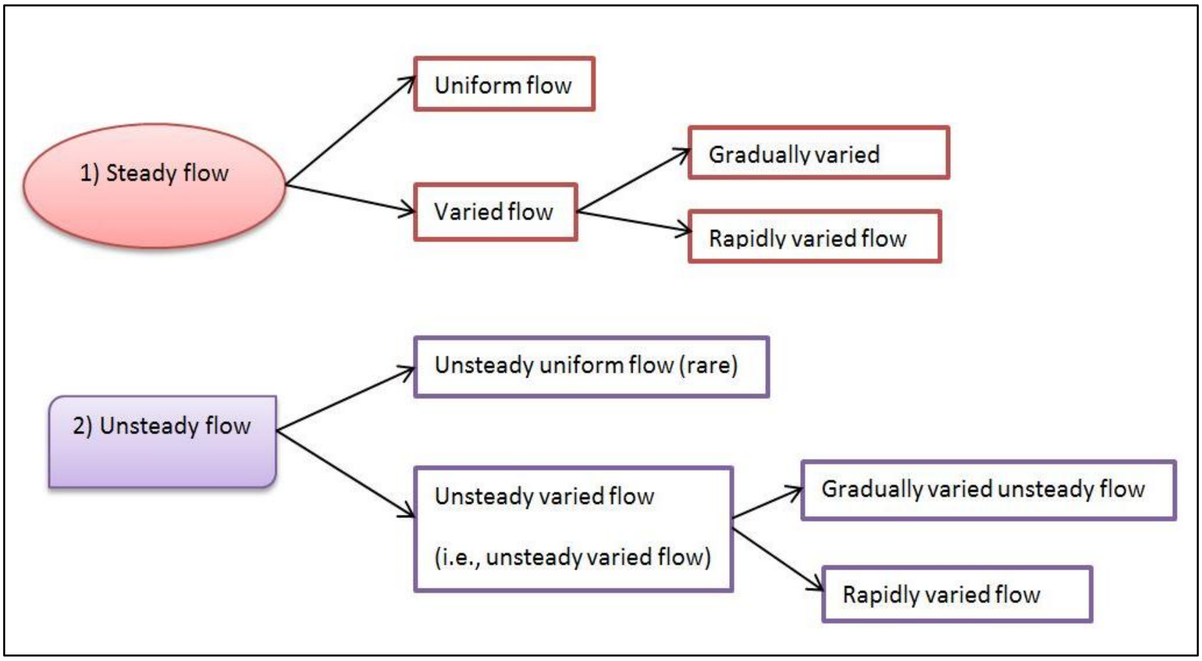


Figure 2.7 Classification of open-channel

Source: Chaudhry (2008)

2.5.2 Sediment transport in rivers

The dimensionless particle diameter d_* is defined from the equation 2.5 (Julien, 2002):

$$d_* = d_s \left[\frac{(G-1)g}{v^2} \right]^{1/3} \qquad 2.5$$

where

d_*= dimensionless particle diameter

d_s = particle size

G = specific gravity of the sediment

g = gravitational acceleration

v = kinematic viscosity of fluid

The settling velocity ω of a sediment particle in still water is defined in equation 2.6:

$$\omega = \frac{8v}{d_*}\left[\left(\frac{(G-1)}{v^2}\right)^{0.5} - 1\right] \quad 2.6$$

where

ω = settling velocity

d_*= dimensionless particle diameter

G = specific gravity of the sediment

v = kinematic viscosity of fluid

The ratio of shear force to bed particle weight defines the Shields parameter, τ_* (equation 2.7)

$$\tau_* = \frac{\tau_0}{(\gamma_s - \gamma)d_s} = \frac{{u_*}^2}{(G-1)gd_s} \quad 2.7$$

where

τ_* = Shields parameter

τ_0= initial shields parameter

γ_s = specific weight of a sediment particles

d_s = particle size

u_* = shear velocity

G = specific gravity of the sediment

g = gravitational acceleration

2.6 TYPE OF FLOW IN BACKWATER

Types of flow are various which may be encountered during bridge waterway design. Type of flow in backwater can be classified into two classes which are Class A and Class B (Yarnell, 1934). Class A is the unchecked flow while class B is the choke flow. The unchocked flow is described as subcritical flow whereas supercritical flow for choked flow (Bradley, 1960). Bradley (1960) added, the types of flow in backwater are labelled as Type I, Type II and .Type III. Type I flow is the subcritical flow while Type II flow is the flow passes through critical. Lastly is Type III which is supercritical flow. Figure 2.8 summarized the type of flow in backwater.

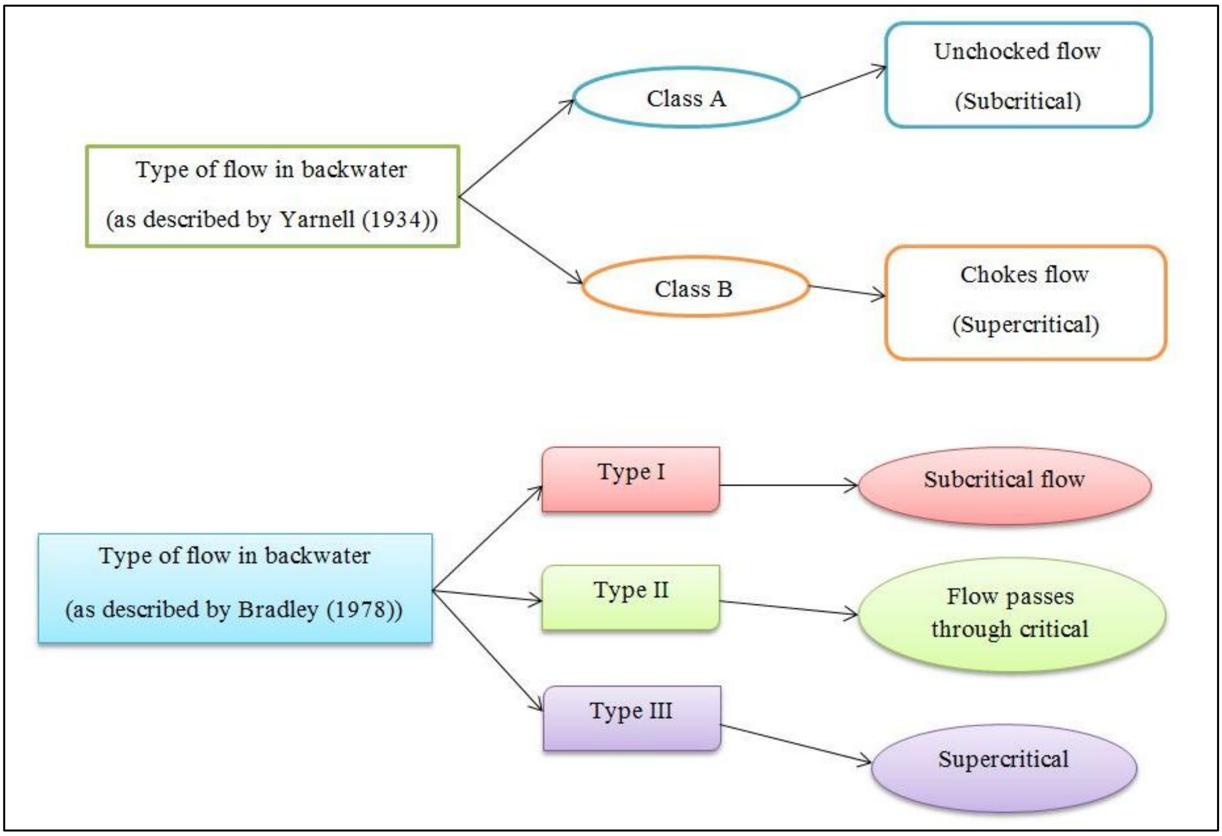


Figure 2.8 Type of flow in backwater
Source: Bradley (1960).

2.6.1 Subcritical flow

A flow is called critical if the flow velocity is equal to the velocity of the gravity wave having small amplitude. A gravity wave may be produced by a change in the flow depth (Chaudhry, 2008). Generally, subcritical flow occurs when the actual water depth is greater than critical depth. El-Alfy (2006) mentioned that the flow between piers is classified as subcritical when the Froude number value at downstream under normal flow conditions (Fr) is less than the critical value of Froude number, Fr_c.

Bradley (1960) had quoted “this type of flow commonly encountered during practice for design of bridge waterway”. Similar statement also stated by Charbeneau & Holley (2001) where subcritical flow exists in most rivers. Therefore all the analysis in this research will be limited to Type 1, subcritical flow as refer to most researches done by researchers such as Bradley (1960), Charbeneau & Holley (2001), Chaudhry (2008) and El-Alfy (2006).

2.6.2 Supercritical flow

Supercritical flow can be defined as a flow which its velocity is larger than the critical velocity (Chaudhry, 2008). When Froude number value at downstream under normal flow conditions (Fr) is greater than the critical value of Froude number, Fr_c. Bradley (1960) described supercritical flow as when the normal water surface is consistently below the critical depth and the flow is throughout. Theoretically, the Froude number, F_r, is equal to the ratio of inertial and gravitational forces and, for a rectangular channel, it is define as equation 2.8 (Chaudhry, 2008):

$$F_r = \frac{V}{\sqrt{gy}} \qquad 2.8$$

where

y = flow depth,

V = velocity

g = gravitational force

2.6.3 Type of channel

A channel having the same cross section and bottom slope throughout is referred to as a prismatic channel, whereas a channel having varying cross section and/or bottom slope is called a non-prismatic channel (Chaudhry, 2008). The depth of flow, y, at a section is the vertical distance of the lowest point of the channel section from the free surface. The depth of flow section, d, is the depth of flow normal to the direction of flow. The stage, Z, is the elevation or vertical distance of free surface above a specified datum (Figure 2.1). The top width, B, is the width of channel section at the free surface. The flow area, A, is the cross-sectional area of flow normal to the direction of flow. The wetted perimeter, P, is defined as the length of line of intersection of channel wetted surface with a cross-sectional plane normal to the flow direction. The hydraulic radius, R, and hydraulic depth, D, are defined in equation 2.9 and 2.10 respectively.

$$R = \frac{A}{P} \tag{2.9}$$

$$D = \frac{A}{B} \tag{2.10}$$

where

A = flow area

P = wetted perimeter

B = top width

Schematic of the values are shown in Figure 2.9.

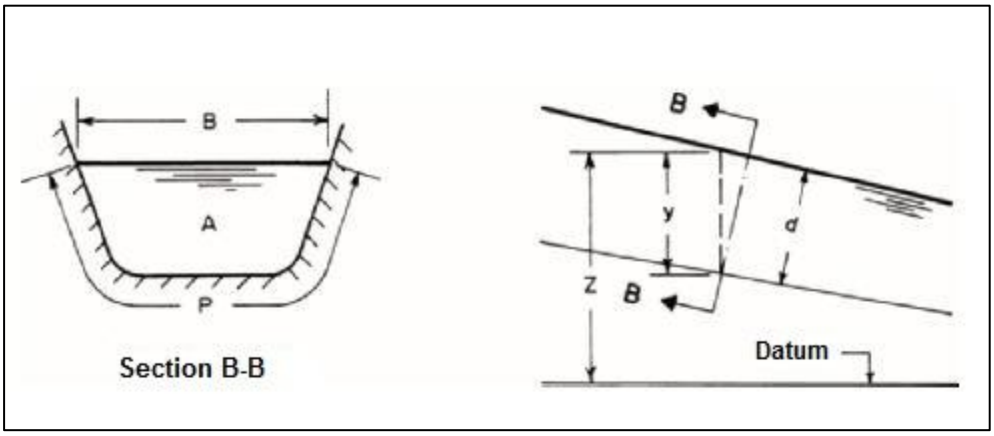


Figure 2.9 Definition sketch

Source: Chaudhry (2008).

Expression for A, P, D and R for typical channel cross sections are presented in Figure 2.10. It can be noted, in this research the type of channel is analysed as a rectangular channel.

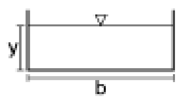
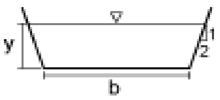
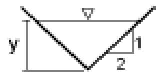
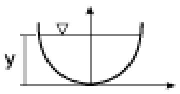
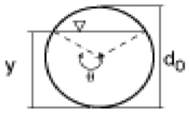
Channel type	Area A	Wetted permiter P	Hydraulic radius R	Top width T	Hydraulic depth D
	by	$\frac{by}{b+2y}$	$\frac{by}{b+2y}$	b	y
	$b+2y$	$b+2y\sqrt{1+z^2}$	$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$	$b+2zy$	$\frac{(b+zy)y}{b+2zy}$
	zy^2	$2y\sqrt{1+z^2}$	$\frac{zy}{2\sqrt{1+z^2}}$	$2zy$	$\frac{1}{2}y$
	$\frac{2}{3}Ty$	$T+\frac{8}{3}\frac{y^2}{T}$	$\frac{2T^2y}{3T^2+8y^2}$	$\frac{3}{2}\frac{A}{y}$	$\frac{2}{3}y$
	$\frac{1}{8}(\theta - \sin\theta)$	$\frac{1}{2}\theta d_0$	$\frac{1}{4}[1-\frac{\sin\theta}{\theta}]d_0$	$2\sqrt{y(d_0-y)}$	$\frac{1}{8}\left(\frac{\theta - \sin\theta}{\sin\frac{\theta}{2}}\right)d_0$

Figure 2.10 Properties of typical channel cross section
Source: Chaudhry (2008).

2.7 BACKWATER

Backwater is one of the hydraulic phenomena related to bridge construction due to piers within the channel or floodplain of natural waterways (Charbeneau & Holley, 2001). Backwater occurs as current hit the bridge's piers which act as obstruction, the water level tends to increase before it takes to lower down to actual level and increase back as flow rate increase. Backwater definition also stated by (Yarnell, 1934) with statement that as velocity increase, water surface in upstream will elevated as piers produce the contraction in area. Piers that act as obstruction can increase the water levels at the upstream of the bridge not only caused by the quantity of the flow, but also the pier's position in the stream, its geometric shape, and contraction of the channel.

Furthermore, Yarnell (1934) also mentioned that the increased in velocity of the river cause a drop in water surface as stream enters the contracted area; which is the space between piers. However, the water surface fails to rise again to the level of water surface upstream from the pier if the stream expands again into the unobstructed channel downstream from the pier (Yarnell, 1934).

Figure 2.11 shows the schematic profile in subcritical open channel. Increase in the water level (Δy) upstream of the obstacle caused by the bridge piers is one of the primary adjustment (Charbeneau & Holley, 2001).

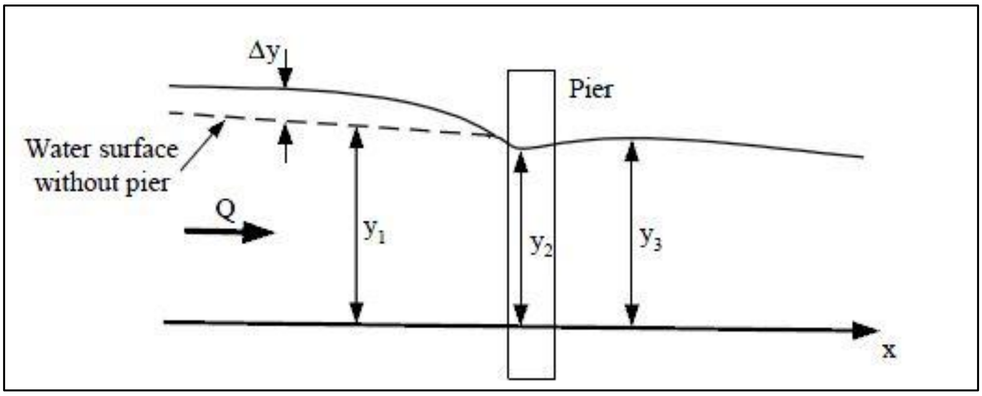


Figure 2.11 Schematic profile of a river in the surrounding area Charbeneau & Holley, (2001).

.

2.7.1 Backwater equation

To compute the magnitude of backwater **Δ**y, commonly used equation is Yarnel equation which focus in calculating the increase in water level due to bridge piers (Yarnell, 1934). The equation can be written as in equation 2.11.

$$\Delta y = 2K\left(K + 5{Fr_3}^2\right)(\alpha + 15\alpha^4)\frac{V_3}{2g} \qquad 2.11$$

Or can be written as the following form (equation 2.12).

$$\frac{\Delta y}{y} = K\left(K + 5{Fr_3}^2 - 0.6\right)(\alpha + 15\alpha^4){Fr_3}^2 \quad 2.12$$

where

y = original (undistributed) local flow depth,

Fr_3 = corresponding Froude number at section 3 downstream of piers,

α = ratio of the flow area obstructed by the piers to the total flow area downstream of the piers

K is summarized in Table 2.3 (noted that L is the distance between the two piers and D is diameter of each pier)

Table 2.3 Bridge pier backwater coefficient

Pier Shape	**K**
Semicircular nose and tail	0.9
Lens-shaped nose and tail	0.9
Twin-cylinder piers with connecting diaphragm (L/D=4)	0.95
90^o triangular nose tail	1.05

Source: Charbeneau & Holley (2001)

Yarnell (1934) had classified equation 2.11 and 2.12 as Class A (Charbeneau & Holley, 2001). To analyse effect of bridge piers in popular computer program such as HEC-RAS and HEC-2, Yarnell equation's is used in spite of the relatively large values of $\propto$ compared to present designs.

There are three part of energy loses computation that caused by structures such as bridges and culverts compute by HEC-RAS program: loss from contraction in the steam immediately upstream from the structure, loss from the expansion in the stream immediately downstream from the structure, and loss at the structure itself (Bedient, et al., 2013). Several different methods can be used in HEC-RAS to analyse a bridge without changing the bridge geometry.

A part from that, Bedient et al (2013) mentioned that the bridge routines have capabilities to model low flow (Class A, B and C), low flow and weir flow (with adjustments for submergence on the weir), pressure flow (orifice and sluice gate equations), weir flow and pressure, and highly submerged flows. The benefit of HEC-

RAS program is that when the flow over the road is highly submerged, it will automatically switch to the energy equation. Figure 2.12 illustrates the general bridge layout for cross section whereas Figure 2.13 and Figure 2.14 shows pressure flow and weir flow through bridges.

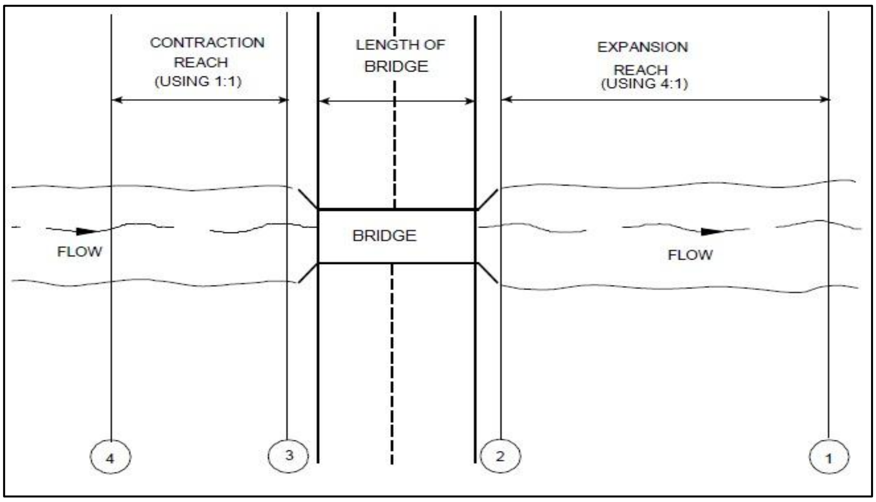


Figure 2.12 Cross section layout for bridge modelling

Source: Bedient (2013).

HEC-RAS considers three conditions, labelled class A, class B, and class C for flow under bridge with piers. It is assume that class A low flow is subcritical, and to compute the change in water surface elevation due to pier, Yarnell equation (equation 2.13) is used (Bedient, et al., 2013) :

$$H_3 = 2K\,(K + 10w - 0.6)(\alpha + 15\,\alpha^4)\,\frac{V_3^{\,2}}{2g} \qquad 2.13$$

where

H_3 = change in water surface elevation through bridge (from sec.3 to sec. 2),

K = pier shape coefficient,

w = ratio of velocity head to depth downstream of bridge at Section 2,

V_2 = velocity downstream from the bridge at Section 2,

$$\alpha = \frac{\text{obstructed area}}{\text{total unobstructed area}} \quad \text{(at Section 2)}$$

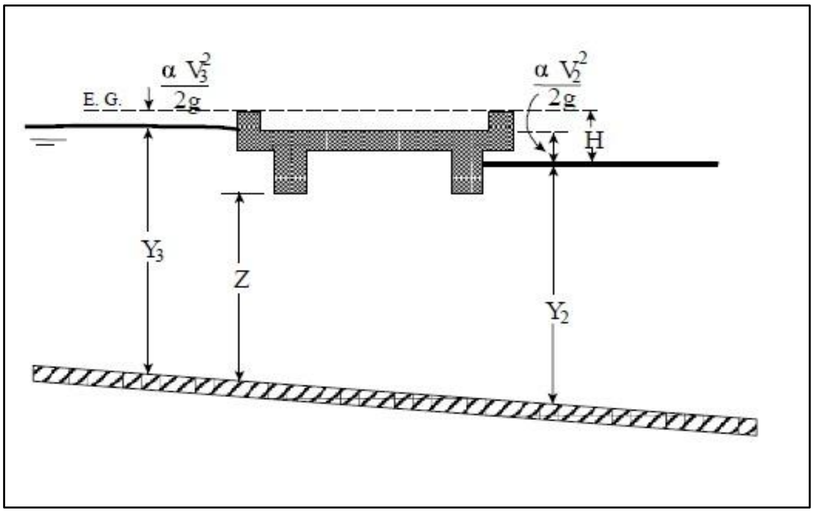


Figure 2.13 Pressure flow and weir flow through bridges
Source: Bedient, et al. (2013).

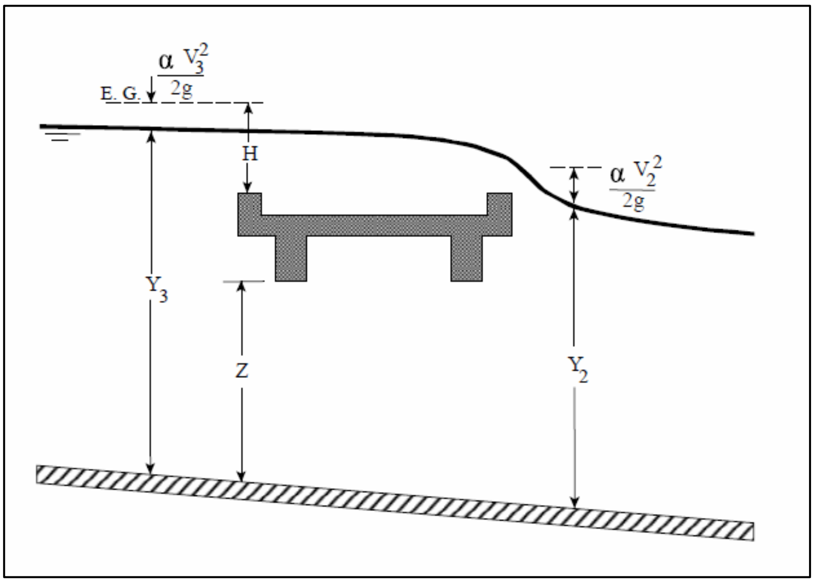


Figure 2.14 Pressure flow and weir flow through bridges.
Source: (Bedient, et al. (2013).

The value of H_3 is added to the downstream water surface elevation after the computation to account for the bridge. Bedient et al (2013) defines class B low flow as a flow that occurs when the water surface profile passes through critical depth underneath the bridge. HEC-RAS uses a momentum balance for cross section adjacent to and under the bridge. When the flow condition is supercritical means that class C low flow occurs (Bedient, et al., 2013).

Occurrence at bridge deck which its becomes submerged such that the low chord is in contact with water and head build up occurs on the upstream side of the bridge is called pressure flow. To handle the pressure flow phenomena, the energy-based method is applied. Bedient et al (2013) stated that we can regard the pressure flow as orifice flow in fluid mechanics when both the upstream and downstream side of bridge are submerged and can describe it by equation 2.14.

$$Q = C_d A(2gH)^{0.5} \quad 2.14$$

where

H= total energy difference upstream and downstream,

Cd = discharge coefficient (0.7 to 0.8),

A = cross-sectional area of the bridge opening,

Q = total orifice flow.

HEC-RAS defines H as the distance from the energy grade line to the centroid of the orifice area. (Bedient, et al., 2013) added that when water begins to flow over the bridge elevated roadways approaches, the occurrence of weir flow happen. The standard weir equation that used in HEC-RAS foe this flow condition is written as in equation 2.15.

$$Q = CLH^{3/2} \quad 2.15$$

where

C = weir discharge coefficient,

L = effective length of weir,

H = total energy difference upstream of the bridge and top of the roadway,

Q = flow over the weir.

2.8 TYPE OF SOFTWARE AVAILABLE

2.8.1 HEC-HMS

The Hydrologic Modeling System (HEC-HMS) is designed to simulate the complete hydrologic processes of dendritic watershed systems (Engineers, 2018). The software includes many traditional hydrologic analysis procedures such as event infiltration, unit hydrographs, and hydrologic routing. HEC-HMS also includes procedures necessary for continuous simulation including evapo-transpiration, snowmelt, and soil moisture accounting. Supplemental analysis tools are provided for model optimization, forecasting streamflow, depth-area reduction, assessing model uncertainty, erosion and sediment transport, and water quality.

A graphical user interface allows the user seamless movement between the different parts of the software. The software features a completely integrated work environment including a database, data entry utilities, computation engine, and results reporting tools. Simulation results are stored in HEC-DSS (Data Storage System) and can be used in conjunction with other software for studies of water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, and systems operation.

2.8.2 HEC-RAS

HEC-RAS is designed to perform one and two-dimensional hydraulic calculations for a full network of natural and constructed channels. The HEC-RAS system contains several river analysis components for: (1) steady flow water surface profile computations; (2) one- and two-dimensional unsteady flow simulation; (3) movable boundary sediment transport computations; and (4) water quality analysis (Hicks & Peacock, 2005).

A key element is that all four components use a common geometric data representation and common geometric and hydraulic computation routines. In addition to these river analysis components, the system contains several hydraulic design features that can be invoked once the basic water surface profiles are computed.

2.8.3 Infoworks-RS

Infoworks-RS is aimed for investigate the element of river which include full solution modelling of open channel, floodplains, and hydraulic structure of the river (Infoworks, 2007). Mah et al (2007) stated that InfoWorks-RS is one of the modelling programs that combine the advanced flow simulation engine, both hydrological and hydraulic models, GIS functionality and database storage within one single environment. The basic system architecture is an "Integrated Network Model" links data storage using a GIS to hydrologic/hydraulic modelling software suite embedded in Infoworks-RS.

Hydrodynamics refers to the motion of a water body through its geo-morphological environment, taking into account the effects of gravity and friction at the water/bed interface (Mah, et al., 2007). A hydrodynamic model simulates these effects, giving the water surface elevation and velocity in response to tidal influence and flows from upstream of the river. The Wallingford Software's Infoworks-RS is an example of one-dimensional hydrodynamic model used for prediction of discharge and water level for a wide range of rivers, reservoirs, complex floodplains under both steady and unsteady conditions. Infoworks computes flow depths and discharges using a method based on the equations for shallow water waves in open channel – the Saint-Venant equations, which consists of the continuity equation, Equation 2.16, and the momentum equation, equation 2.17, respectively

$$\beta \frac{\partial y}{\partial t} + \frac{\partial Q}{\partial x} = q \qquad 2.16$$

where

y = stage

Q = discharge

β = stream top width

q = lateral flow channel per unit length of channel

x = distance along the channel

t = time

$$S_f = \frac{Q^2}{K^2} = \frac{n^2Q^2}{A^2R^{4/3}} \qquad 2.17$$

where

A = flow area

K = conveyance

R = hydraulic radius

n = manning roughness coeffiecient

The cross sectional average flow velocity used hereafter is defined as

V = Q/A

Infoworks model uses a four-point, implicit, finite difference approximation to solve the Saint-Venant equations in full, together with proper boundary conditions. The scheme is structured so as to be independent of the wave description specific (kinematics, diffusive or dynamics).

2.8.4 MIKE

Mike software is used within all water environments anywhere in the world. They cover oceans and coastlines, rivers and reservoirs, ecology, groundwater and water distribution (MIKE, 2017). The two main software that can be used for coastal modelling are MIKE 21 for 2D modelling and M IKE 3 for 3D modelling. Both products have an ample range of application possibilities, but are typically used for coastal engineering studies (MIKE, 2017).

Software that can be used for groundwater modelling involves all about groundwater flow, groundwater age, contaminant or heat transport processes is FEFLOW. FEFLOW has links with MIKE 11 to model the interactions of groundwater with surface water bodies such as rivers, lakes, and floodplains area to consider water quality and water quantity issues of the coupled system (MIKE, 2017).

MIKE HYDRO is for river modelling that enables user to model a variety of tasks related to river hydraulics, water quality, flooding, forecasting, navigation as well as catchment dynamics and runoff. It provides the largest diversity in calculation features and add-on module enables river engineer to conduct all required modelling activities within one modelling package.

2.8.5 AGIS Software

AGIS Software can help user to plot their geographical information or download and display data from a variety of sources on the web. A part from that, the software has a multi-document interface and can be used to add high-quality vector map displays to documents such as Microsoft Word (AGIS, 2003).

Type of format supported for the data that want to be imported include MapInfo, Garmin GPS and ARCInfo. Furthermore, a built-in scripting language can be used to create animation; automate map displays using data from other sources, such as an Microsoft Access database. This software features also allows user to fully control over virtually every aspect of a map display, many layers of maps and data, thematic mapping, a number of map projections, simplified access to digital charts of the world, and world base-map data (AGIS, 2003).

2.9 GUIDELINES

2.9.1 Urban Stormwater Management Manual for Malaysia (MSMA)

The Stormwater Management Manual is prepared by Department of Irrigation and Drainage Malaysia (DID) which function in providing guidance to all regulators, designers, and planners who are involve in stormwater management. The manual prepared is very useful as a guidance related to issues that need to take into consideration. For example, act as a guidance to solve problem facing by nation such as flash flood, river pollution, development in the highlands and low lands, and soil erosion (MSMA, 2012).

A part from that, MSMA also used by hydrologists to investigate and identify a new direction of stormwater management in urban areas in Malaysia. Example of documentation that had been documented in MSMA is the latest development based on control at source approaCH

This manual has also been reviewed by various agencies, organizations and foreign experts. Reviewed by the agencies will be taken into consideration in preparing the final document as the review is also importance part of documentation. Of course MSMA also facing some challenge which is to ensure that the administration of the planning, design and maintenance of stormwater management systems is consistent across the relevant local, environmental and civil engineering, landscape architecture, state and federal authorities and the professions of planning (MSMA, 2012).

The objectives of MSMA are to:

i. Ensure the safety of the public.
ii. Protect property.
iii. Stabilise the landform and control erosion.
iv. Control nuisance flooding and provide the safe passage of less frequent and larger flood events.
v. Enhance the urban landscape.
vi. Minimise the environmental impact of urban runoff on water quality.
vii. Enhance the urban landsape.

2.9.2 Government of Malaysia Department of Irrigation and Drainage (DID)

This Volume of the DID Manual focuses on topics related to the Flood Management function of the Department of Irrigation and Drainage Malaysia (DID). It covers the broad spectrum of the technical and non-technical aspects of flood management as practiced in the country. This includes the engineering aspects of planning and design and the principles of flood management (DID, 1997) .

This manual also serves as a reference for flood management practices in the country. In some ways, it is also a record of the history of flood management practices in Malaysia. This manual serves as a reference for DID engineers and staffs involved in the planning and design of flood management systems as well as those managing the DID offices in the States and Districts (DID, 1997).

2.9.2.1 Freeboard

Freeboard (F) is defined as the vertical distance between top of the channel and the water surface when the channel is carrying the design flow at a normal depth. Figure 2.15 shows the distance between the top of the channel lining or bank and the calculated water surface (DID, 1997).

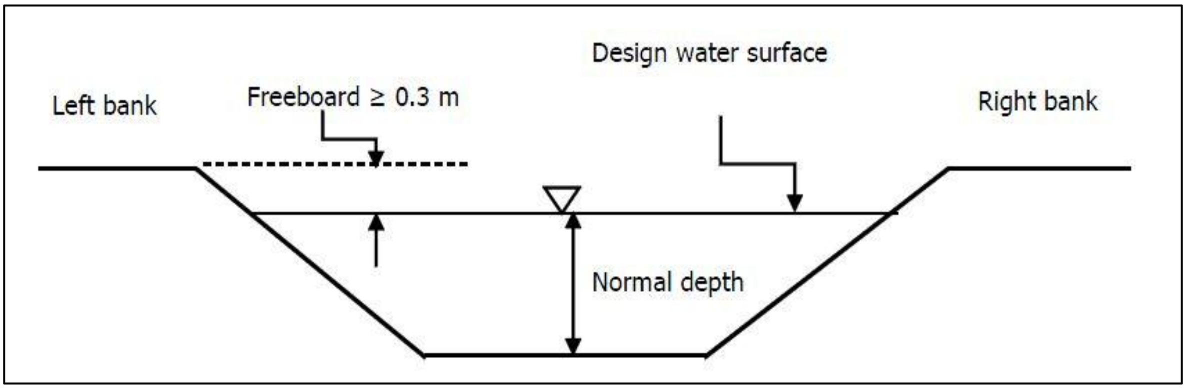


Figure 2.15 Freeboard of the channel design
Source: DID (1997).

The use of freeboard is as protection to the structure against uncertainty in the design parameters. The minimum freeboard is normally 30 cm at the maximum design water surface elevation (DID, 1997). However, and additional freeboard equal to the super-elevation of the water surface should be provided around bends. Floodway has been arbitrarily defined as that part of the cross-section that includes the channel and will pass the 100-year return period flood without increasing the water level more than 0.3m above the existing 100-year flood level (DID, 1997).

2.9.2.2 Freeboard for hydraulic structure

For hydraulic structure, freeboard is the vertical distance between a design maximum water level and the top of structure such as bund, channel, floodwall, and dam. In bridge design, there is also a consideration of freeboard since the freeboard act as a safety factor intended to accommodate the possible effect of unpredictable obstruction such as bridge’s piers and debris blockage that could increase water levels above the design water surface (DID, 1997).

Bridge freeboard should be based on 50 year flood frequency. Additional freeboard functions to protect the structure if the drainage area produces unusually large debris. Moreover, if the drainage area produces very little debris, the freeboard criteria may be reduced.

Freeboard is the required clearance between the lower limit of superstructure and the design high water surface elevation. The minimum freeboard for river crossing structure is shown in Figure 2.16 while the minimum clearance recommended for river crossing structures is shown in the Figure 2.17 (DID, 1997).

Structure Type	Minimum Freeboard
Bridges with drainage area > 2.6 km^2	0.6 m
Bridges with drainage area < 2.6 km^2	0.3 m
Temporary bridges	0.3 m

Figure 2.16 Minimum freeboard for river crossing structures

Source: DID (1997)

River Width	Minimum Height Clearance	Minimum Width Clearance
< 15 meter	3.5 m	8 m from river banks (left and right)
15 meter – 20 meter	4.0 m	10 m from river banks (left and right)
> 20 meter	4.5 m	10 m from river banks (left and right)

Figure 2.17 Minimum clearance for river crossing structure

Source: DID (1997)

Figure 2.18 illustrates the freeboard for the bridge crossing with the minimum freeboard design.

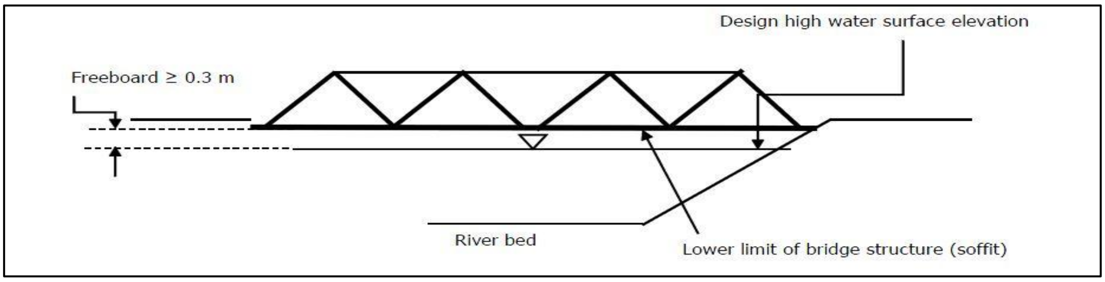


Figure 2.18 Freeboard for the bridge crossing
Source: (DID, 1997).

2.9.3 Hydrological Procedure No. 27

The design of many engineering works requires the consideration of storage upstream of the structure. For example are retention ponds and spillways. Thus, it it necessary to determine the relationship between the inflow, outflow and storage for the study area. This procedure gives a method for the estimation of design flood hydrographs for rural catchments in Peninsular Malaysia (DID, 2010).

The procedure uses three components; the design storm, the rainfall-runoff relationship and the equations for Clark parameters in the development of design flood hydrograph. Apart from that, the reliability and limitation of the procedure are discussed and worked examples using a computer programme illustrating the uses of the procedure are also presented (DID, 2010).

2.9.3.1 Equation development

Equation relating time of concentration, T_c, coefficient of determination, R and catchment characteristics are required to estimate T_c and R for ungauged catchments. A multiple linear regreesion program was used to determine the mathematical relationships of T_c and R with catchment characteristics such as area, slope and length of mainstream for the 43 catchments of Peninsular Malaysia (DID, 2010). For simplicity and consistency, equations relating T_c, R, catchment area, stream slope and main stream length are used to estimate T_c and R for this procedure. Equations 2.18 and 2.19 show the relationship among the characteristics required.

$$T_c = 2.32A^{-0.1188}\,L^{0.9573}\,S^{-0.5074} \qquad 2.18$$

$R^2 = 0.7883$

$SE = 0.2116$

$$R = 2{,}976A^{-0.1943}\,L^{0.9995}\,S^{-0.4588} \qquad 2.19$$

$R^2 = 0.7656$

$SE = 0.2024$

where

A = catchment area in km^2

L = main stream length in km

S = weighted slope of main stream in m/km

R^2 = coefficient of determination

SE = standard error or the root mean square error

The catchments were subdivided into east and west coast catchments and the same multiple linear correlations carried out to derive T_c and R on a regional basis (DID, 2010). It was found that there is no better correlations can be obtained. Equation 2.18 and 2.19 are used to estimate T_c and R since attempts to obtain better correlations by further dividing the catchments into smaller regional groups for regression analysis are not successful.

CHAPTER 3

METHODOLOGY

3.1 INTRODUCTION

To conduct this study, several methods of analysis need to be taken and take as consideration in in order to achieve the objectives and for a good result. The objectives includes to determine the backwater effect due to presence of bridge piers and to understand the extended flow at upstream and downstream of bridge. In addition, the simulation was also carried out to determine the water profile of the Rasau River as flow hit the bridge piers. The method conducted includes data collection, data analysis, simulation of the river, and analysis of the present data.

This chapter describes the application of HEC-HMS and HEC-RAS software in determining all of the objectives. In using the application of the software, all the input data must be accurately follow the specification of the software in order to give an accurate simulation. There are many applications in the HEC-HMS and HEC-RAS software for different purposes, but only certain application in taken into action in order to achieve the purpose of the study. Figure 3.1 summarized the methodology for this study

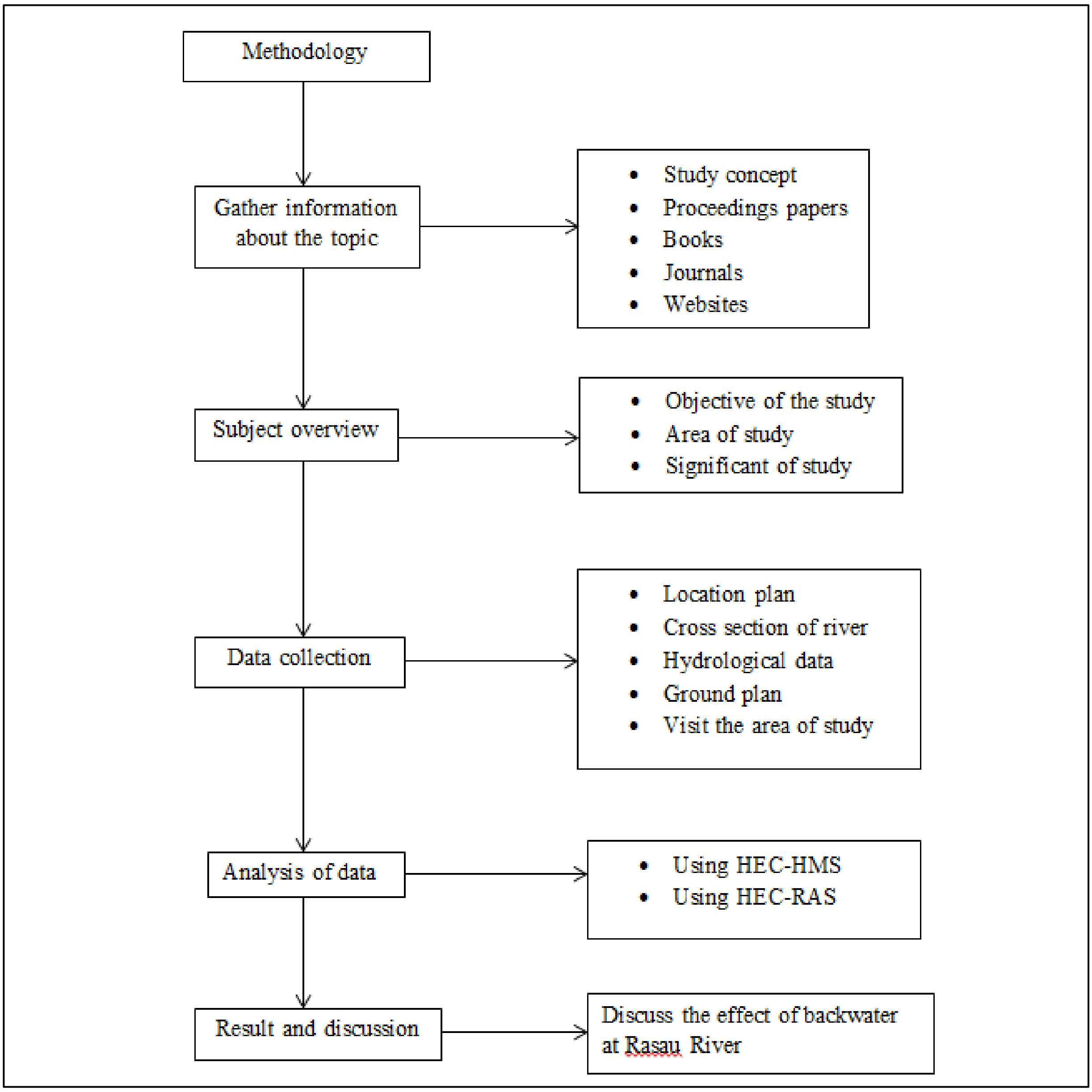


Figure 3.1 Methodology of the study

3.2 REFERENCE AND PRELIMINARY STUDY

At the beginning of the study, most information was collected from previous research by researcher all around the world including books, websites and journal related to the topic. Other than that, data such as river cross section, hydraulics and hydrological data related to the Rasau River was collected from the corresponding official departments and contractor of the bridge project at the Rasau River. Data analysis by the sources is very useful to be used throughout the study.

3.3 DATA COLLECTION

All the data collected were the crucial inputs for the HEC-HMS and HEC-RAS software application. The data included are the hydrological properties, river characteristics, and features involves of the Rasau River were obtained in order to run the simulation successfully. The primary data required are listed in the Table 3.1 (Hassan, 2009):

Table 3.1 The necessary information and data required

No	Type of information	Format	Data properties	Sources
1	Topographical map	Digital or hard copy	Identification of catchment area and river channel	JUPEM
2	River cross sections	Digital or hard copy	Main input	ATZ Consultant
3	Location plan	Digital or hard copy	Reference	ATZ Consultant
5	Satellite image	Digital	Catchment activities	MACRES
6	Rainfall data	Digital or hard copy	Hydrological analysis	DID
7	Land use plan	Digital or hard copy	Hydrological analysis	DOA
8	Soil properties map	Digital or hard copy	Hydrologic analysis	DOA

NOTE:

DID : Department of irrigation and Drainage Malaysia

JUPEM : Surveying and Mapping Department Malaysia

MACRES : Malaysia Remote Sensing Centre

DOA : Department of Agriculture Malaysia

3.3.1 Cross section of the river

The main input in running the simulation is the cross section of the river. The data is available at the Department of Irrigation and Drainage (DID) which is collected in the form of digital and hard copy. In this software application the digital cross section is used which are presented in Microsoft Excel from and it is very suitable for analysis process.

3.3.2 Digital map

In order to determine the boundary of the river row and the catchment area, digital map is required. In this study, the map and the location plan was obtained from The Department of Survey and Mapping Malaysia (JUPEM). Normally, the map is in GIS form, CAD form or map in a hardcopy is required for analysis such as to determine the elevation of the catchment area.

3.3.3 Hydrological data

A real event rainfall data was obtained from DID from 29th April 2003 to 5th May 2009. The data showed the rainfall depth for every 5 minute. From the hydrograph, simulations of the river were carried out. Rainfall data were needed as a rainfall profile for the catchment area. Table 3.2 shows the rainfall data taken from Sungai Lembing, Kuantan rainfall station.

Table 3.2 Example of rainfall data

Bil.	Station name	Date	Time	RF Daily (mm)
1	Sg. Lembing	29/12/2003	16:56:00	0.10
2	Sg. Lembing	29/12/2003	17:01:00	0.10
3	Sg. Lembing	29/12/2003	17:06:00	0.10
4	Sg. Lembing	29/12/2003	17:11:00	0.20
5	Sg. Lembing	29/12/2003	17:16:00	0.15
6	Sg. Lembing	29/12/2003	17:21:00	0.15
7	Sg. Lembing	29/12/2003	17:26:00	0.10

Source: (DID)

3.4 ESTIMATION OF RAINFALL DEPTH

Firstly, time of concentration, T_c and rainfall intensity were required to estimate the 100 year rainfall depth. For natural catchment, the time of concentration can be estimated by referring the formula in the Hydrological Procedure No.27. Based on the parameter shown in the Figure 3.2, the value of Tc is:

$$T_c = 2.32(13.65)^{-0.1188}(5.35)^{0.9573}(5.05)^{-0.5074}$$

$$T_c = 3.73\text{hr}$$

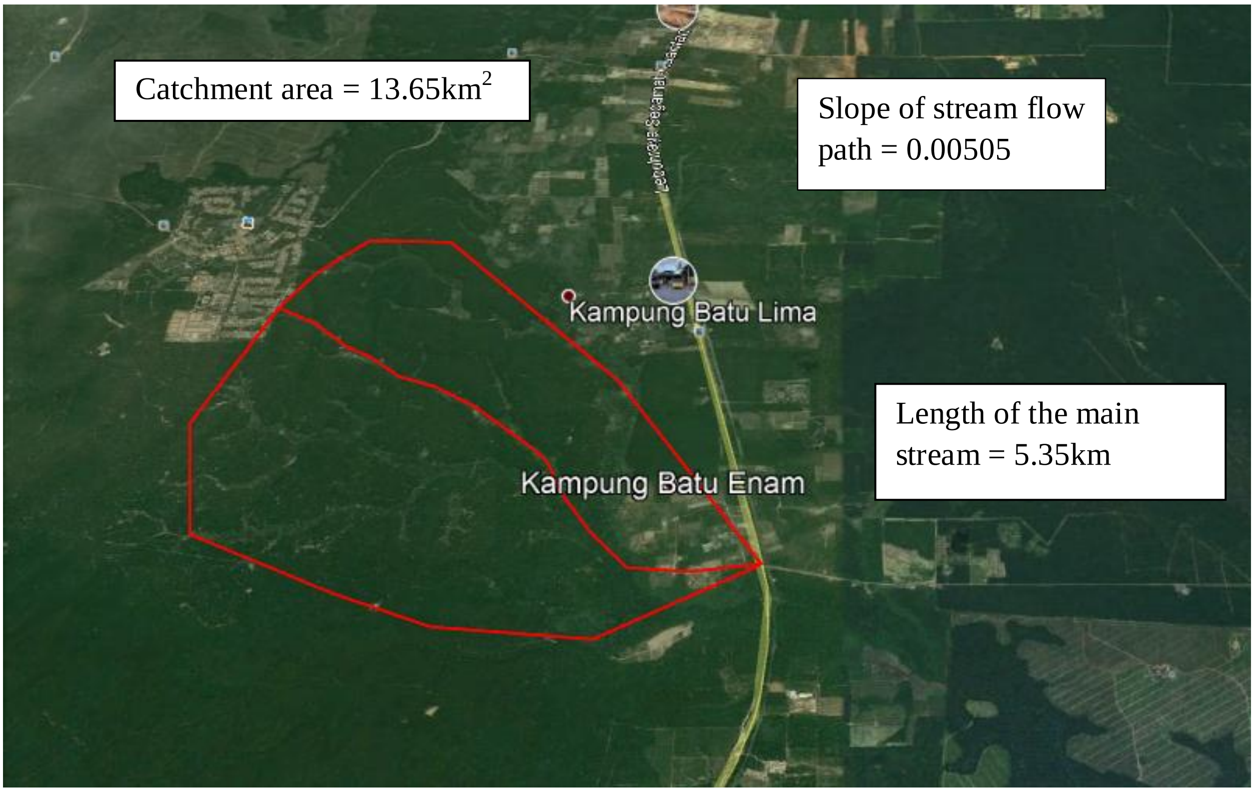


Figure 3.2 Illustration of Rasau River catchment area

Rainfall intensity is used in order to calculate the flow rate. The rainfall intensity, i in the rational formula represent the average rainfall intensity over duration equal to the time of concentration for the catchment.

According to MSMA, the location of the nearest rainfall station with the study area is Sungai Lembing station. Thus the fitting constant value for the IDF Empirical equation for Sungai Lembing station can be used in this study. The value of the constants is as listed:

$\lambda = 45.999$

$K = 0.210$

$\theta = 0.074$

$\eta = 0.59$

By insert the constant value, the rainfall intensity for 100-year ARI for the study area is:

$$i = \frac{\lambda T^K}{(d + \theta)^\eta}$$

$$i = \frac{(45.999)(100)^{0.21}}{(3.73 + 0.074)^{0.59}}$$

$$i = 55.00\ mm/hr$$

The average 100 year rainfall is required to be inputted in the HEC-HMS in order to estimate the 100 year design flood hydrograph for site catchment. Thus, by referring to normalised design rainfall temporal pattern, the 100 year rainfall depth with 5min time interval is tabulated in the Table 3.3.

Table 3.3 Rainfall depth

Time (min)	Rainfall depth (mm)
5	10.87
10	12.10
15	12.92
20	17.84
25	21.13
30	31.39
35	22.57

40	18.05
45	14.16
50	12.31
55	11.69
60	9.44

3.5 DATA ANALYSIS

3.5.1 HEC-HMS

In this study, HEC-HMS is used to compute the 100 year peak flow of the river. The peak flow or peak discharge only can be determined if the catchment area, time of concentration and rainfall data is known and calculated. The procedure below need to be followed to achieved the peak flow by using HEC-HMS application:

1. Create a new project by inserting the title and name of the project as shown in the Figure 3.3.

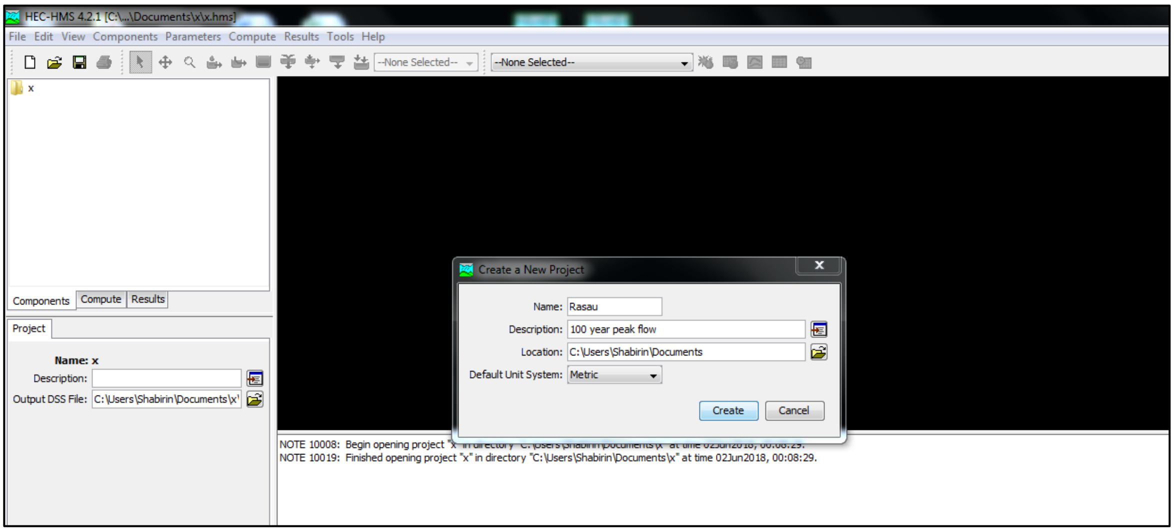


Figure 3.3 Create a new project with a proper name

2. Next is create a new basin model by go to **Components** and click on **Basin Model Manager** as shown in the Figure 3.4.

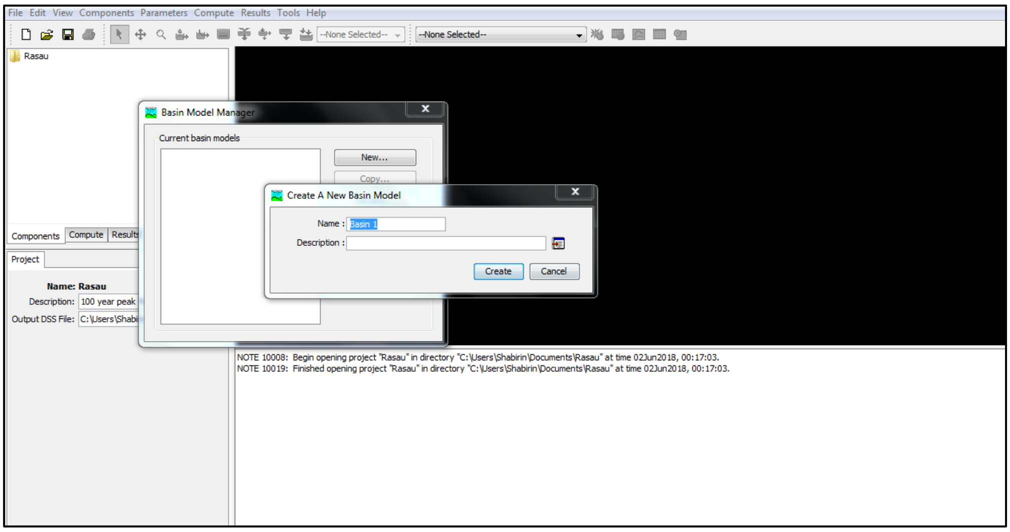


Figure 3.4 Create a new basin model

3. After that, click on **Sub-basin Creation Tool** before a new sub-basin can be created as shown in the Figure 3.5.

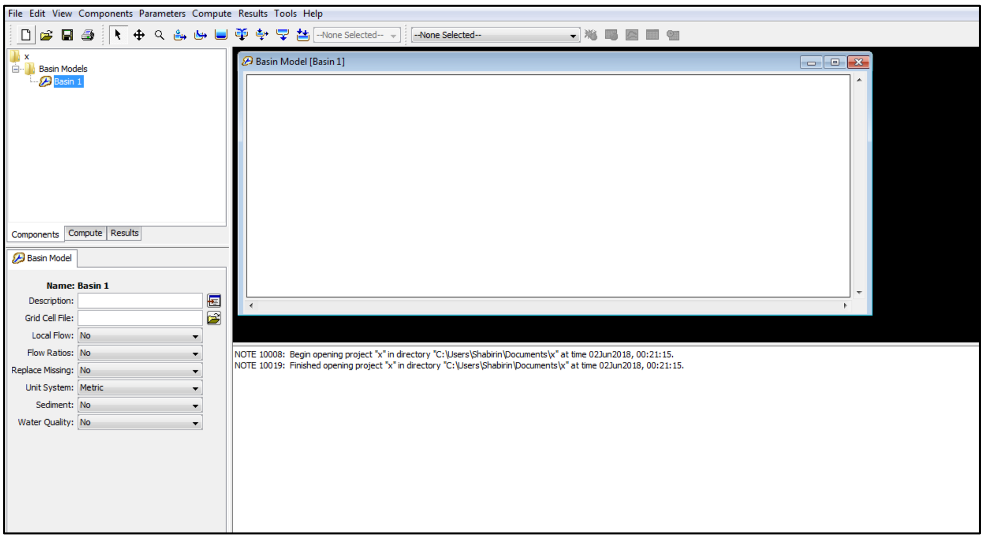


Figure 3.5 Click on Sub-basin Creation Tool to start create a new sub-basin.

4. Next, click on the **Basin Model** commend to make one sub-basin for the catchment area as shown in the Figure 3.6.

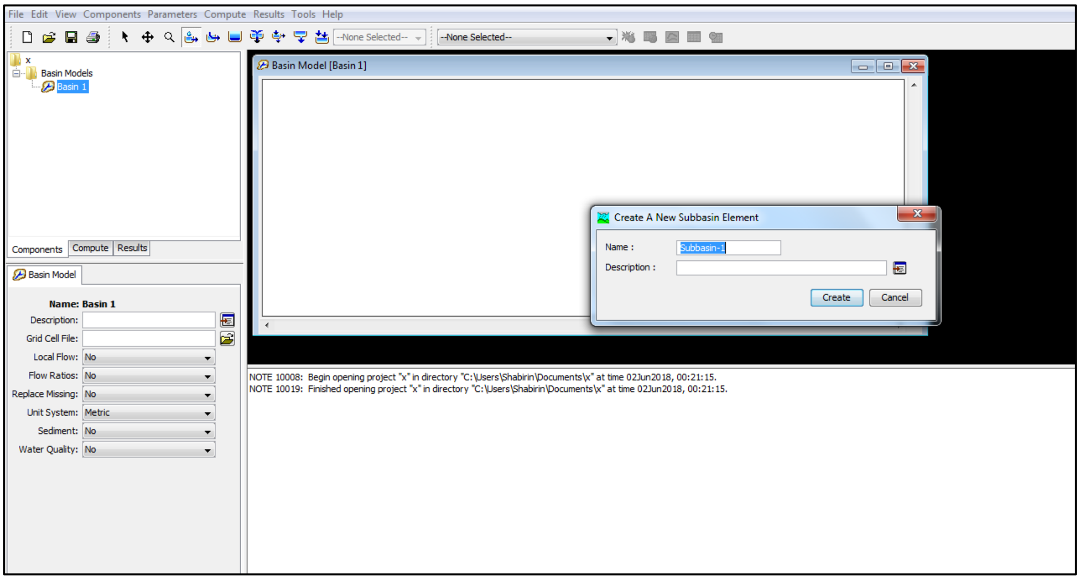


Figure 3.6 Create a new sub-basin

5. After a new sub-basin created, fill in the component for the sub-basin. Put the catchment area in the **Area** column as shown in the Figure 3.7.

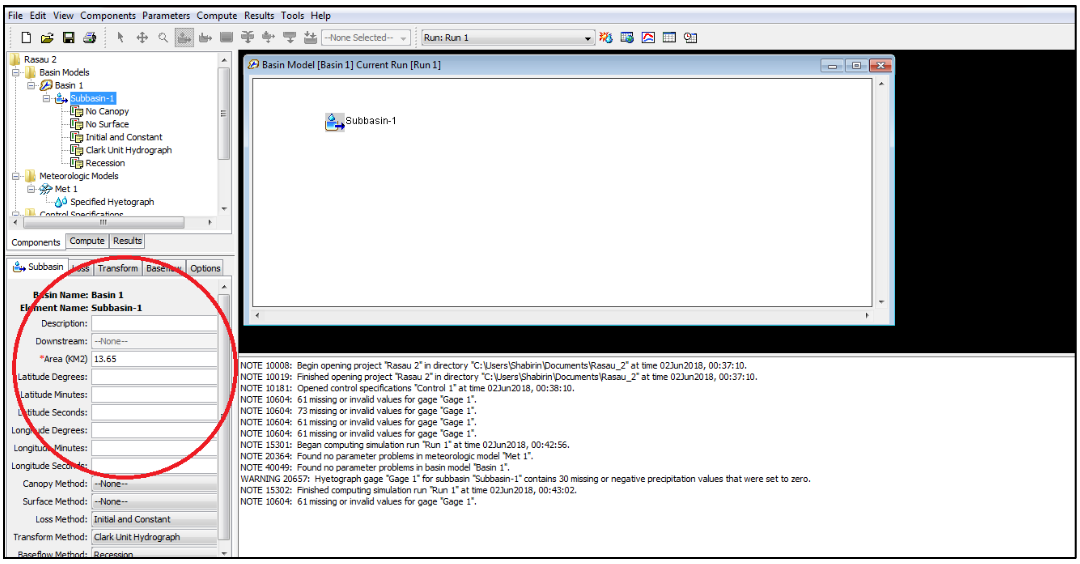


Figure 3.7 Fill in the component for the sub-basin

6. Again, click on the **Component** to select **Time Series Data Manager**. Then create a new **Time Series Data Manager** as shown in the Figure 3.8.

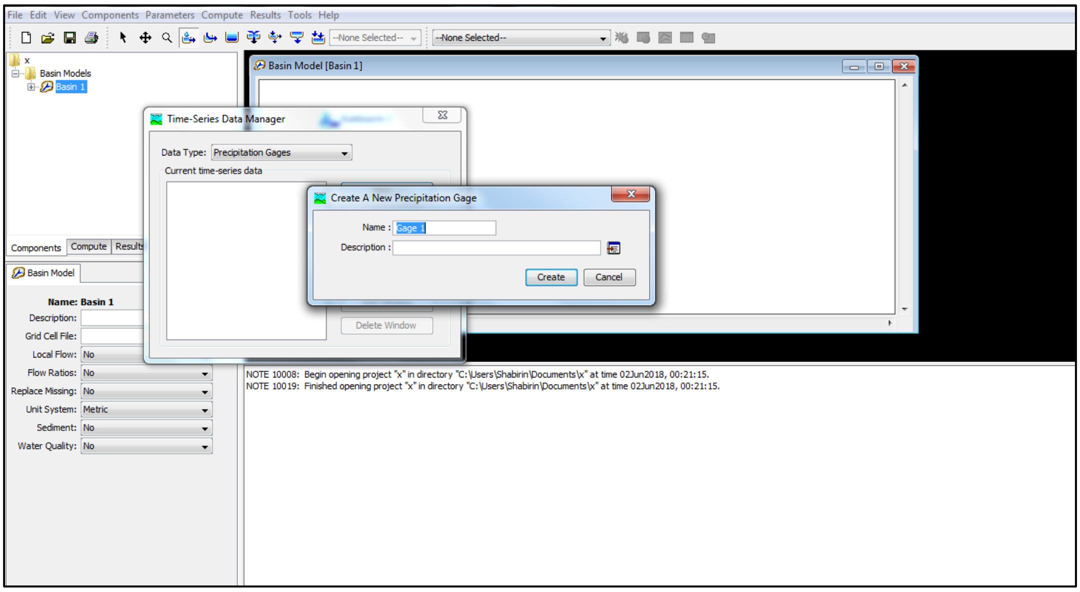


Figure 3.8 Click Time Series Data Manager to create a new time Series data Manager

7. At **Time Series Gage** field, choose 5 minute for the time interval. Then choose any date and time for that interval. After that, fill in the precipitation value as shown in the Figure 3.9 and Figure 3.10.

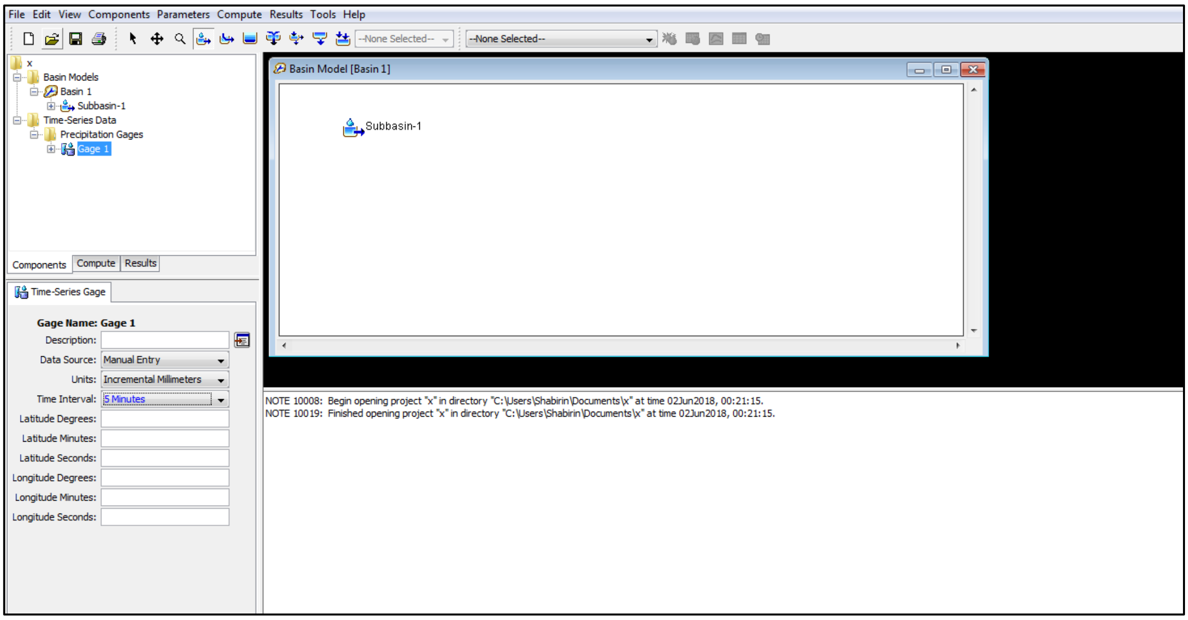


Figure 3.9 Fill in the requirement data in Time Series Gage field

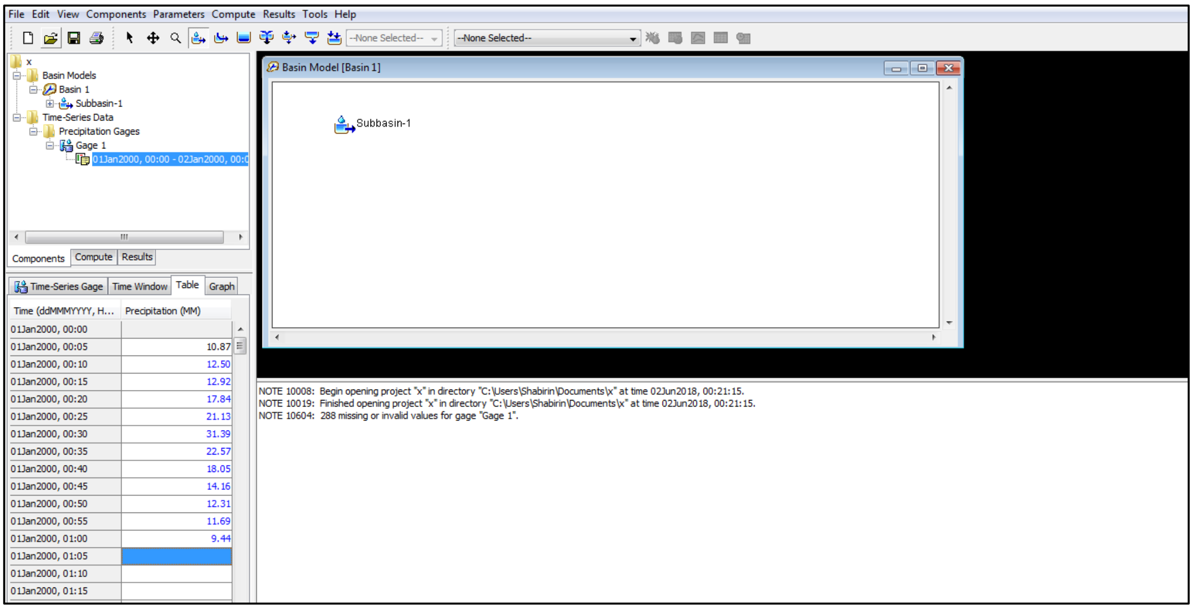


Figure 3.10 Insert the precipitation value

8. After that, choose the **Meteorology Model Manager** to create a new Met. At the **Meteorology Model** field, change the **Replace Missing** to **Set to Default**. In the **Specified Hydrograph**, choose everything as **Gage 1** as shown in Figure 3.11, Figure 3.12 and Figure 3.13.

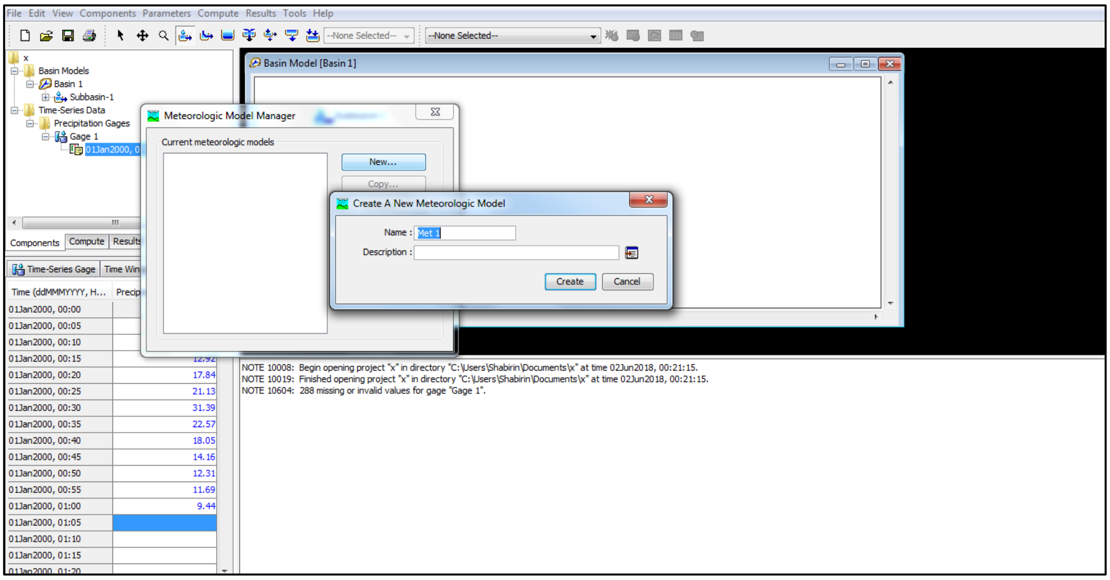


Figure 3.11 Create a new Met

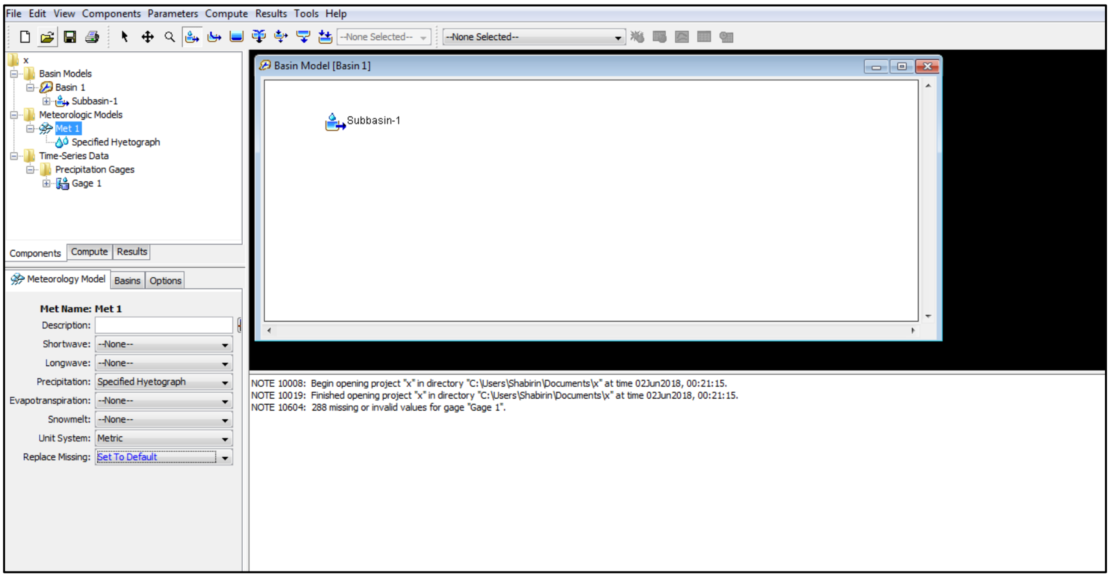


Figure 3.12 Change the Replace Missing to Set to Default

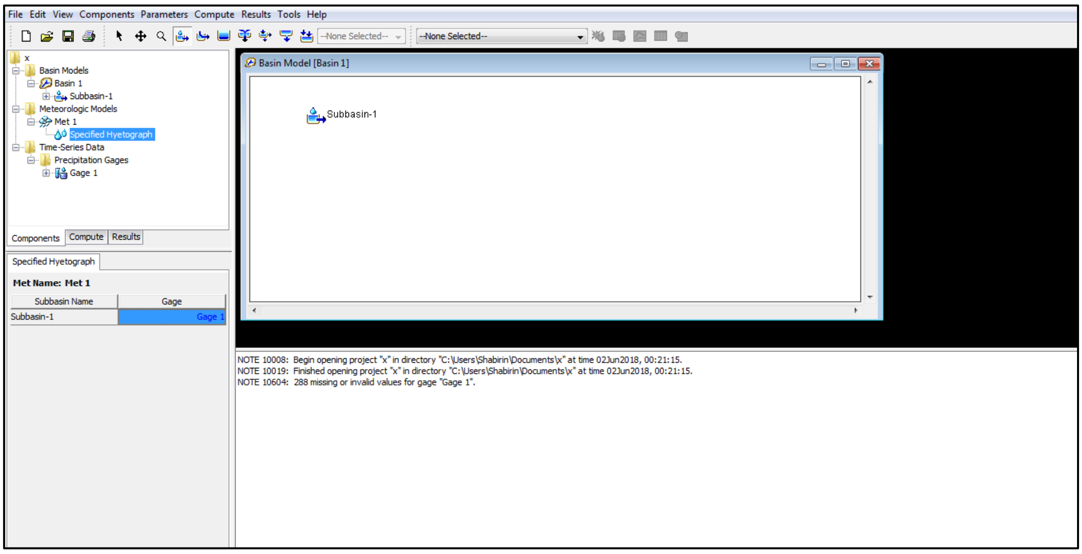


Figure 3.13 Change every column to Gage 1

9. Next, choose **Control Specification Manager** in the Components to create a new Control as shown in the Figure 3.14. Then, put a same date and time as before with time interval 5 minutes as shown in the Figure 3.15.

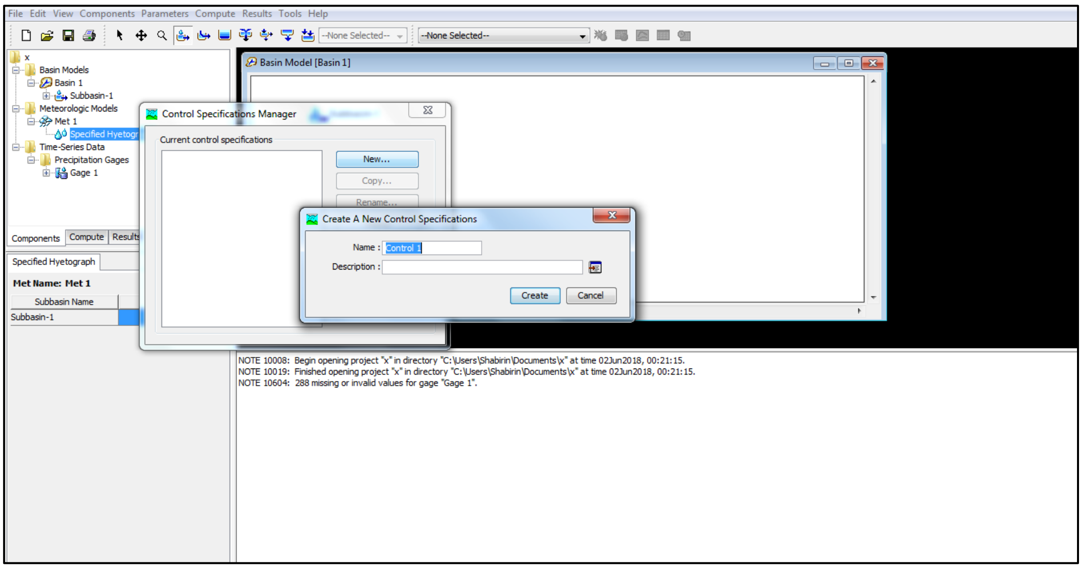


Figure 3.14 Choose Control Specification Manager to start create a new Control

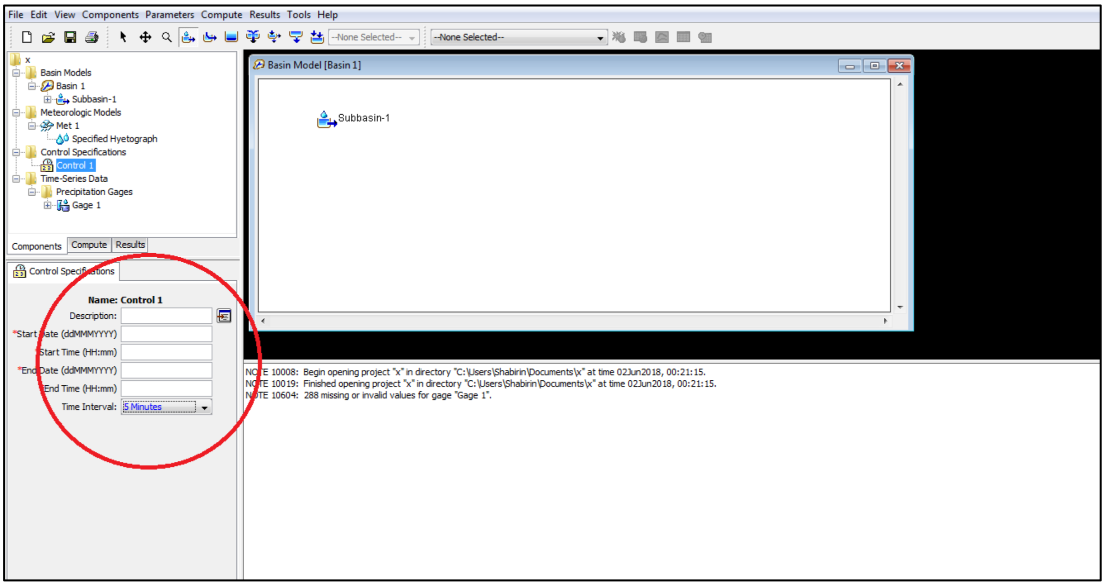


Figure 3.15 Put same date and time in the Control field

10. Next is to create a **Compute** and run the program. A **Compute** can be created by go to **Compute** and click **Compute**>**Create Compute**>**Simulation Run**. A window will pop up as shown in Figure 3.16. The click **Next** to finish creates a **Compute.**

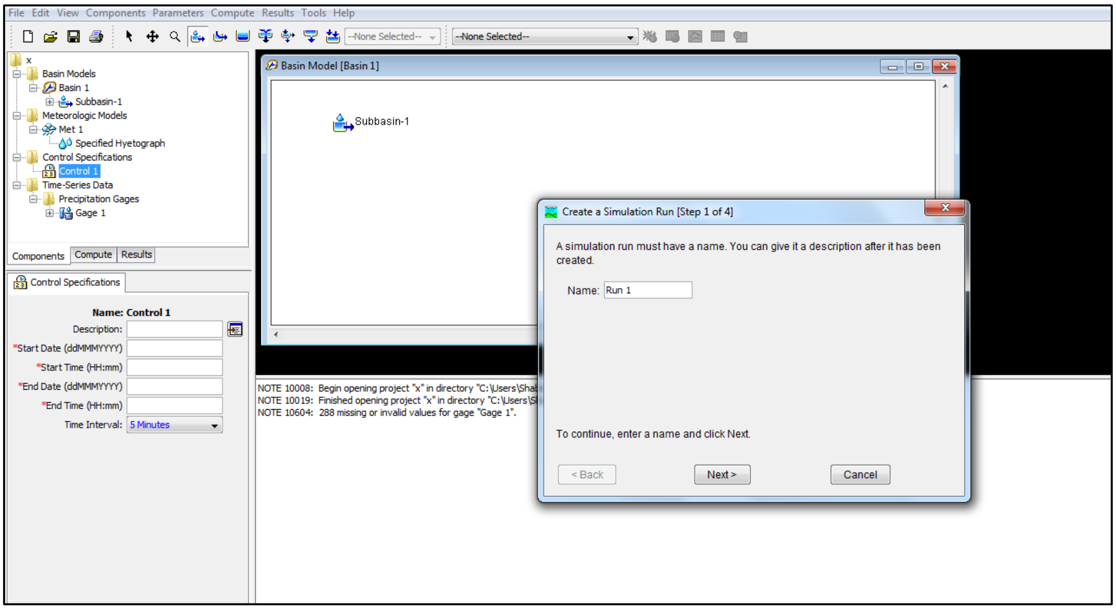


Figure 3.16 Create a Compute

11. Finally, the peak discharge can be known by run the program. By click **Compute** in Compute window, we can see the result in **Global Summary** as shown in the Figure 3.17.

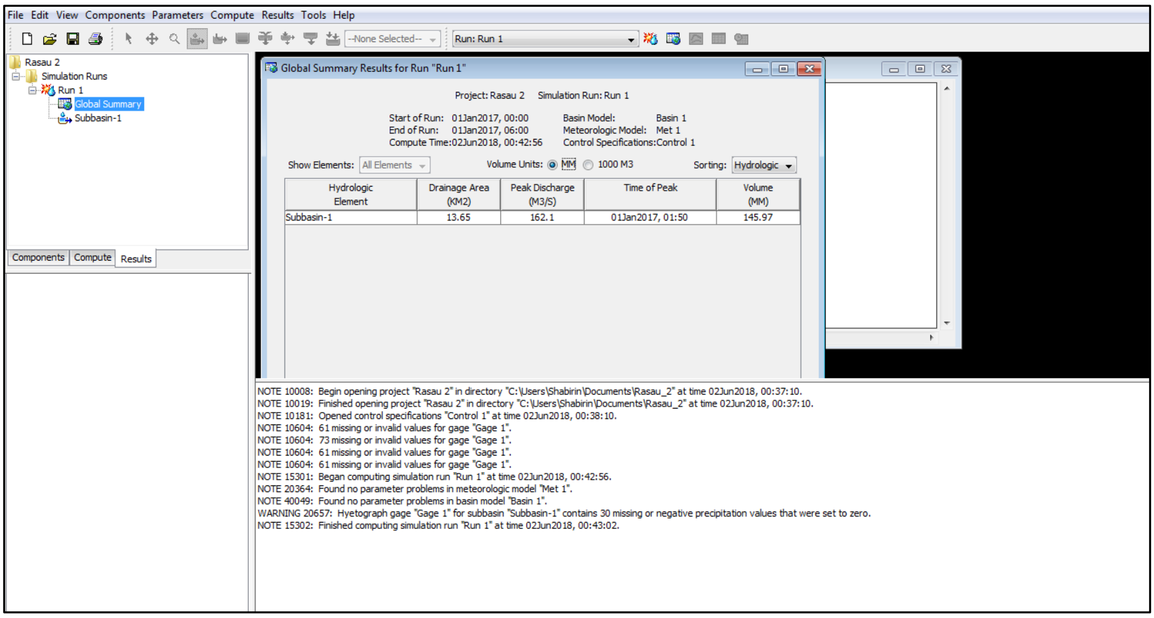


Figure 3.17 Peak discharge in shown in the Global Summary

12. Repeat all the steps above to compute the 5 year, 10 year, 20 year, and 50 year peak flow.

3.5.2 HEC-RAS

HEC-RAS divides the necessary input into two categories: geometric data and flow data. Both can be accessed through the Edit menu in the main program window or by clicking one of the shortcut buttons in the same window. In using HEC-RAS application, the steps outlined below need to be followed to achieve the desired result.

3.5.2.1 Geometric Data

i. The **Geometric Data Editor** is where all physical and topographical data are input.

ii. The first step in creating a model is to click on a button on the left side of the window labelled **River ReaCH** The user then able to draw the river riCH

iii. Next, the cross sections can be entered. As cross sections are created, they are automatically placed on the drawing of the river reaCH

iv. To create a new cross section, click on **Add a new Cross Section** in the Options menu. After river station number is entered, one can input data for the cross section into the program.

v. Distance to the next downstream cross section is needed along the left overbank (LOB), the channel, and the right overbank (ROB). Channel bank stations as well as contraction and expansion coefficients are also necessary.

vi. The Manning *n* values can be entered in one of the two different ways.

a) If there is no variation in the n values within a portion of the cross section, then the *n* values can be directly entered into the existing fields.

b) If there is variation in *n* values within a part of the cross section, choosing **Horizontal Variation in *n* Values from the** Options menu creates a new column next to the cross-sectional elevation field.

vii. Figure 3.18 below shows an example of a complete Cross Section data input window

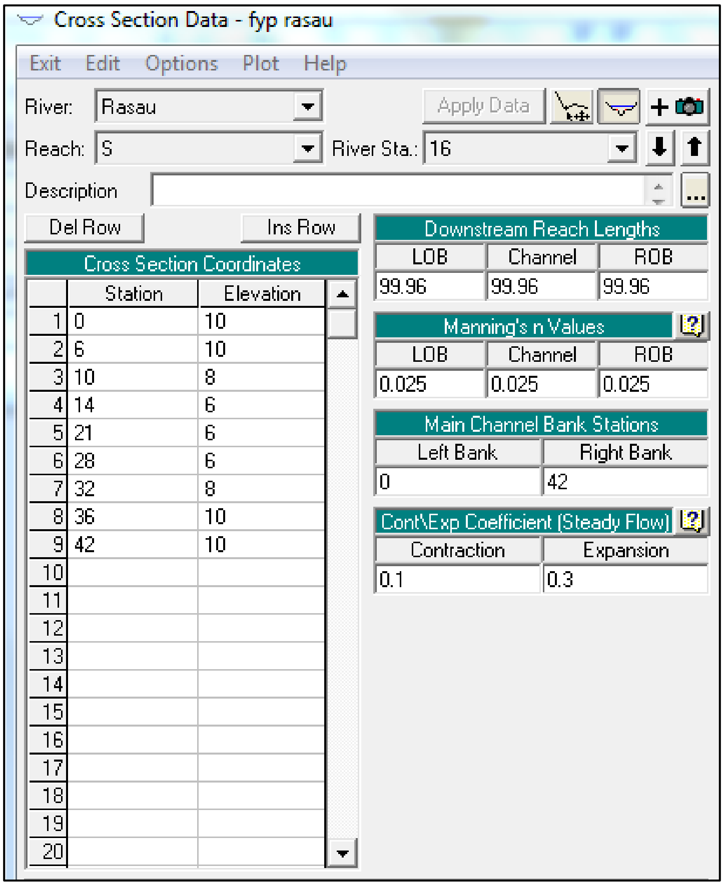


Figure 3.18 Input window for cross section

viii. Other important options are accessed from the **Cross Section Data** window. Areas of ineffective flow as well as levees and blocked obstructions are defined from the Options menu.

ix. Next is click on **Plot Cross Section** in the Plot menu brings up a plot of the cross section as shown in the Figure 3.19.

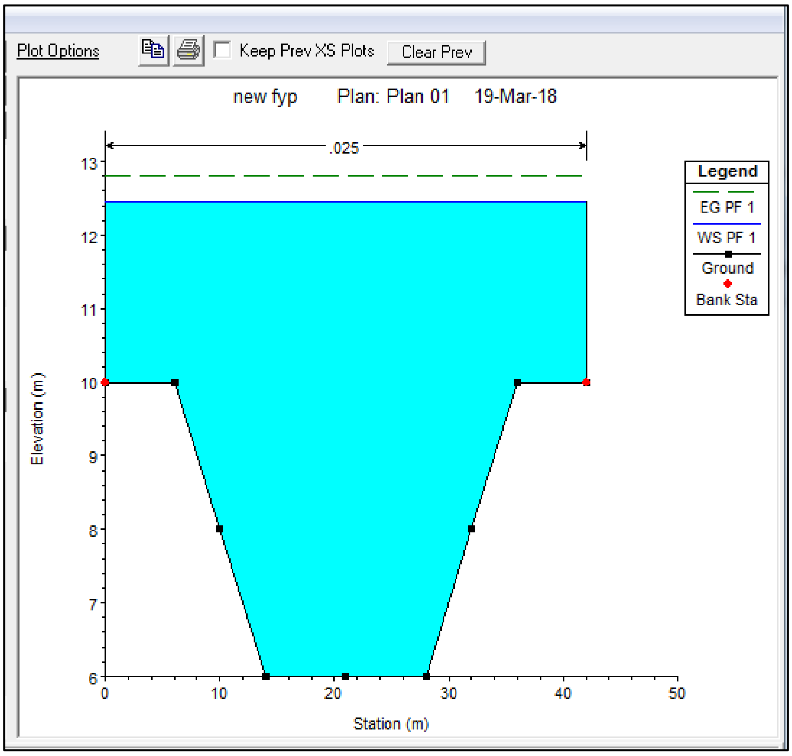


Figure 3.19 Cross section with water surface profiles

x. HEC-RAS cross section data can be checked by simply clicking through the cross section plots on the screen using up and down arrows.

xi. Bridges in HEC-RAS required four cross section: two just a few feet away from each face of the bridge, one far enough upstream that flow has not yet begun to contract, and one far enough downstream that flow has completely expanded.

xii. Then, click on the **Brdg/Culv** button to opens the Bridge Culvert window as shown in Figure 3.20.

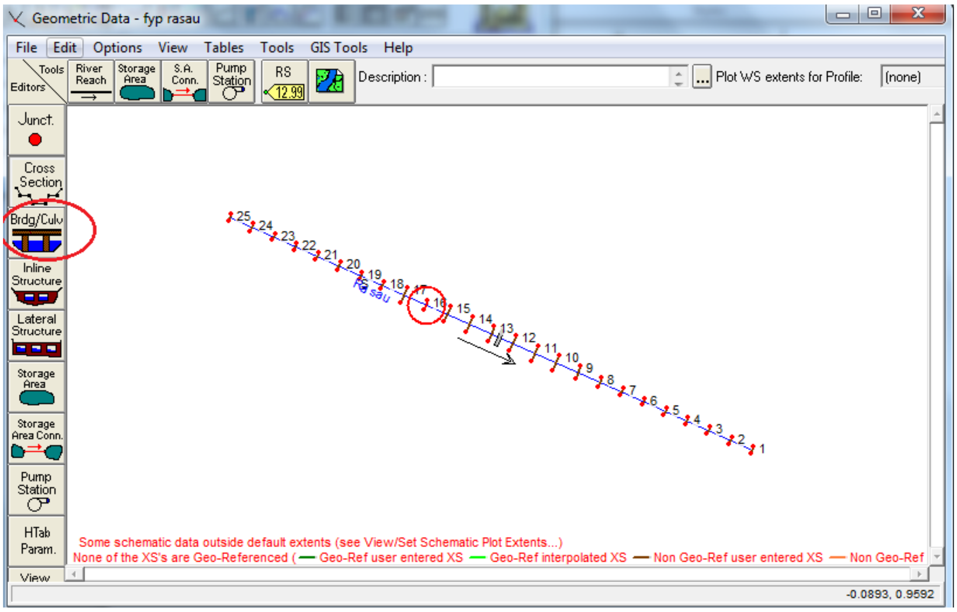


Figure 3.20 Brdg/Culv button

xiii. Click on Options and select **Add a Bridge and/or Culvert** to begin the process of creating the bridge as shown in Figure 3.21.

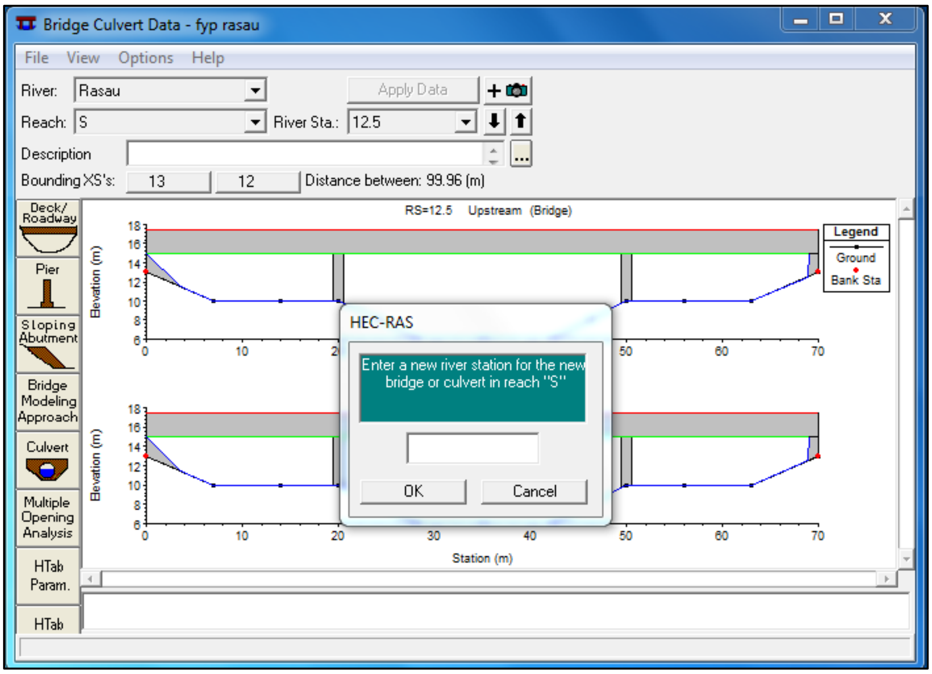


Figure 3.21 Process of creating a bridge by click Options >Add a bridge and/or culvert

xiv. HEC-RAS allows user to input individual bridge piers and to define the height and width of each pier. The **Pier** window is used for defining the size and location of the piers as shown in Figure 3.22 and Figure 3.23.

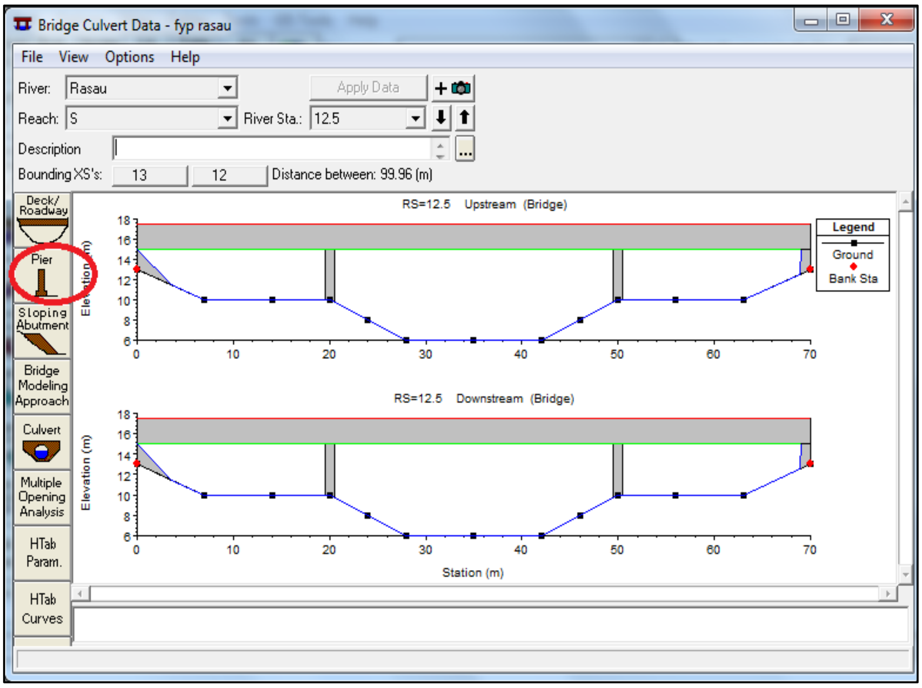


Figure 3.22 Pier window is used for defining the size and location of the piers

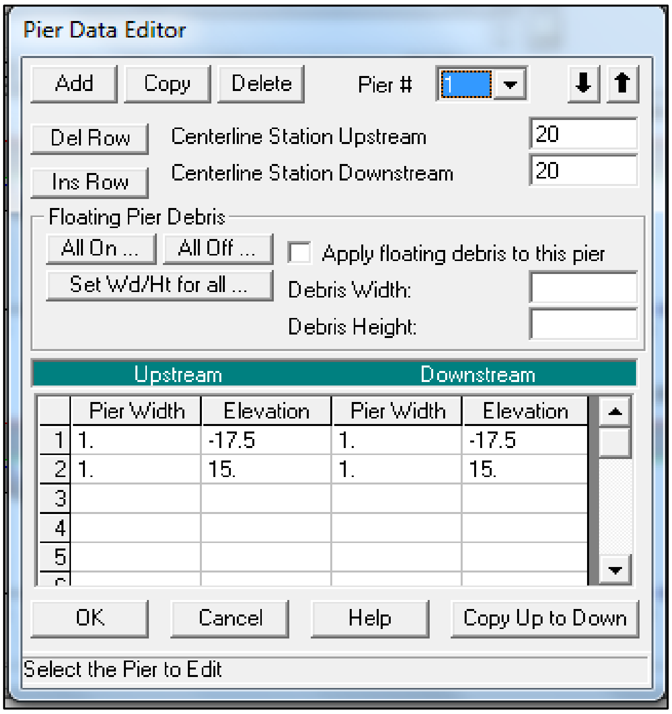


Figure 3.23 Insert required information in Pier Data Editor

xv. Next, one can input deck and roadway information by click **Deck/Roadway** window. Then, insert all required information in Deck/Roadway Data Editor. Everything is shown in Figure 3.24 and Figure 3.25.

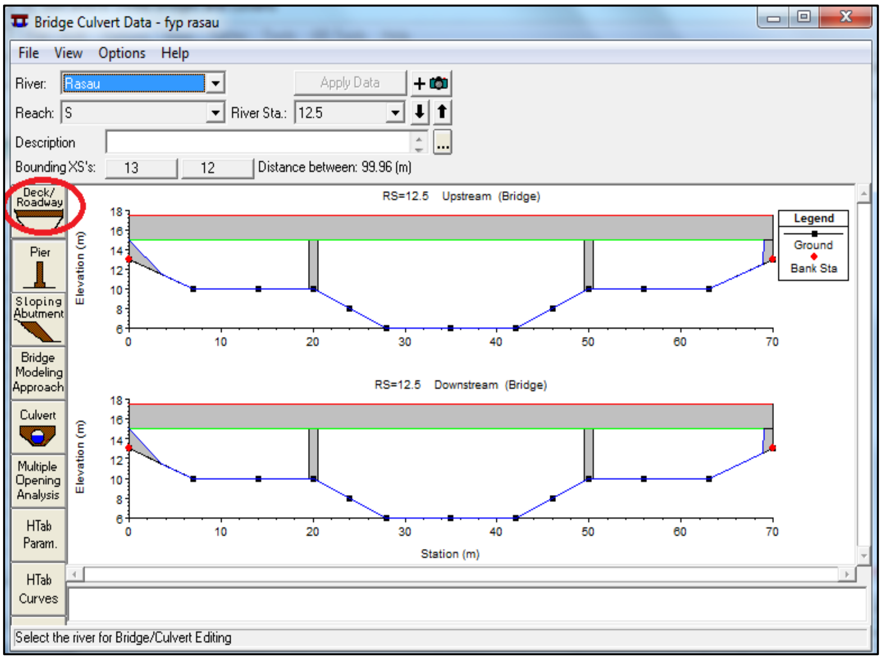


Figure 3.24 Click Deck/Roadway window to start insert required data

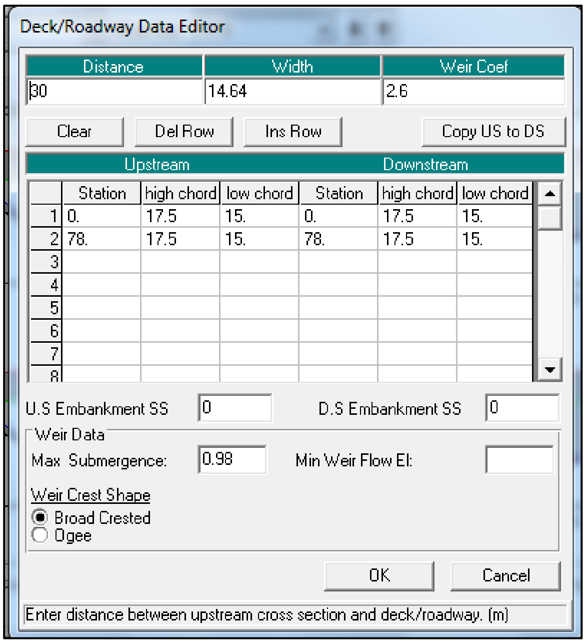


Figure 3.25 Insert all required data in Deck/Roadway Data Editor

xvi. Next is inserting the required information for the sloping abutment by clicks the **Sloping Abutment** editor as shown in the Figure 3.26.

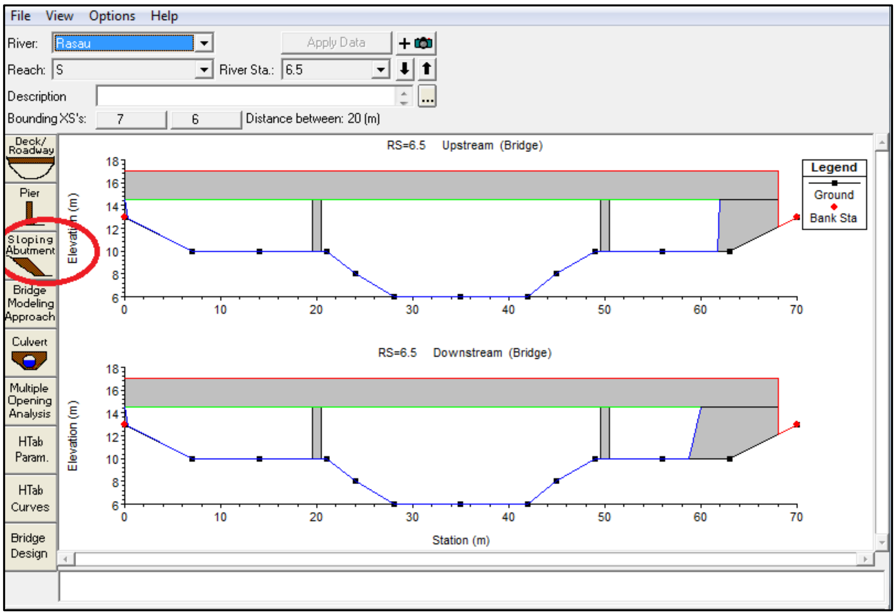


Figure 3.26 Click Sloping Abutment to start insert required information

xvii. Window sloping abutment editor will appear as shown as Figure 3.27. Then, insert all the known information about the sloping abutment.

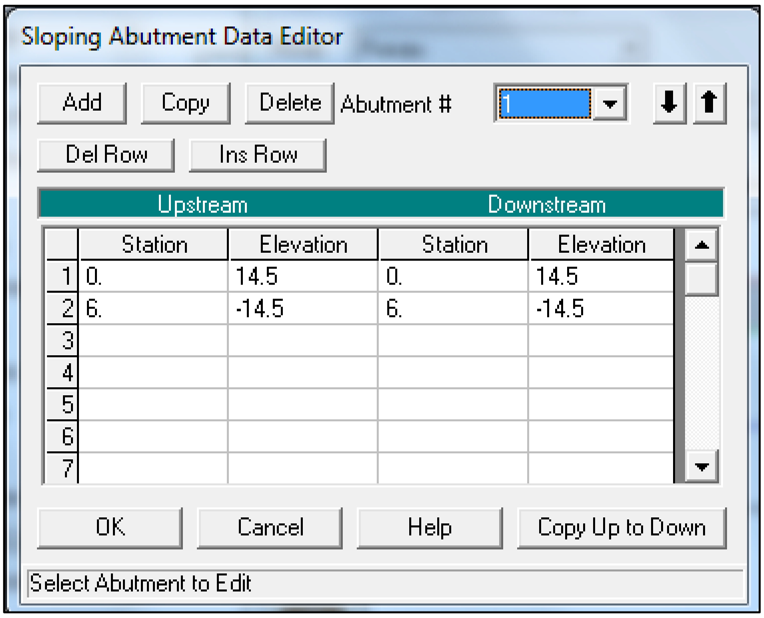


Figure 3.27 Insert the known information about sloping abutment

3.5.2.2 Flow Data

i. HEC-RAS requires the user to select the reach and all the cross sections where a change in flow occurs.

ii. HEC-RAS also maintains the ability to model multiple profiles simultaneously. This allows the user to easily compare, for example, the 5-yr, 10-yr, 20-yr, 50-yr, and 100-yr floods on one graph.

iii. The Steady Flow Data Editor is accessed by clicking Steady Flow Data as shown in Figure 3.28

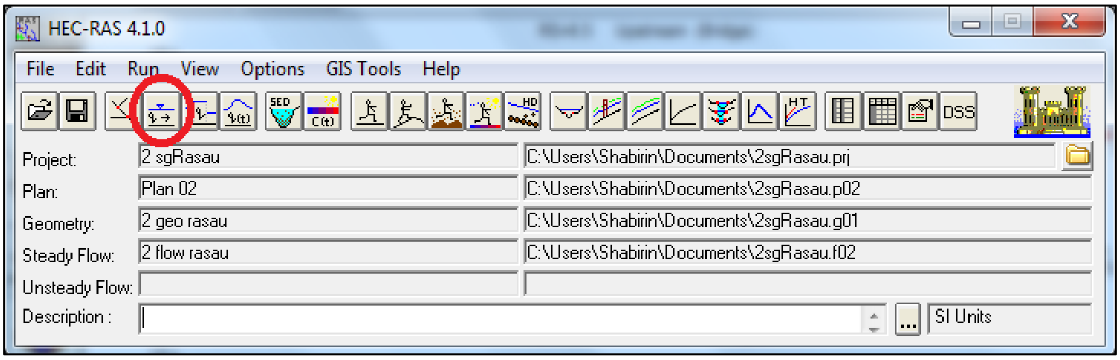


Figure 3.28 Click steady flow icon to start insert steady flow data

iv. **Steady Flow** window will appear as shown in the Figure 3.29. Then insert the value of the flow rate(s).

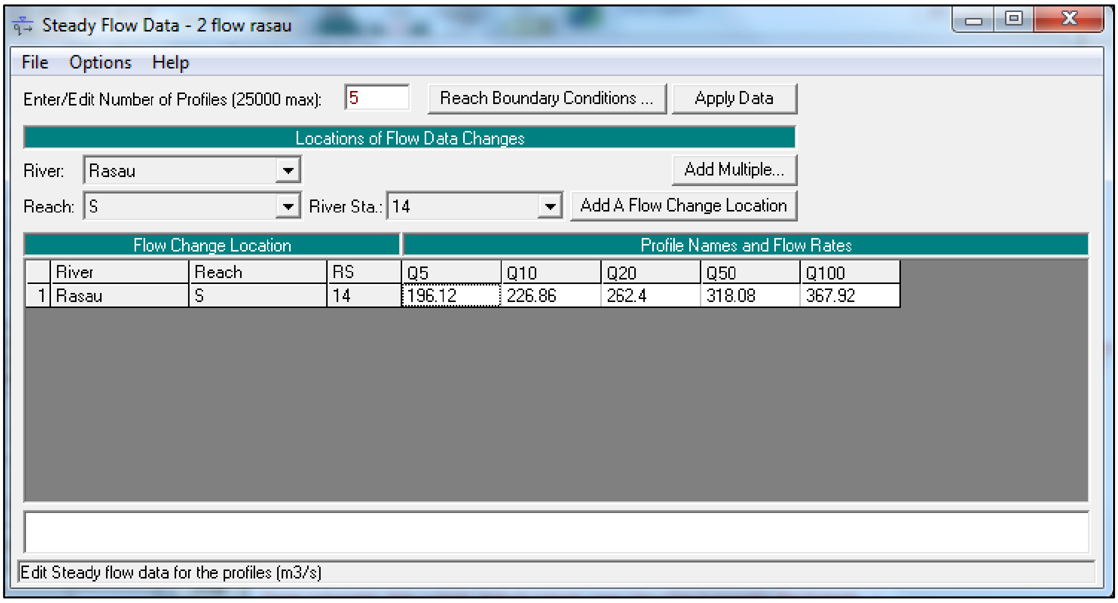


Figure 3.29 Insert the flow rate,Q data in the steady flow window

v. After include all the necessary information in the **Reach Boundary Condition** field, click **Apply Data** to finish the step in steady flow requirement.

vi. In order to get the steady flow simulation, click '**perform a steady flow simulation**' icon as shown in the Figure 3.30. Then tick the **Subcritical** condition before click **Compute** button as shown in the Figure 3.31.

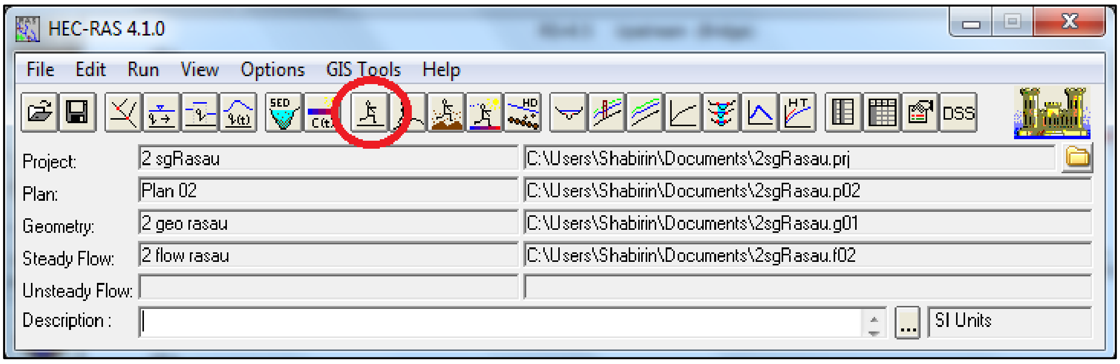


Figure 3.30 Click 'perform steady flow simulation' to start the simulation

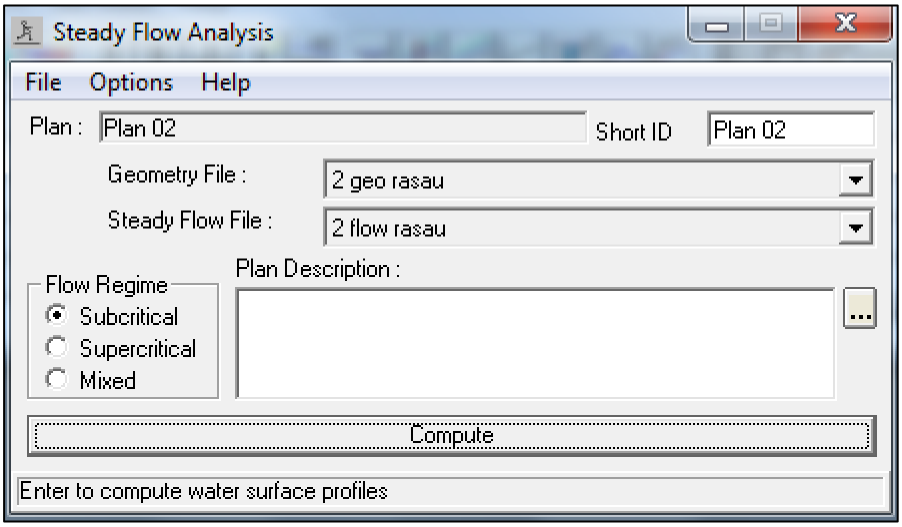


Figure 3.31 Click compute button to compute steady flow simulation

3.5.2.3 Running and Viewing Results

i. Click on **Run** and then **Steady Flow Analysis** in the main program window to run the simulation after all geometric data, flow data and boundary conditions have been entered.

ii. Results of HEC-RAS are useful in their ability to create various plots and tables of the output results.

iii. In order to view the water profile with corresponding cross section, click '**view cross section**' icon as shown in the Figure 3.32.

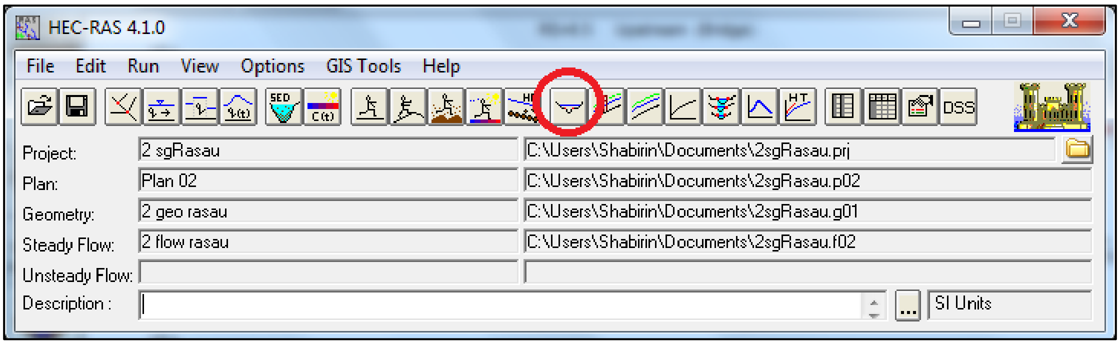


Figure 3.32 'View cross section' icon which use to view water profile at specific cross section

iv. Then, the result of water level profile will appear as shown in Figure 3.33.

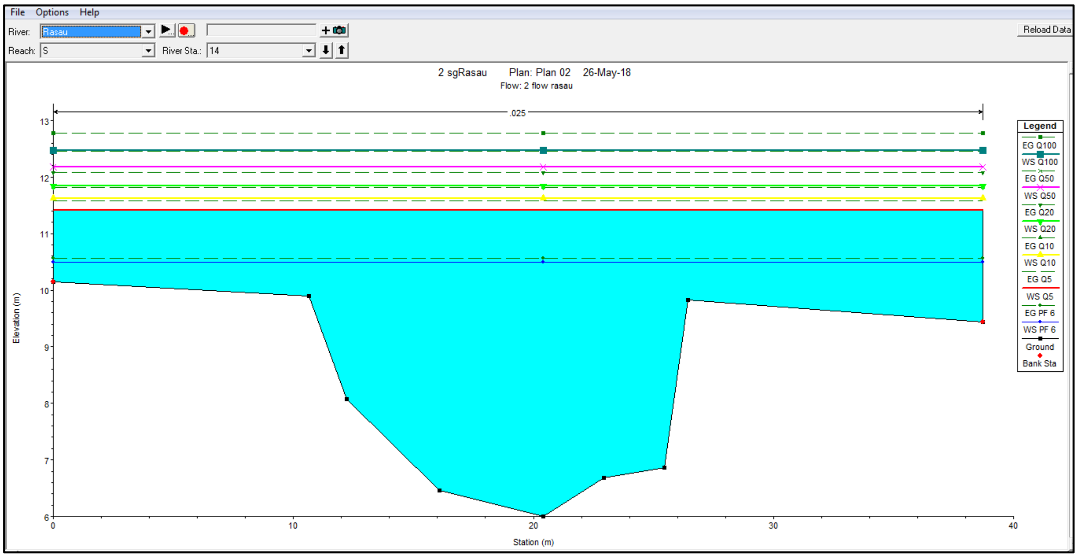


Figure 3.33 Result of water level profile at specific cross section

v. To view result in the form of longitudinal cross section, click '**view profile'** icon as shown in the Figure 3.34.

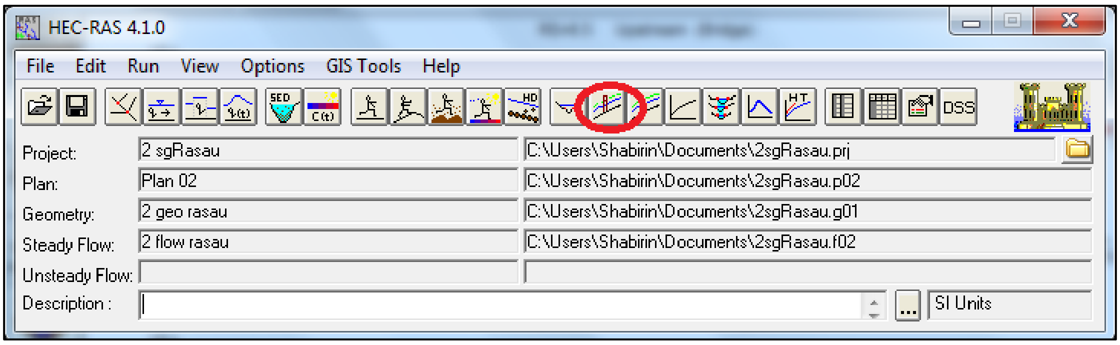


Figure 3.34 'View profile' icon

vi. The water level profile with the longitudinal cross section will appear as shown in the Figure 3.35

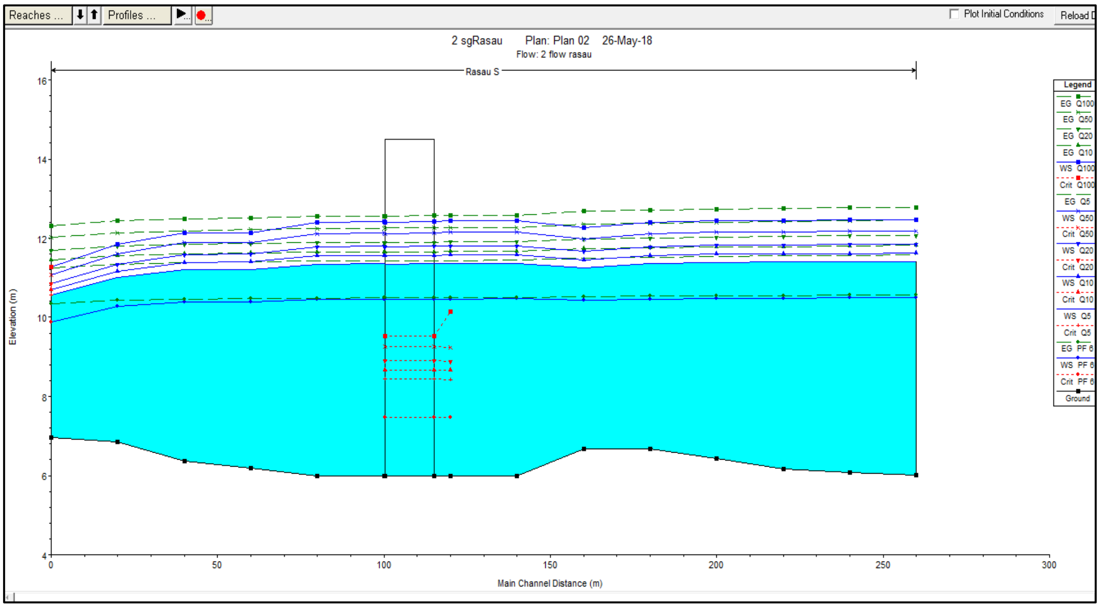


Figure 3.35 Longitudinal cross section with water level profile

vii. Next, one also can view the result in perspective plot or 3D form by click at the '**View 3D multiple cross section plot**' as shown in the Figure 3.36.

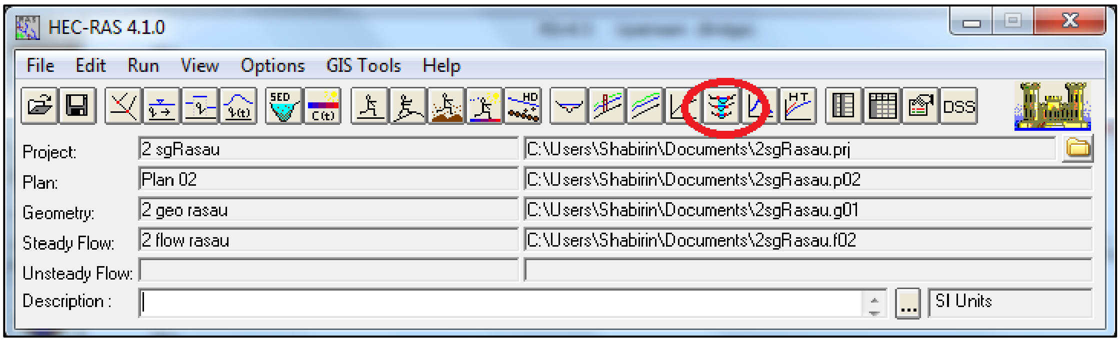


Figure 3.36 'View 3D multiple cross section plot" is to view the 3D plot of water profile

viii. The result of the3D XYZ perspective plot will appear as shown in the Figure 3.27.

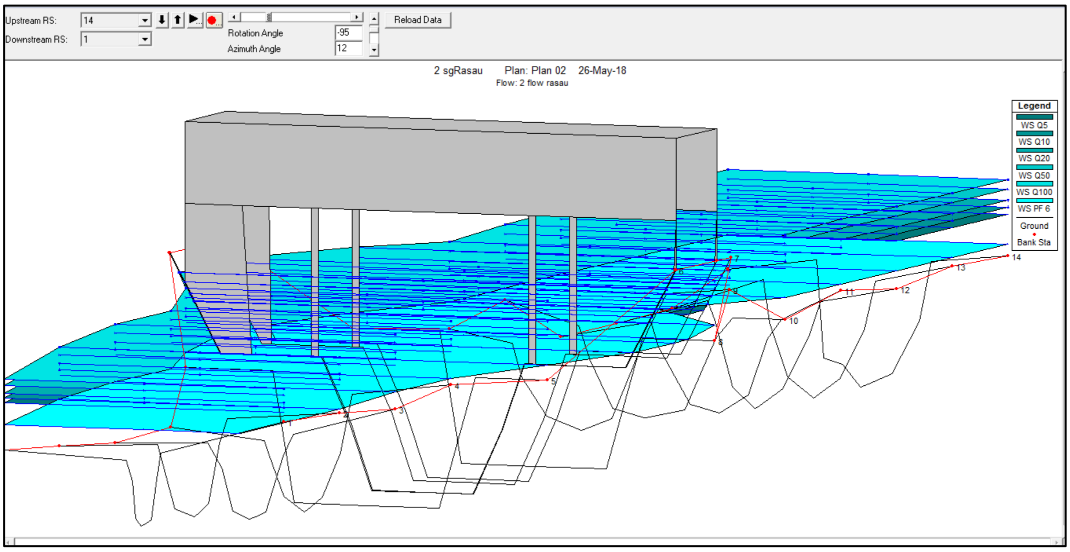


Figure 3.37 3D XYZ perspective plot of reach

ix. Another output or result is cross section output which is in the form of tables as shown in the Figure 3.38. All results can be found by clicking on the View menu in the main program window.

Cross Section Output

File Type Options Help

River: Rasau Profile: Q5

Reach S RS: 14 Plan: Plan 02

Plan: Plan 02 Rasau S RS: 14 Profile: Q5

		Element	Left OB	Channel	Right OB
E.G. Elev (m)	11.58	Element	Left OB	Channel	Right OB
Vel Head (m)	0.17	Wt. n-Val.		0.025	
W.S. Elev (m)	11.41	Reach Len. (m)	20.00	20.00	20.00
Crit W.S. (m)		Flow Area (m2)		106.29	
E.G. Slope (m/m)	0.000684	Area (m2)		106.29	
Q Total (m3/s)	196.12	Flow (m3/s)		196.12	
Top Width (m)	38.72	Top Width (m)		38.72	
Vel Total (m/s)	1.85	Avg. Vel. (m/s)		1.85	
Max Chl Dpth (m)	5.41	Hydr. Depth (m)		2.74	
Conv. Total (m3/s)	7496.8	Conv. (m3/s)		7496.8	
Length Wtd. (m)	20.00	Wetted Per. (m)		45.40	
Min Ch El (m)	6.00	Shear (N/m2)		15.71	
Alpha	1.00	Stream Power (N/m s)	1854.02	0.00	0.00
Frctn Loss (m)	0.01	Cum Volume (1000 m3)		30.83	
C & E Loss (m)	0.00	Cum SA (1000 m2)		11.34	

Errors, Warnings and Notes

Figure 3.38 Cross section output

3.6 DISCUSSION

The results from the modelling process were presented in a form of a water level profile and simulation. From the simulation, the water level of the river was determined in the event of 5-year, 10-year, 20-year, 50-year and 100-year ARI. In order to determine the backwater effect at the bridge which resulted in overflow to the catchment area, the difference between the water level with and without the presence of bridge piers is determined.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 INTRODUCTION

This chapter discussed the analysis of the simulation results base on the event cases that were carried out. Apart from that, this chapter also explained the achievement of the simulations which focused at the bridge.

The result and analysis for this study were focused on the flow of the river channel and water profile at upstream of the bridge, under the bridge and at the downstream of the bridge. Flow rate and water level of the river were simulated and recorded.

In order to determine the backwater effect at the bridge, the peak flow is required which is computed using the HEC-HMS software. From the peak flow computed, the water level profile of the river can be determined using the HC-RAS software.

4.2 SIMULATION RESULTS

The simulations were carried out for two different conditions:

i. Flow without the bridge piers.
ii. Flow with the bridge and its piers.

The water level data were recorded for one day duration and flow rate for each scenario was calculated with rainfall intensity for 55.0 minutes storm duration and 5-year, 10-year, 20-year, 50-year and 100-year ARI. With these flows, the water profile of the river for each cases were produced. Outcome of the analysis obtained from the simulation were then analysed.

4.3 TYPES OF SIMULATION

In using HEC-RAS software, a total of two simulations cases were carried out in the experiment. The discussion and analysis of the cases are stated in Table 4.1.

Table 4.1 Type of analysis

No. of Analysis	Case	Event	Analysis Type
1	Without bridge piers	Flow rate,Q with 5-year, 10-year, 20-year, 50-year and 100-year ARI	Elevation vs. Station (14 chainage)
2	With bridge piers	Flow rate,Q with 5-year, 10-year, 20-year, 50-year and 100-year ARI	Elevation vs. Station (14 chainage)

4.4 HEC-HMS

HEC-HMS is used to estimate the 5-year, 10-year, 20-year, 50-year, and 100-year ARI which to be inputted into HEC-RAS for river simulation. From HEC-HMS peak flow estimation, the result is summarized as Table 4.2.

Table 4.2 Results from HEC-HMS

Event (ARI)	Q_{peak} (m^3/s)
5-year	92.21
10-year	106.3
20-year	122.7
50-year	148.6
100-year	172.0

4.5 ANALYSIS OF SIMULATION RESULTS (HEC-RAS)

This study involved 260m length of the Rasau River included presence of a bridge. A part from that, there are total of 14 chainage along the 260m river length with maximum distance between chainage is 20m. The direction of flow is from CH0 to CH260. Figure 4.1 illustrates the cross section plan with respective chainage along the Rasau River. The levels of the left and right bank along the chainage were between 9.11m to 10.23m above sea level.

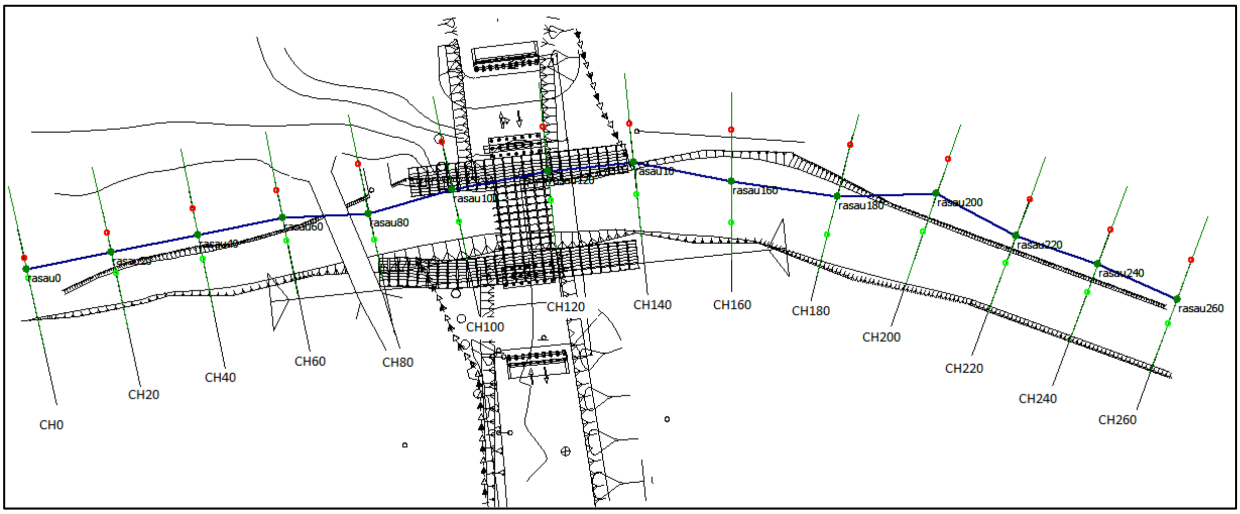


Figure 4.1 Cross section plan of Rasau River.

4.5.1 Water level for Q_5 with and without bridge piers

In this study, water level profiles were determined as the output from HEC-RAS software. Other than that, the backwater effect at the bridge was determined by knowing the difference in water levels at each cross section or chainage. The difference in water levels of Rasau River along the chainage with 5-year ARI is tabulated in Table 4.3 and water levels profiles together with backwater effect were discussed.

Table 4.3 Difference of water levels with Q_5

$Q_5 = 92.10\ m^3/s$						
Chainage	**Before bridge construction**		**After bridge construction**		**Difference of water levels (m)**	
	Water levels (m)		**Water levels (m)**			
	Left bank	**Right bank**	**Left bank**	**Right bank**	**Left bank**	**Right bank**
CH 0	9.57	9.57	9.94	9.94	0.37	0.37
CH 20	9.91	9.91	10.32	10.32	0.41	0.41
CH 40	10.00	10.00	10.45	10.45	0.45	0.45
CH 60	10.01	10.01	10.45	10.45	0.44	0.44
CH 80	10.01	10.01	10.52	10.52	0.51	0.51
CH 100	9.99	9.99	10.52	10.52	0.53	0.53
CH 110 U	9.99	9.99	10.52	10.52	0.53	0.53
CH 110 D	9.99	9.99	10.52	10.52	0.53	0.53
CH 120	9.99	9.99	10.52	10.52	0.53	0.53
CH 140	10.07	10.07	10.53	10.53	0.46	0.46
CH 160	10.11	10.11	10.48	10.48	0.37	0.37
CH 180	10.12	10.12	10.52	10.52	0.40	0.40
CH 200	10.14	10.14	10.55	10.55	0.41	0.41
CH 220	10.14	10.14	10.55	10.55	0.41	0.41
CH 220	10.15	10.15	10.55	10.55	0.40	0.40
CH 240	10.15	10.15	10.56	10.56	0.41	0.41

4.5.1.1 Water level for Q_5 at upstream of the bridge

Based on Table 4.3 and Figure 4.2 to Figure 4.13 the water levels were in between 9.57m to 10.01m above sea level at the upstream of the bridge without the bridge piers presence while with the bridge piers presence the water levels were between 9.94m to 10.52m above sea level at the upstream of the bridge. It can be said that the water levels has surpassed the river bank along CH 0 to CH 100 during rainfall event of 5-year ARI. In other words, the water is overflown onto both the left bank and right bank except at CH 20(without bridge piers) and CH 80(without bridge piers) due to its incline bank surface.

Besides, the differences in water levels along the upstream chainage which start from CH 0 to CH 100 were between 0.37m to 0.53m. This indicates that with the presence of Bridge 3 at the Rasau River, an increase of 0.53m in water level at CH 100 where the bridge piers are located.

Moreover, this situation shows that the water levels had overflow onto the left and right bank higher than normal condition at the same chainage without the bridge piers presence. Even though along the studied cross section should be already flooded with water but the level still at the safest level if compared with the water levels at each chainage or cross section with the presence of bridge piers.

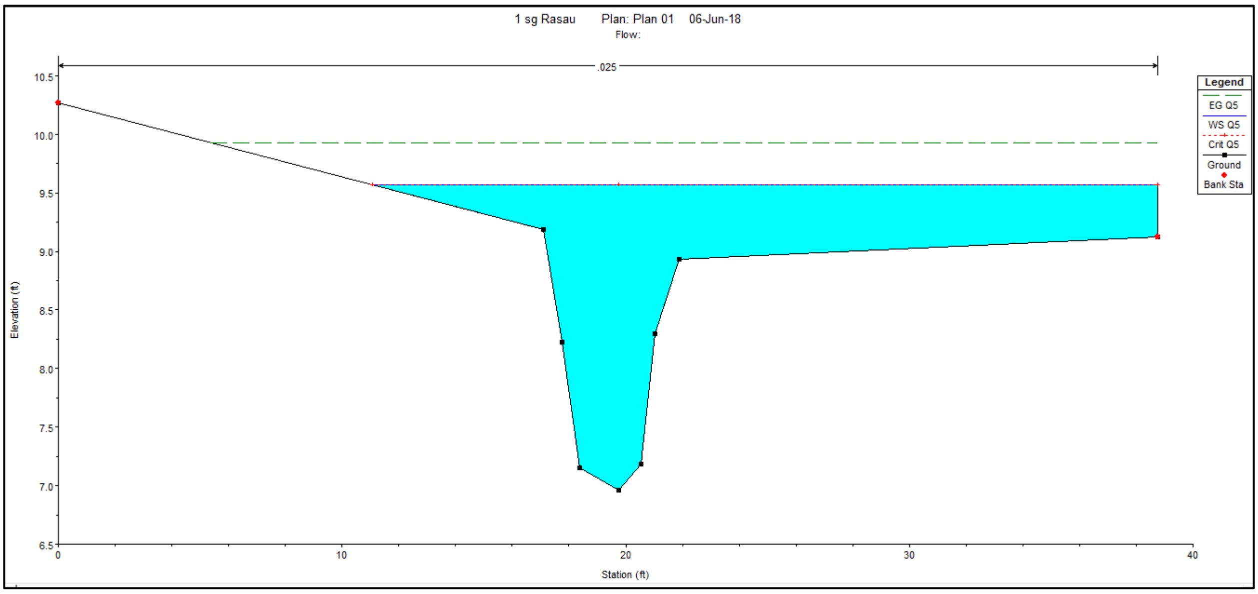


Figure 4.2 Water level at CH0 of the Rasau River (without bridge piers) – Q_5

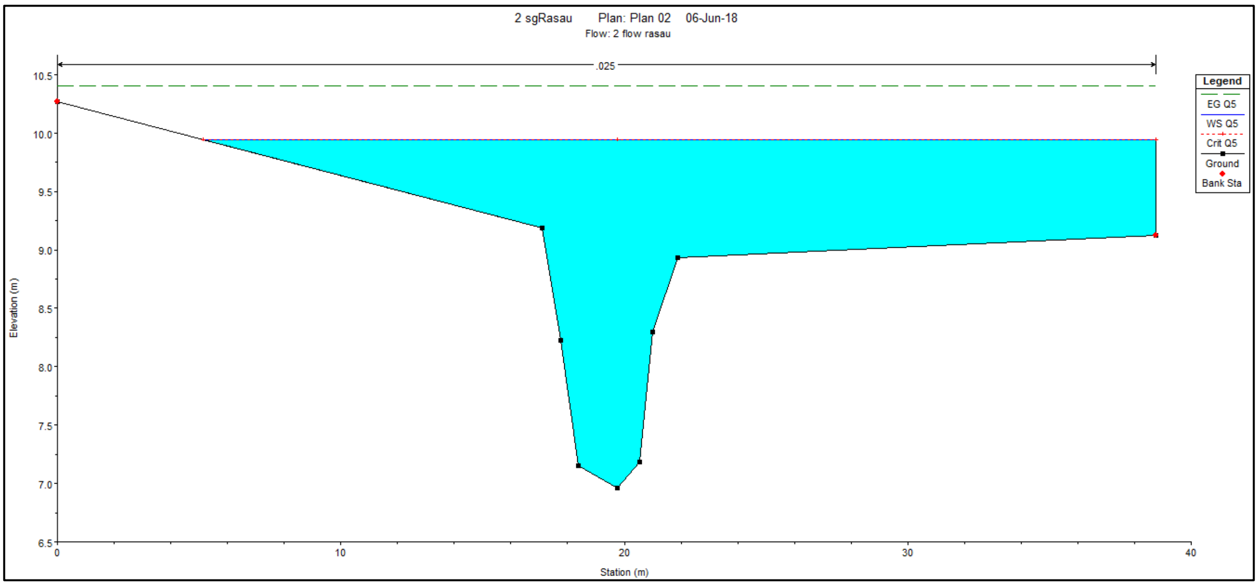


Figure 4.3 Water level at CH0 of the Rasau River (with bridge piers) – Q_5

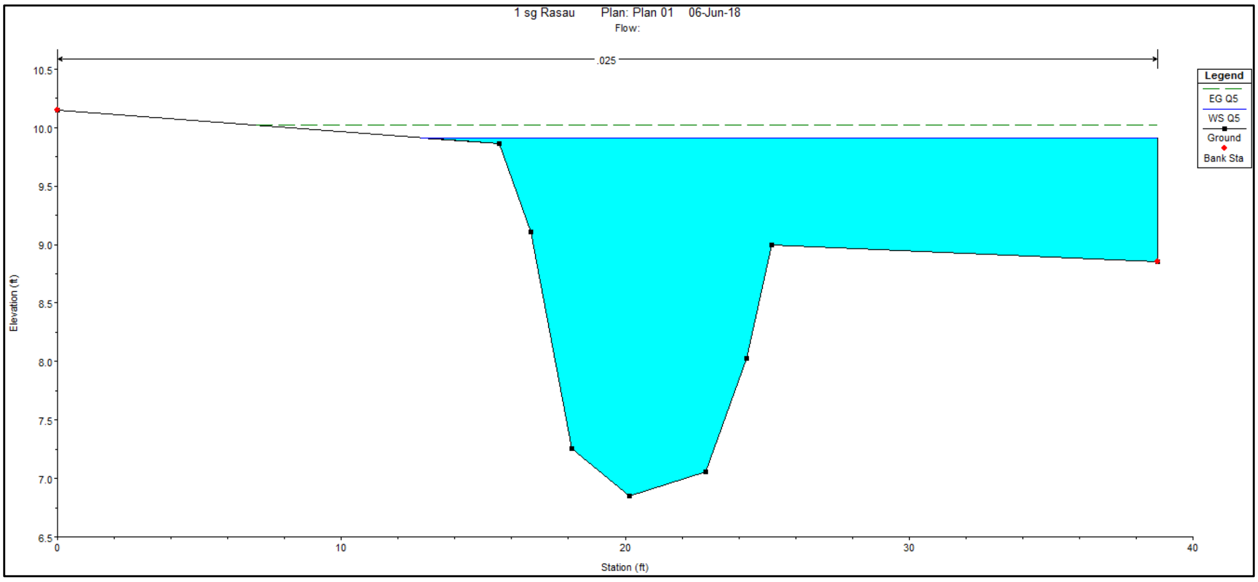


Figure 4.4 Water level at CH20 of Rasau River (without bridge piers) – Q_5

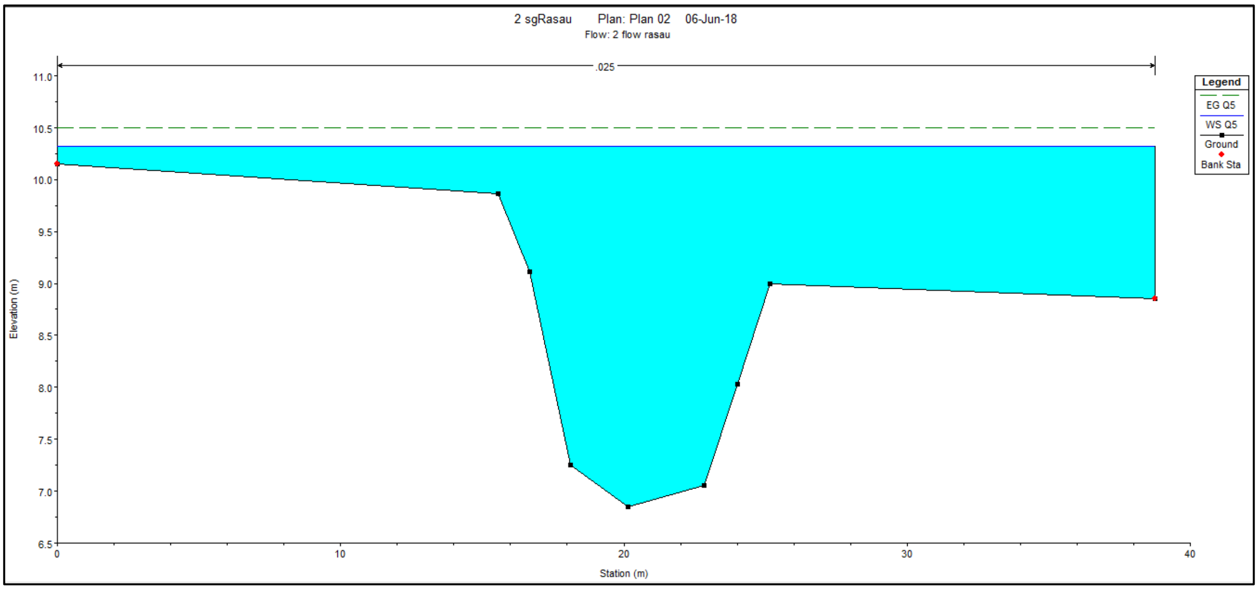


Figure 4.5 Water level at CH20 of Sg Rasau (with bridge piers) – Q_5

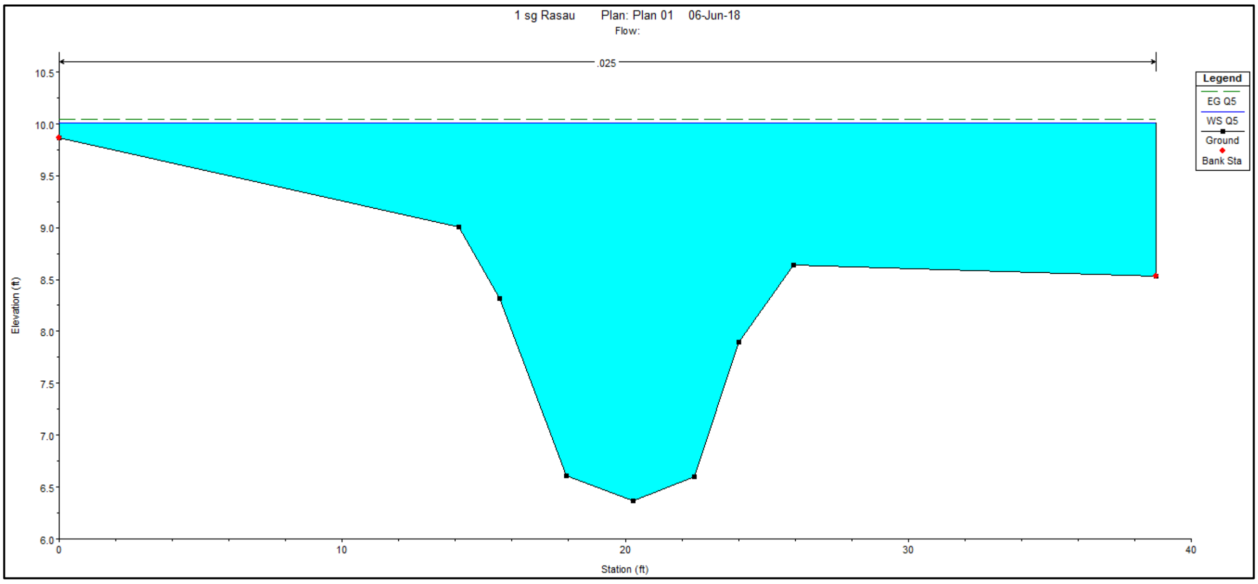


Figure 4.6 Water level at CH40 of Rasau River (without bridge piers) – Q_5

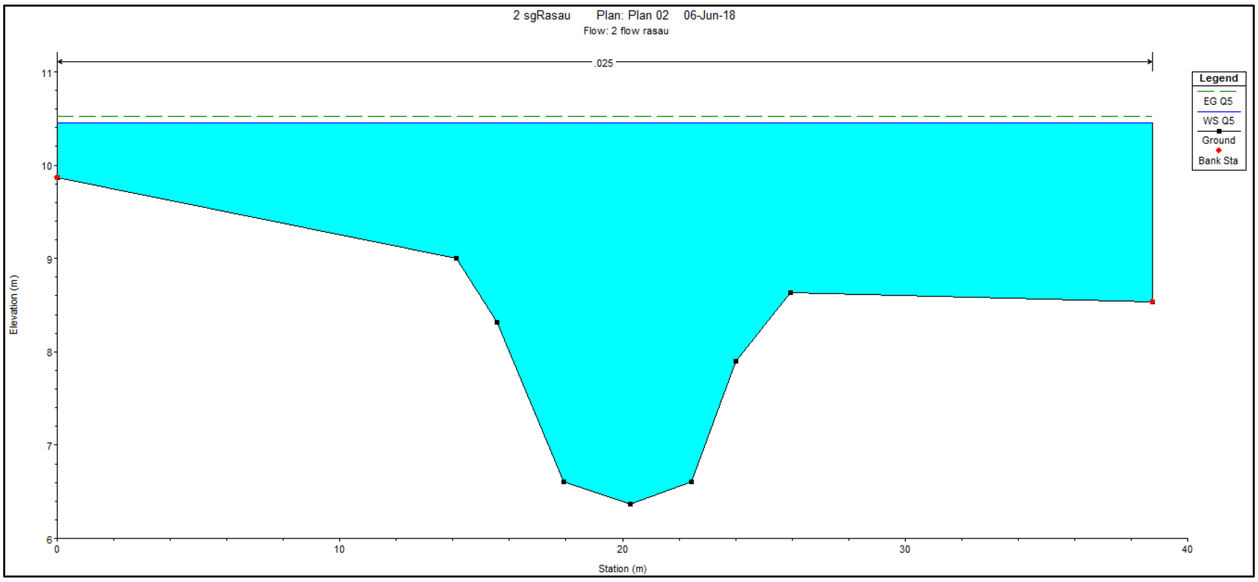


Figure 4.7 Water level at CH 40 of Rasau River (with bridge piers) – Q_5

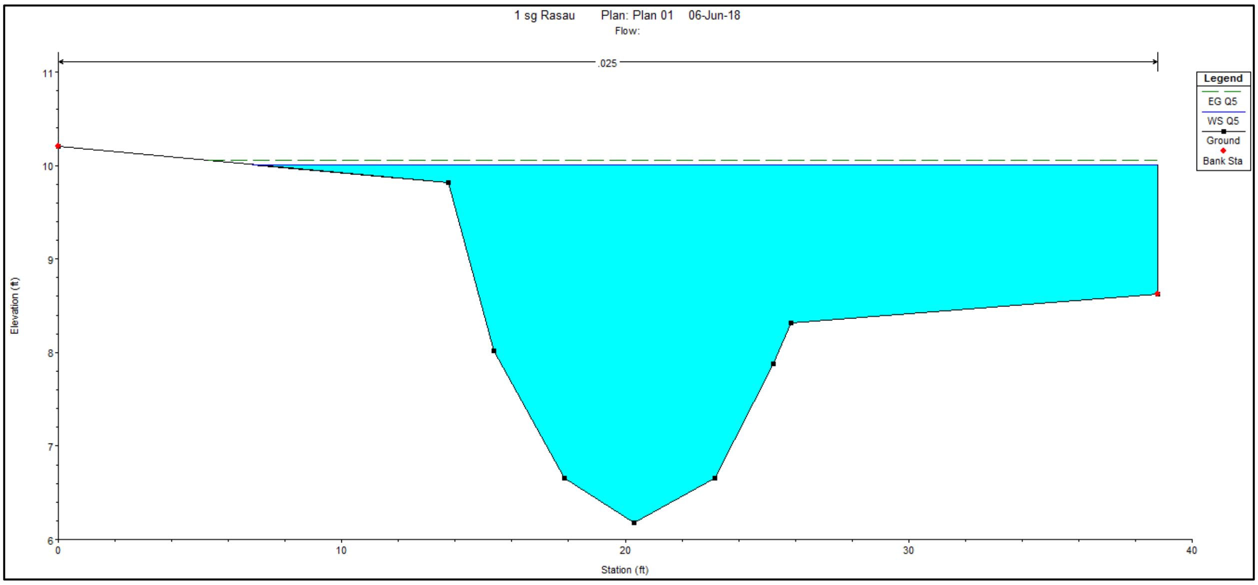


Figure 4.8 Water level at CH60 of Rasau River (without bridge piers) – Q_5

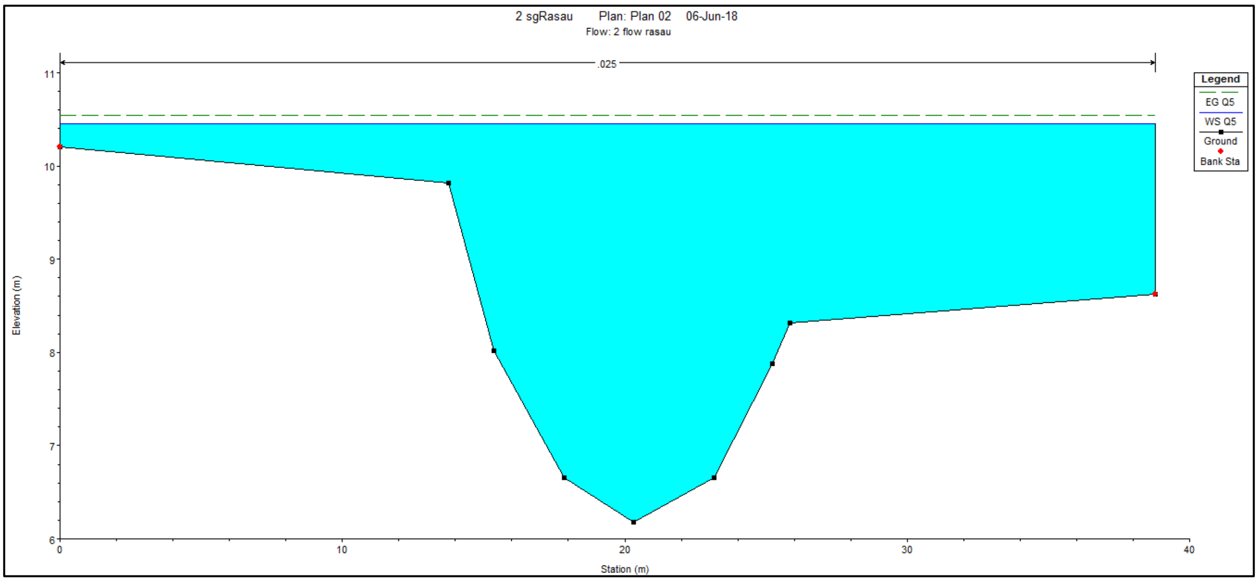


Figure 4.9 Water level at CH60 of Rasau River (with bridge piers) – Q_5

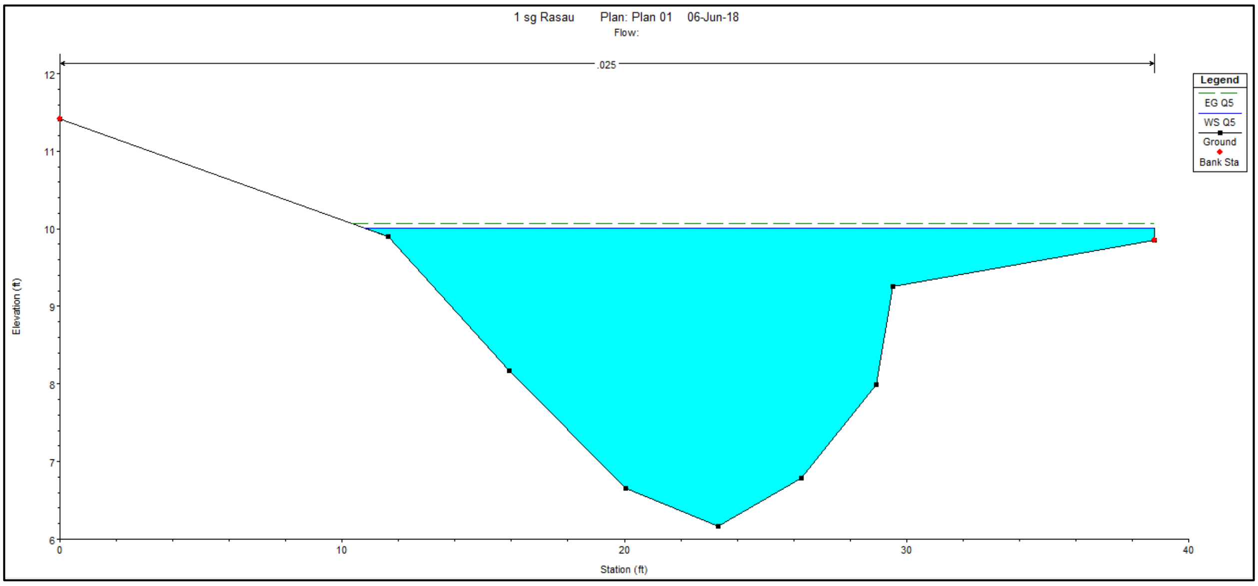


Figure 4.10 Water level at CH80 of Rasau River (without bridge piers) – Q_5

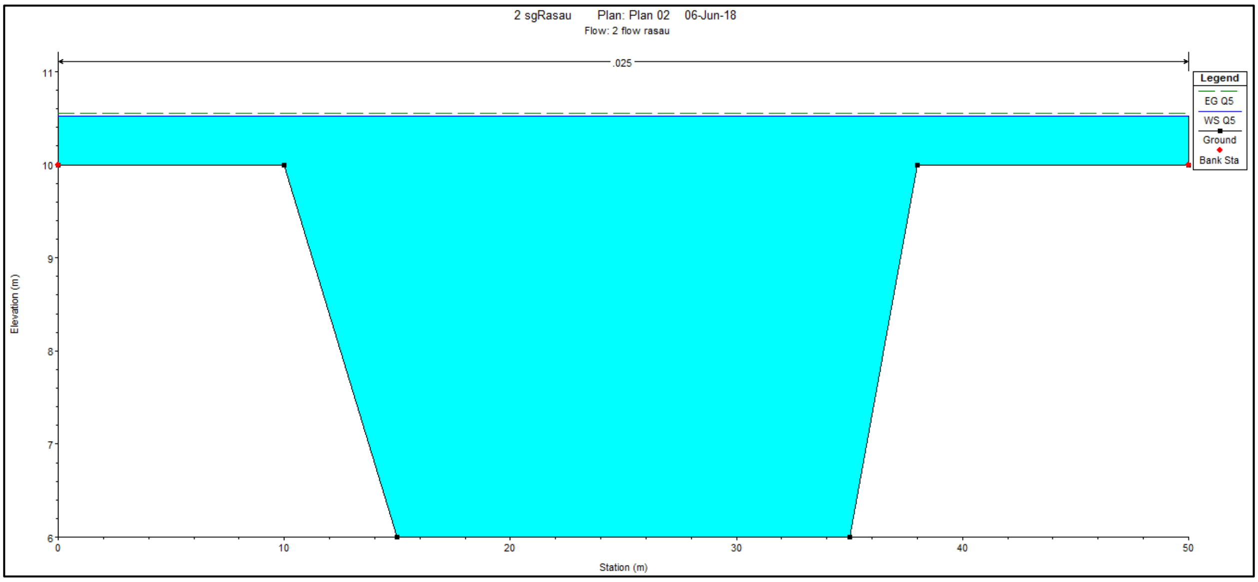


Figure 4.11 Water level at CH80 of Rasau River (with bridge piers) – Q_5

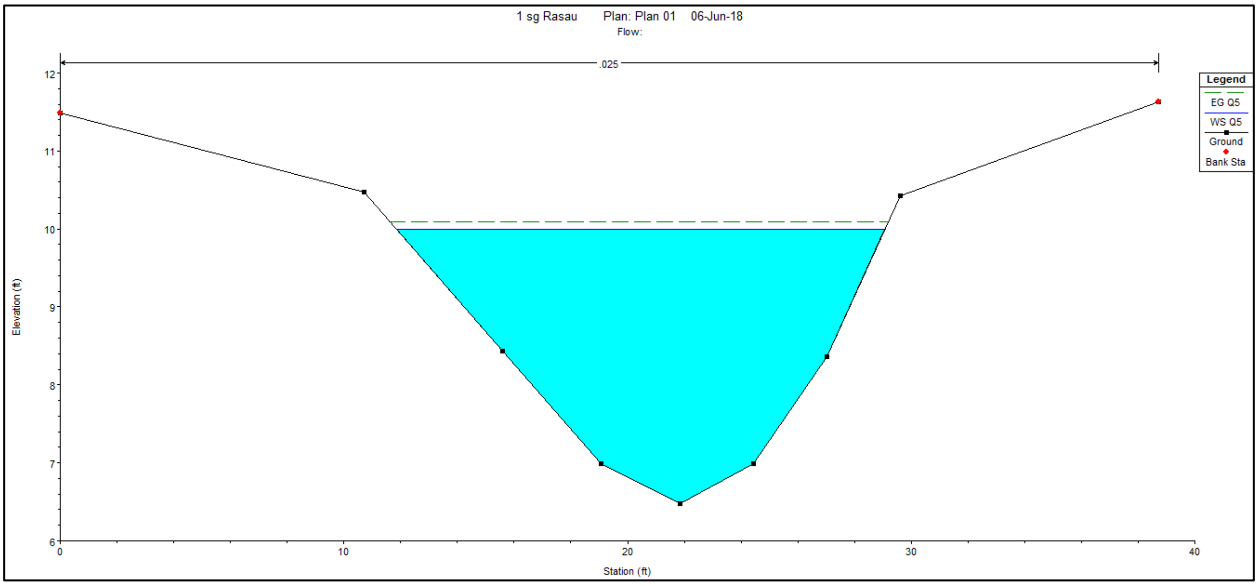


Figure 4.12 Water level at CH100 of Rasau River (without bridge piers) – Q_5

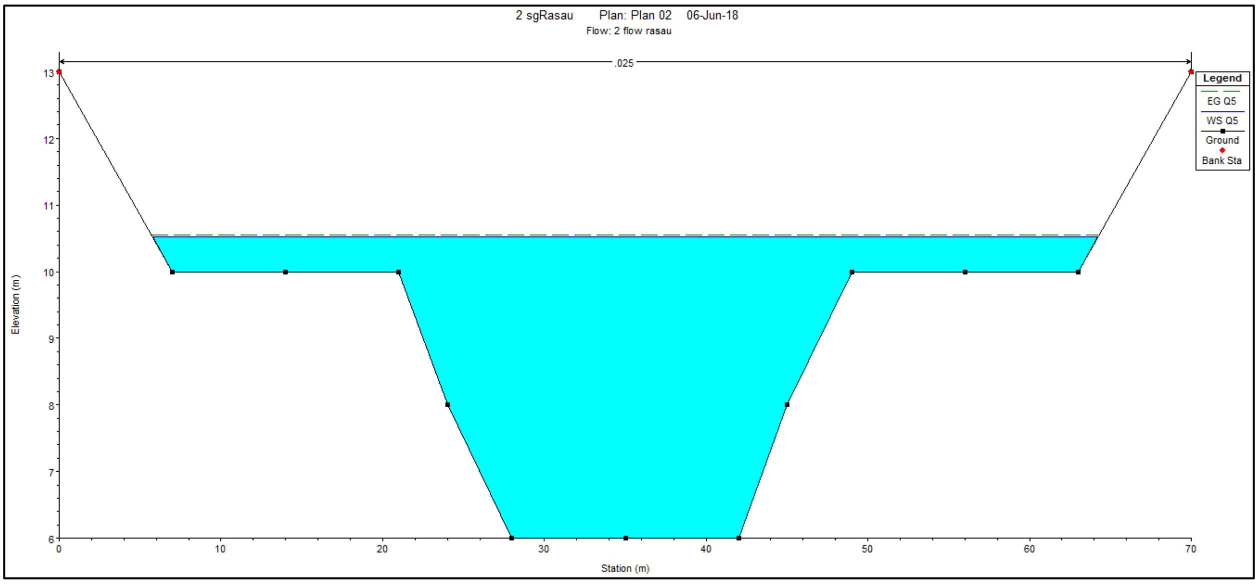


Figure 4.13 Water level at CH100 of Rasau River (with bridge piers) – Q_5

4.5.1.2 Water level for Q_5 under the bridge

The Bridge 3 is located between CH 100 and CH 120. Table 4.3 and Figure 4.14 to Figure 4.17 presented the water profiles at the bridge areas. These simulations showed that the water level at this chainages with 5-year ARI is 10.01m above sea level without the bridge piers presence and 10.52m with the bridge piers presence. Before the bridge construction, the water levels were contained in the river channel without overflowing to the left and right bank. After the bridge construction, the water levels were overflown onto the left and right bank due to changes made along the cross section at CH 100 and CH 120.

Apart from that, during rainfall event of 5-year ARI, the water level under the bridge cross section which starts from CH 100 to CH 120 has the difference of 0.53m. Water levels condition below the bridge can be indicated that the backwater effect has increases the water levels 0.53m more but only at level of 10.52m above sea level which is still not overflow onto the road level which at 15.92m.

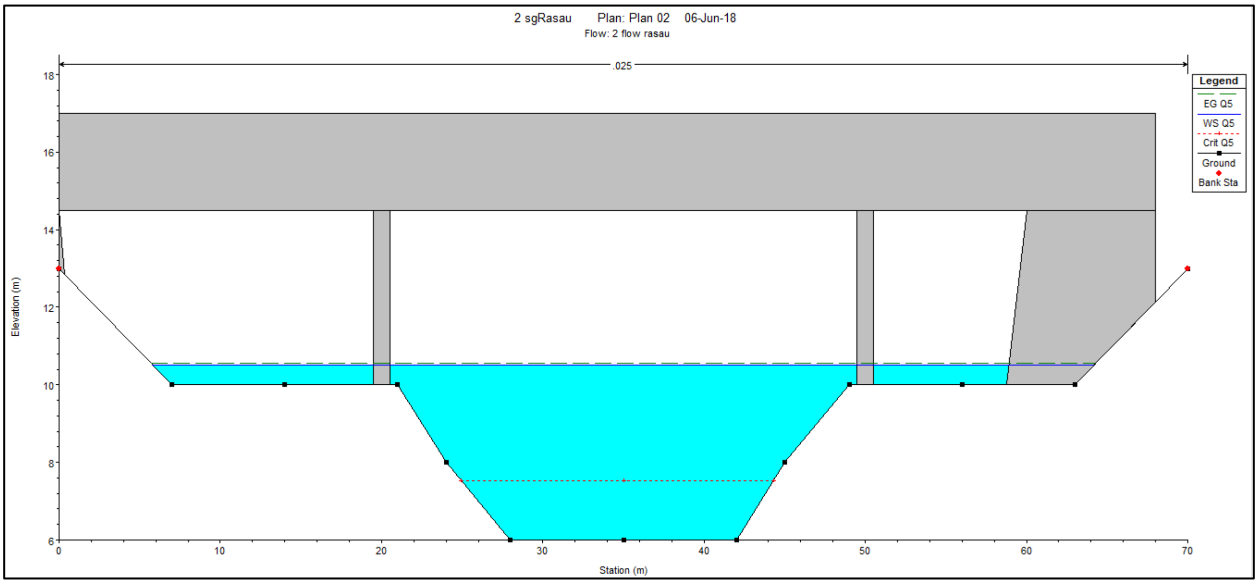


Figure 4.14 Water level at CH110 downstream of Rasau River (with bridge piers) – Q_5

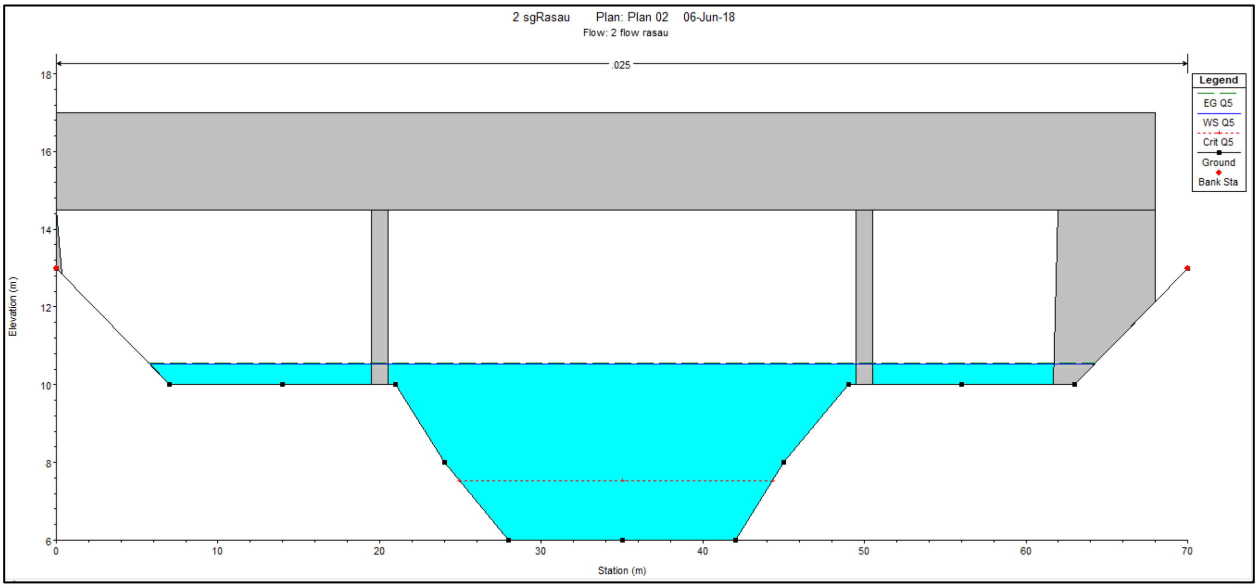


Figure 4.15 Water level at CH110 upstream of Rasau River (with bridge piers) – Q_5

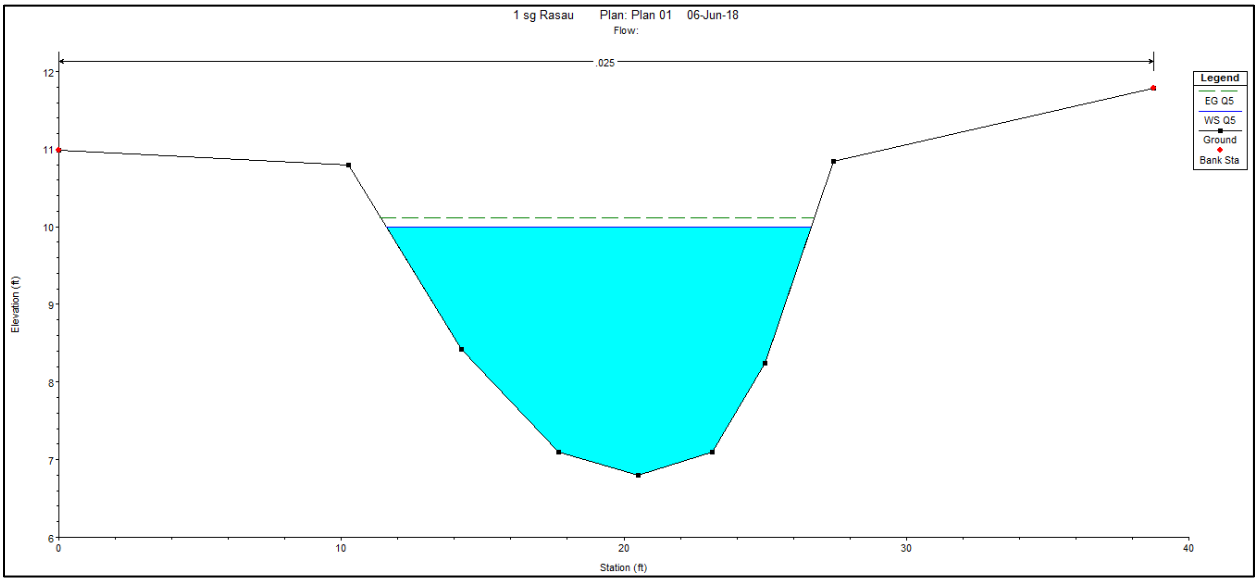


Figure 4.16 Water level at CH120 of Rasau River (without bridge piers) – Q_5

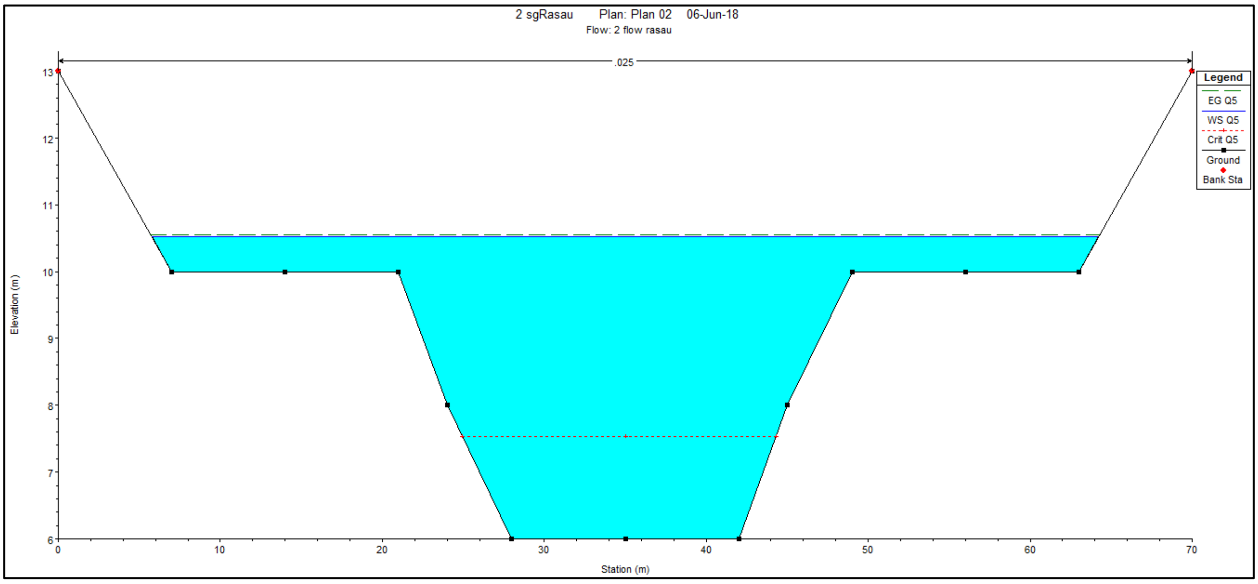


Figure 4.17 Water level at CH120 of Rasau River (with bridge piers) – Q_5

4.5.1.3 Water level for Q_5 at downstream of the bridge

Based on Table 4.3 and Figure 4.18 to Figure 4.30, the water levels were overflown along CH 140 to CH 260 except at CH 140(without bridge piers) and CH 160(without bridge piers) because of its higher level of left banks. Water levels along the downstream were between 10.07m to 10.15m above sea level without bridge piers condition but the water levels were between 10.48m to 10.56m above sea level with bridge piers presence.

Another part of concerned is the water levels due to backwater effect along the downstream cross section of the river. This study shows that the increases in water levels from CH 120 to CH 240 had slightly fluctuated. The water levels are from 0.37m to 0.53m different along the chainage with 5-year ARI condition.

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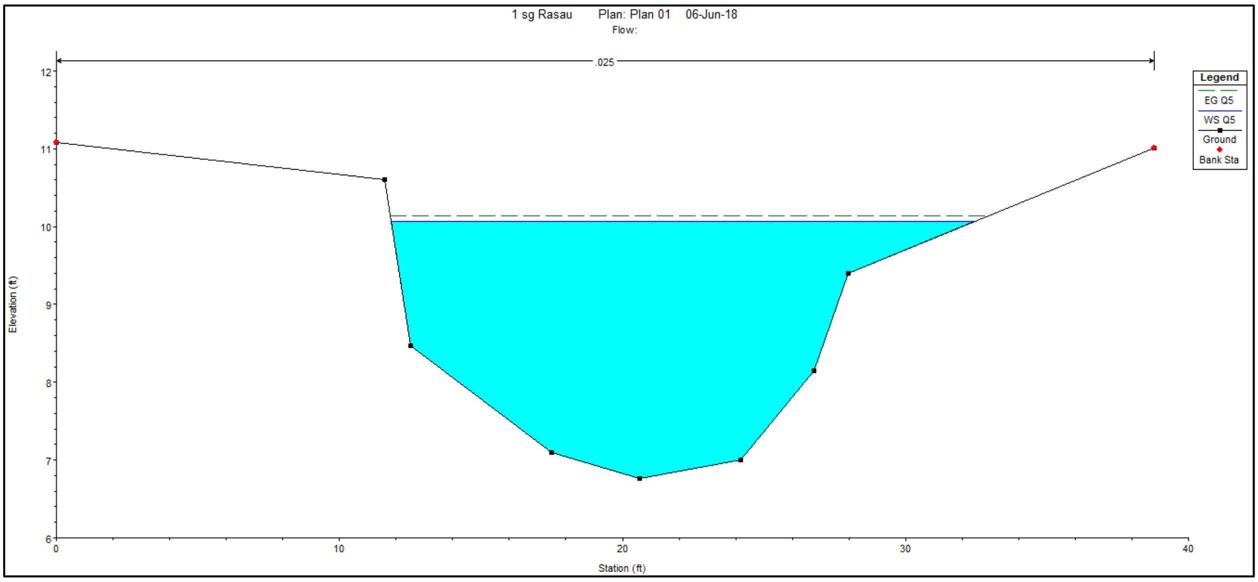

Figure 4.18 Water level at CH140 of Rasau River (without bridge piers) – Q_5

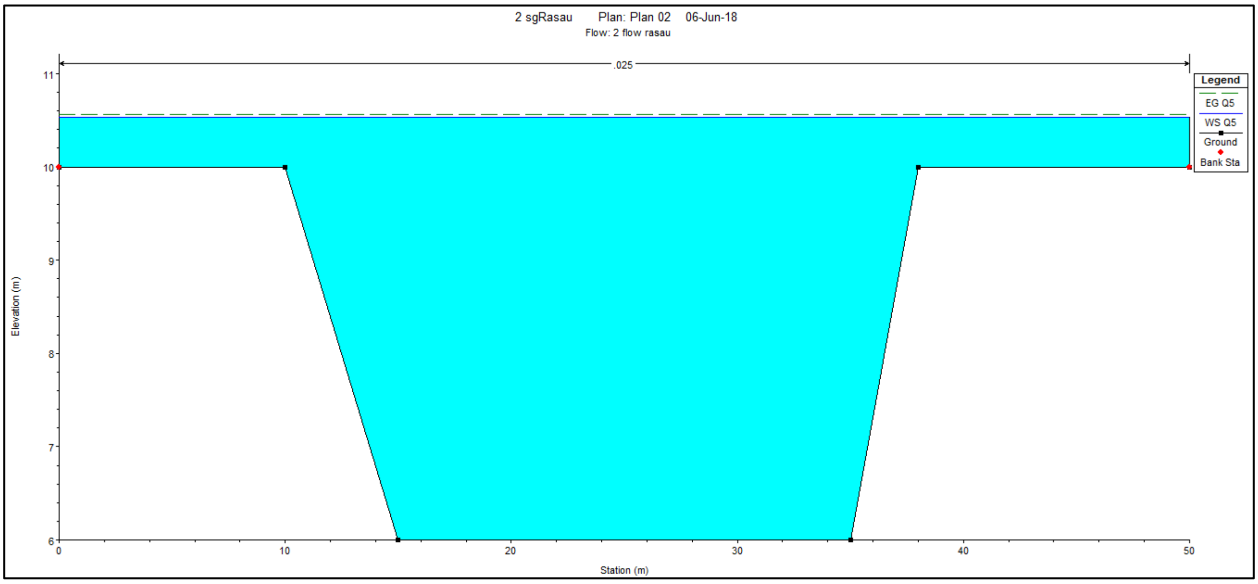

Figure 4.19 Water level at CH140 of Rasau River (with bridge piers) – Q_5

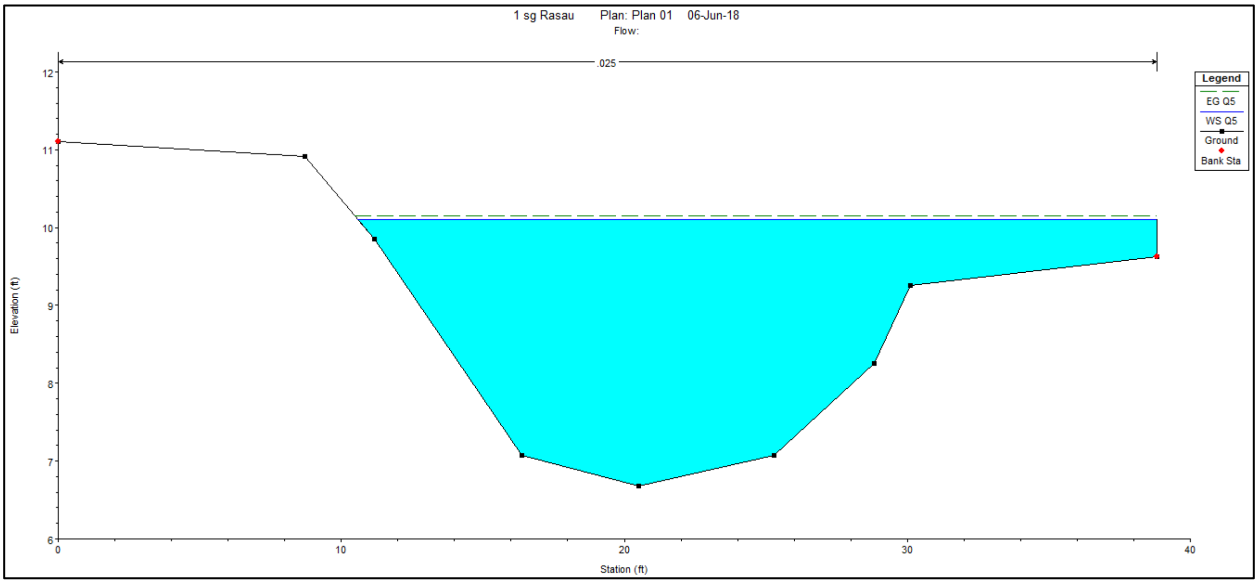

Figure 4.20 Water level at CH160 of Rasau River (without bridge piers) – Q_5

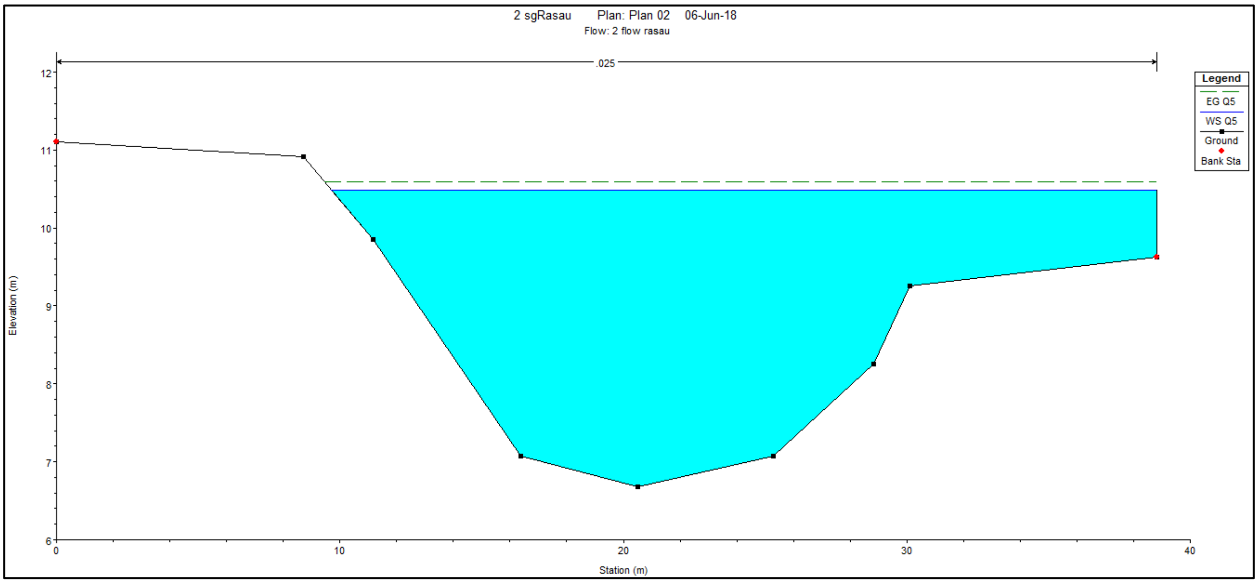

Figure 4.21 Water level at CH160 of Rasau River (with bridge piers) – Q_5

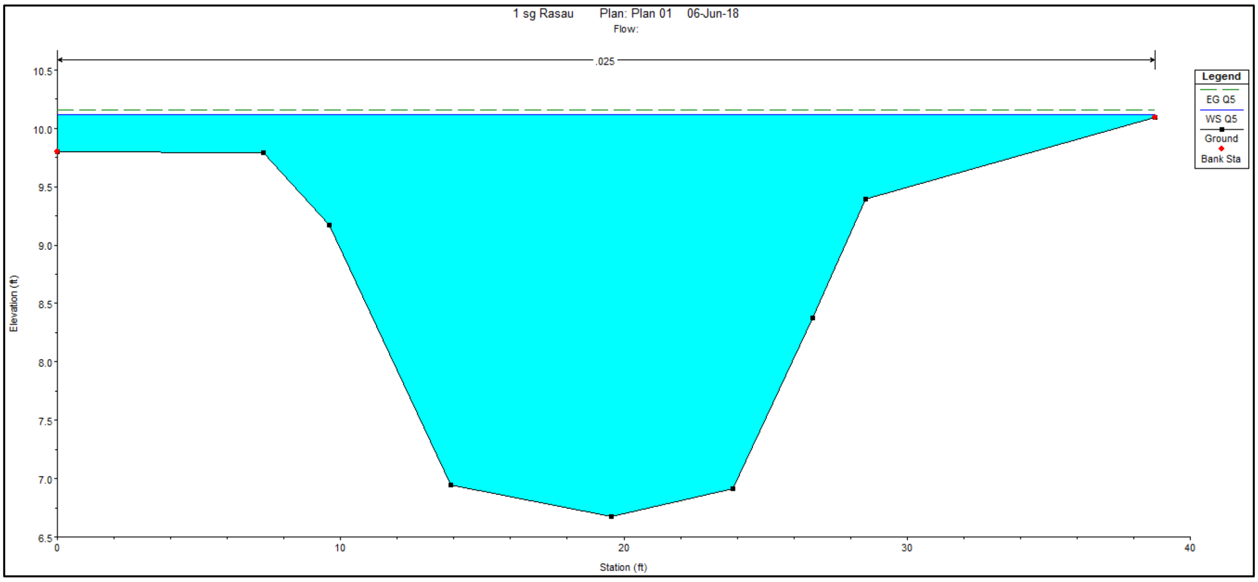

Figure 4.22 Water level at CH180 of Rasau River (without bridge piers) – Q_5

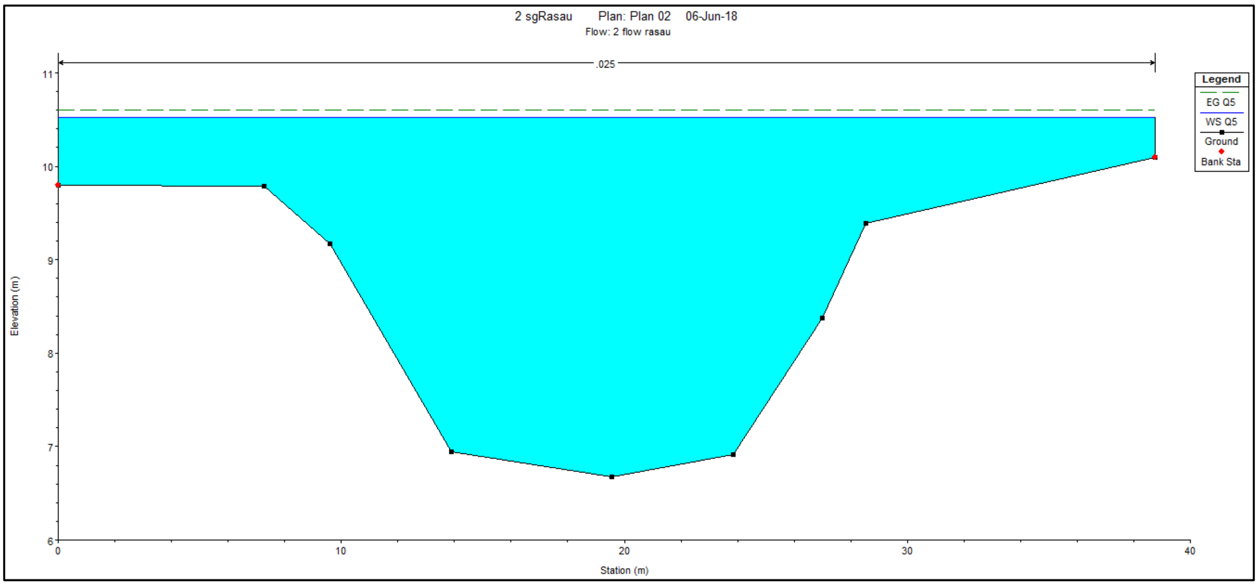

Figure 4.23 Water level at CH180 of Rasau River (with bridge piers) – Q_5

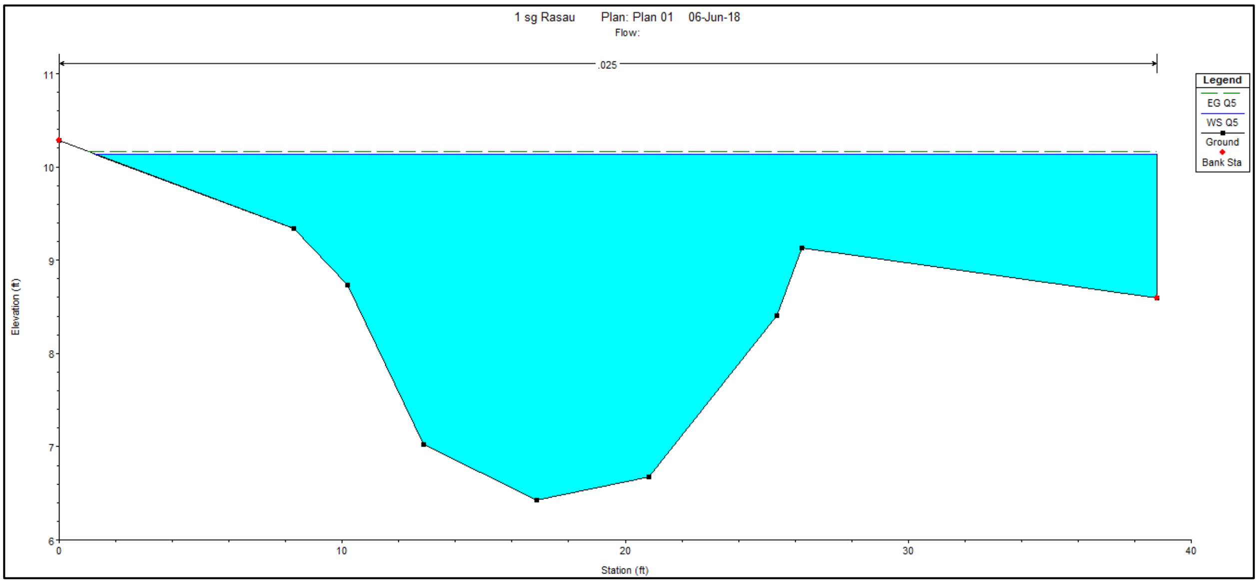

Figure 4.24 Water level at CH200 of Rasau River (without bridge piers) – Q_5

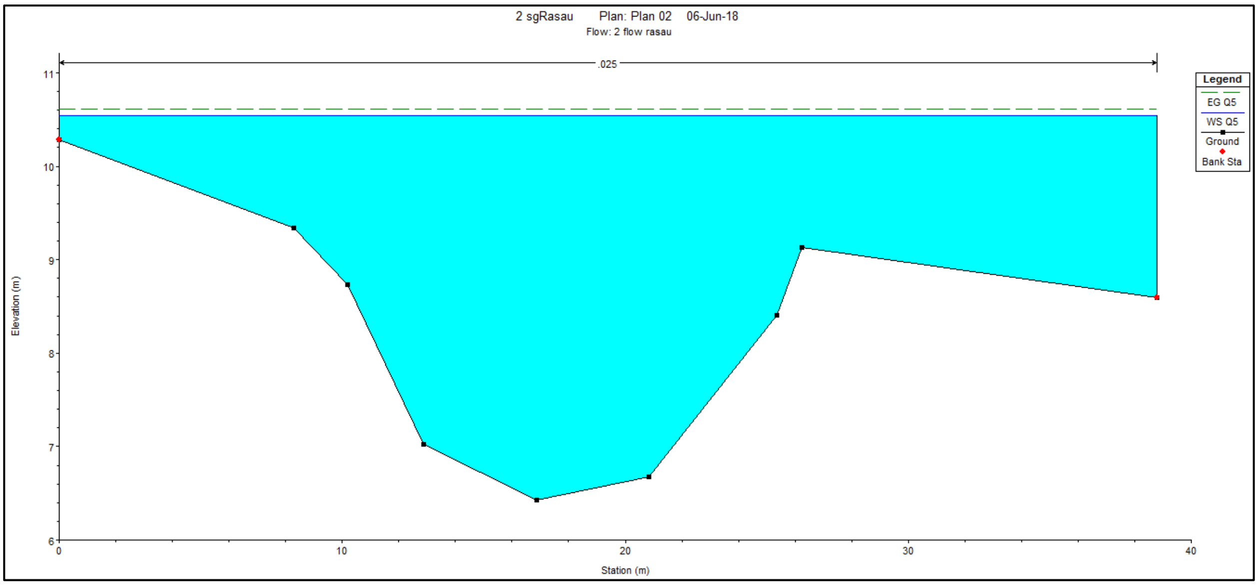

Figure 4.25 Water level at CH200 of Rasau River (with bridge piers) – Q_5

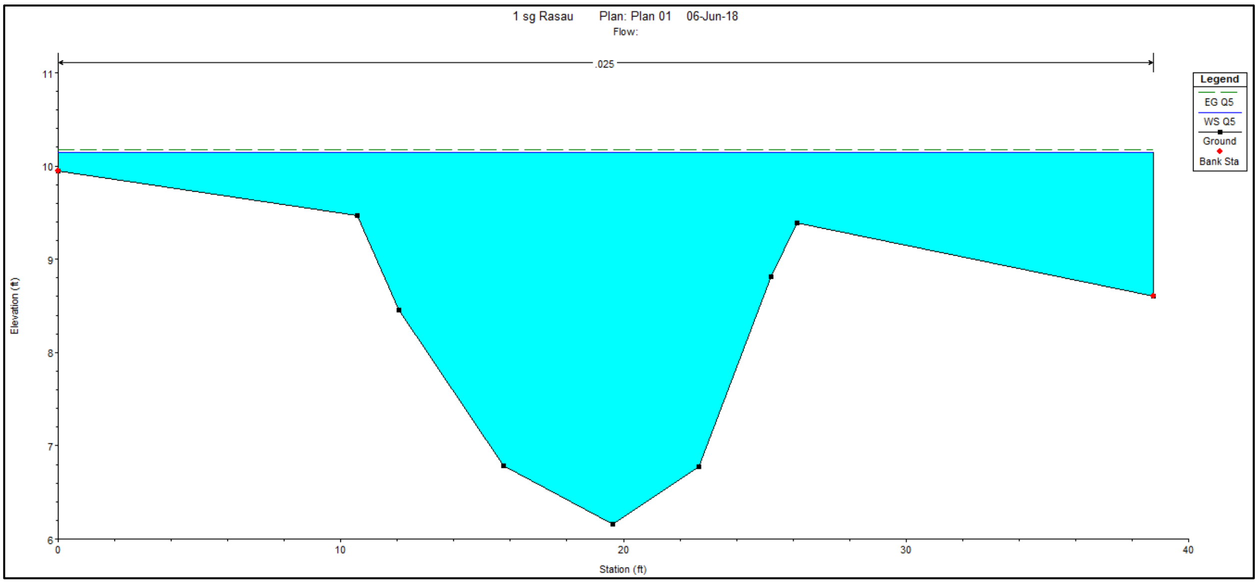

Figure 4.26 Water level at CH220 of Rasau River (without bridge piers) – Q_5

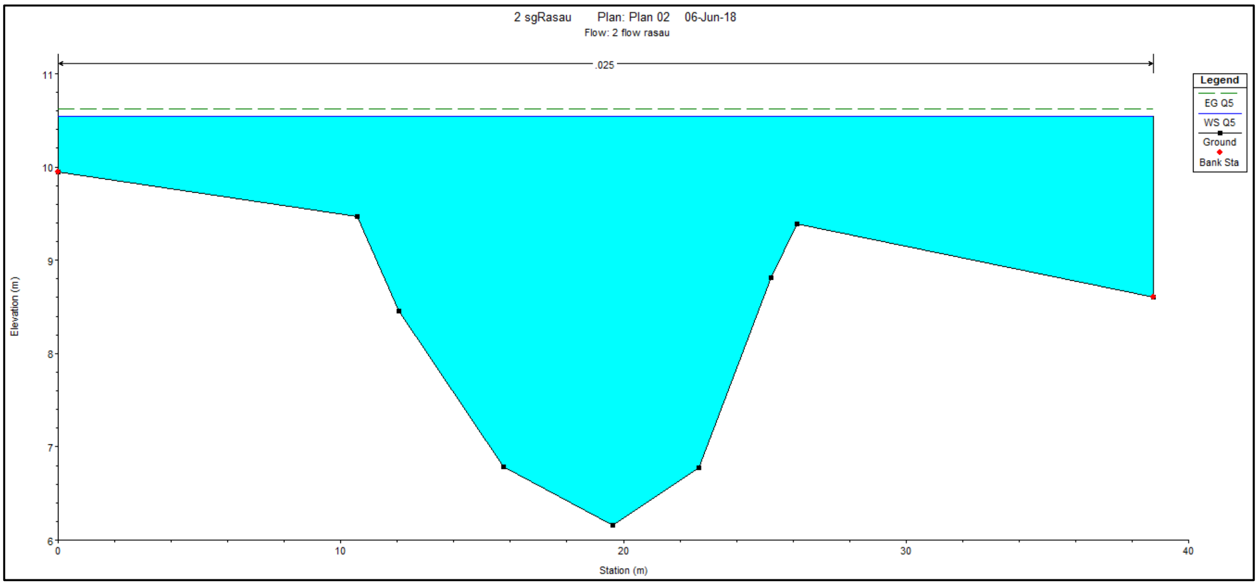

Figure 4.27 Water level at CH220 of Rasau River (with bridge piers) – Q_5

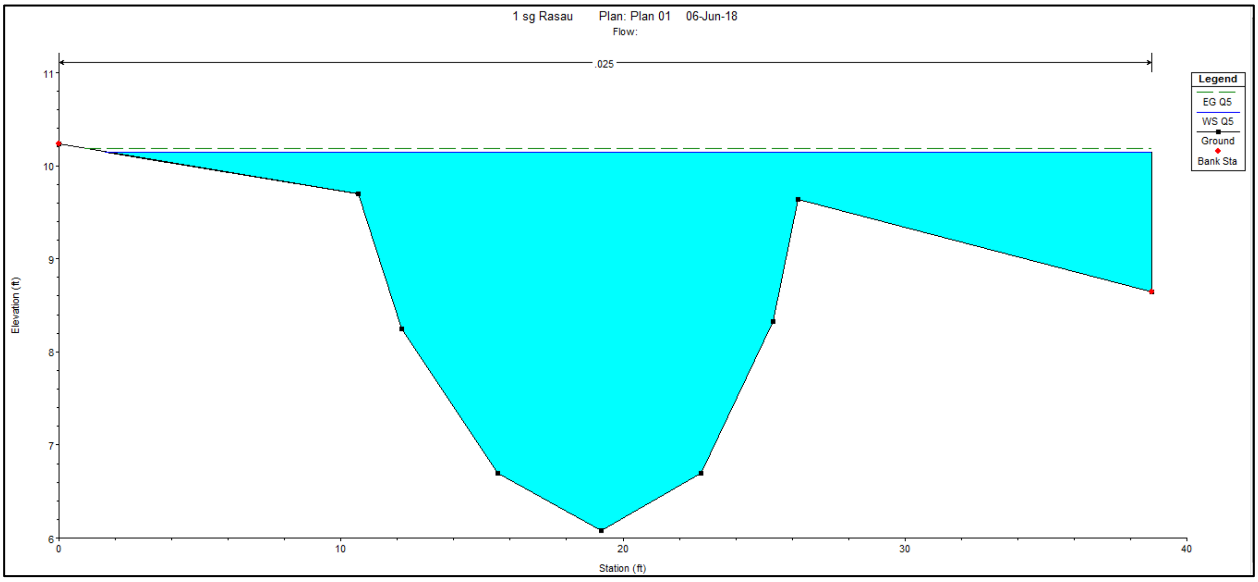

Figure 4.28 Water level at CH240 of Rasau River (without bridge piers) – Q_5

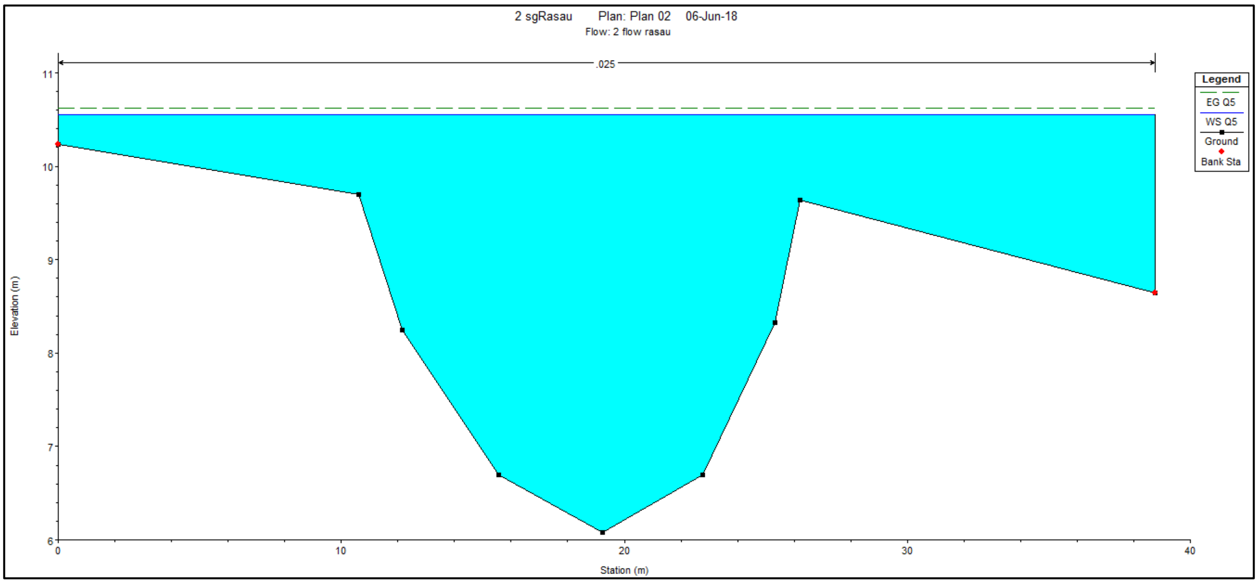

Figure 4.29 Water level at CH240 of Rasau River (with bridge piers) – Q_5

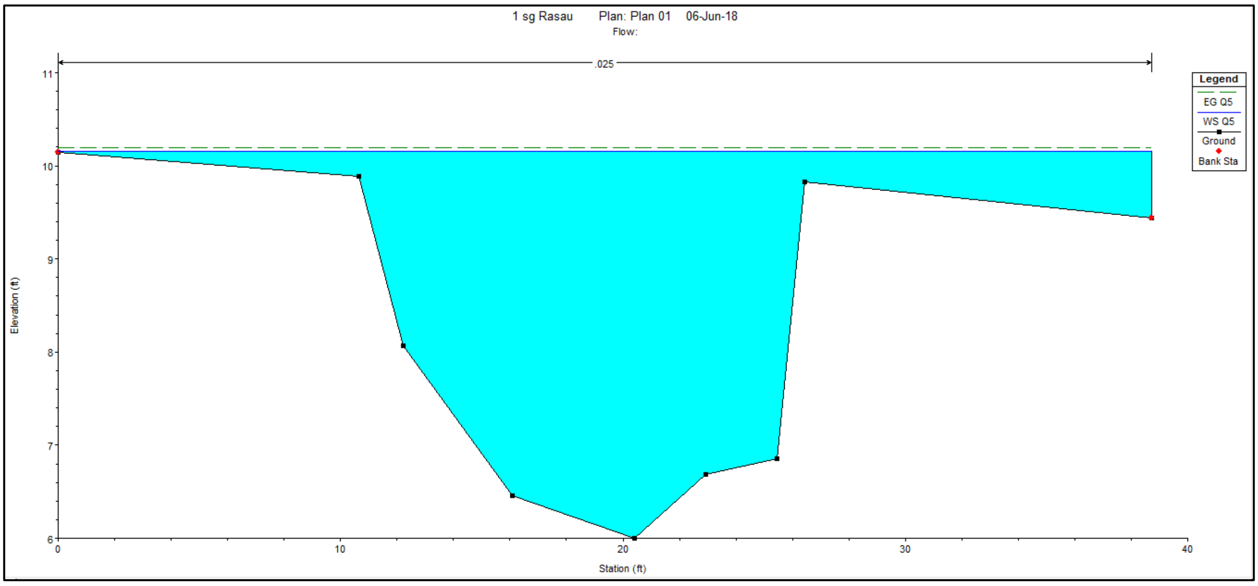

Figure 4.30 Water level at CH260 of Rasau River (without bridge piers) – Q_5

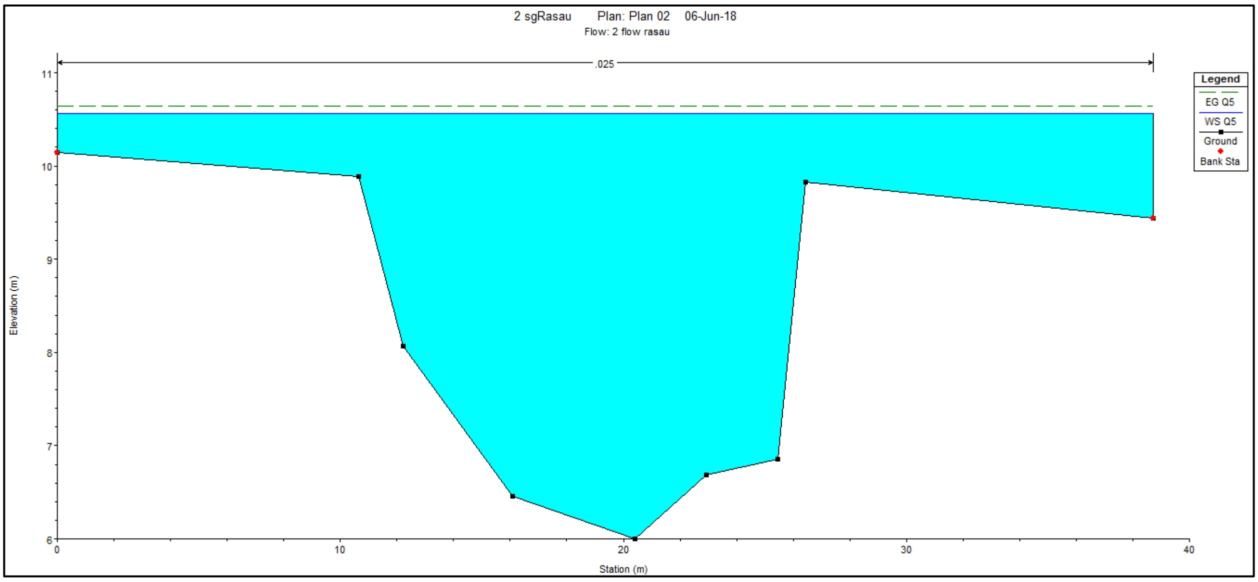

Figure 4.31 Water level at CH260 of Rasau River (with bridge piers) – Q_5

4.5.2 Water level for Q_{10} with and without bridge piers

The difference in water levels of the Rasau River along the chainage with 10-year ARI is tabulated in Table 4.4 and water level profiles together with backwater effect were discussed.

Table 4.4 Difference of water level with Q_{10}

Q_{10} = 106.3 m³/s						
Chainage	**Before bridge construction**		**After bridge construction**		**Difference of water level (m)**	
	Water level (m)		**Water level (m)**			
	Left bank	**Right bank**	**Left bank**	**Right bank**	**Left bank**	**Right bank**
CH 0	9.66	9.66	10.05	10.05	0.39	0.39
CH 20	10.01	10.01	10.43	10.43	0.42	0.42
CH 40	10.12	10.12	10.57	10.57	0.45	0.45
CH 60	10.12	10.12	10.58	10.58	0.46	0.46
CH 80	10.13	10.13	10.65	10.65	0.52	0.52
CH 100	10.10	10.10	10.65	10.65	0.55	0.55
CH 110 U	10.10	10.10	10.65	10.65	0.55	0.55
CH 110 D	10.10	10.10	10.66	10.66	0.56	0.56
CH 120	10.10	10.10	10.66	10.66	0.56	0.56
CH 140	10.19	10.19	10.67	10.67	0.48	0.48
CH 160	10.24	10.24	10.61	10.61	0.37	0.37
CH 180	10.26	10.26	10.66	10.66	0.40	0.40
CH 200	10.27	10.27	10.68	10.68	0.41	0.41
CH 220	10.28	10.28	10.68	10.68	0.40	0.40
CH 220	10.28	10.28	10.69	10.69	0.41	0.41
CH 240	10.29	10.29	10.70	10.70	0.41	0.41

4.5.2.1 Water level for Q_{10} at upstream of the bridge

With reference to the Table 4.4 and Figure 4.32 to Figure 4.43, it can be said that the water levels at the upstream of the river were between 9.66m to 10.13m above sea level without the bridge piers presence and between 10.05m to 10.65m above sea level with the bridge piers presence. This shows that, during rainfall event of 10-year ARI, the water levels were surpassed the river bank along CH 0 to CH 100.

Next, the differences in water levels along the upstream chainage were between 0.39m to 0.55m. This indicated that the upstream channel which starts from CH 0 to CH 100 has affected the increases in water levels up to 0.55m at CH 100 where the presence of the Bridge 3 is located.

Apart from that, without the bridge piers presence the water levels had overflown onto the left and right bank higher than normal condition at the same chainage. This situation shows that even though along the studied cross section should be already flooded with water as it without bridge piers presence, but the level still at the safest level if compared with the water levels at each cross section with the presence of bridge piers.

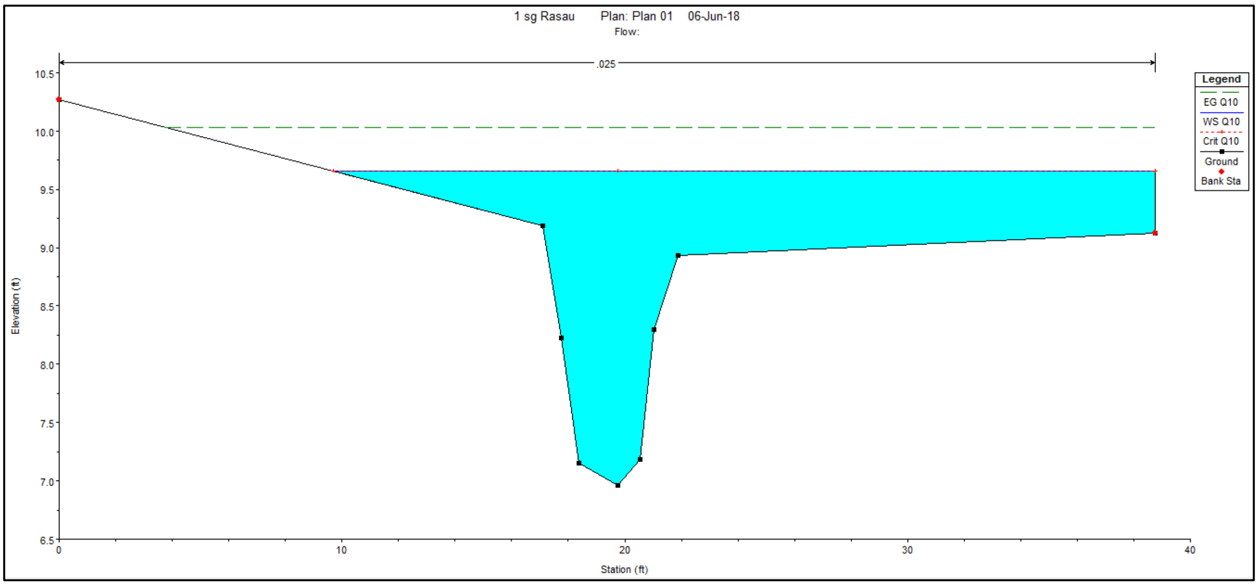

Figure 4.32 Water level at CH0 of the Rasau River (without bridge piers) – Q_{10}

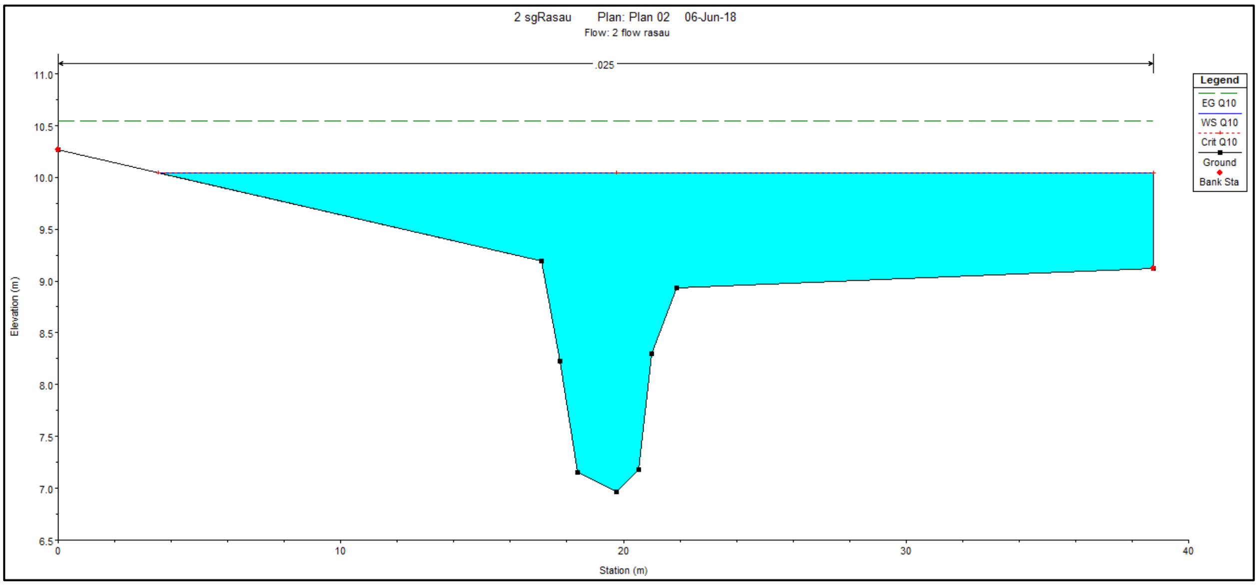

Figure 4.33 Water level at CH0 of the Rasau River (with bridge piers) – Q_{10}

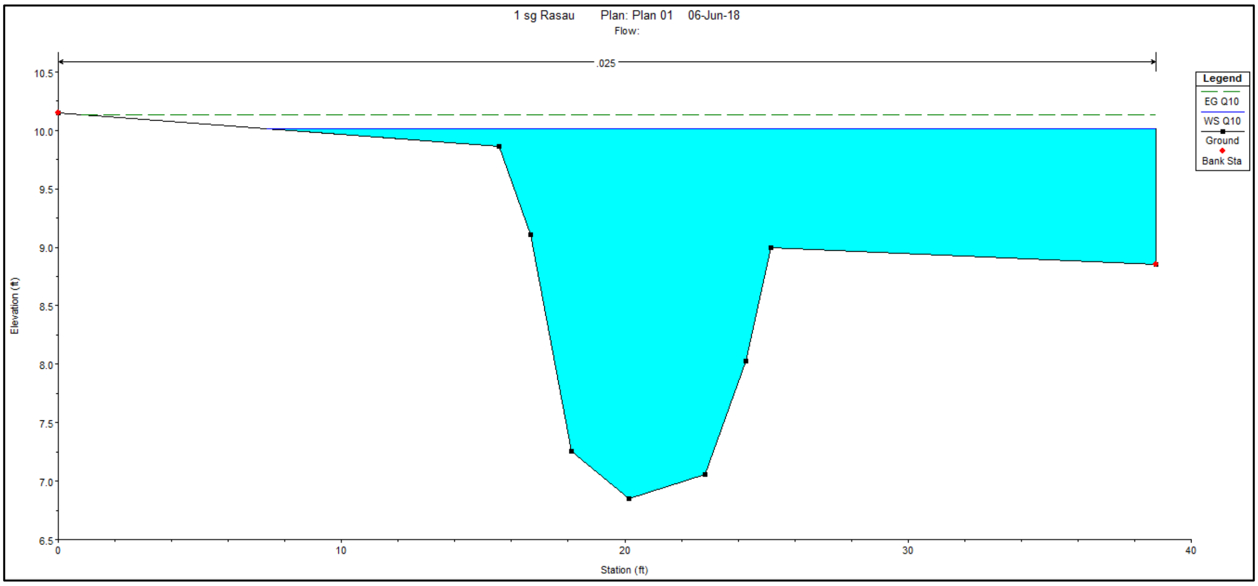

Figure 4.34 Water level at CH20 of Rasau River (without bridge piers) – Q_{10}

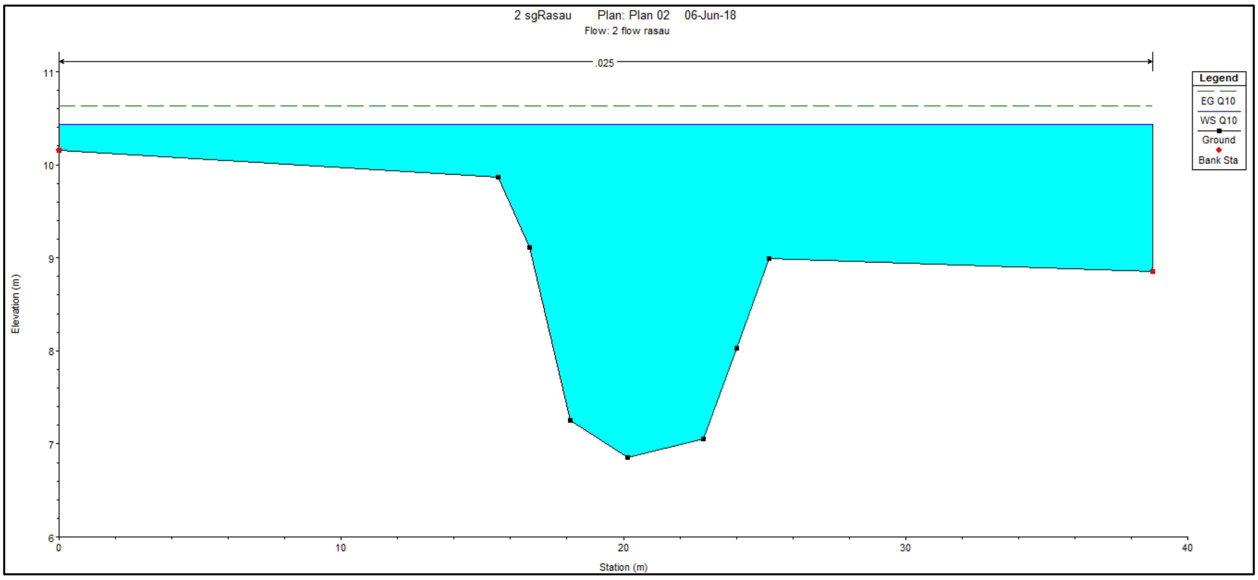

Figure 4.35 Water level at CH20 of Sg Rasau (with bridge piers) – Q_{10}

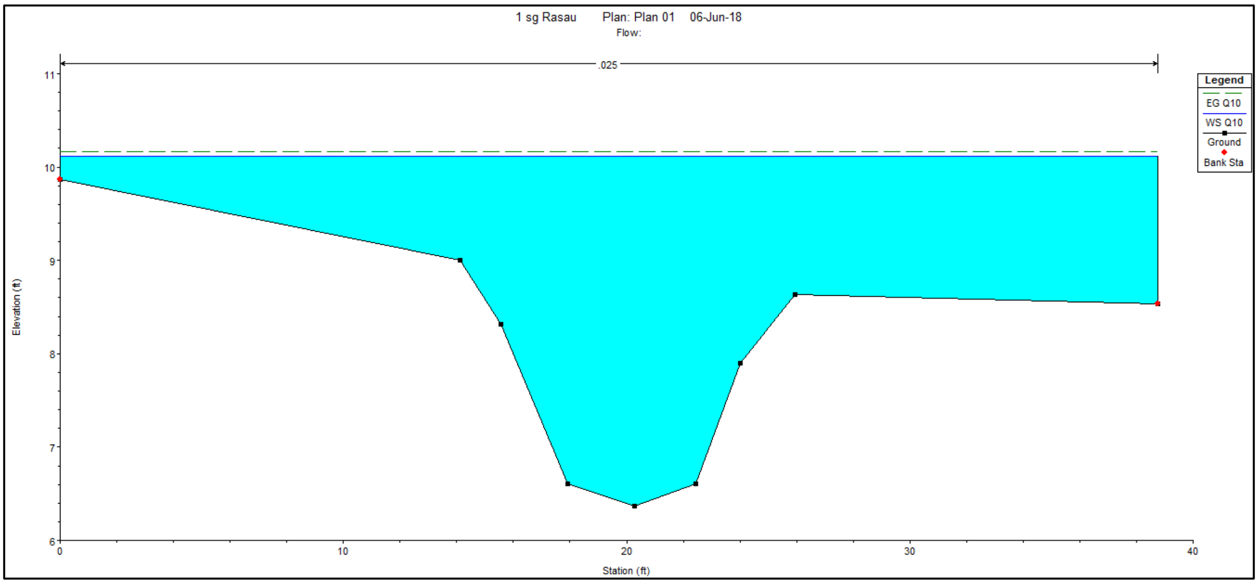

Figure 4.36 Water level at CH40 of Rasau River (without bridge piers) – Q_{10}

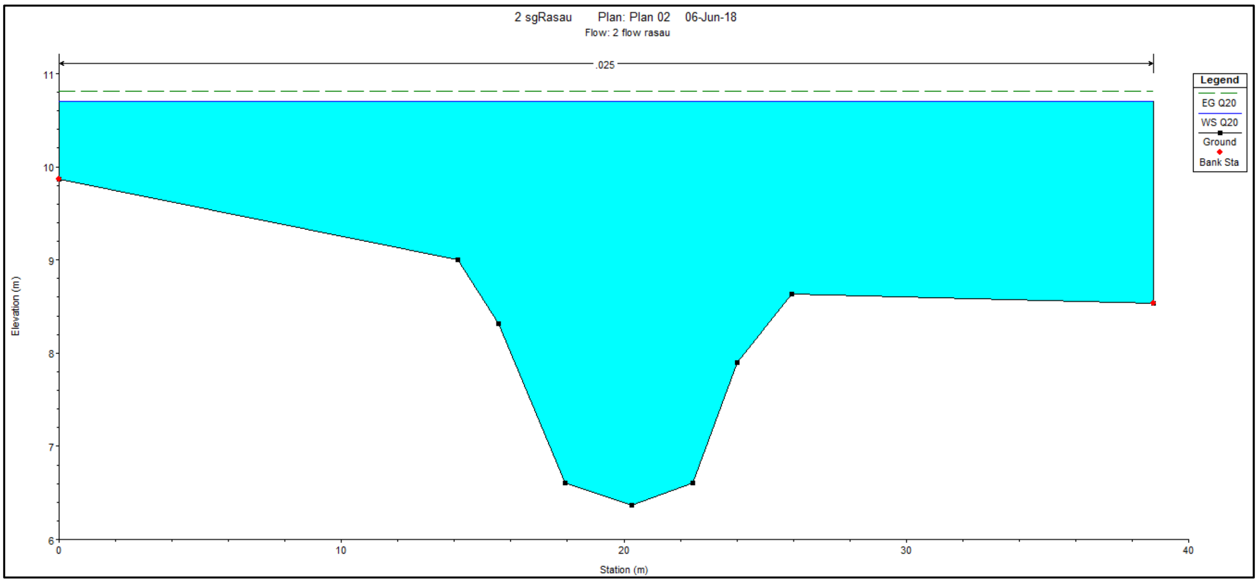

Figure 4.37 Water level at CH 40 of Rasau River (with bridge piers) – Q_{10}

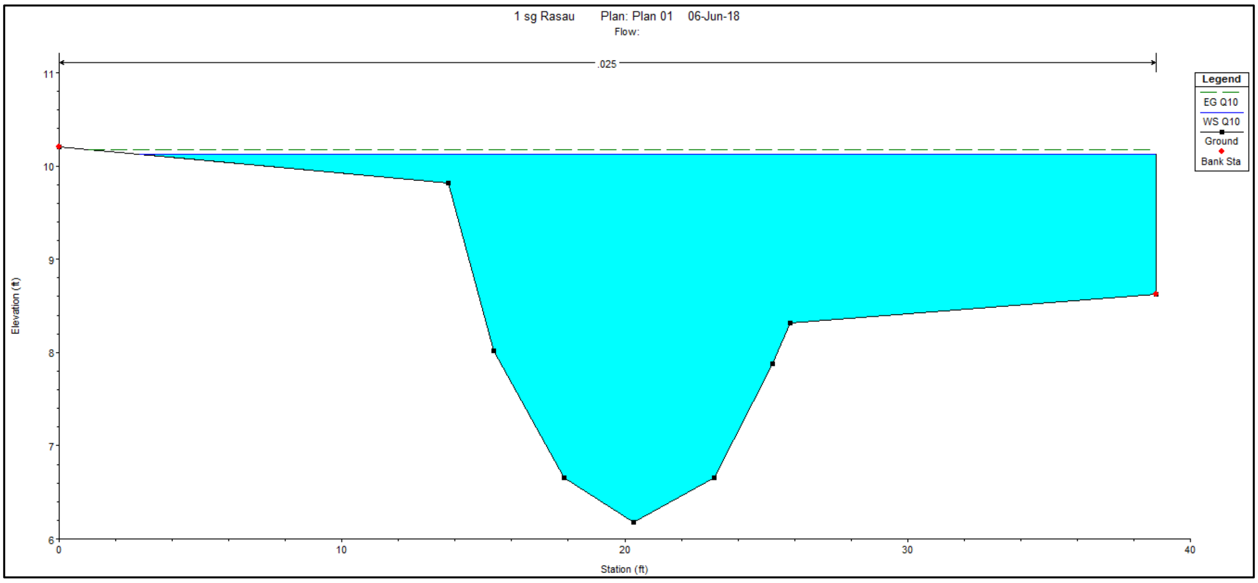

Figure 4.38 Water level at CH60 of Rasau River (without bridge piers) – Q_{10}

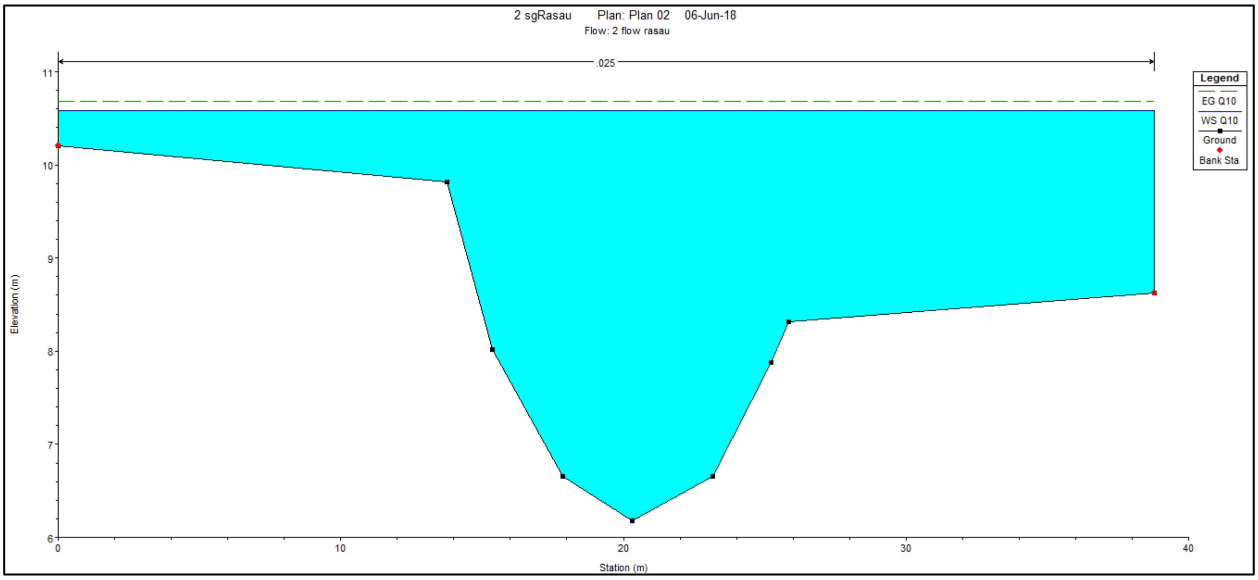

Figure 4.39 Water level at CH60 of Rasau River (with bridge piers) – Q_{10}

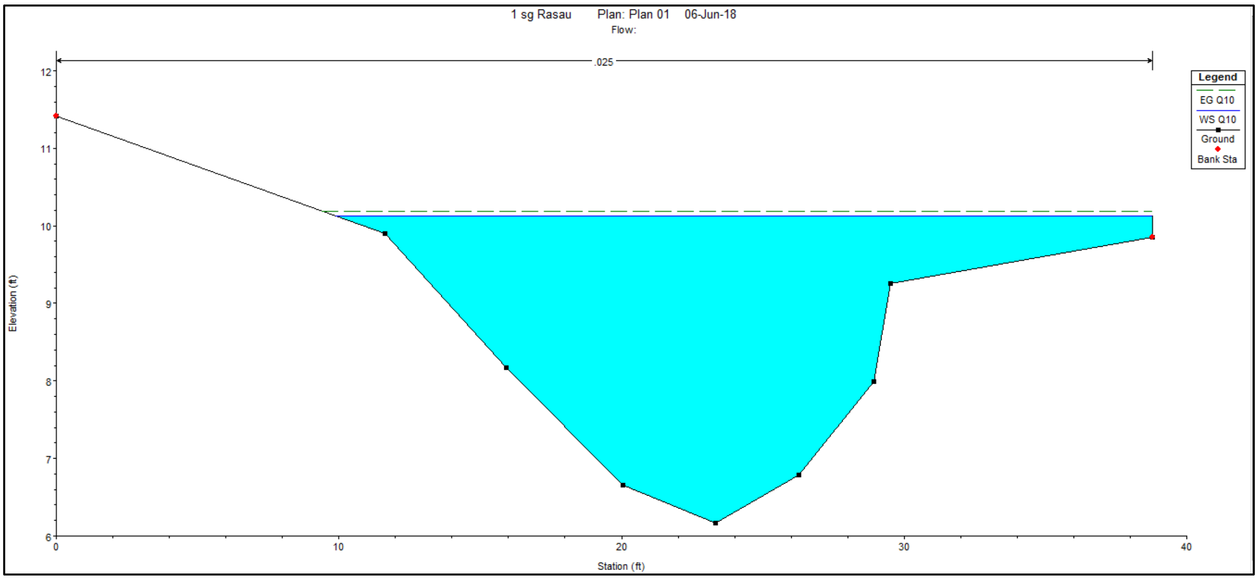

Figure 4.40 Water level at CH80 of Rasau River (without bridge piers) – Q_{10}

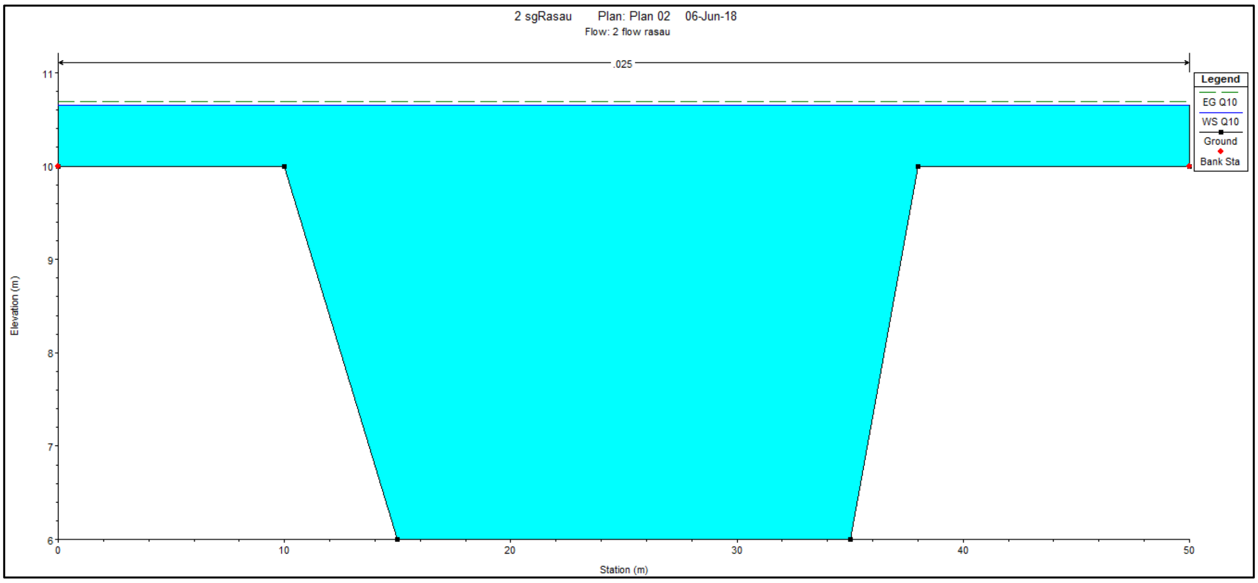

Figure 4.41 Water level at CH80 of Rasau River (with bridge piers) – Q_{10}

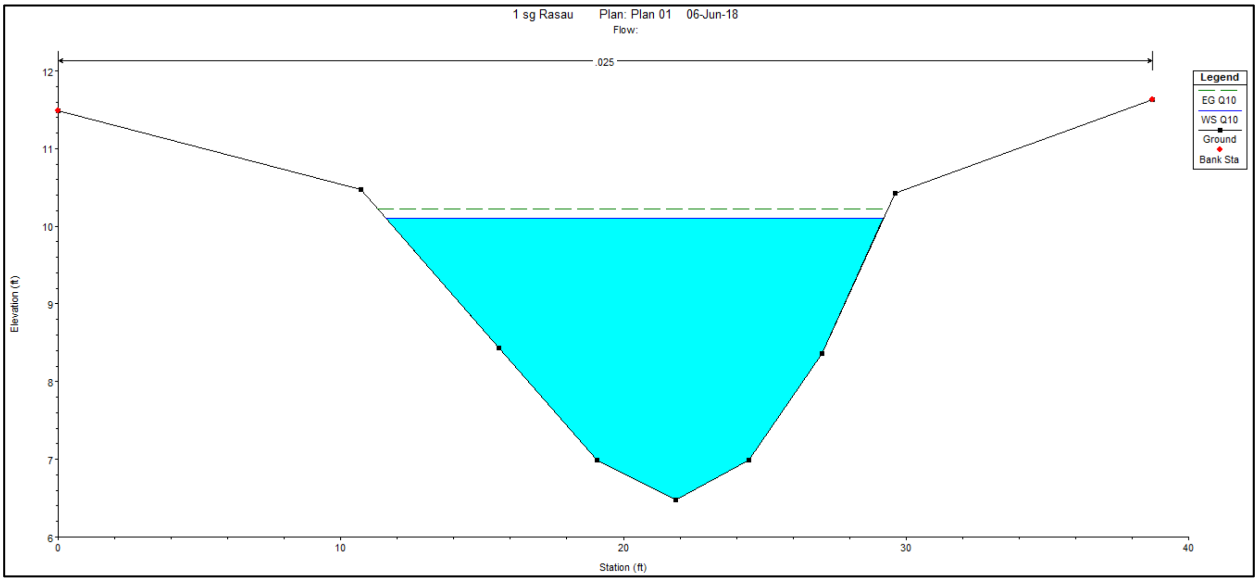

Figure 4.42 Water level at CH100 of Rasau River (without bridge piers) – Q_{10}

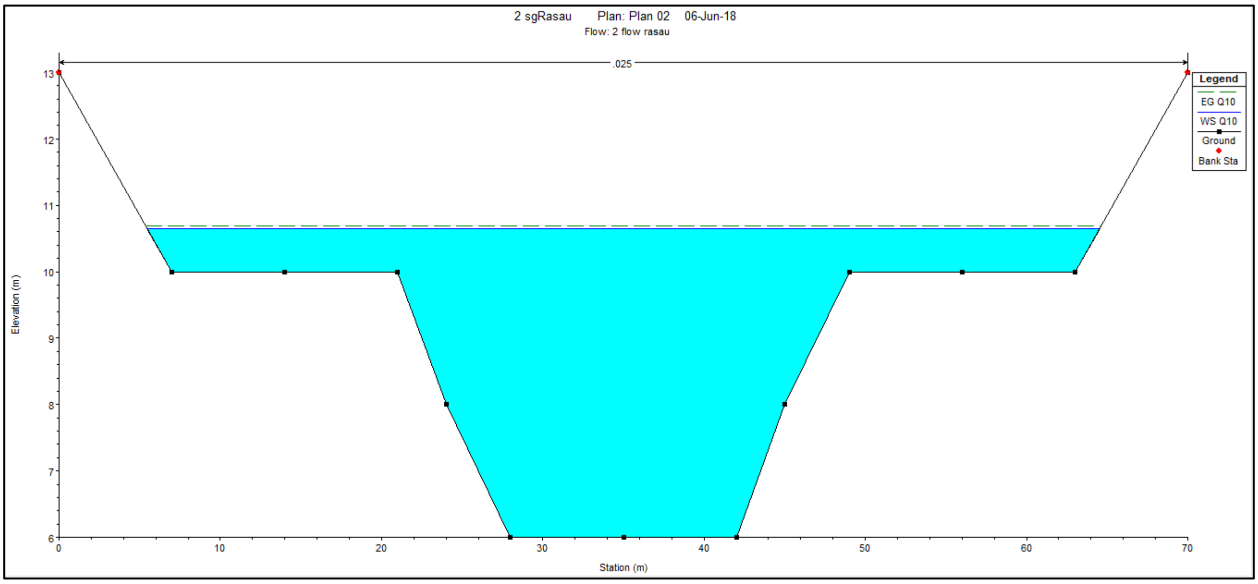

Figure 4.43 Water level at CH100 of Rasau River (with bridge piers) – Q_{10}

4.5.2.2 Water level for Q_{10} under the bridge

With reference to the Table 4.4 and Figure 4.44 to Figure 4.47, these simulations showed that the water levels at chainages where the bridge will be located were 10.10m above sea level and as the bridge is constructed, the water levels were between 10.65m to 10.66m above sea level. The water levels were contained in the river channel without overflown to the left and right bank as without the bridge but it were overflown to the left and right bank under the bridge cross section with bridge piers presence.

Next, during rainfall event of 10-year ARI, the water levels under the bridge cross section which starts from CH 100 to CH 120 showed that the difference in water levels between the two conditions were between 0.55m to 0.56m. Water level condition below the bridge can be indicated that the backwater effect increased the water levels up to 0.56m but only at level of 10.66m above sea level. This means that the water would not overflow onto the road level which at 15.92m.

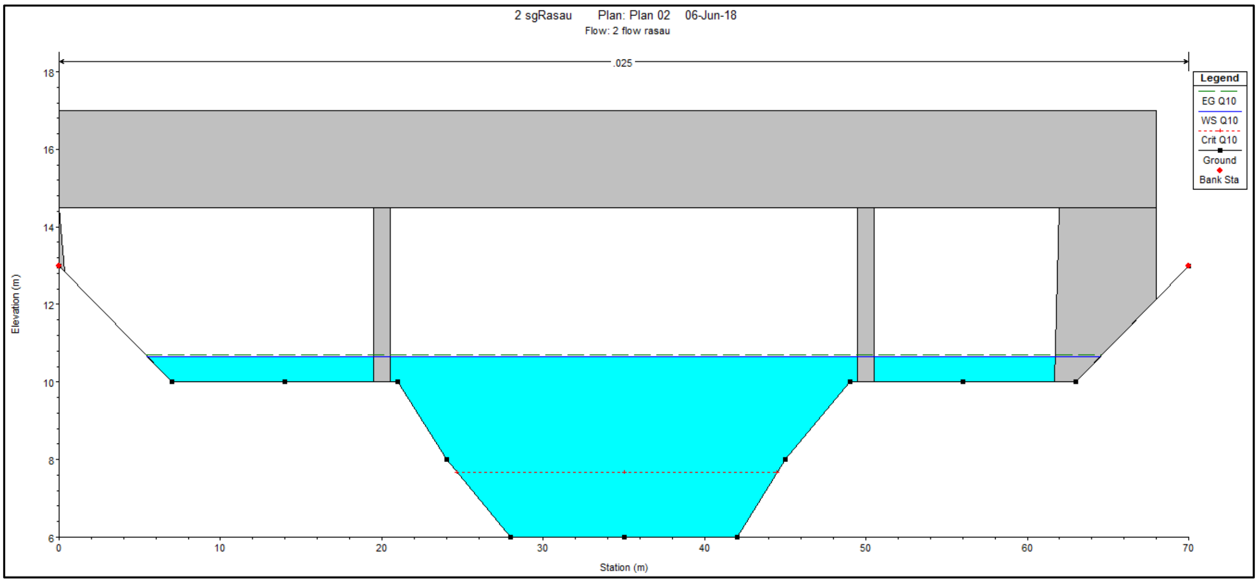

Figure 4.44 Water level at CH110 downstream of Rasau River (with bridge piers) – Q_{10}

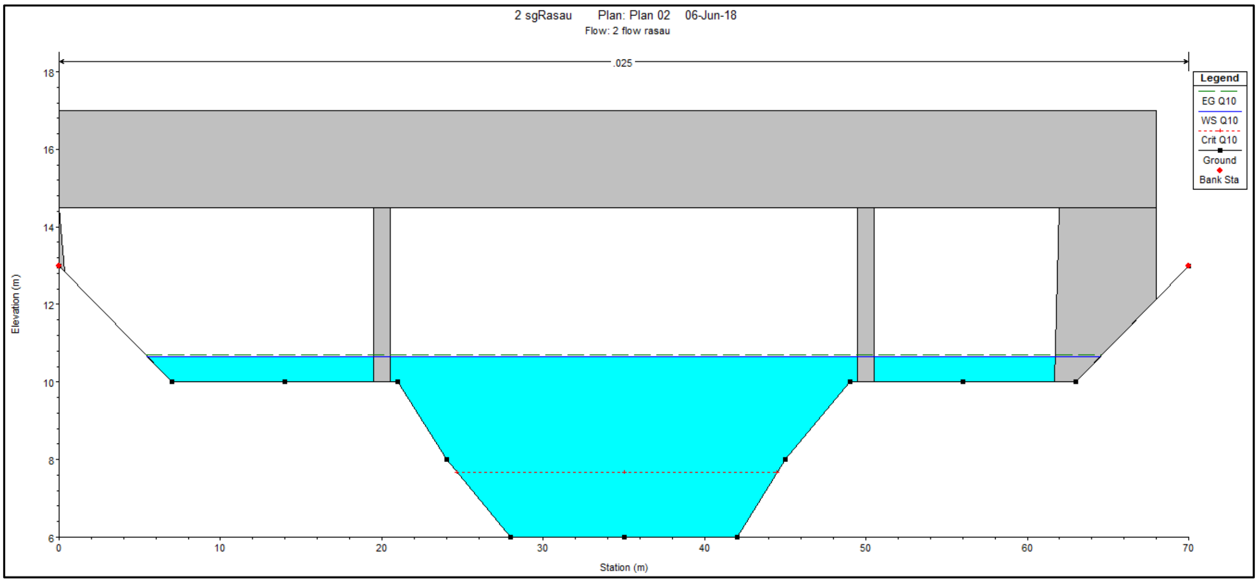

Figure 4.45 Water level at CH110 upstream of Rasau River (with bridge piers) – Q_{10}

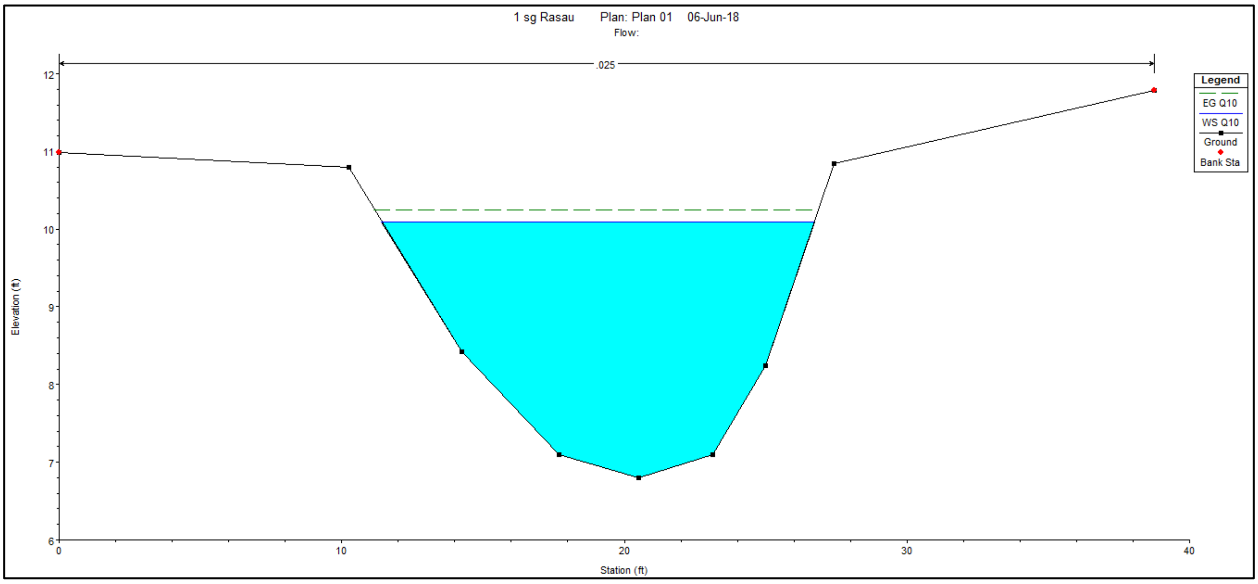

Figure 4.46 Water level at CH120 of Rasau River (without bridge piers) – Q_{10}

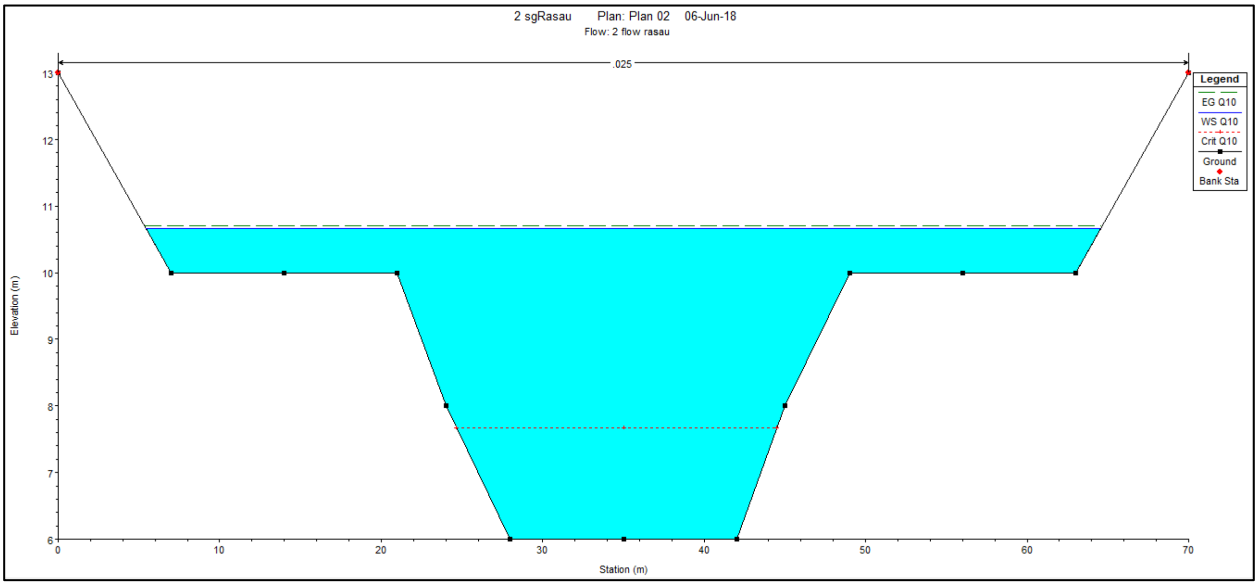

Figure 4.47 Water level at CH120 of Rasau River (with bridge piers) – Q_{10}

4.5.2.3 Water level for Q_{10} at downstream the bridge

Table 4.4 and Figure 4.48 to Figure 4.61 shown that the river simulations produced an output as such the water levels were overflown along CH 140 to CH 260 except at CH 140(without bridge piers) and CH 160(without bridge piers) because of its higher level of left banks. Water levels along the downstream chainages were between 10.19m to 10.29m above sea level without the bridge piers presence and between 10.61m to 10.7m with the bridge piers presence.

In addition, the water levels along the downstream cross section of the river also can be discussed. This study shows that the increases in water levels from CH 120 to CH 240 had slightly fluctuated. The water levels are from 0.37m to 0.56m different along the chainage with 10-year ARI condition.

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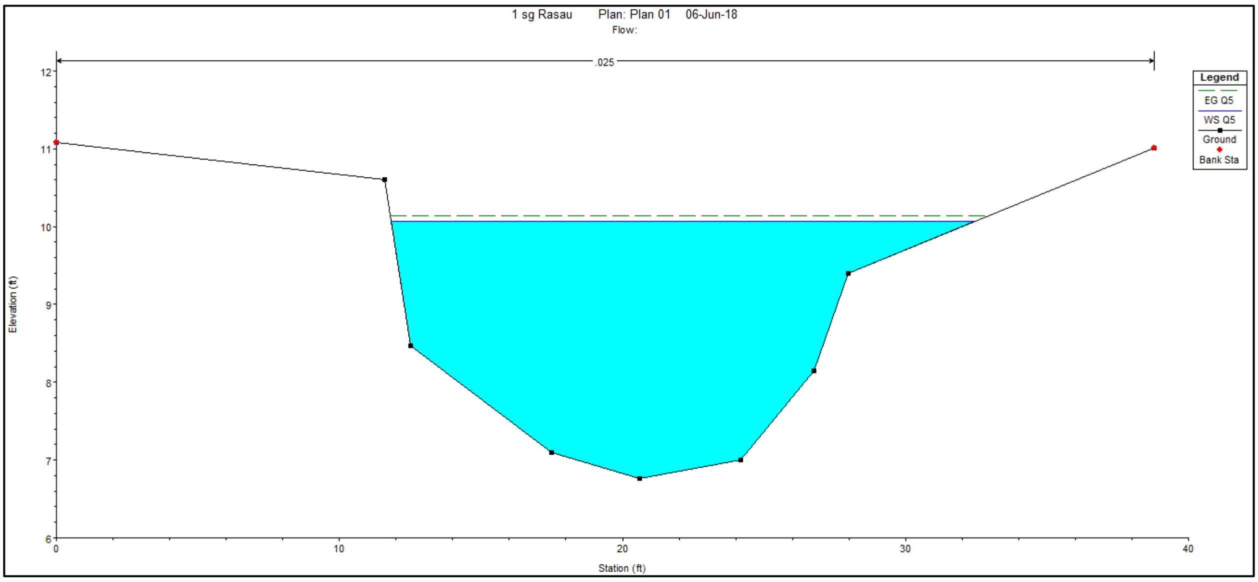

Figure 4.48 Water level at CH140 of Rasau River (without bridge piers) – Q_{10}

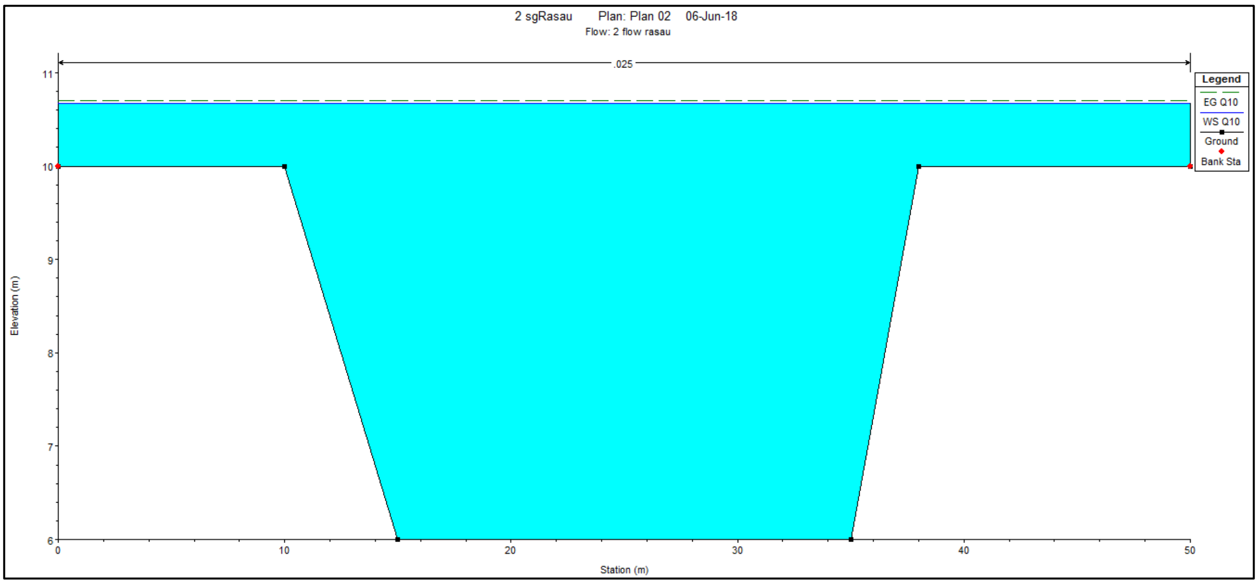

Figure 4.49 Water level at CH140 of Rasau River (with bridge piers) – Q_{10}

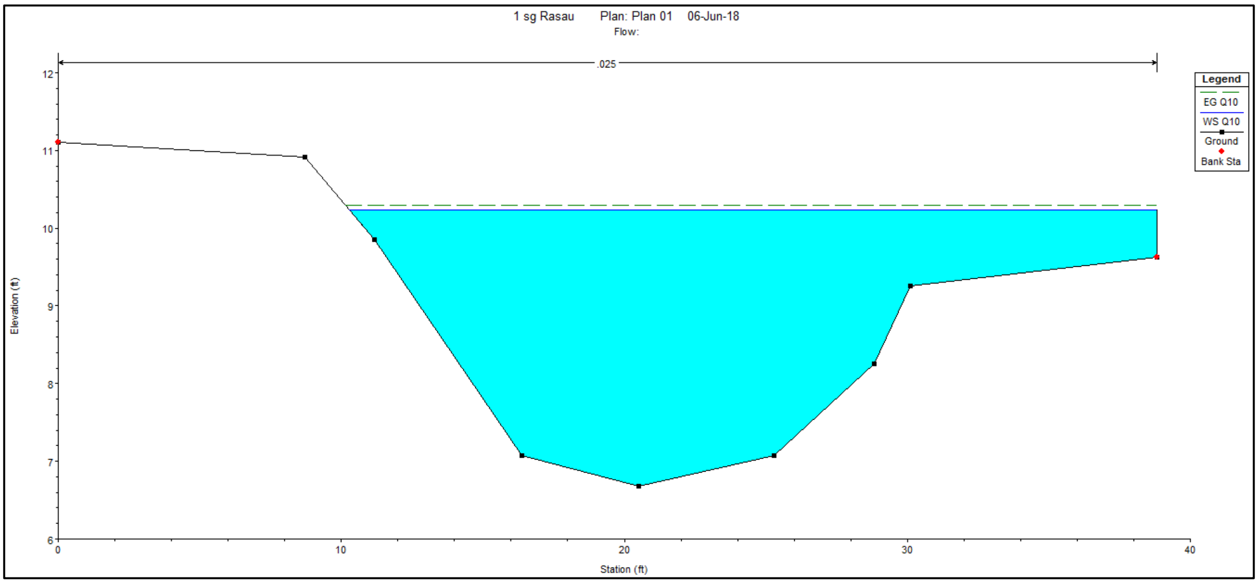

Figure 4.50 Water level at CH160 of Rasau River (without bridge piers) – Q_{10}

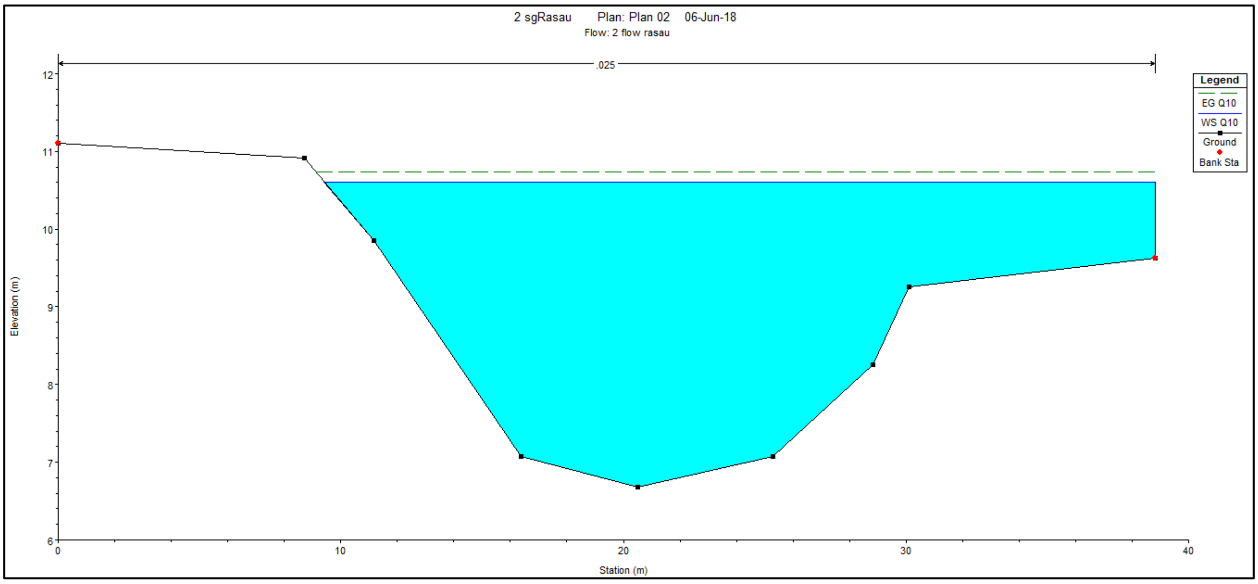

Figure 4.51 Water level at CH160 of Rasau River (with bridge piers) – Q_{10}

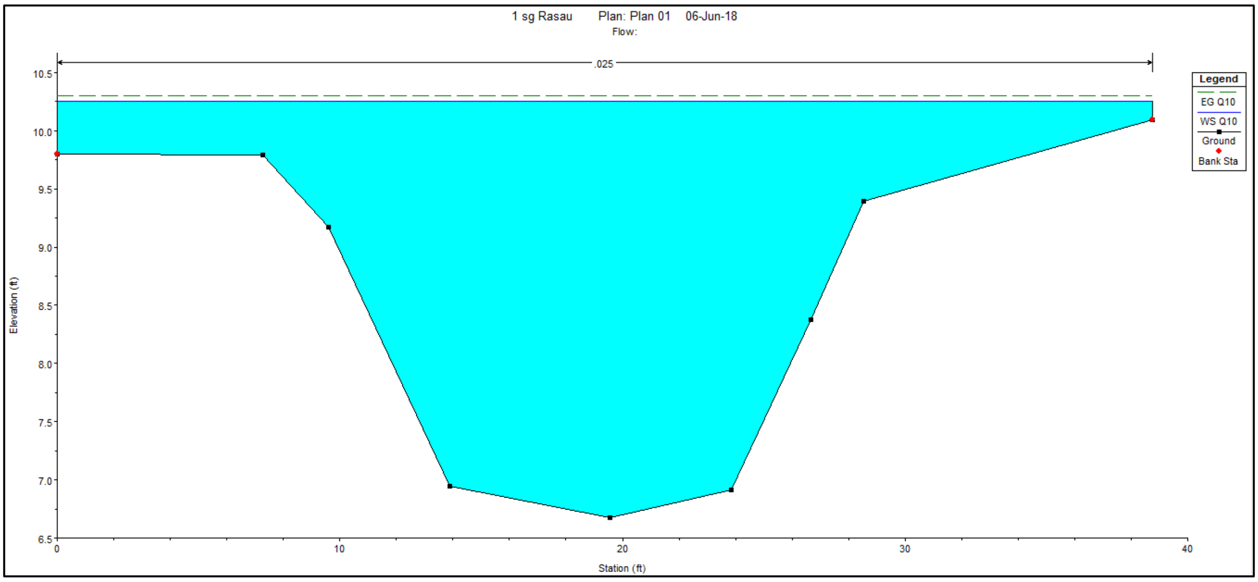

Figure 4.52 Water level at CH180 of Rasau River (without bridge piers) – Q_{10}

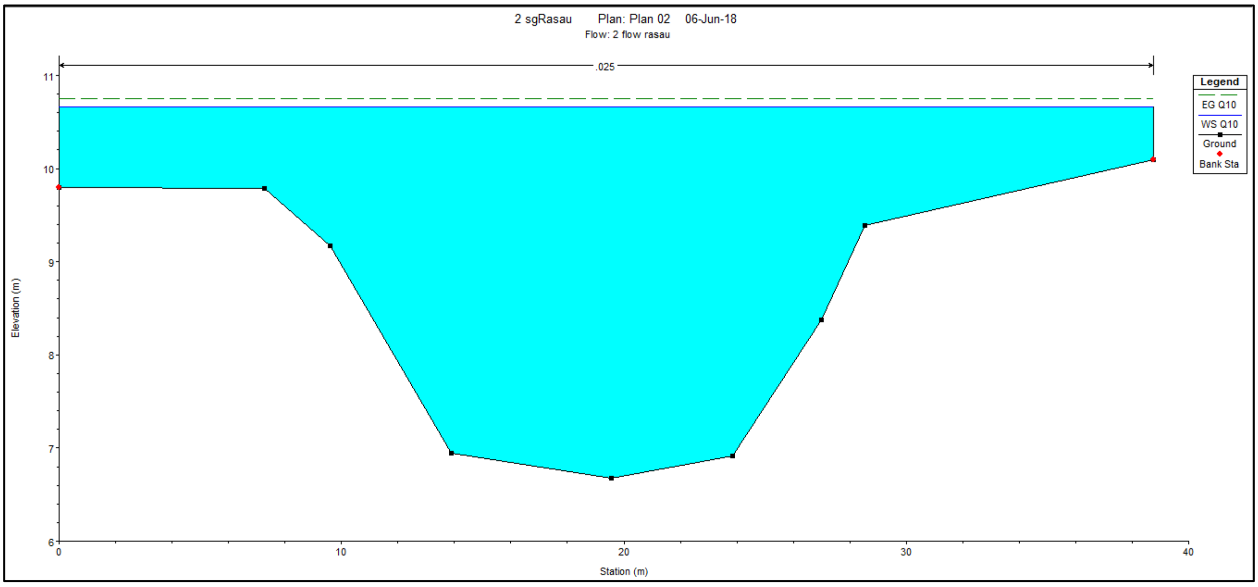

Figure 4.53 Water level at CH180 of Rasau River (with bridge piers) – Q_{10}

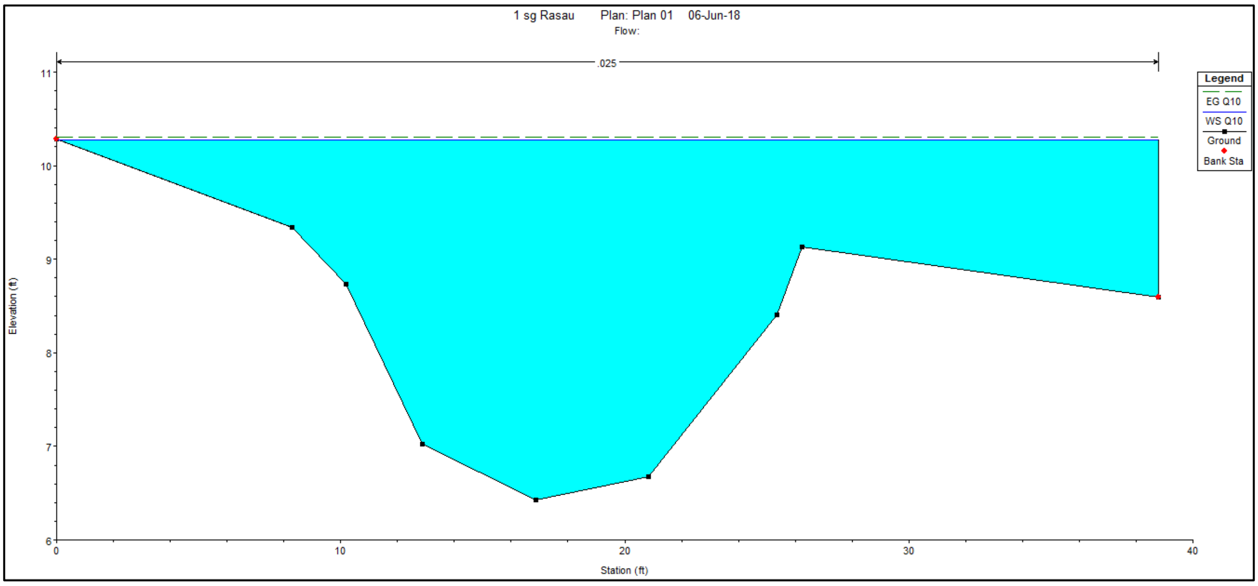

Figure 4.54 Water level at CH200 of Rasau River (without bridge piers) – Q_{10}

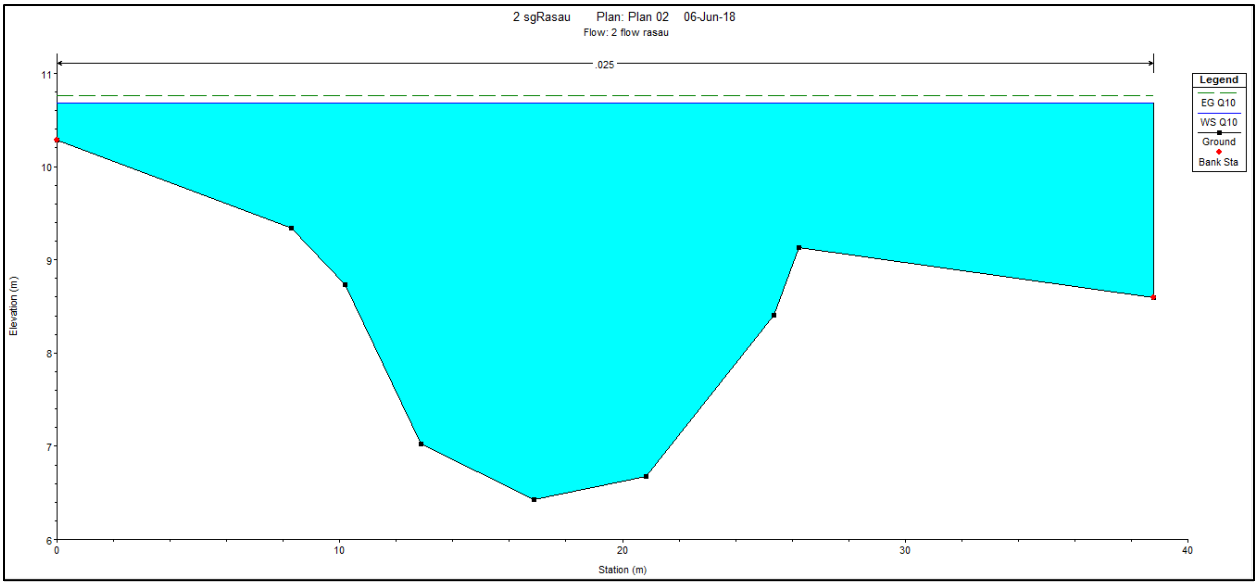

Figure 4.55 Water level at CH200 of Rasau River (with bridge piers) – Q_{10}

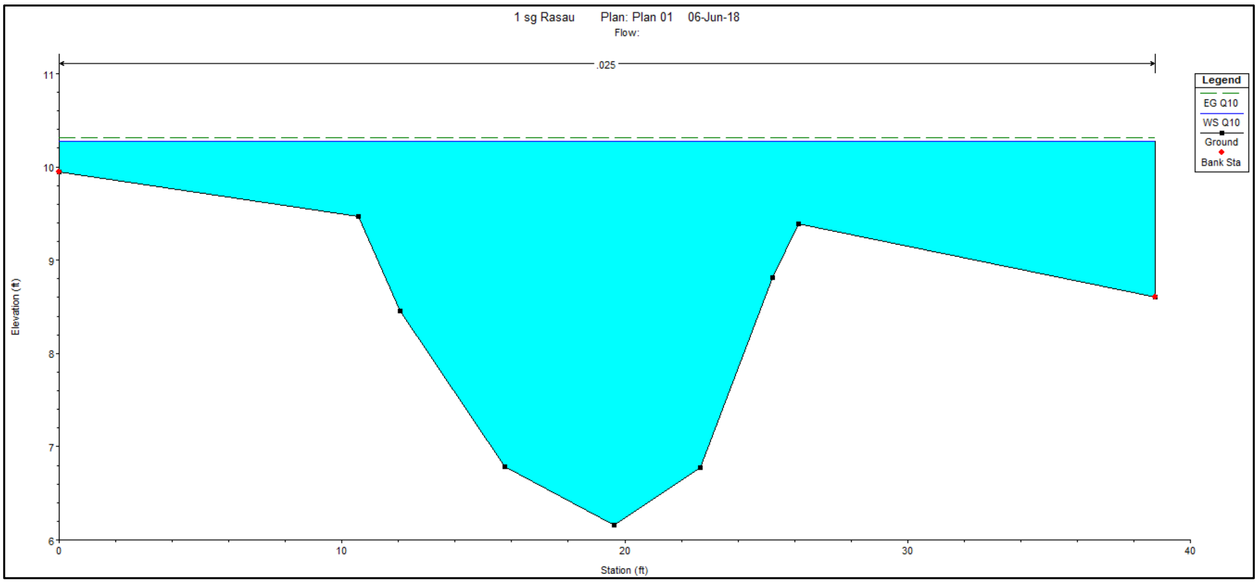

Figure 4.56 Water level at CH220 of Rasau River (without bridge piers) – Q_{10}

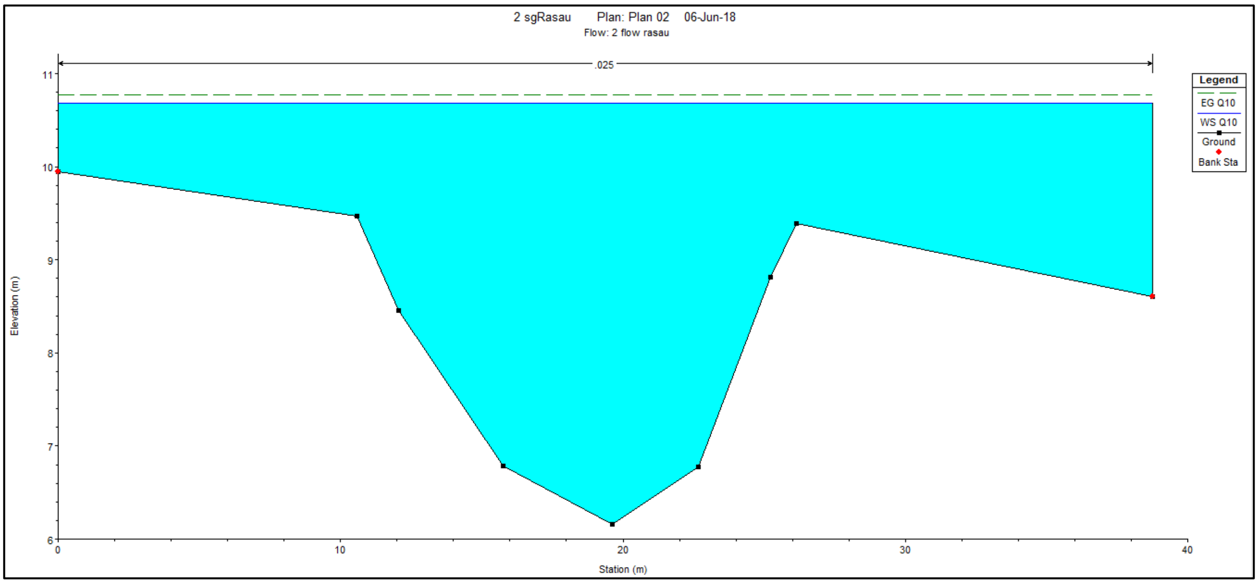

Figure 4.57 Water level at CH220 of Rasau River (with bridge piers) – Q_{10}

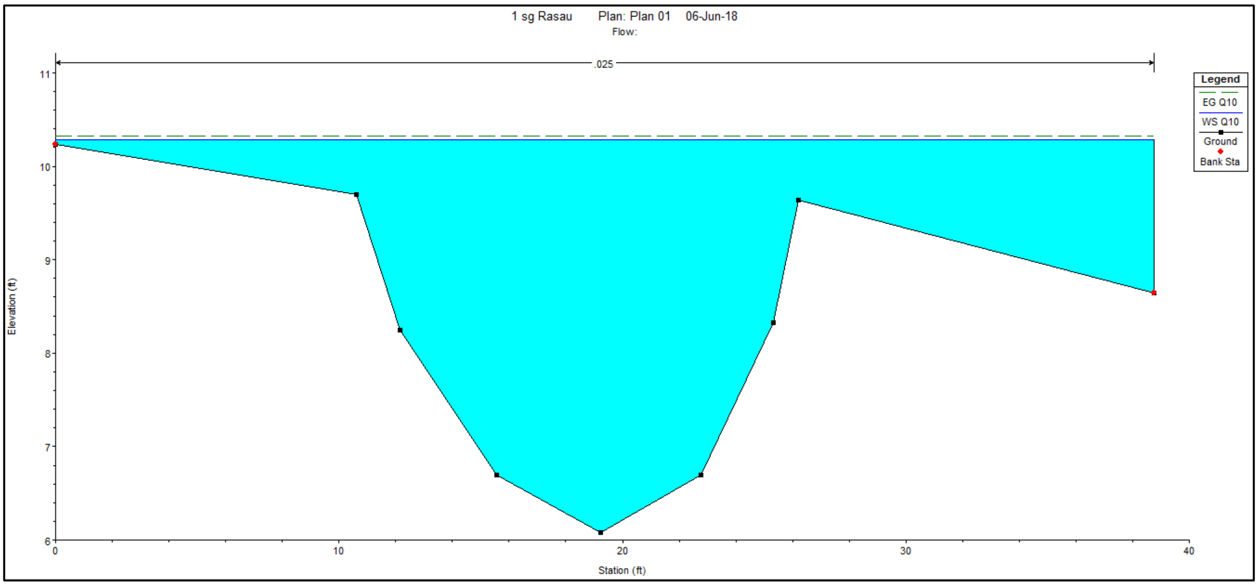

Figure 4.58 Water level at CH240 of Rasau River (without bridge piers) – Q_{10}

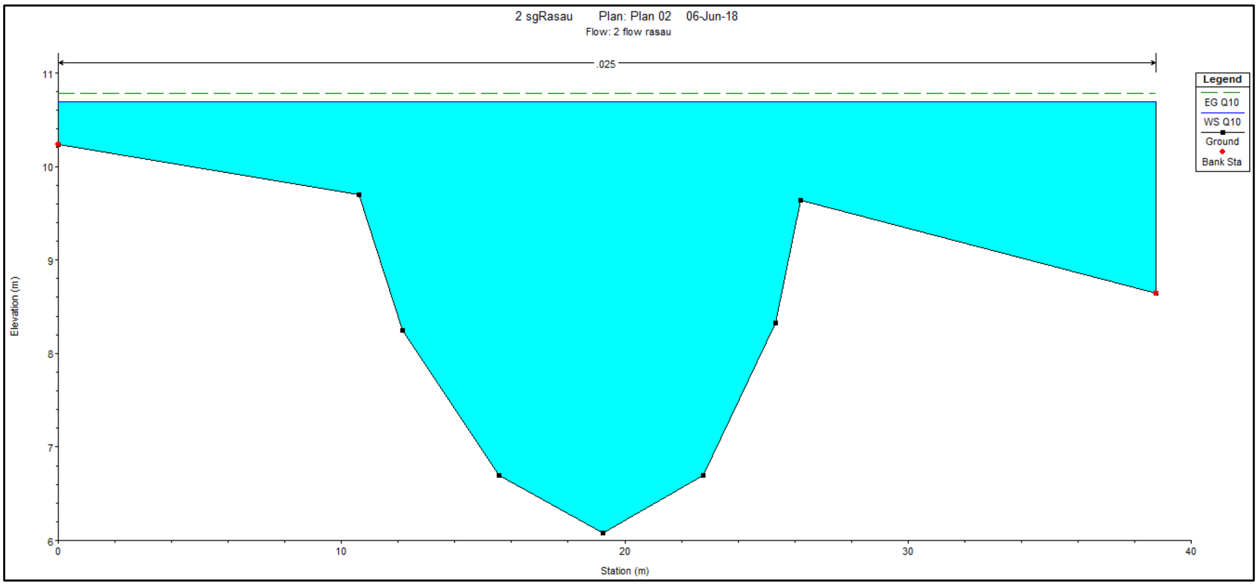

Figure 4.59 Water level at CH240 of Rasau River (with bridge piers) – Q_{10}

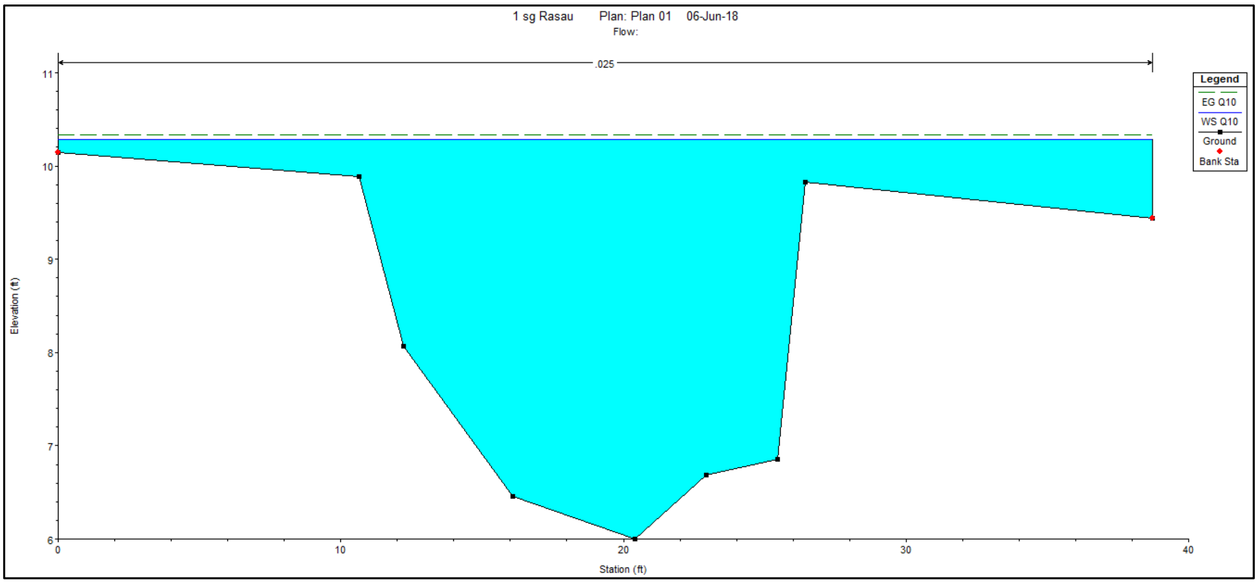

Figure 4.60 Water level at CH260 of Rasau River (without bridge piers) – Q_{10}

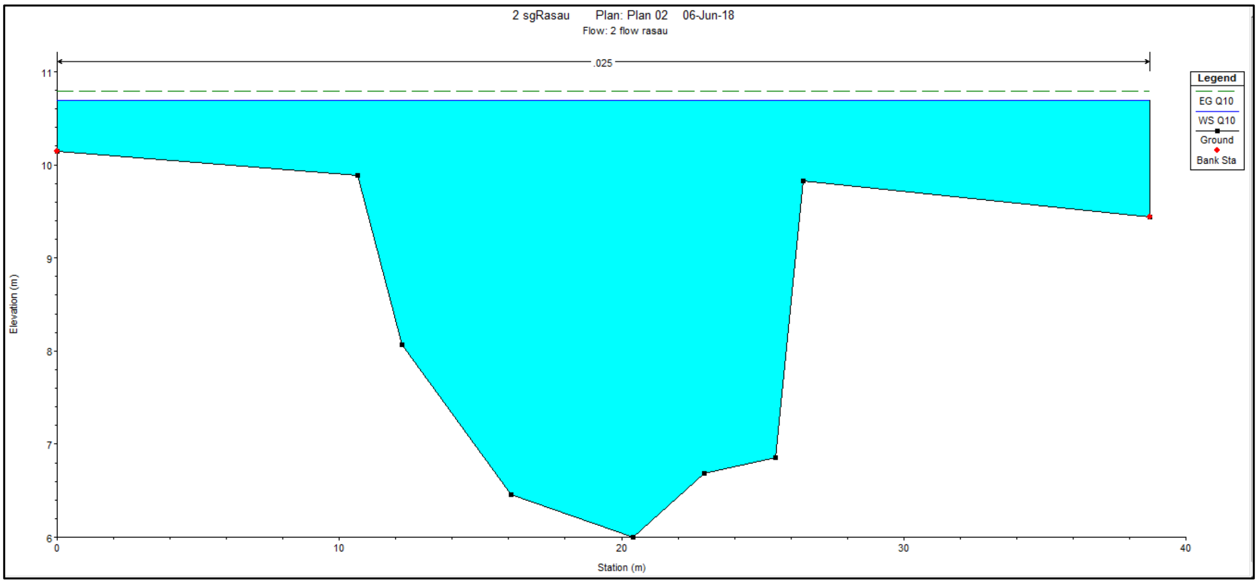

Figure 4.61 Water level at CH260 of Rasau River (with bridge piers) – Q_{10}

4.5.3 Water level for Q_{20} with and without bridge piers

The difference in water levels of Rasau River along the chainage with 20-year ARI is tabulated in Table 4.5 and water level profiles together with backwater effect were discussed.

Table 4.5 Difference of water level with Q_{20}

Q_{20} = 122.7 m^3/s						
Chainage	**Before bridge construction**		**After bridge construction**		**Difference of water level (m)**	
	Water level (m)		**Water level (m)**			
	Left bank	**Right bank**	**Left bank**	**Right bank**	**Left bank**	**Right bank**
CH 0	9.74	9.74	10.16	10.16	0.42	0.42
CH 20	10.12	10.12	10.55	10.55	0.43	0.43
CH 40	10.23	10.23	10.71	10.71	0.48	0.48
CH 60	10.24	10.24	10.71	10.71	0.47	0.47
CH 80	10.24	10.24	10.80	10.80	0.56	0.56
CH 100	10.21	10.21	10.80	10.80	0.56	0.56
CH 110 U	10.21	10.21	10.80	10.80	0.56	0.56
CH 110 D	10.21	10.21	10.80	10.80	0.56	0.56
CH 120	10.20	10.20	10.80	10.80	0.60	0.60
CH 140	10.32	10.32	10.81	10.81	0.49	0.49
CH 160	10.37	10.37	10.74	10.74	0.37	0.37
CH 180	10.40	10.40	10.80	10.80	0.40	0.40
CH 200	10.42	10.42	10.83	10.83	0.41	0.41
CH 220	10.42	10.42	10.83	10.83	0.41	0.41
CH 220	10.43	10.43	10.84	10.84	0.41	0.41
CH 240	10.43	10.43	10.84	10.84	0.41	0.41

4.5.3.1 Water level for Q_{20} at upstream of the bridge

Based on Table 4.5 and Figure 4.62 to Figure 4.73, the water levels were in between 9.74m to 10.24m above sea level at the upstream of the river without the bridge piers presence and 10.16m to 10.80m with the presence of the bridge piers. It can be said that the water levels were overflown onto both the left bank and right bank during rainfall event of 20-year ARI. In other words, the water is no longer contained in the river channel along CH 0 to CH 100 for both conditions of simulations.

Next, the differences in water levels along the upstream chainage which start from CH 0 to CH 100 were between 0.42m to 0.56m. These indicated that with the presence of Bridge 3 at the Rasau River, it increased the water level up to 0.56m at CH 100 where the bridge piers are located.

Moreover, these situations showed that the water levels had overflown onto the left and right bank that was higher than the normal condition at the same chainage without the bridge piers presence. Even though along the studied cross section should be already flooded, with water but the level still at the safest level if compared with the water level at each chainage or cross section with the presence of bridge piers.

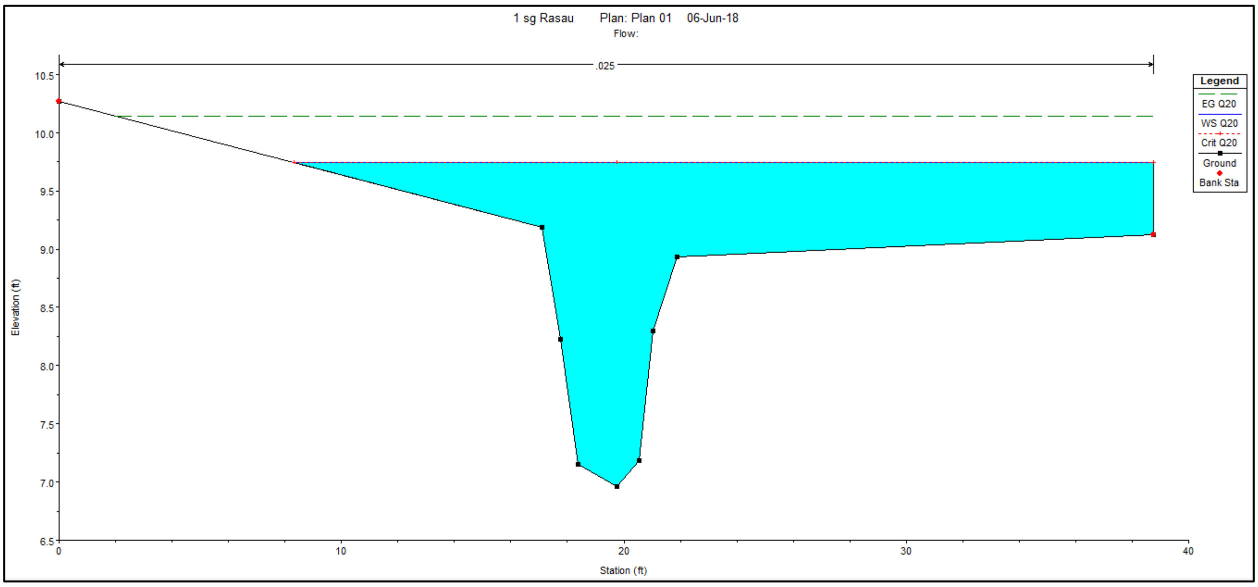

Figure 4.62 Water level at CH0 of the Rasau River (without bridge piers) – Q_{20}

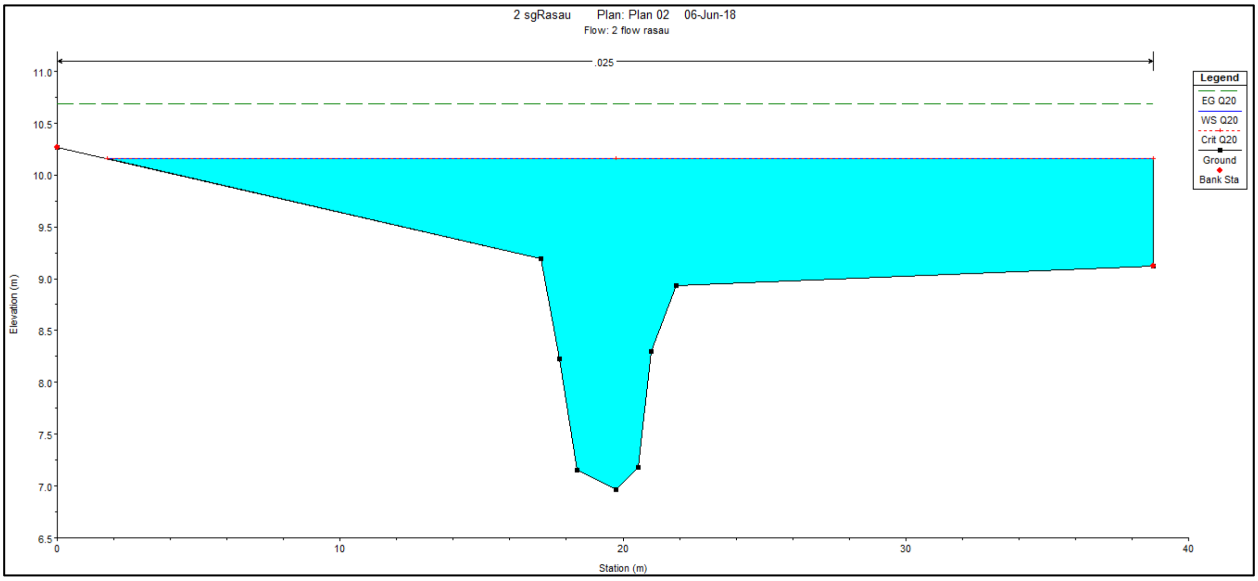

Figure 4.63 Water level at CH0 of the Rasau River (with bridge piers) – Q_{20}

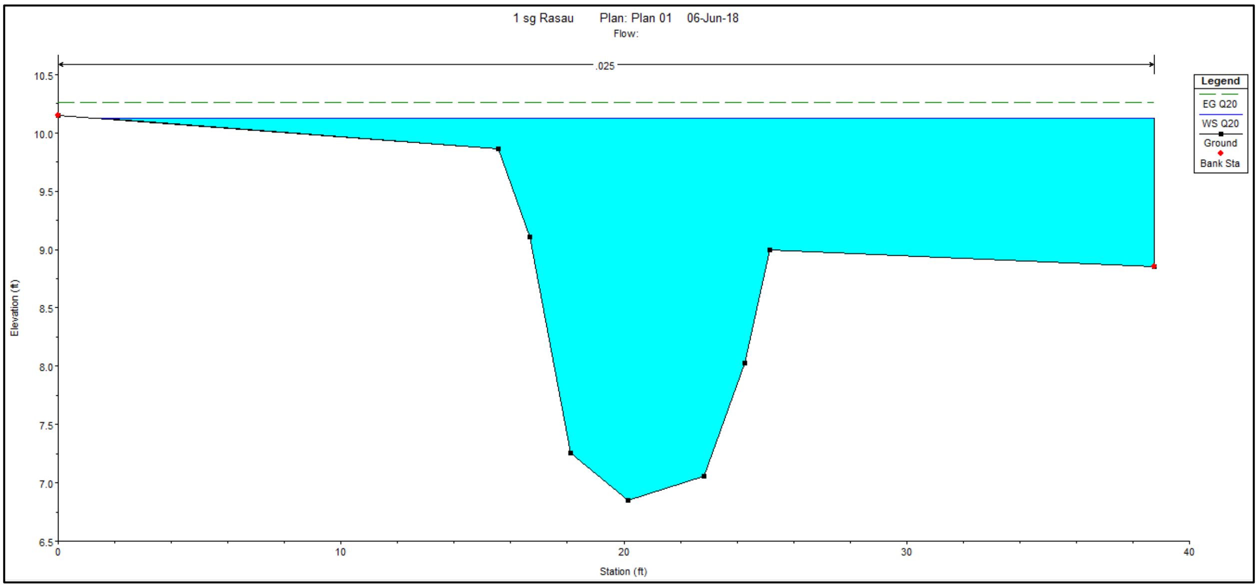

Figure 4.64 Water level at CH20 of Rasau River (without bridge piers) – Q_{20}

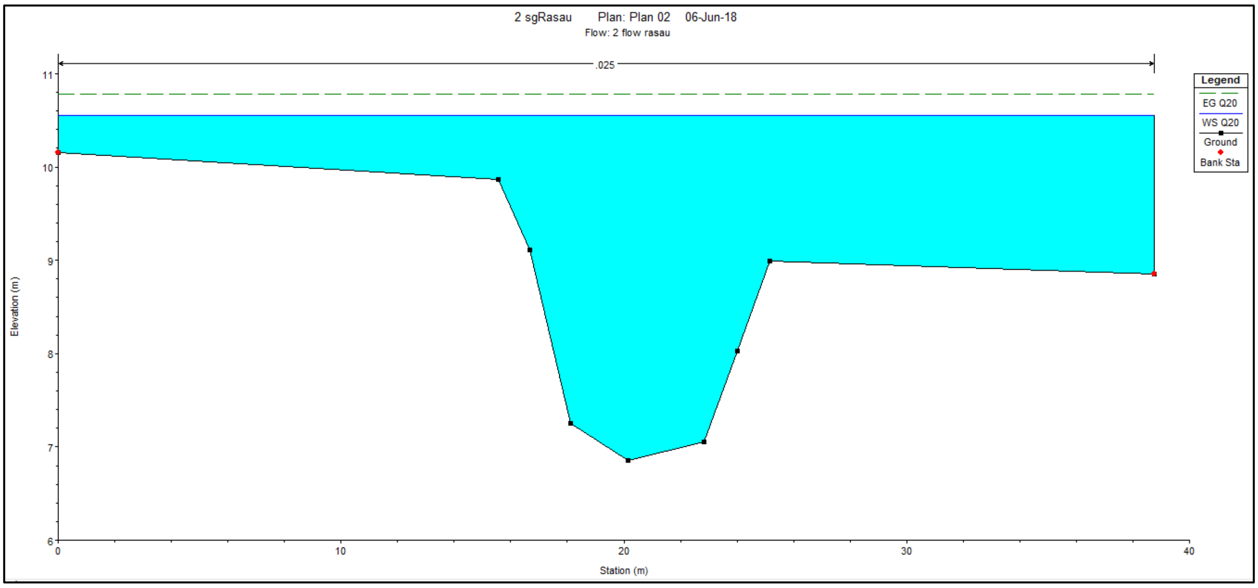

Figure 4.65 Water level at CH20 of Sg Rasau (with bridge piers) – Q_{20}

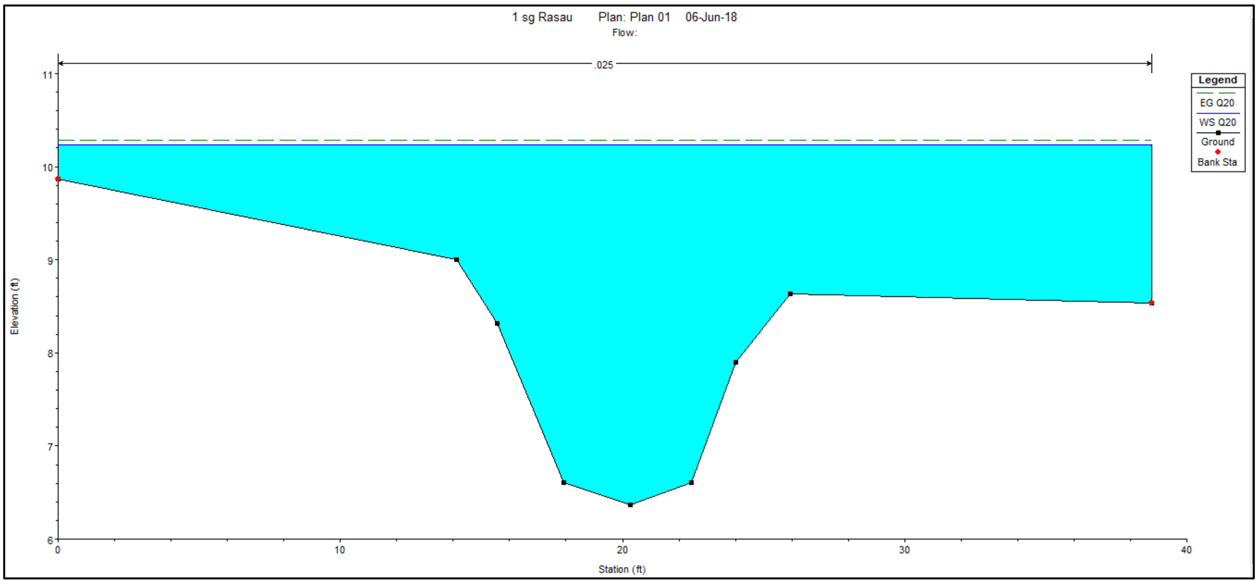

Figure 4.66 Water level at CH40 of Rasau River (without bridge piers) – Q_{20}

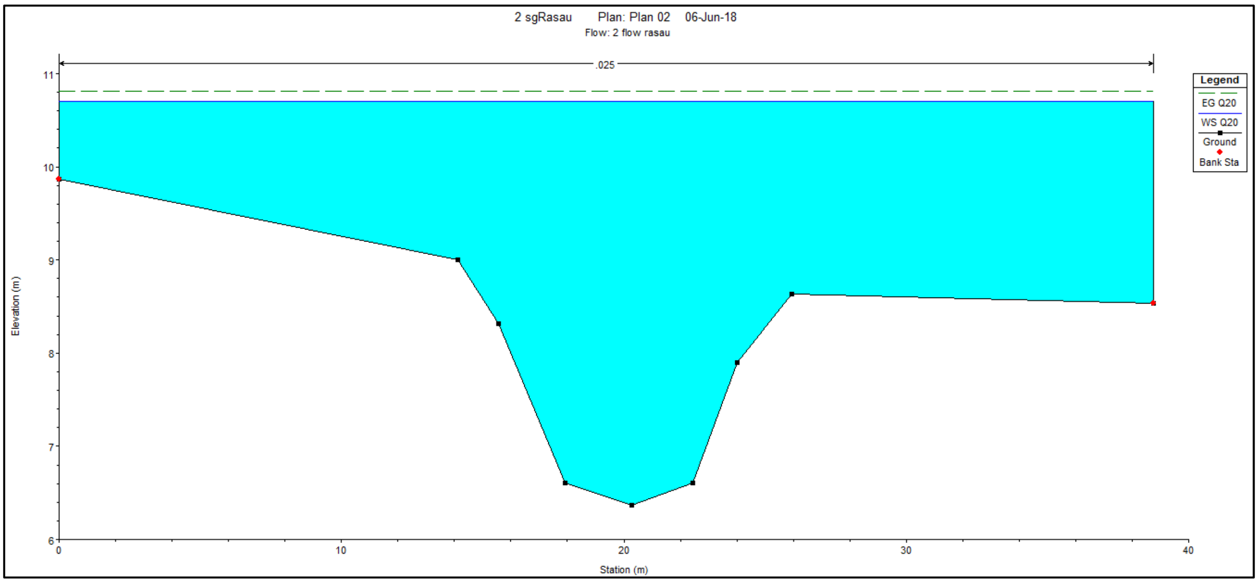

Figure 4.67 Water level at CH 40 of Rasau River (with bridge piers) – Q_{20}

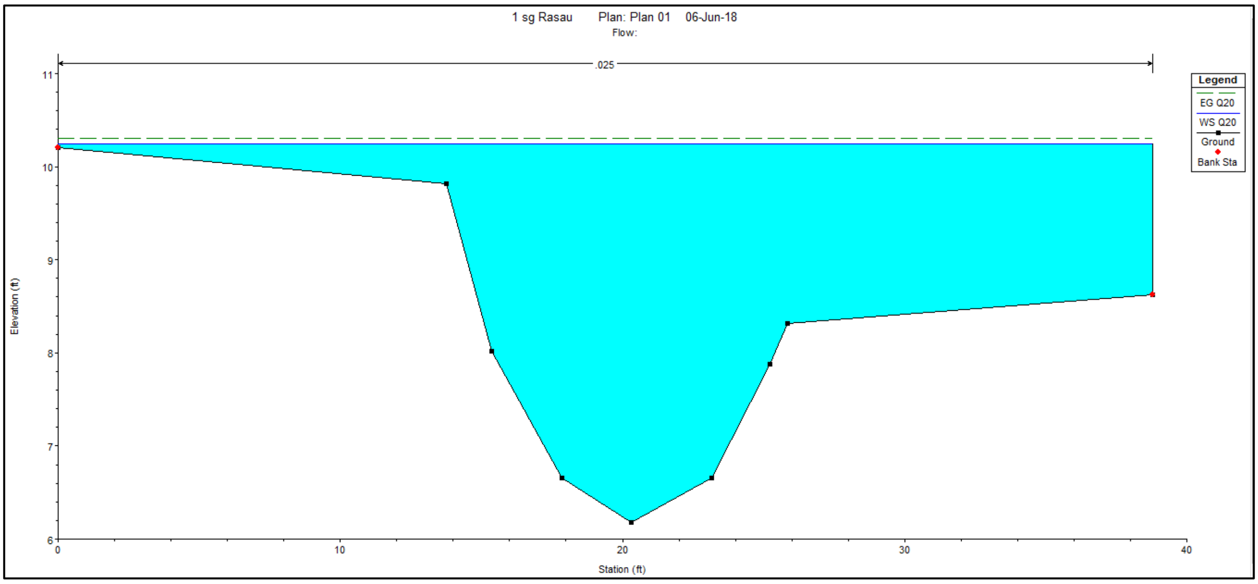

Figure 4.68 Water level at CH60 of Rasau River (without bridge piers) – Q_{20}

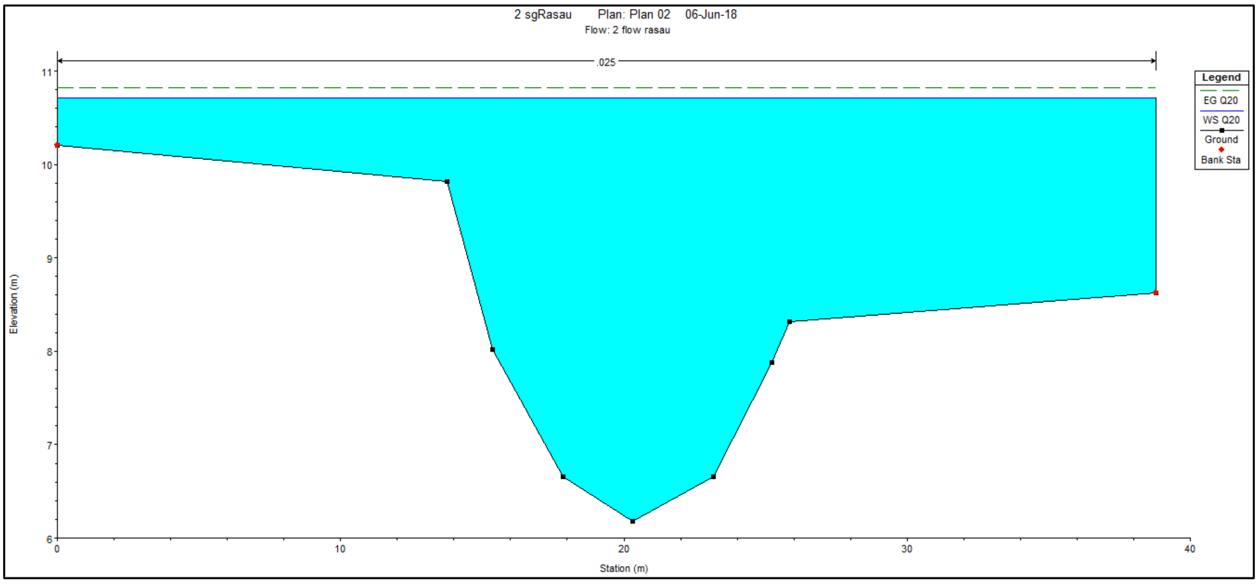

Figure 4.69 Water level at CH60 of Rasau River (with bridge piers) – Q_{20}

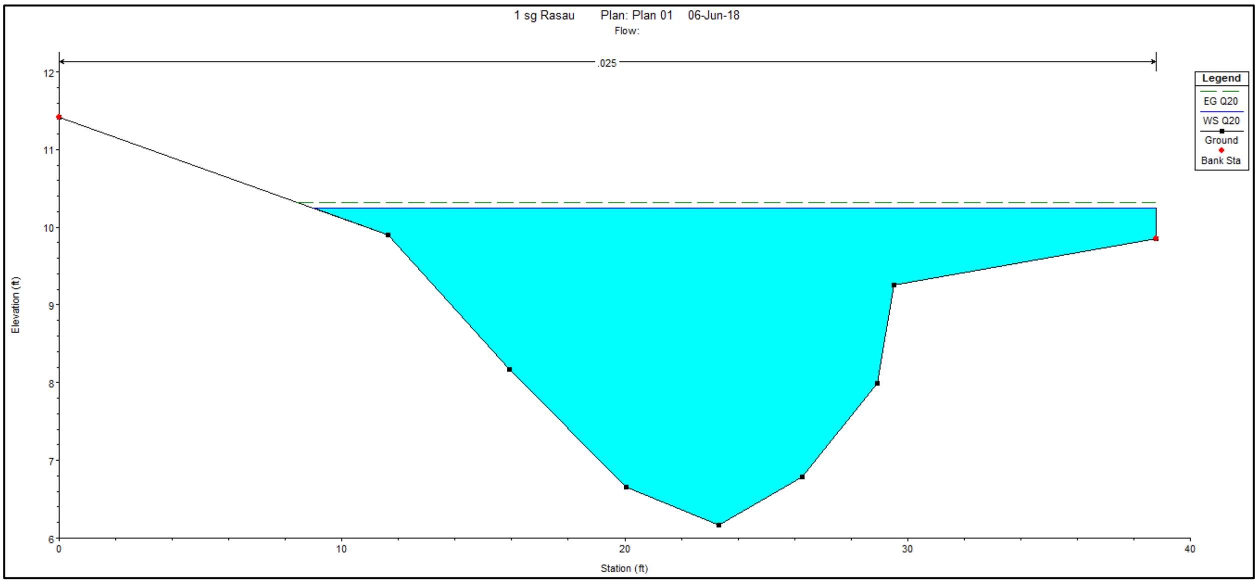

Figure 4.70 Water level at CH80 of Rasau River (without bridge piers) – Q_{20}

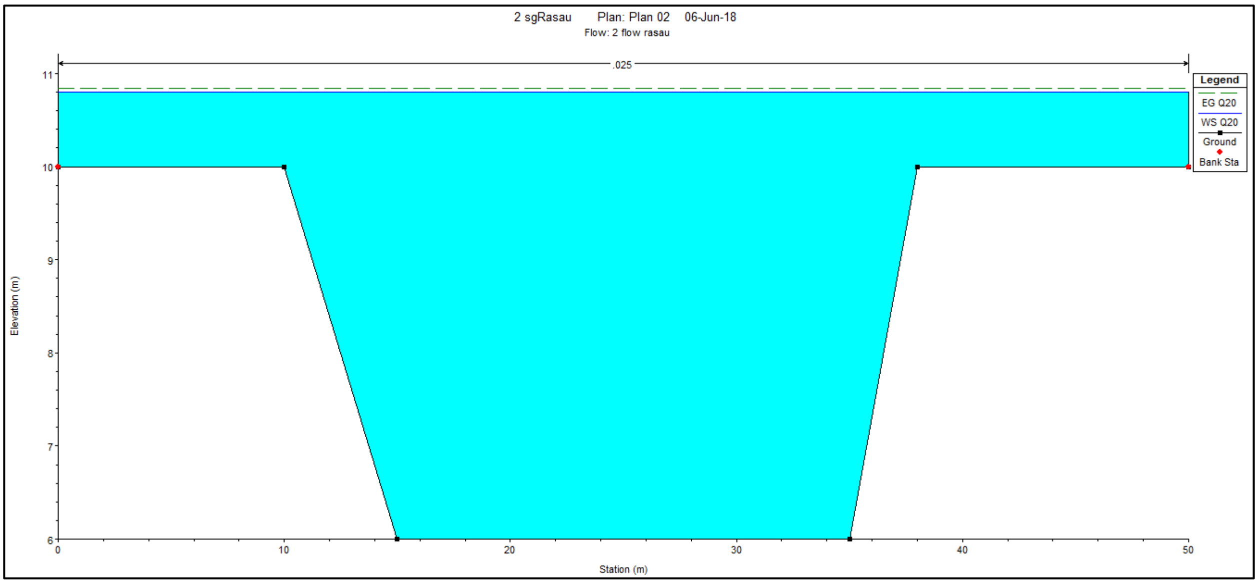

Figure 4.71 Water level at CH80 of Rasau River (with bridge piers) – Q_{20}

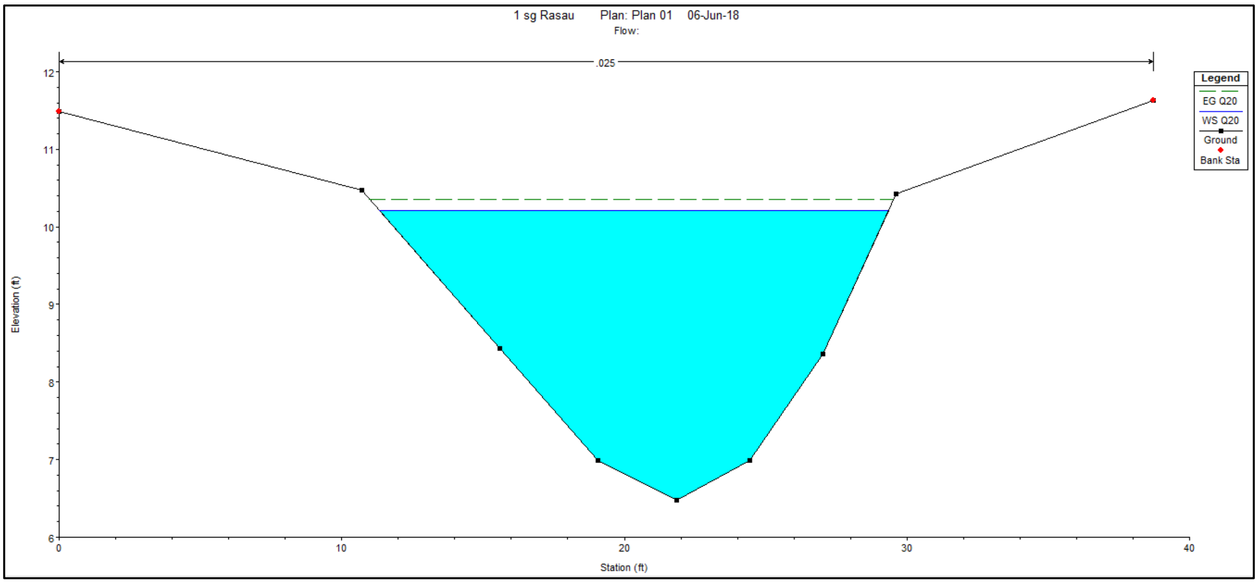

Figure 4.72 Water level at CH100 of Rasau River (without bridge piers) – Q_{20}

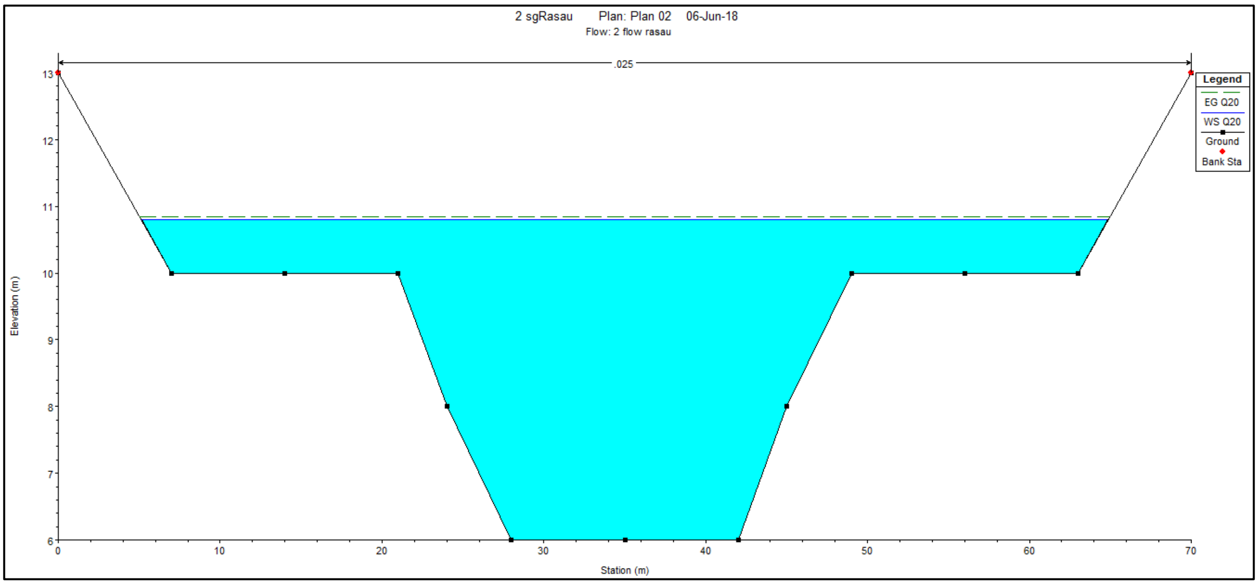

Figure 4.73 Water level at CH100 of Rasau River (with bridge piers) – Q_{20}

4.5.3.2 Water level for Q_{20} under the bridge

With reference to the Table 4.5 and Figure 4.74 to Figure 4.77 and at CH 100 and CH 120 where the bridge will be located, the result showed that the water levels at these chainages with 20-year ARI were 10.20m to 10.21m above sea level without the bridge piers presence and 10.8m with the presence of bridge piers. The water levels were contained in the river channel without overflow to the left and right bank.

Apart from that, during rainfall event of 20-year ARI, the water level under the bridge cross section which starts from CH 100 to CH 120 showed that the difference in water levels between the two conditions was 0.56m. Water levels condition below the bridge indicated that the backwater effect increased the water levels 0.56m more but only at level of 10.80m above sea level which is still below the road level (15.92m).

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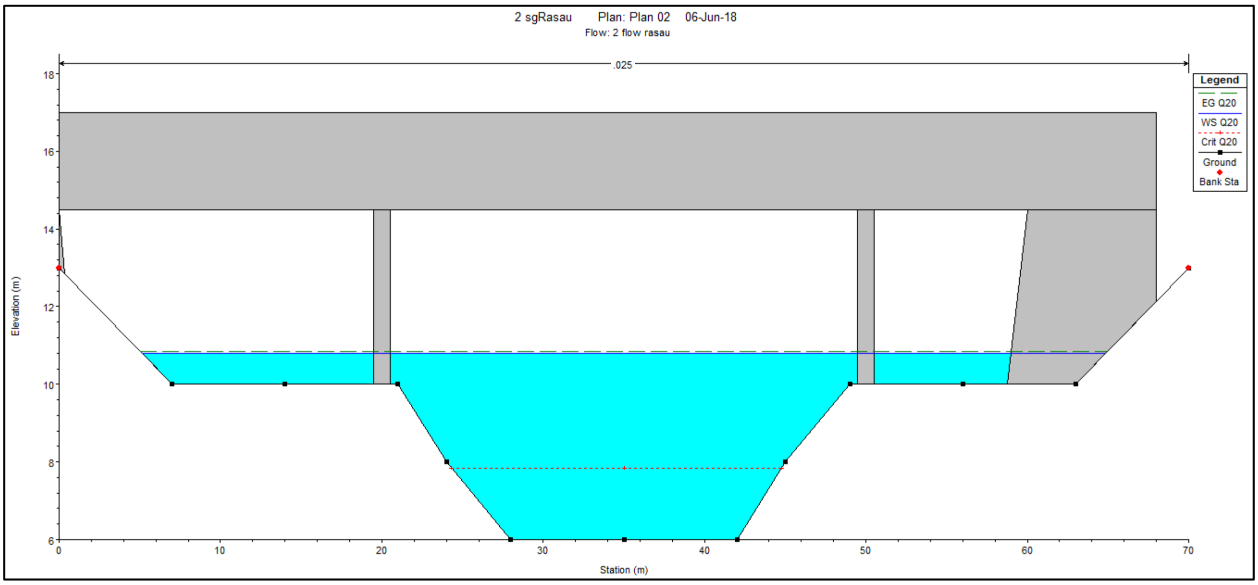

Figure 4.74 Water level at CH110 downstream of Rasau River (with bridge piers) – Q_{20}

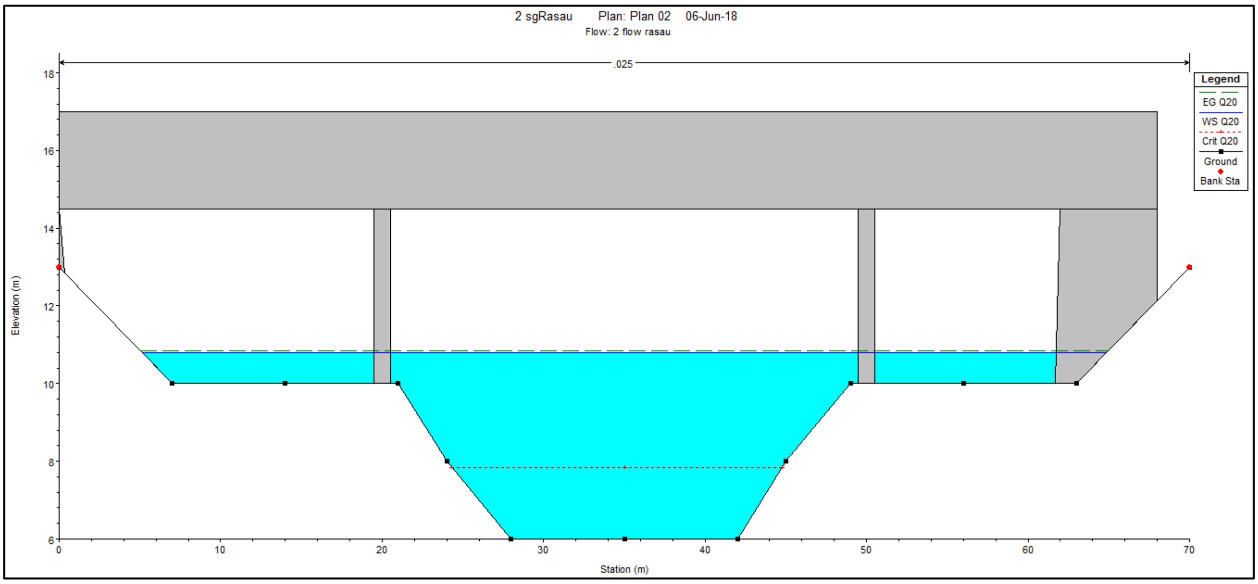

Figure 4.75 Water level at CH110 upstream of Rasau River (with bridge piers) – Q_{20}

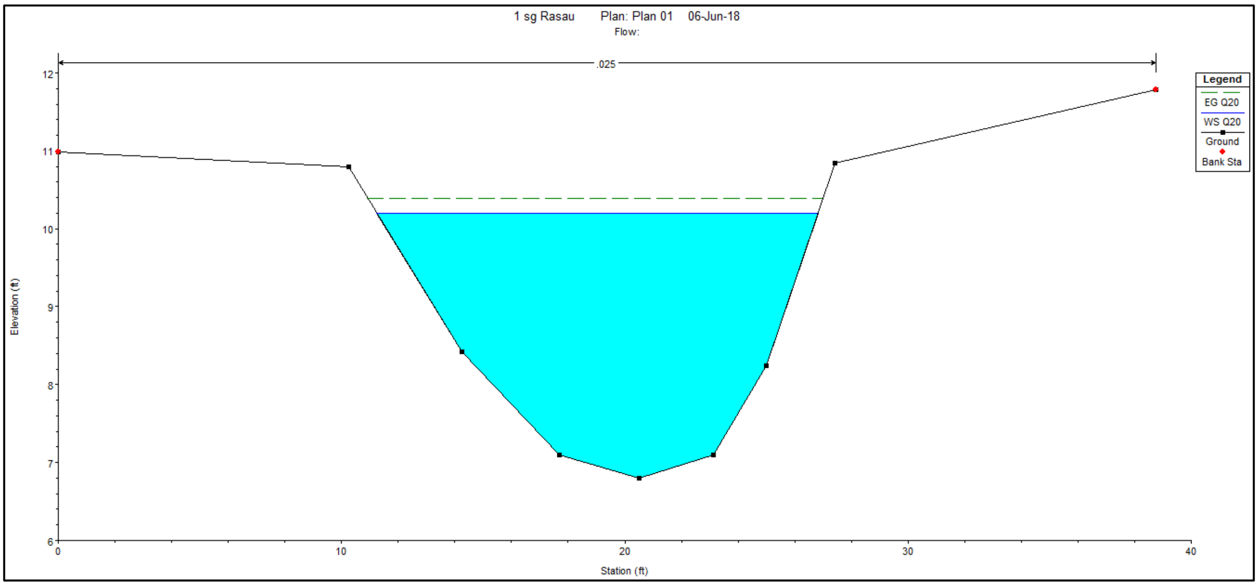

Figure 4.76 Water level at CH120 of Rasau River (without bridge piers) – Q_{20}

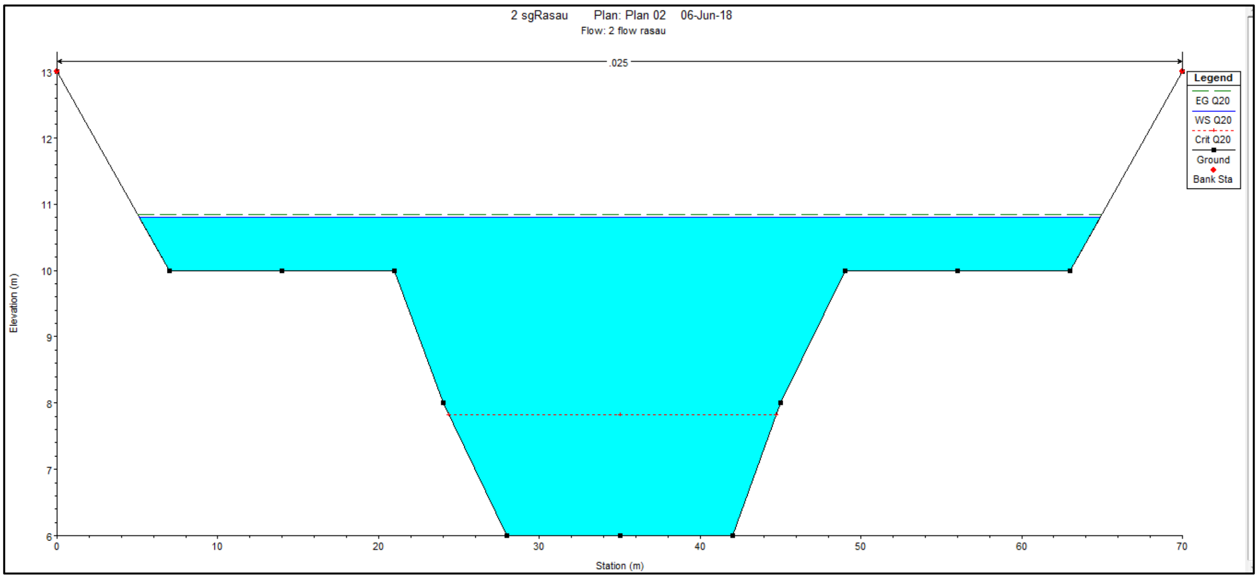

Figure 4.77 Water level at CH120 of Rasau River (with bridge piers) – Q_{20}

4.5.3.3 Water level for Q_{20} at downstream of the bridge

Based on Table 4.5 and Figure 4.78 to Figure 4.91, water levels were overflown along CH 140 to CH 260 except at CH 140(without bridge piers) and CH 160(without bridge piers) because of its higher level of left banks. Water levels along the downstream chainages were between 10.32m to 10.43m above sea level without the bridge piers presence and between 10.74m to 10.84m with the bridge piers presence.

Another area of concerned is the water levels along the downstream cross section of the river. These studies show that there was an increase in water levels from CH 120 to CH 240. The differences in water levels were from 0.37m to 0.60m different along the chainage with 20-year ARI condition.

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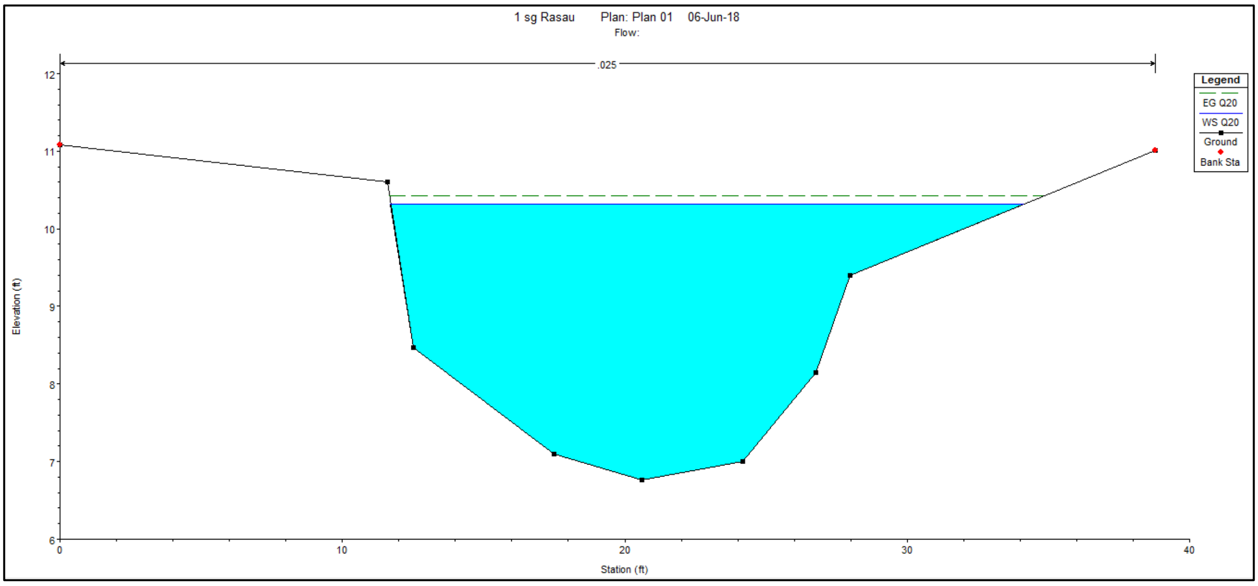

Figure 4.78 Water level at CH140 of Rasau River (without bridge piers) – Q_{20}

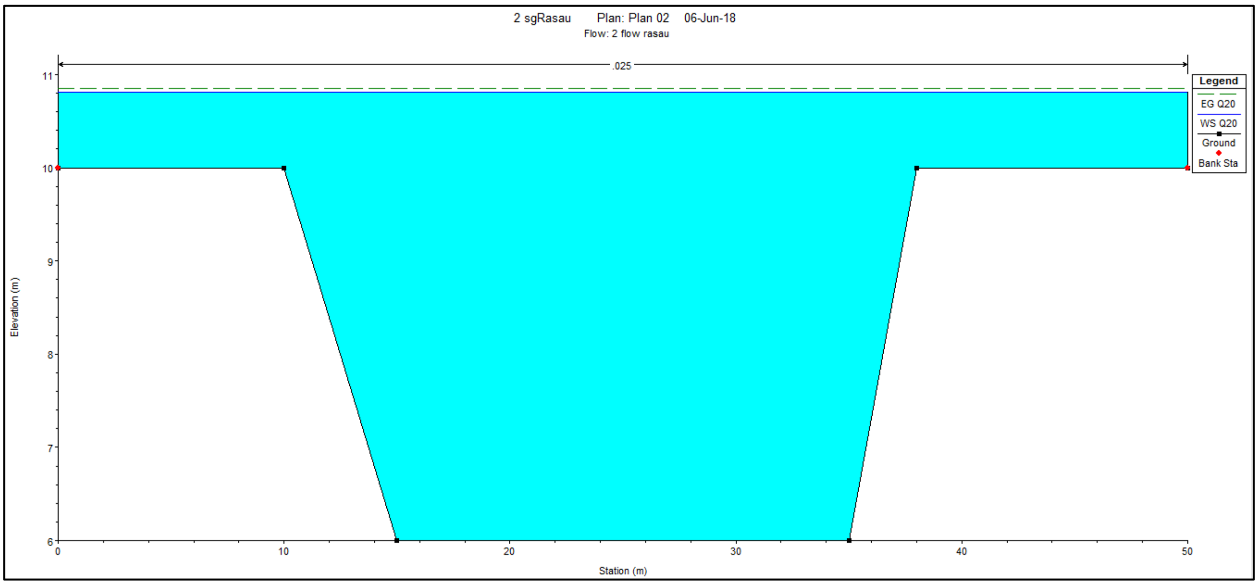

Figure 4.79 Water level at CH140 of Rasau River (with bridge piers) – Q_{20}

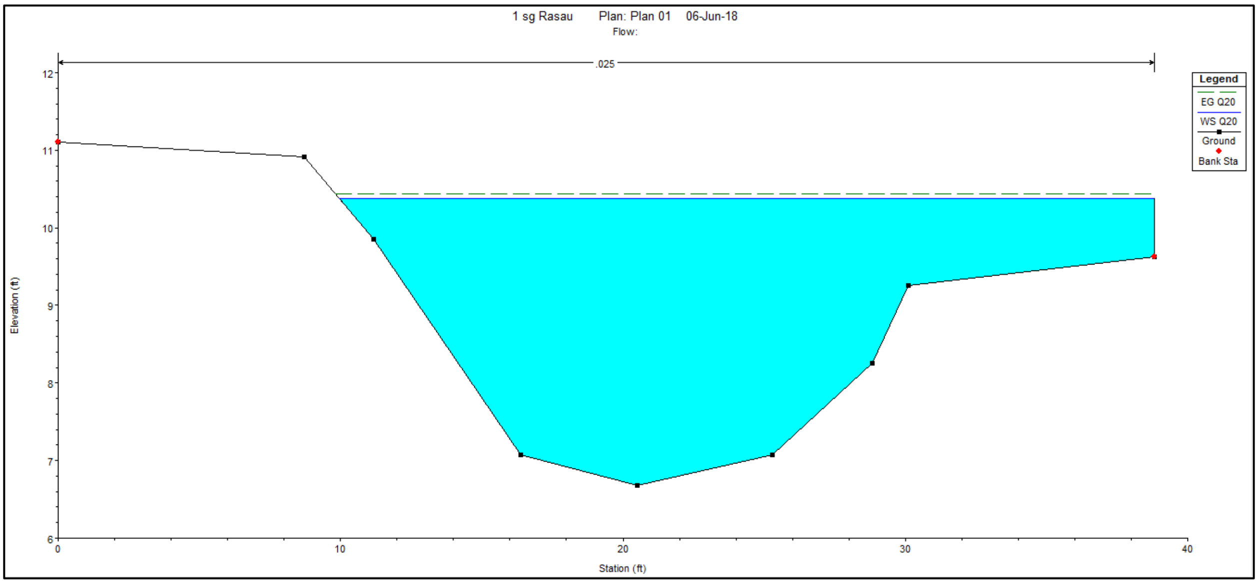

Figure 4.80 Water level at CH160 of Rasau River (without bridge piers) – Q_{20}

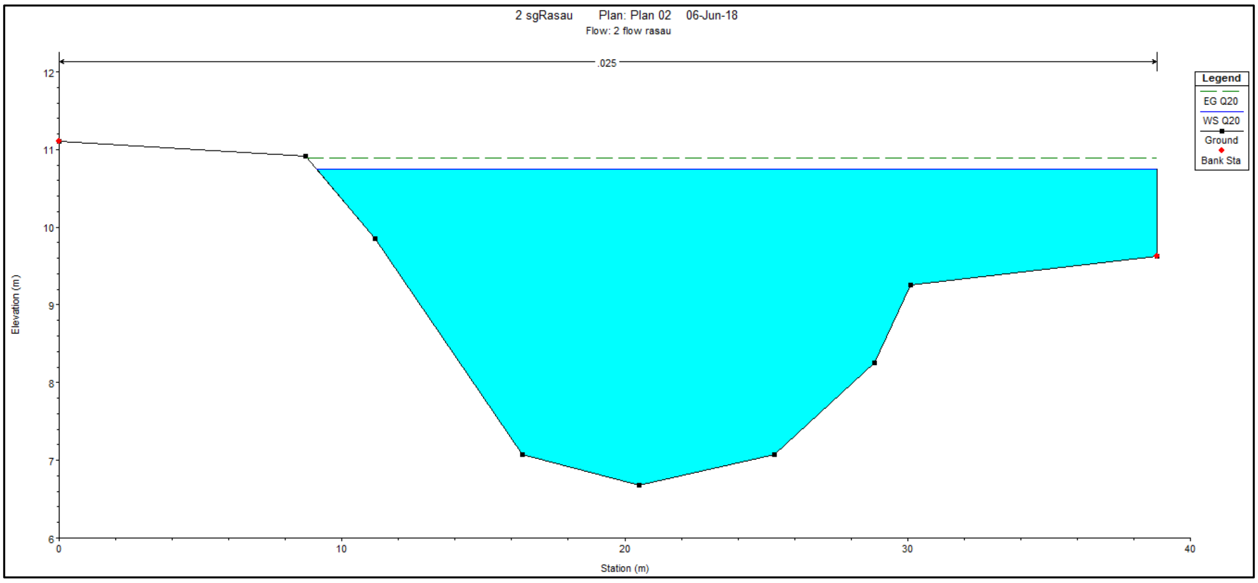

Figure 4.81 Water level at CH160 of Rasau River (with bridge piers) – Q_{20}

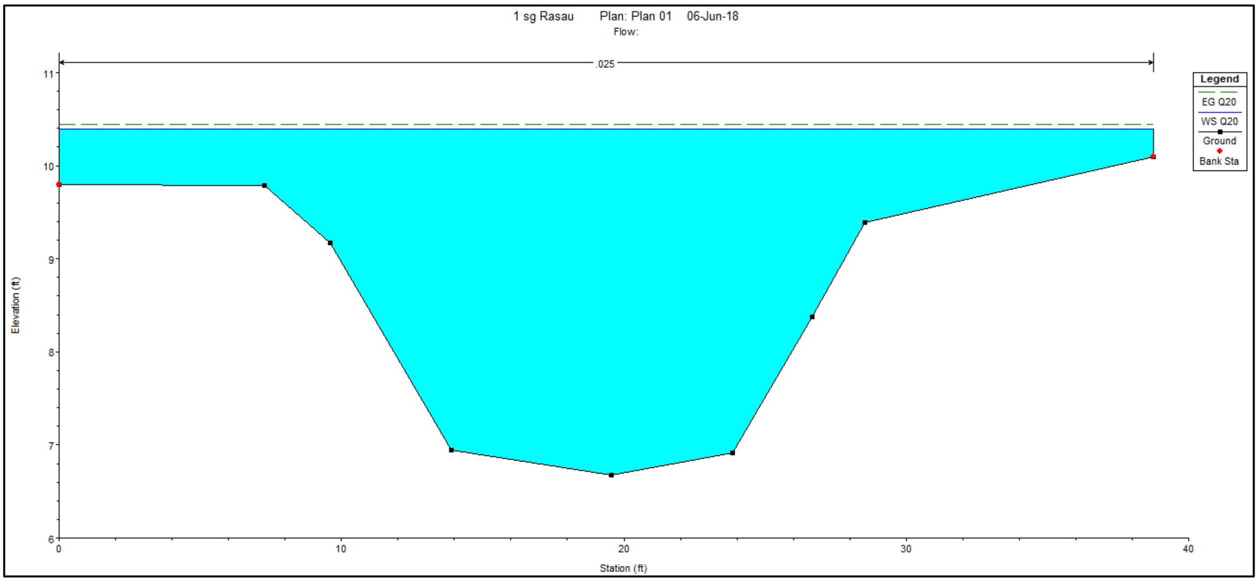

Figure 4.82 Water level at CH180 of Rasau River (without bridge piers) – Q_{20}

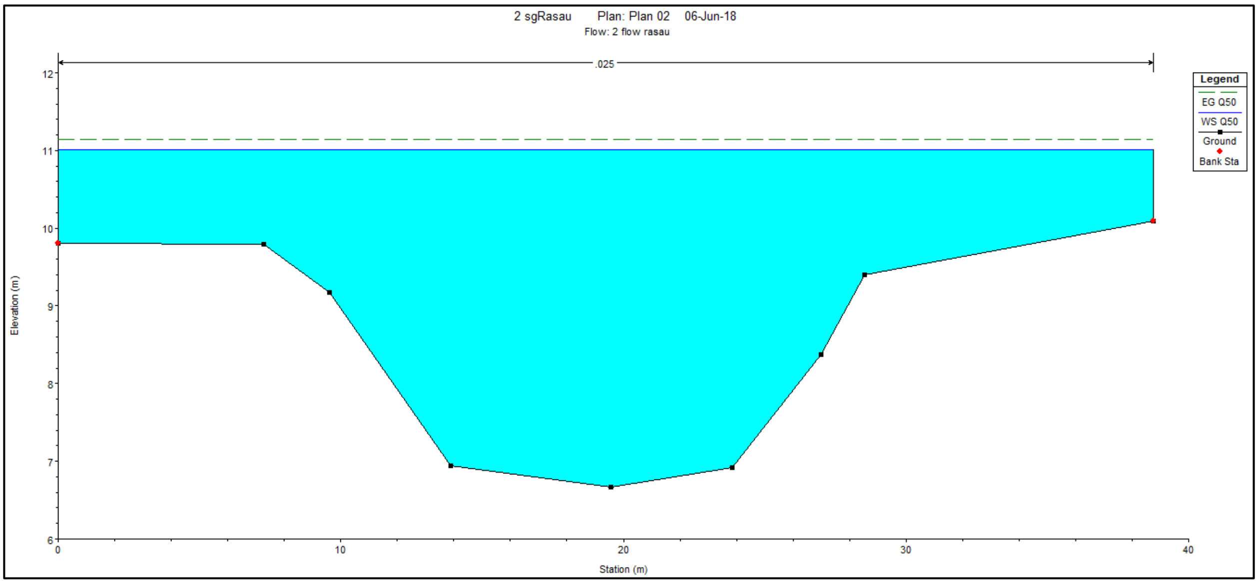

Figure 4.83 Water level at CH180 of Rasau River (with bridge piers) – Q_{20}

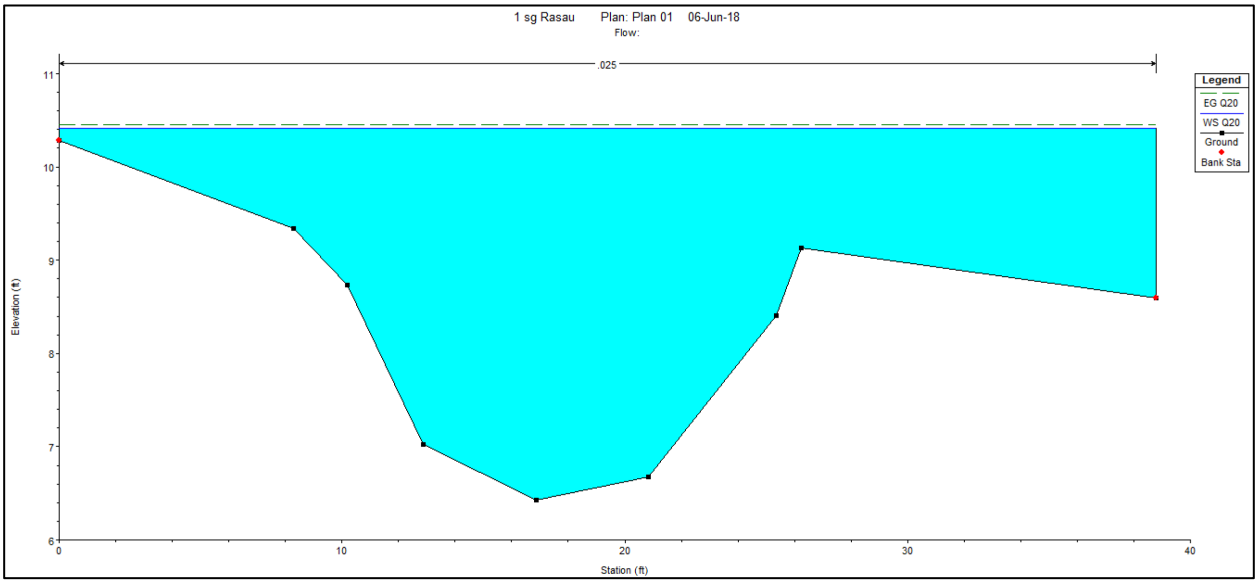

Figure 4.84 Water level at CH200 of Rasau River (without bridge piers) – Q_{20}

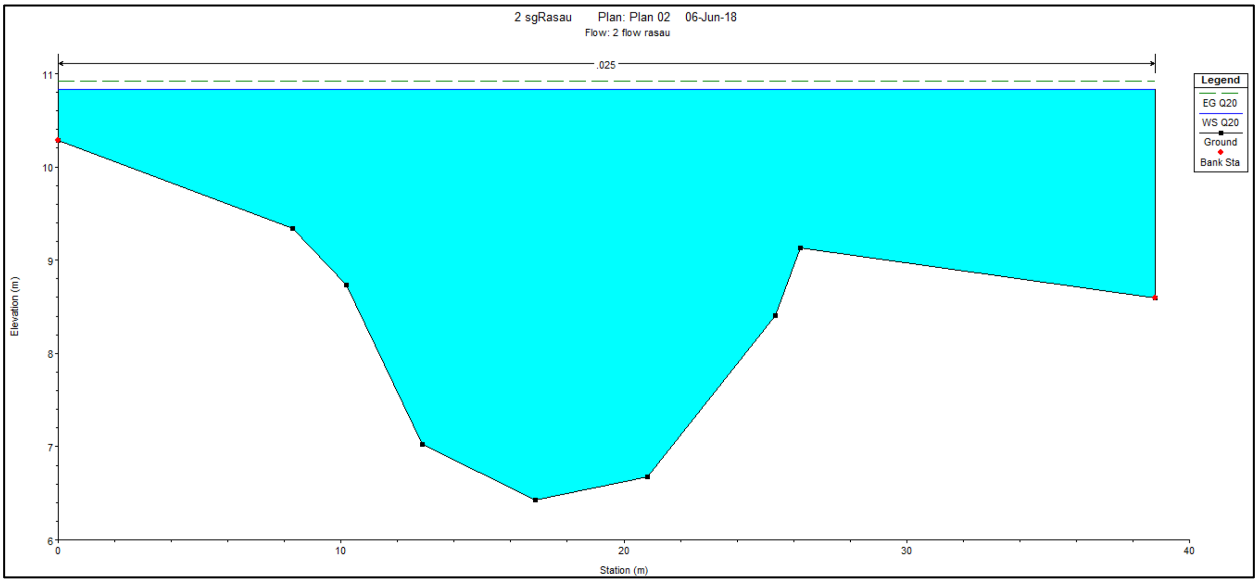

Figure 4.85 Water level at CH200 of Rasau River (with bridge piers) – Q_{20}

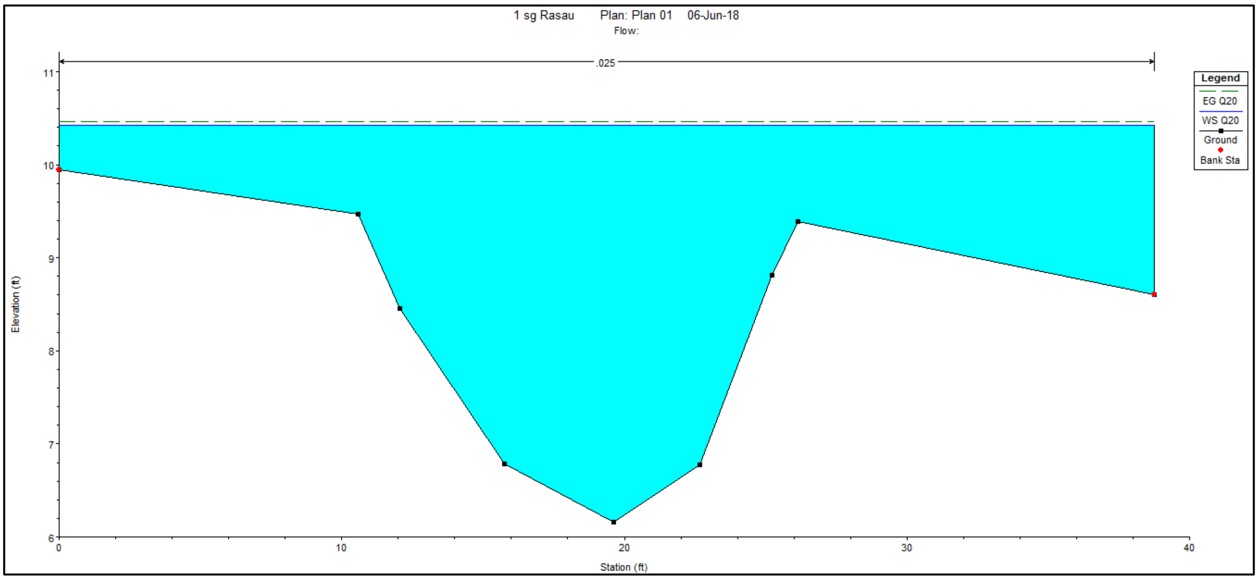

Figure 4.86 Water level at CH220 of Rasau River (without bridge piers) – Q_{20}

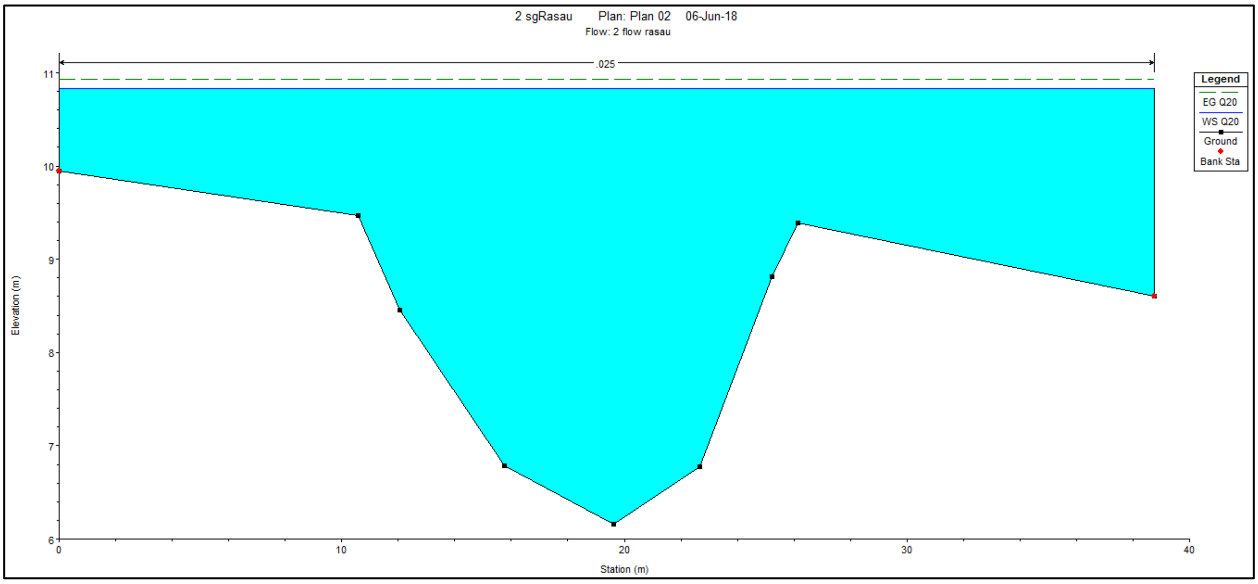

Figure 4.87 Water level at CH220 of Rasau River (with bridge piers) – Q_{20}

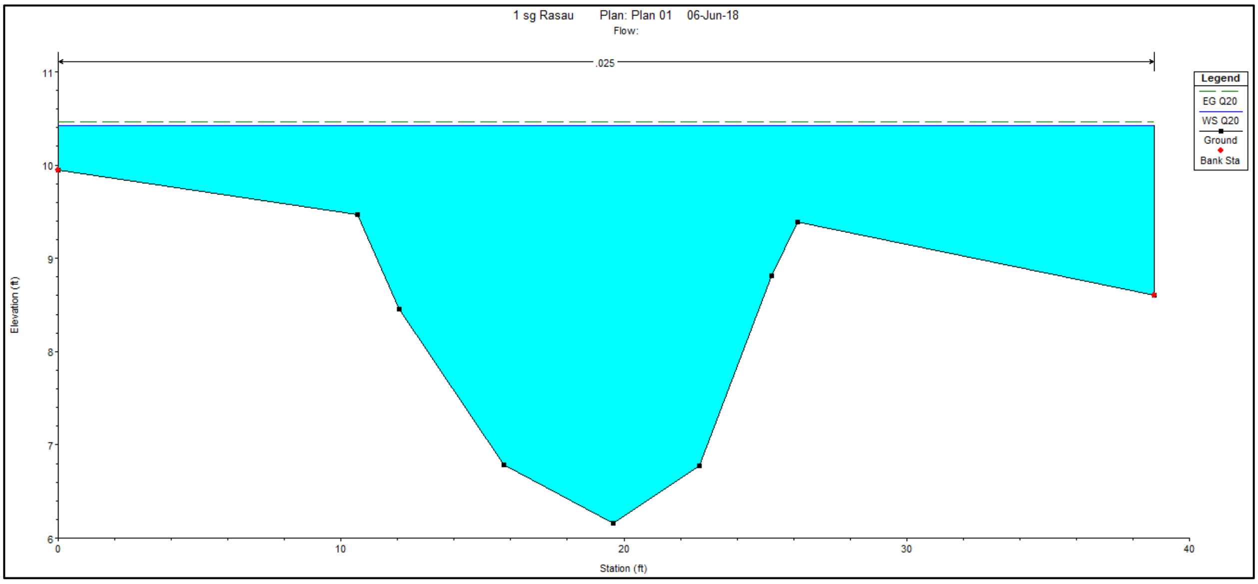

Figure 4.88 Water level at CH240 of Rasau River (without bridge piers) – Q_{20}

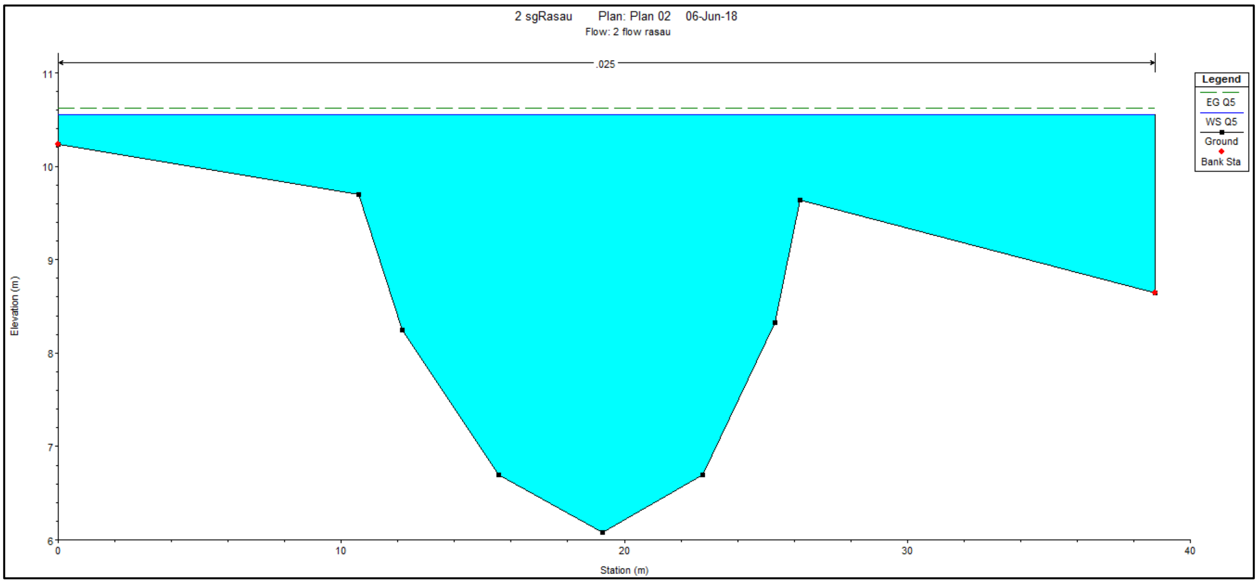

Figure 4.89 Water level at CH240 of Rasau River (with bridge piers) – Q_{20}

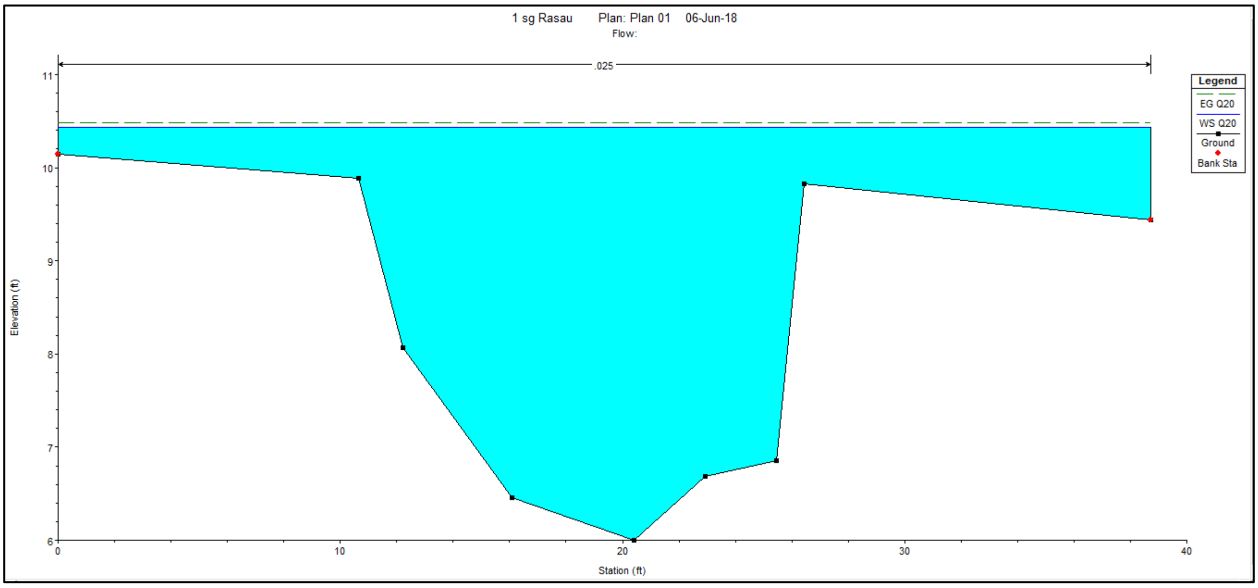

Figure 4.90 Water level at CH260 of Rasau River (without bridge piers) – Q_{20}

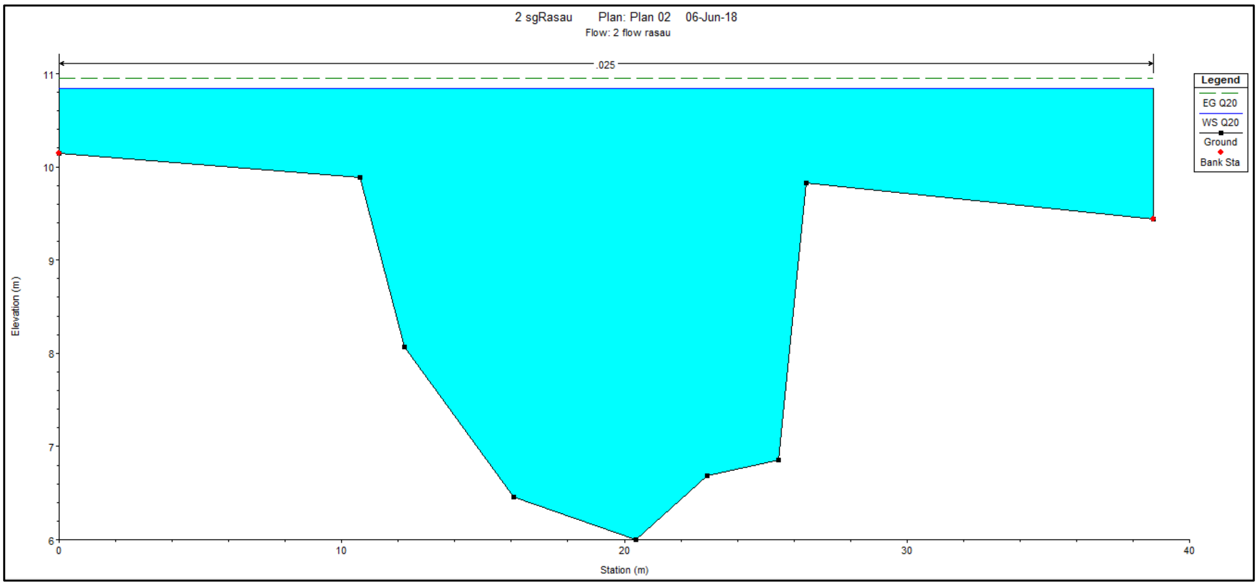

Figure 4.91 Water level at CH260 of Rasau River (with bridge piers) – Q_{20}

4.5.4 Water level for Q_{50} with and without bridge piers

The difference in water levels of Rasau River along the chainage with 50-year ARI is tabulated in Table 4.6 and water level profiles together with backwater effect were discussed.

Table 4.6 Difference of water level with Q_{50}

Q_{50} = 148.6 m^3/s						
Chainage	**Before bridge construction Water level (m)**		**After bridge construction Water level (m)**		**Difference of water level (m)**	
	Left bank	**Right bank**	**Left bank**	**Right bank**	**Left bank**	**Right bank**
CH 0	9.87	9.87	10.32	10.32	0.45	0.45
CH 20	10.26	10.26	10.72	10.72	0.46	0.46
CH 40	10.38	10.38	10.84	10.84	0.46	0.46
CH 60	10.39	10.39	10.9	10.90	0.51	0.51
CH 80	10.39	10.39	11.00	11.00	0.61	0.61
CH 100	10.34	10.34	11.00	11.00	0.66	0.66
CH 110 U	10.34	10.34	11.00	11.00	0.66	0.66
CH 110 D	10.33	10.33	11.01	11.01	0.68	0.68
CH 120	10.33	10.33	11.01	11.01	0.68	0.68
CH 140	10.49	10.49	11.02	11.02	0.53	0.53
CH 160	10.59	10.59	10.93	10.93	0.34	0.34
CH 180	10.59	10.59	11.01	11.01	0.42	0.42
CH 200	10.61	10.61	11.04	11.04	0.43	0.43
CH 220	10.61	10.61	11.04	11.04	0.43	0.43
CH 220	10.62	10.62	11.05	11.05	0.43	0.43
CH 240	10.63	10.63	11.05	11.05	0.42	0.42

4.5.4.1 Water level for Q_{50} at upstream of the bridge

Based on Table 4.6 and Figure 4.92 to Figure 4.103, it can be said that the water levels at the upstream of the river were between 9.87m to 10.34m above sea level without the bridge piers presence and between 10.32m to 11m with the presence of the bridge piers. This shows that, during rainfall event of 50-year ARI, the water levels were no longer contained in the river channel along CH 0 to CH 100 and the water were overflown onto both the left bank and right bank at all chainages.

Moreover, the difference in water levels along the upstream chainage is between 0.45m to 0.66m. This indicates that the upstream channel which starts from CH 0 to CH

100 has affected the increases in water levels up to 0.66m at CH 100 where the presence of Bridge 3 at Rasau River is located.

Apart from that, without the bridge piers presence, the water levels had overflown onto the left and right bank higher than normal condition at the same chainage. This situation shows that even though along the studied cross section should be already flooded with water as it without bridge piers presence,but the level was still at the safest level if compared with the water levels at each cross section with the presence of bridge piers.

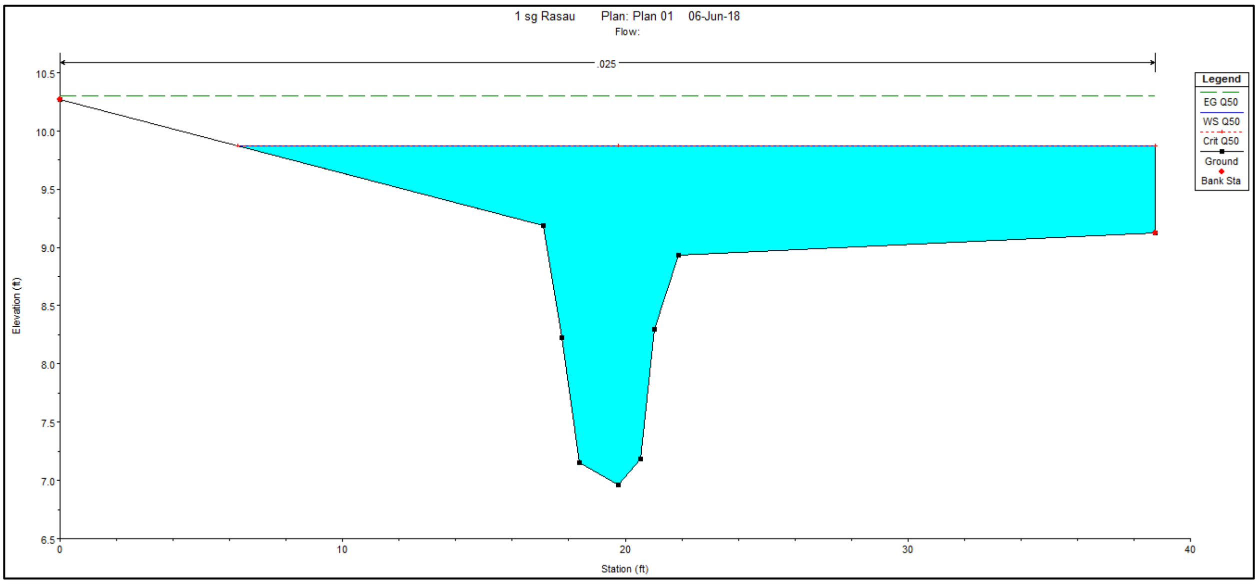

Figure 4.92 Water level at CH0 of the Rasau River (without bridge piers) – Q_{50}

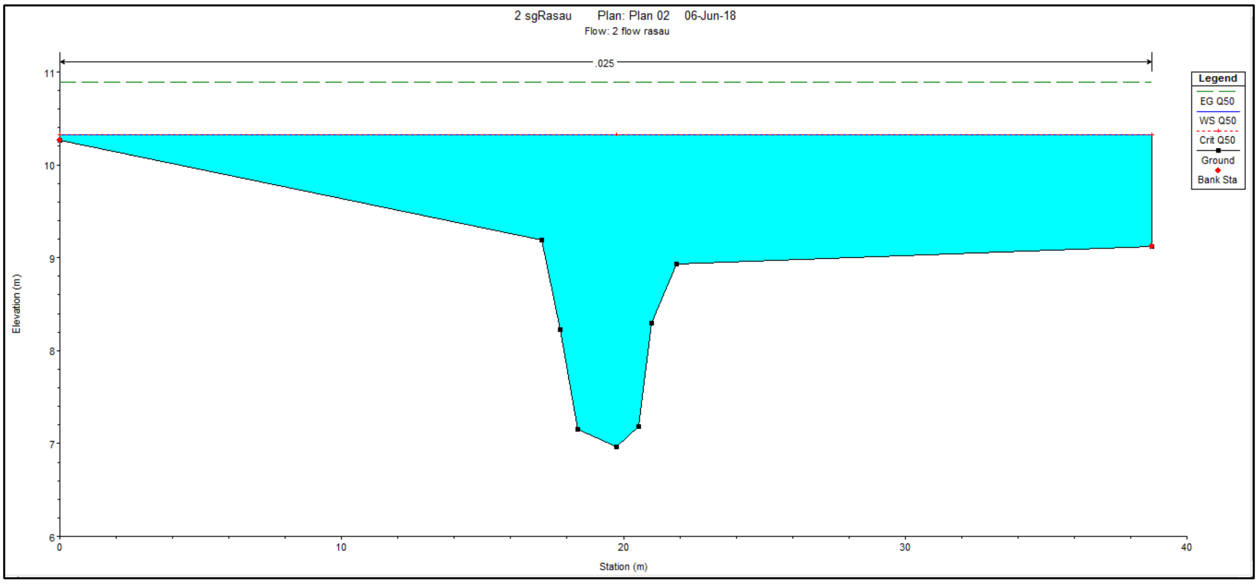

Figure 4.93 Water level at CH0 of the Rasau River (with bridge piers) – Q_{50}

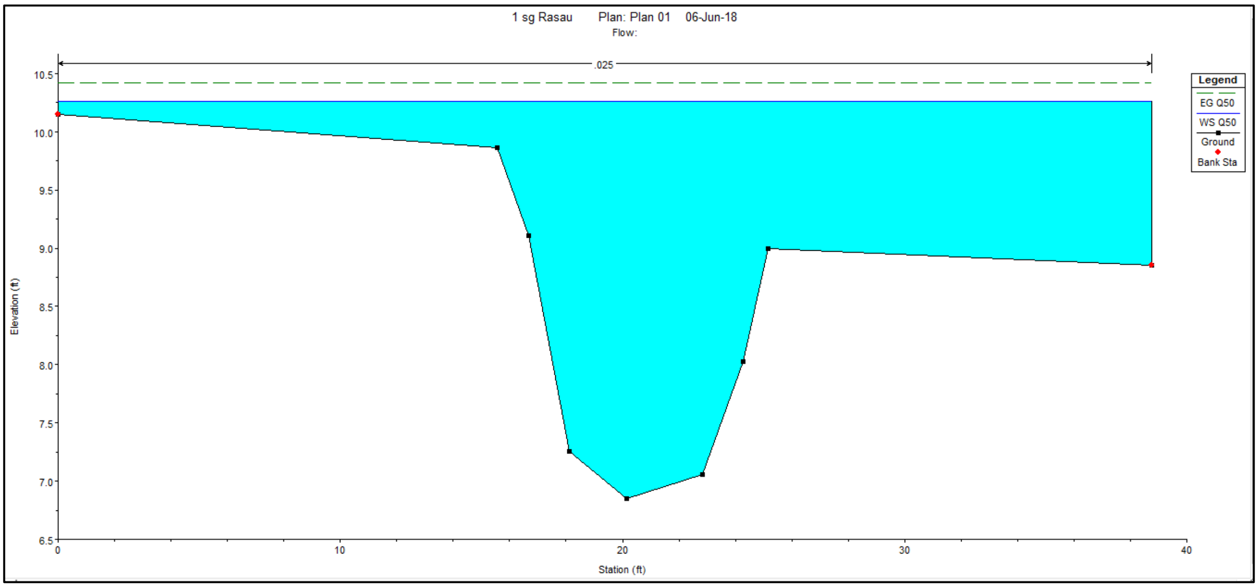

Figure 4.94 Water level at CH20 of Rasau River (without bridge piers) – Q_{50}

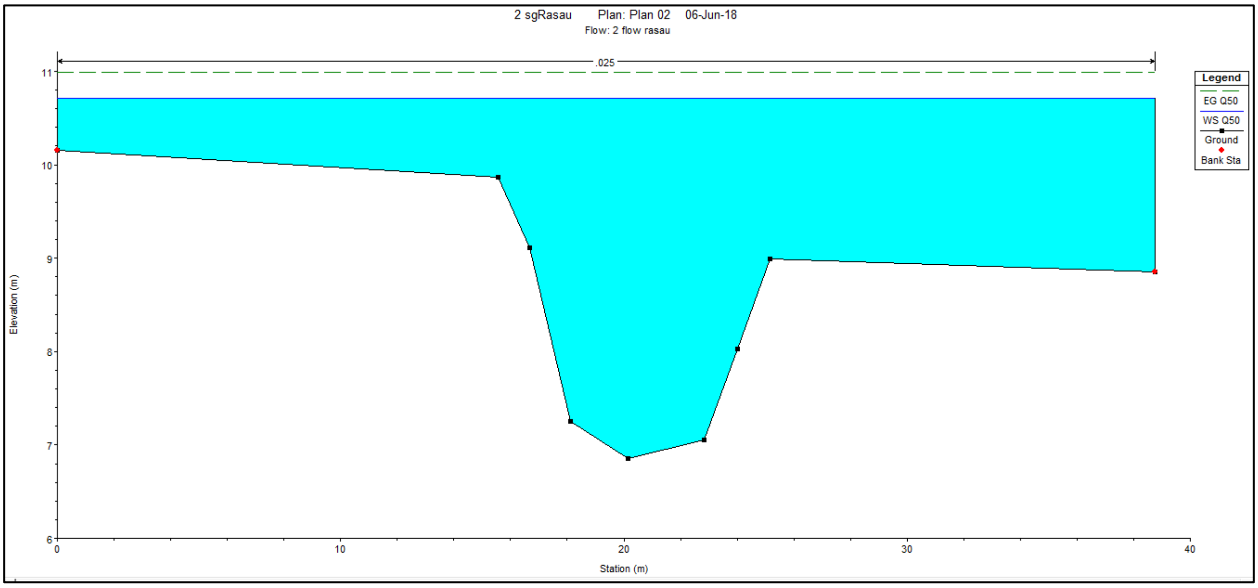

Figure 4.95 Water level at CH20 of Sg Rasau (with bridge piers) – Q_{50}

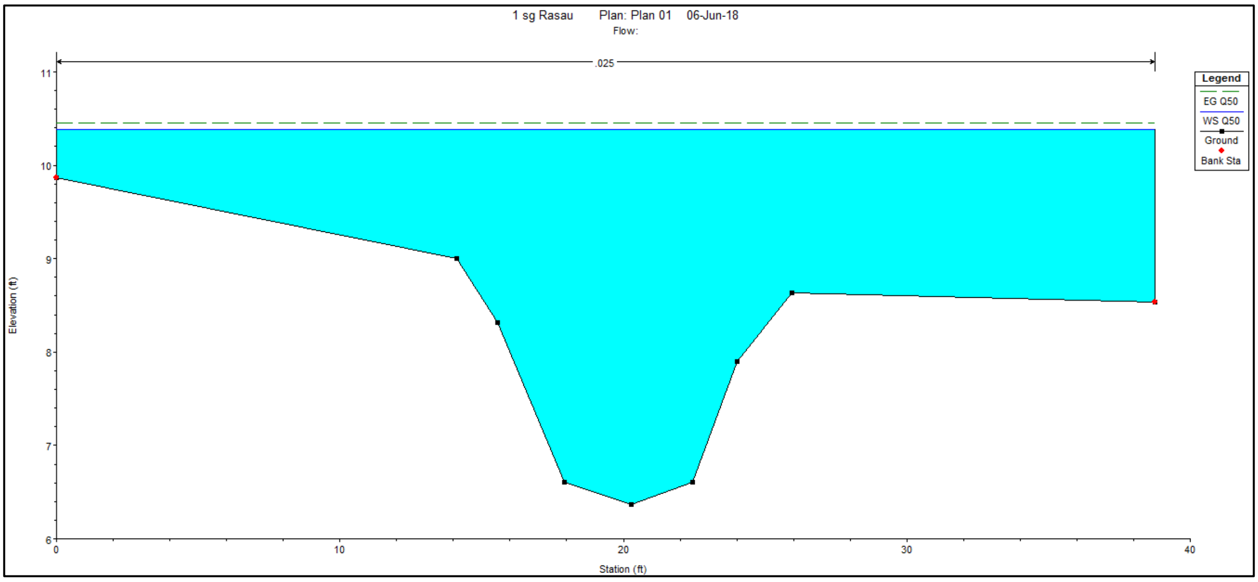

Figure 4.96 Water level at CH40 of Rasau River (without bridge piers) – Q_{50}

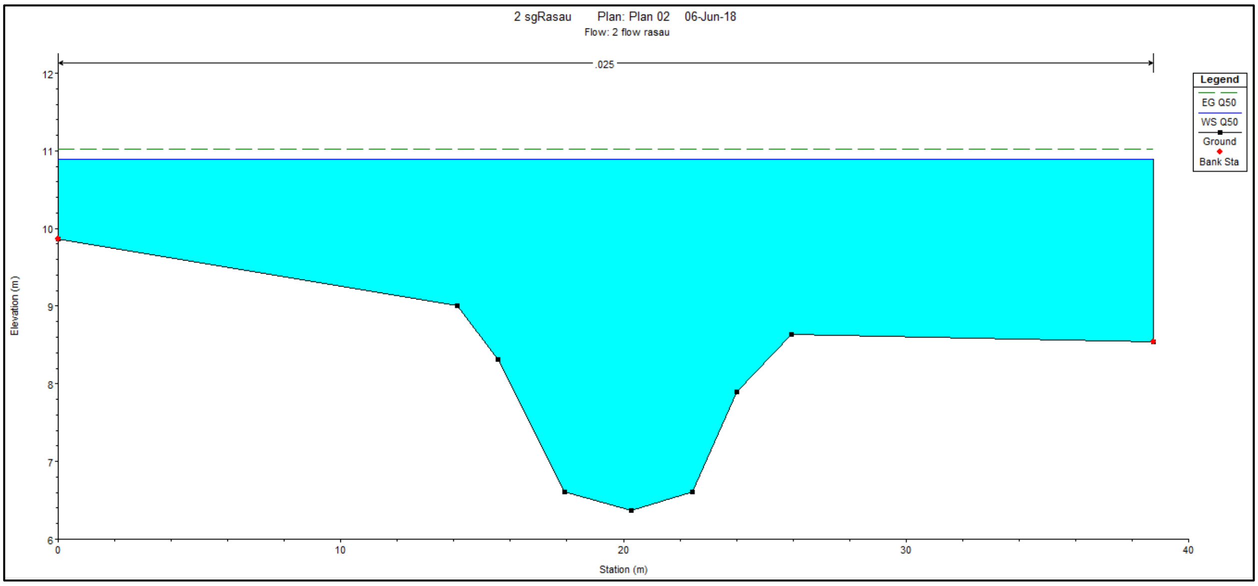

Figure 4.97 Water level at CH 40 of Rasau River (with bridge piers) – Q_{50}

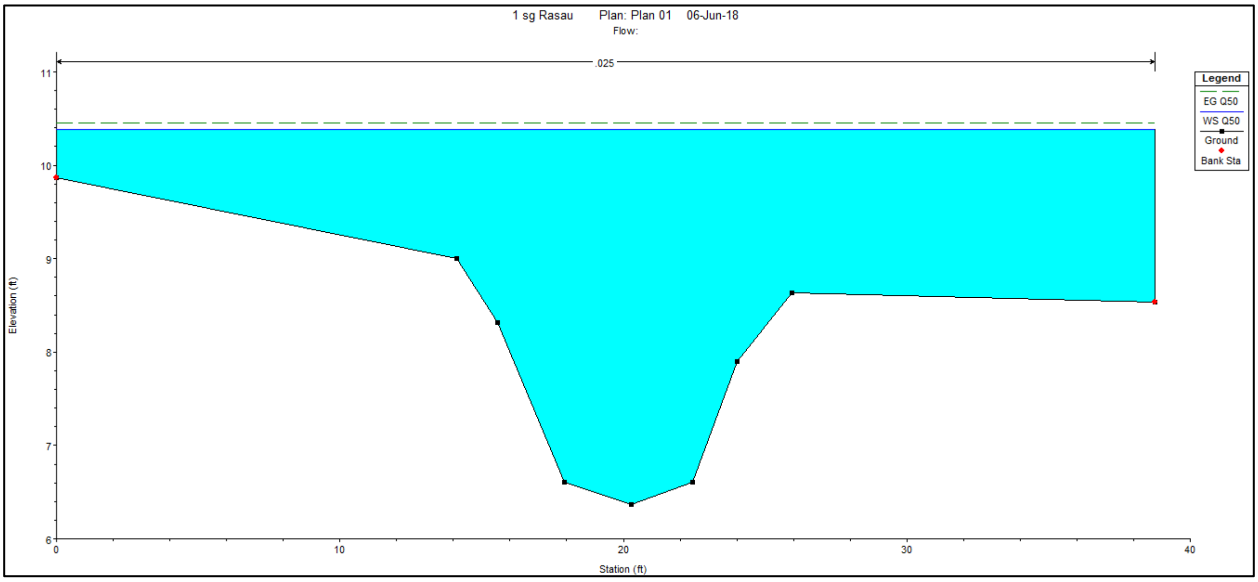

Figure 4.98 Water level at CH60 of Rasau River (without bridge piers) – Q_{50}

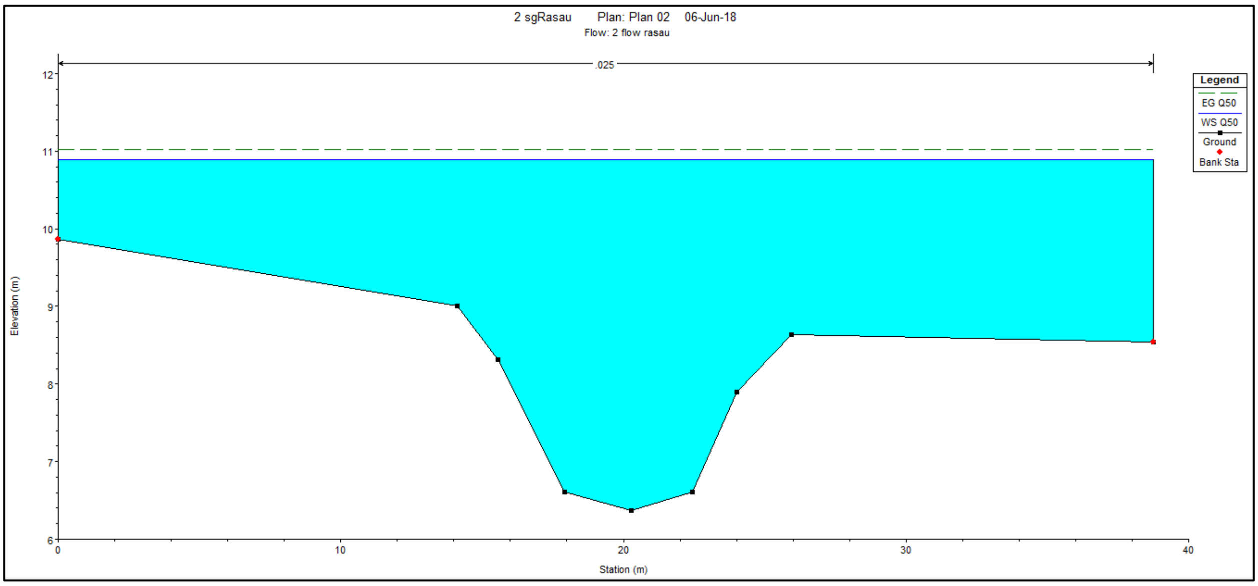

Figure 4.99 Water level at CH60 of Rasau River (with bridge piers) – Q_{50}

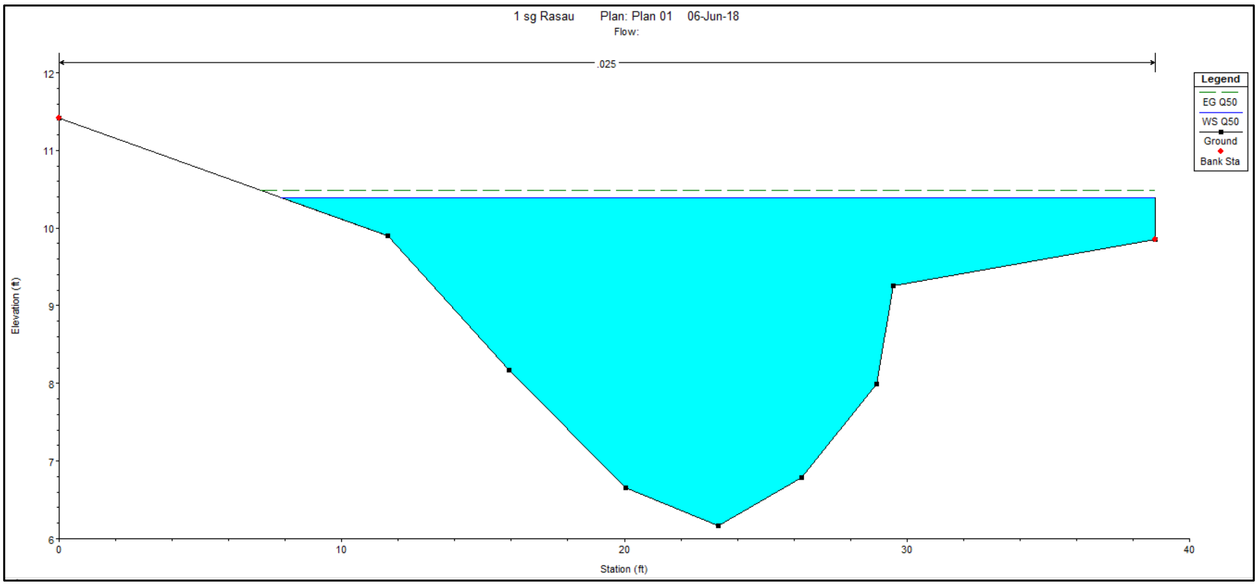

Figure 4.100 Water level at CH80 of Rasau River (without bridge piers) – Q_{50}

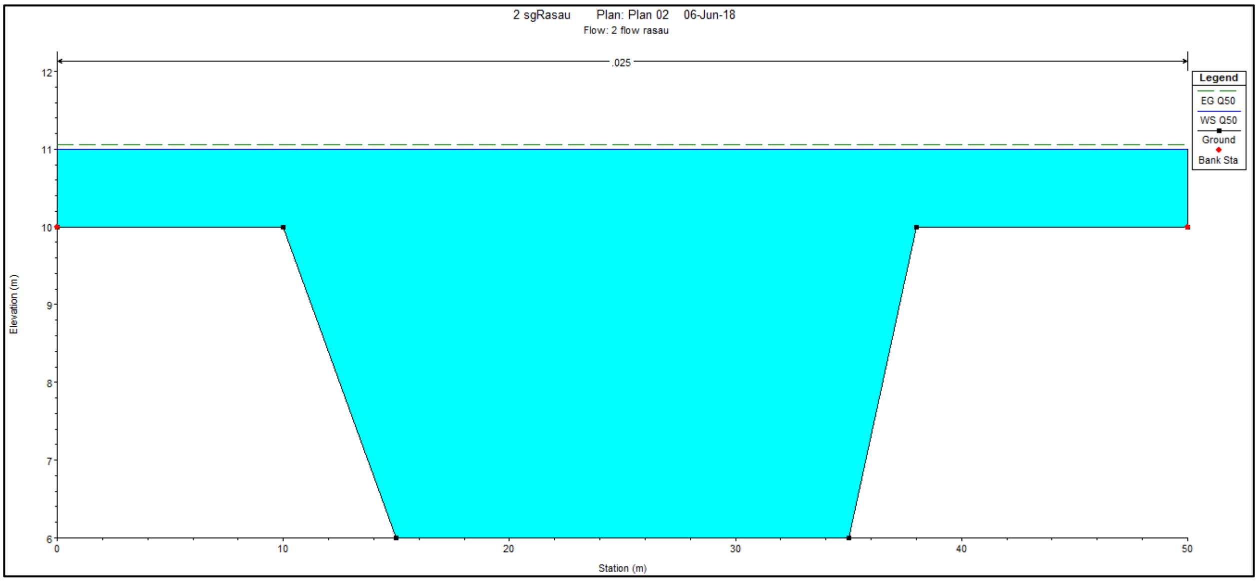

Figure 4.101 Water level at CH80 of Rasau River (with bridge piers) – Q_{50}

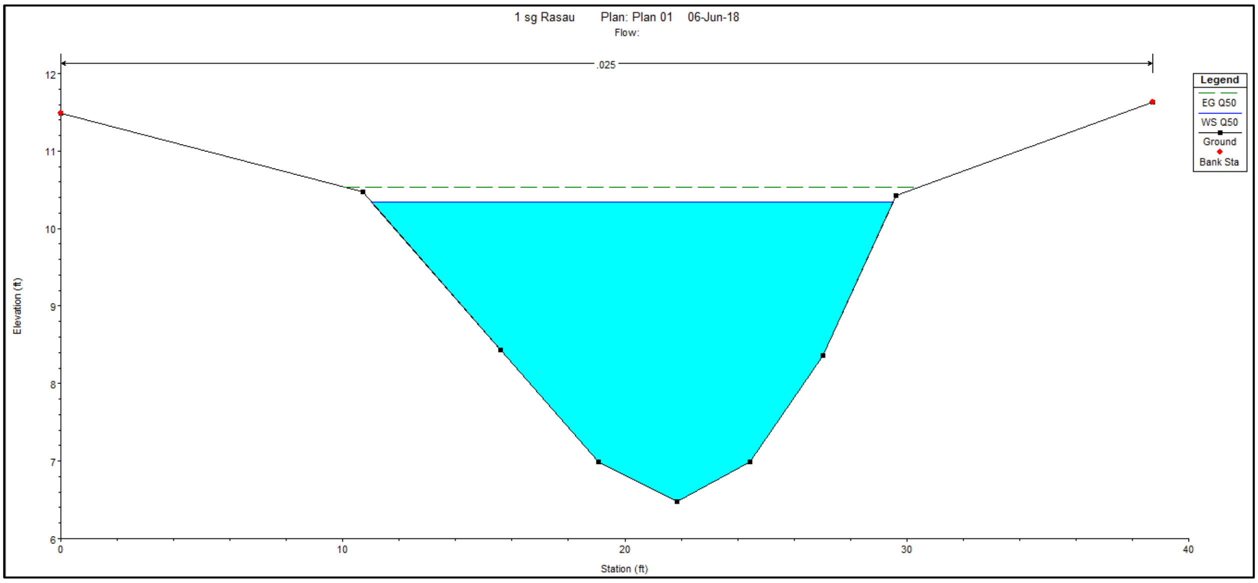

Figure 4.102 Water level at CH100 of Rasau River (without bridge piers) – Q_{50}

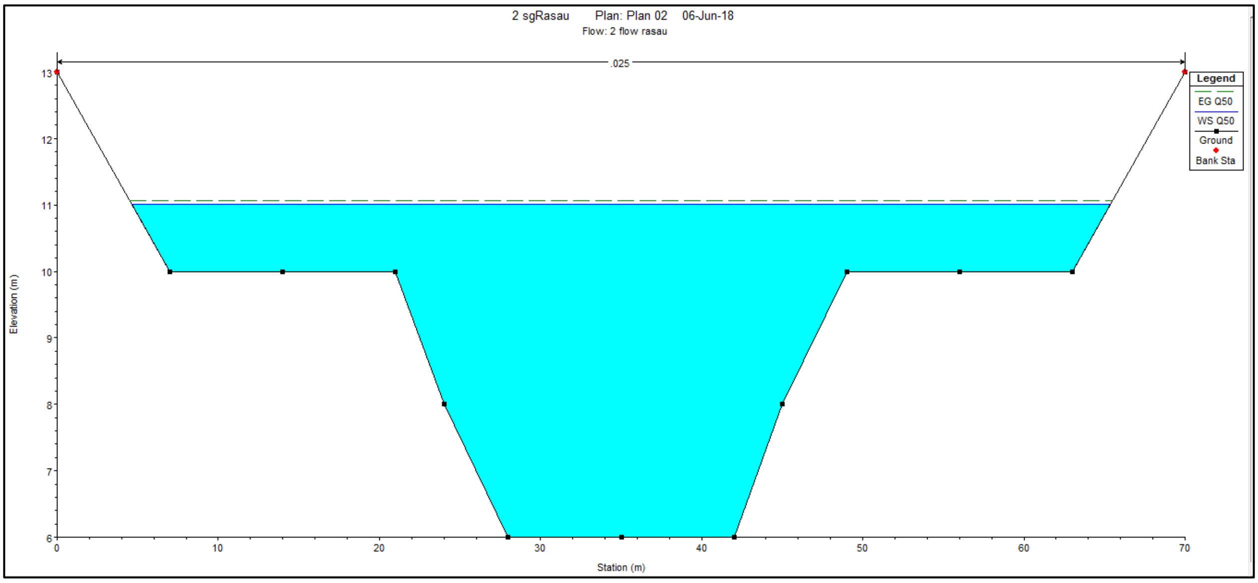

Figure 4.103 Water level at CH100 of Rasau River (with bridge piers) – Q_{50}

4.5.4.2 Water level for Q_{50} under the bridge

With reference to the Table 4.6 and Figure 4.104 to Figure 4.107, these simulations showed that the water levels at chainages where the bridge will be located were 10.33m to 10.34m above sea level. The water levels were contained in the river channel without overflown to the left and right bank. With the presence of bridge piers the water levels were between 11.0m to 11.01m.

Next, during rainfall event of 50-year ARI, the water levels under the bridge cross section which starts from CH 100 to CH 120 has shown that the difference in water levels between the two conditions were between 0.66m to 0.68m. Water levels condition below the bridge indicated that the backwater effect increased the water levels up to 0.68m more but only at level of 11.01m above sea level. This means that the water will not overflow onto the road level which at 15.92m.

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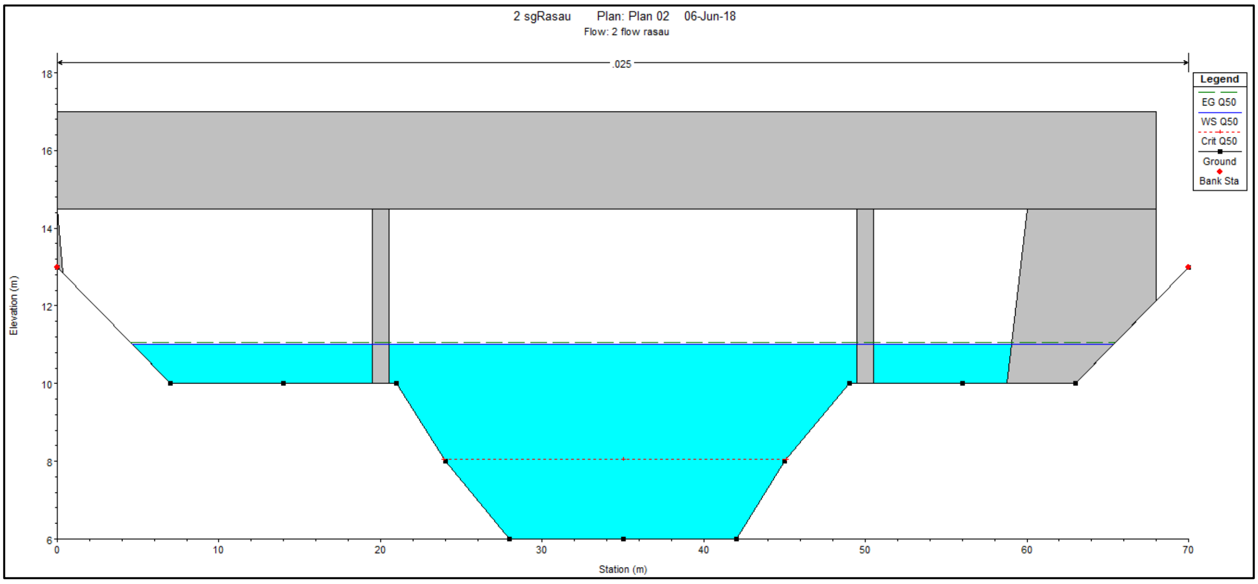

Figure 4.104 Water levels at CH110 downstream of Rasau River (with bridge piers) – Q_{50}

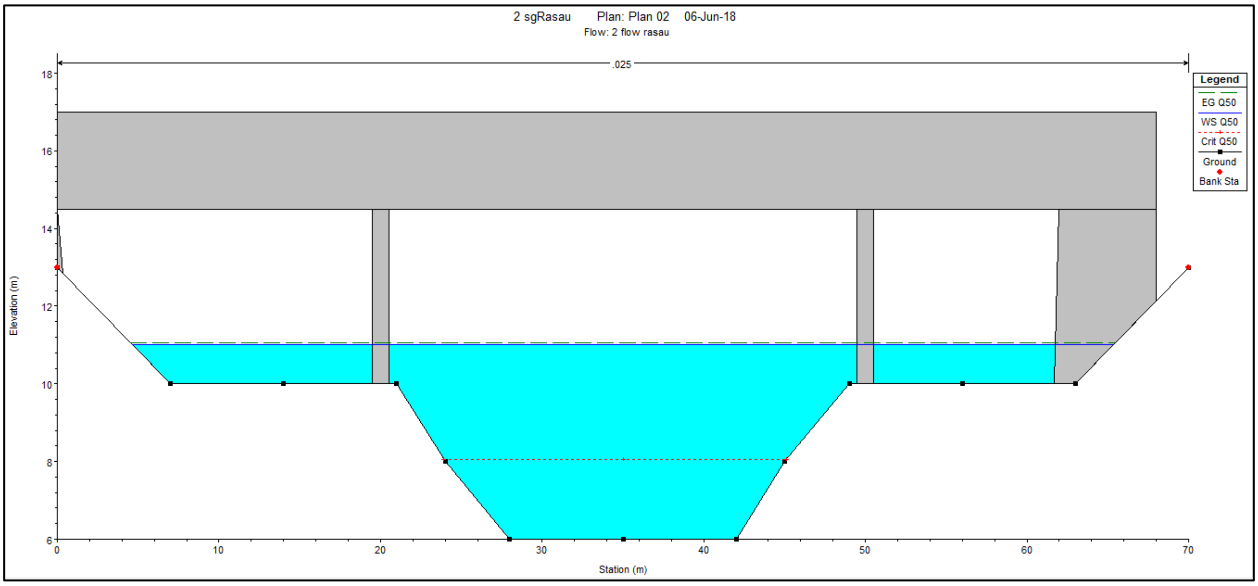

Figure 4.105 Water level at CH110 upstream of Rasau River (with bridge piers) – Q_{50}

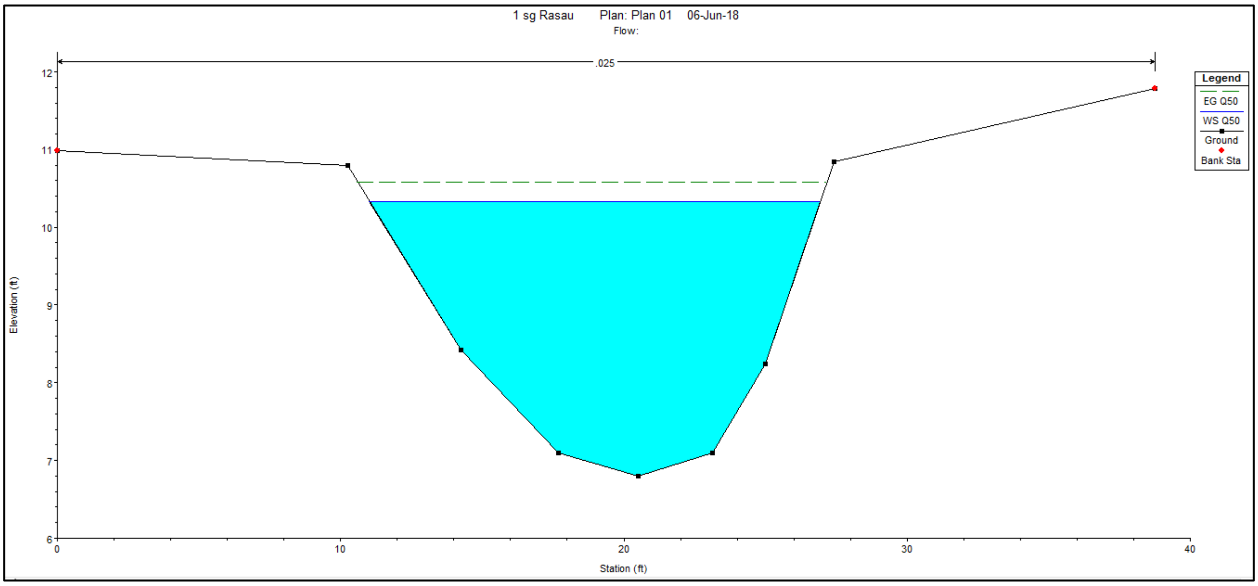

Figure 4.106 Water level at CH120 of Rasau River (without bridge piers) – Q_{50}

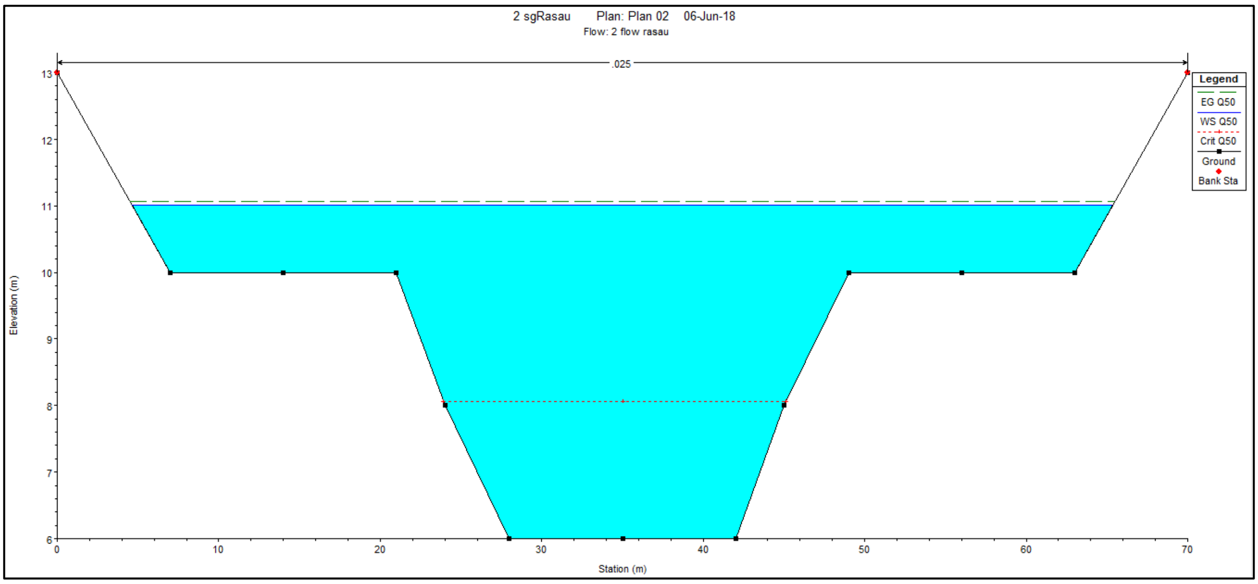

Figure 4.107 Water level at CH120 of Rasau River (with bridge piers). – Q_{50}

4.5.4.3 Water level for Q_{50} at downstream of the bridge

Based on Table 4.6 and Figure 4.108 to Figure 4.121, water levels were overflown along CH 140 to CH 260 except at CH 140(without bridge piers) and CH 160(without bridge piers) because of its higher level of left banks. Water levels along the downstream chainages were between 10.49m to 10.63m above sea level without the bridge piers presence and between 10.93m and 11.05m with the bridge piers presence.

In addition, the water level along the downstream cross section of the river also can be discussed. This study shows that the water levels from CH 120 to CH 240 had slightly fluctuated. The differences in water levels are from 0.34m to 0.68m different along the chainage with 50-year ARI condition.

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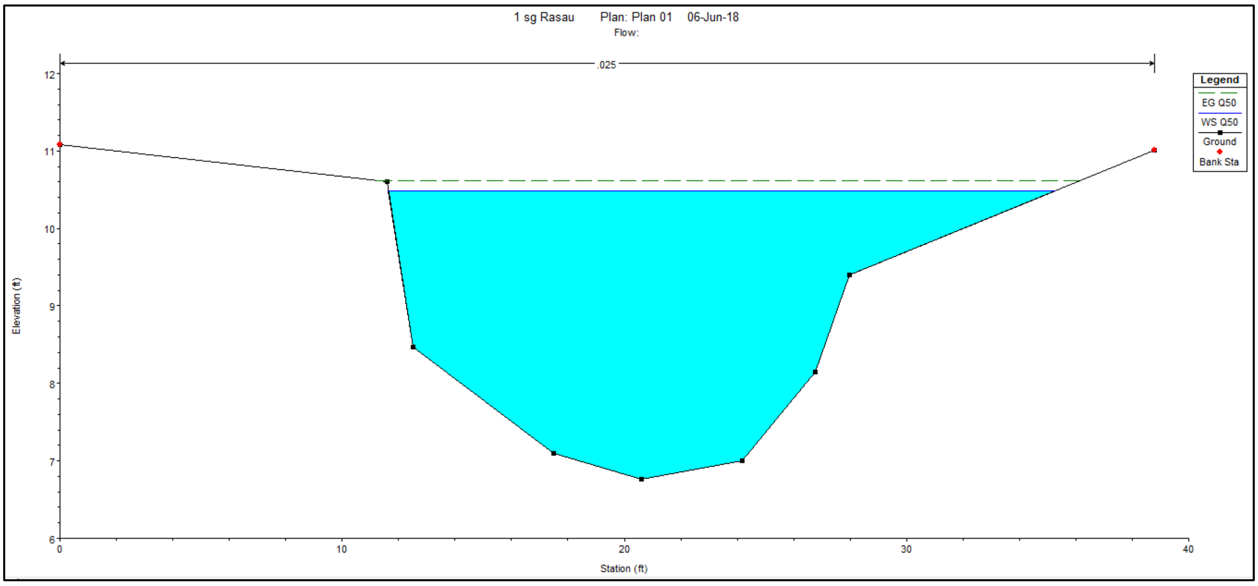

Figure 4.108 Water level at CH140 of Rasau River (without bridge piers) – Q_{50}

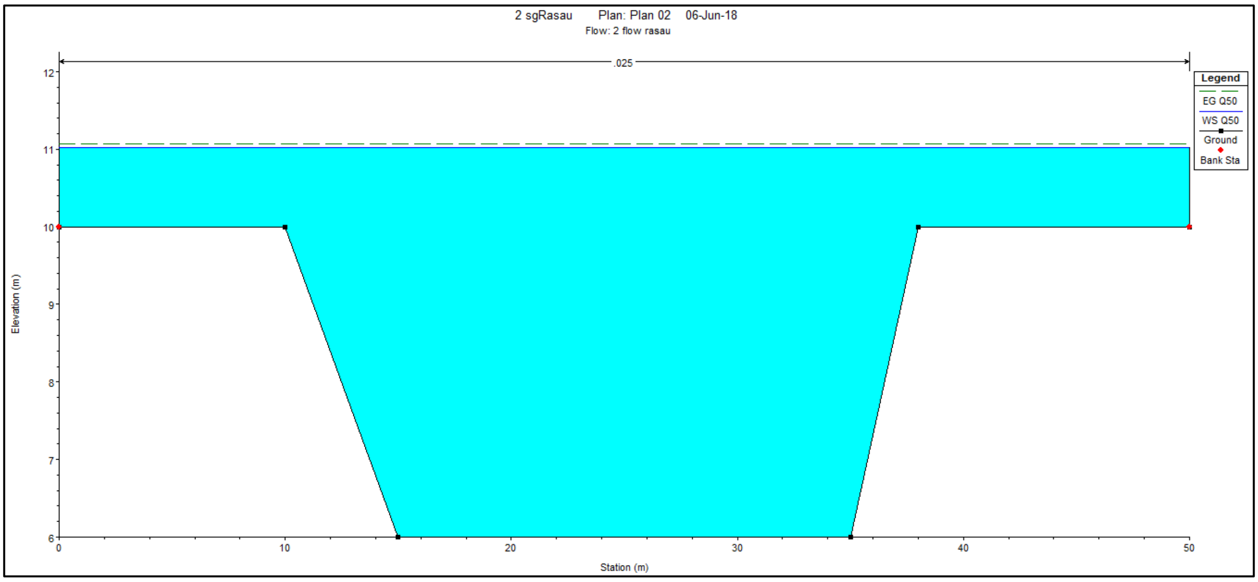

Figure 4.109 Water level at CH140 of Rasau River (with bridge piers) – Q_{50}

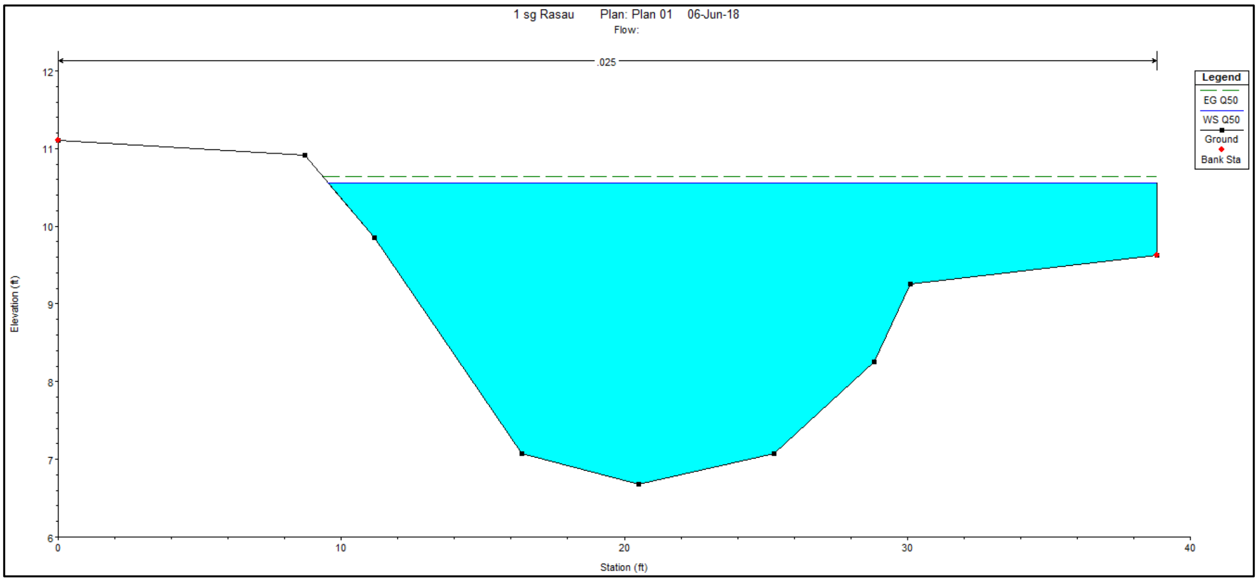

Figure 4.110 Water level at CH160 of Rasau River (without bridge piers) – Q_{50}

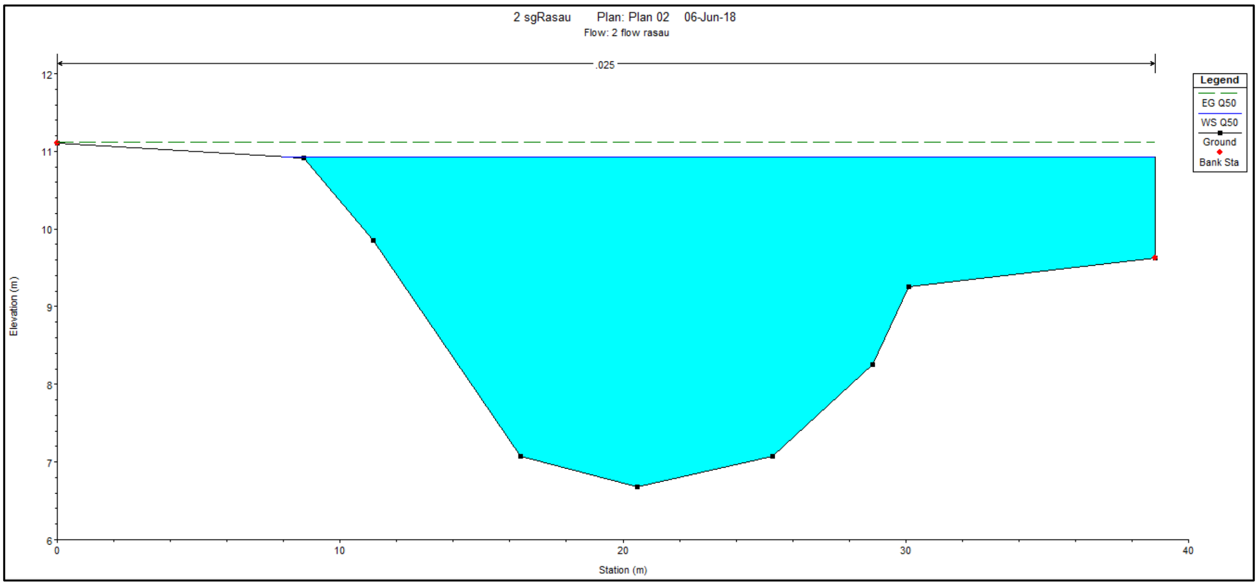

Figure 4.111 Water level at CH160 of Rasau River (with bridge piers) – Q_{50}

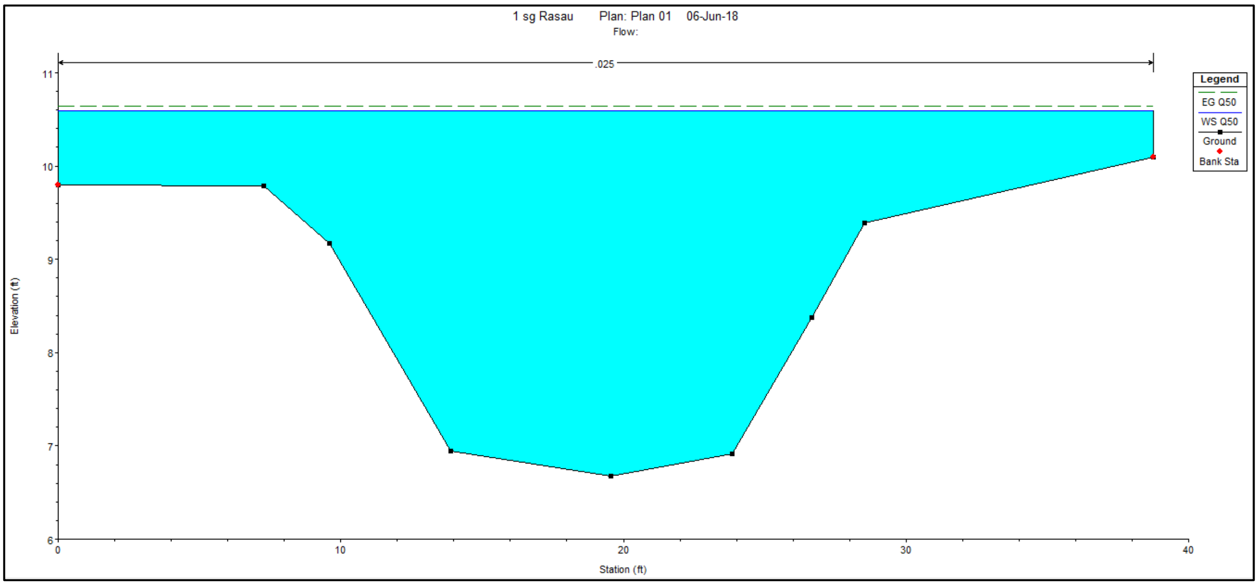

Figure 4.112 Water level at CH180 of Rasau River (without bridge piers) – Q_{50}

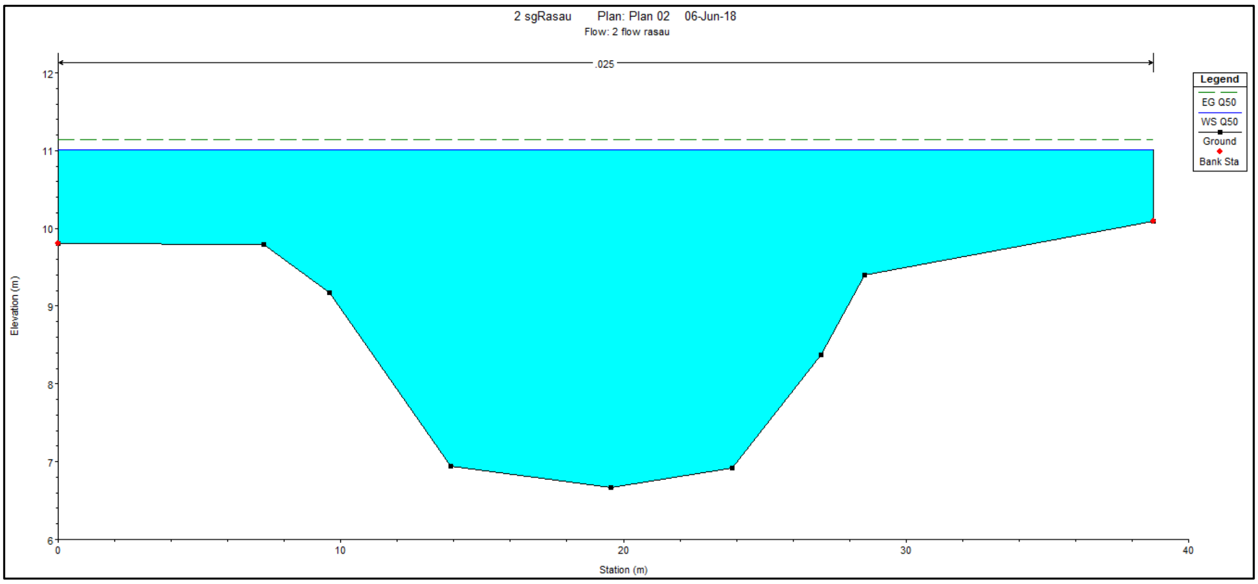

Figure 4.113 Water level at CH180 of Rasau River (with bridge piers) – Q_{50}

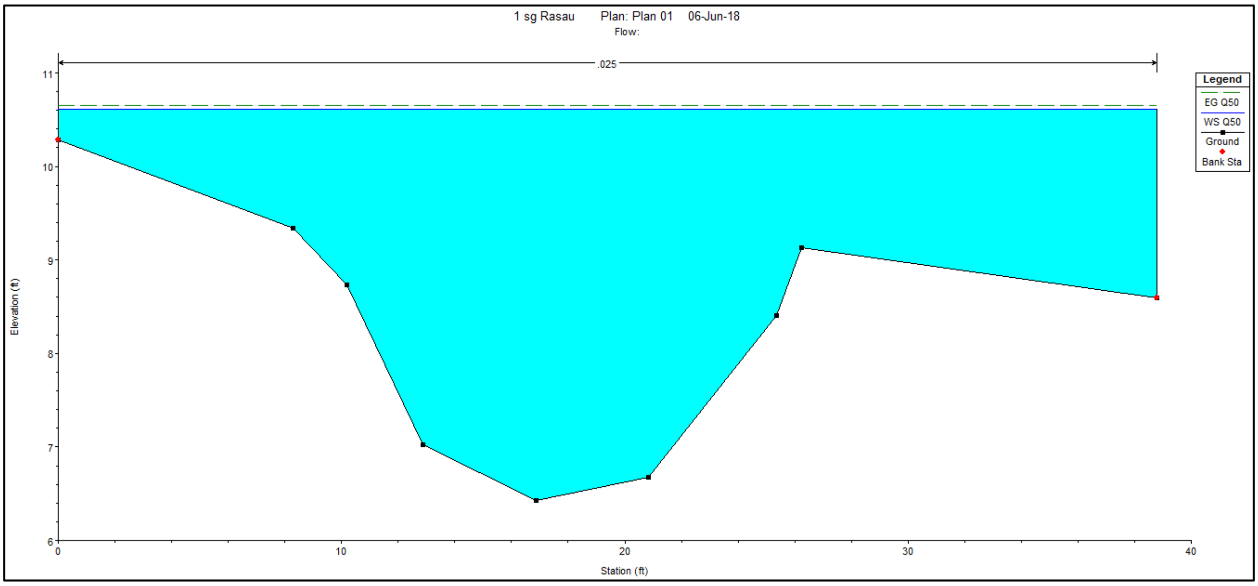

Figure 4.114 Water level at CH200 of Rasau River (without bridge piers) – Q_{50}

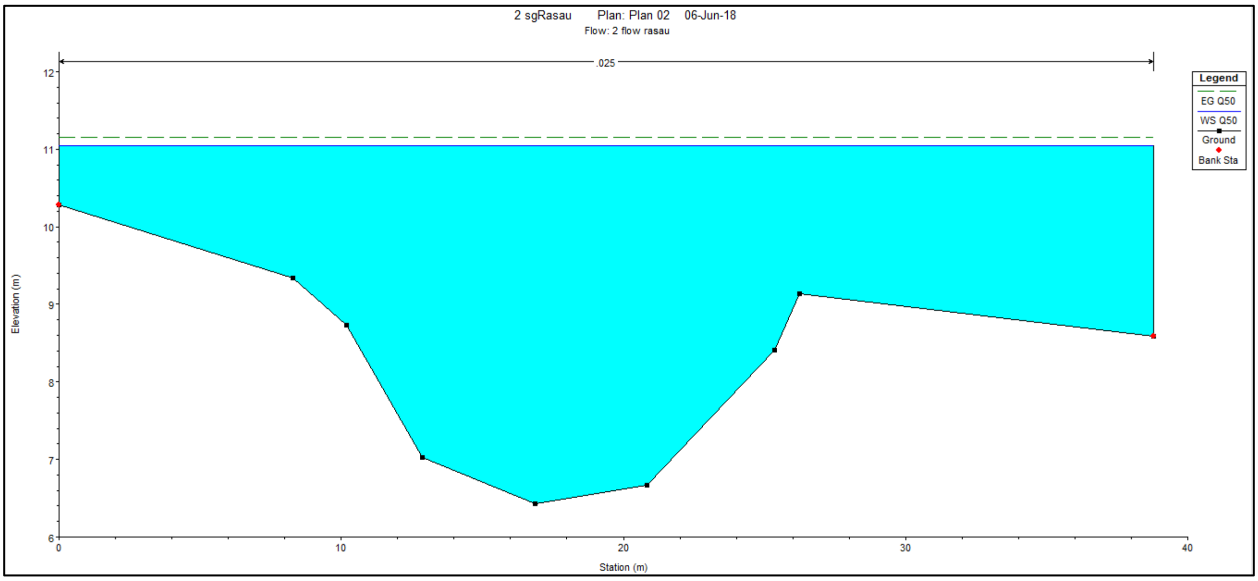

Figure 4.115 Water level at CH200 of Rasau River (with bridge piers) – Q_{50}

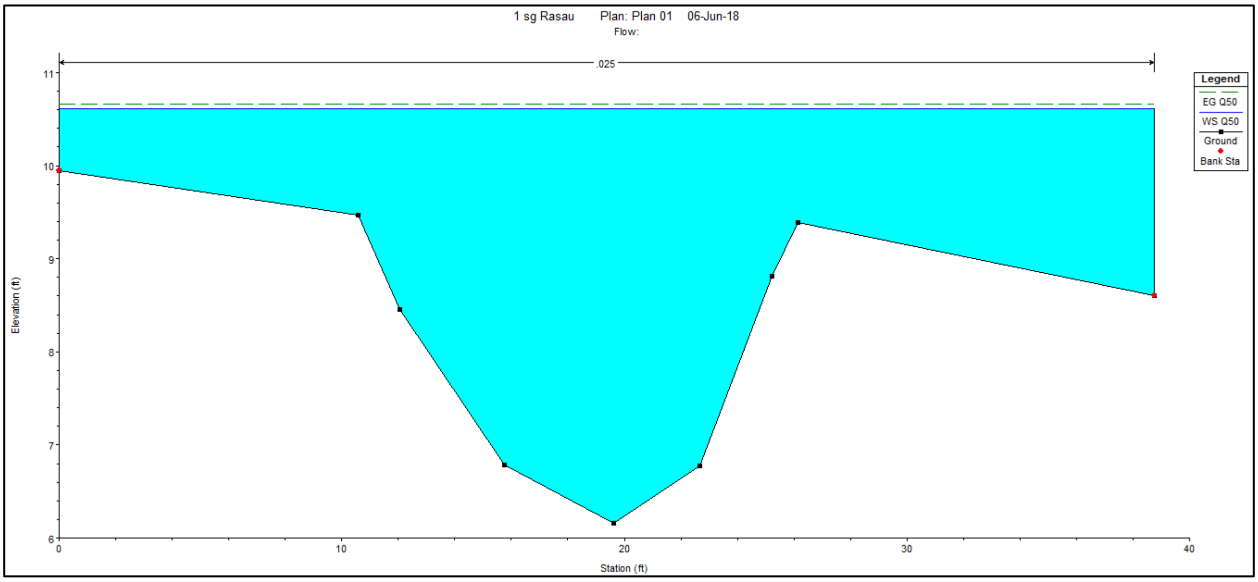

Figure 4.116 Water level at CH220 of Rasau River (without bridge piers) – Q_{50}

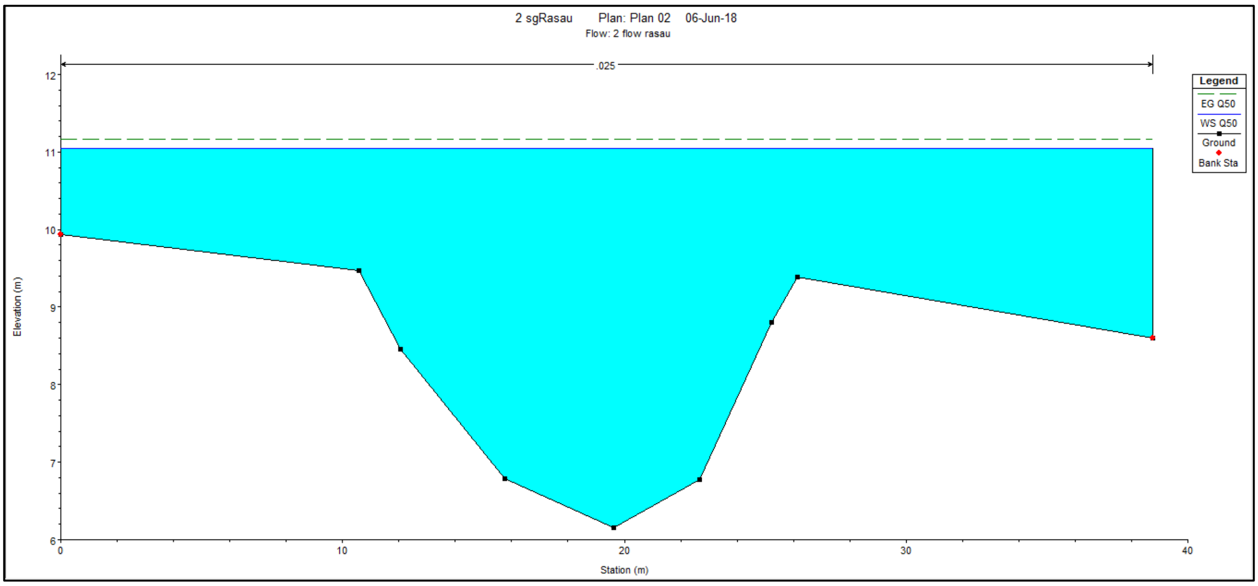

Figure 4.117 Water level at CH220 of Rasau River (with bridge piers) – Q_{50}

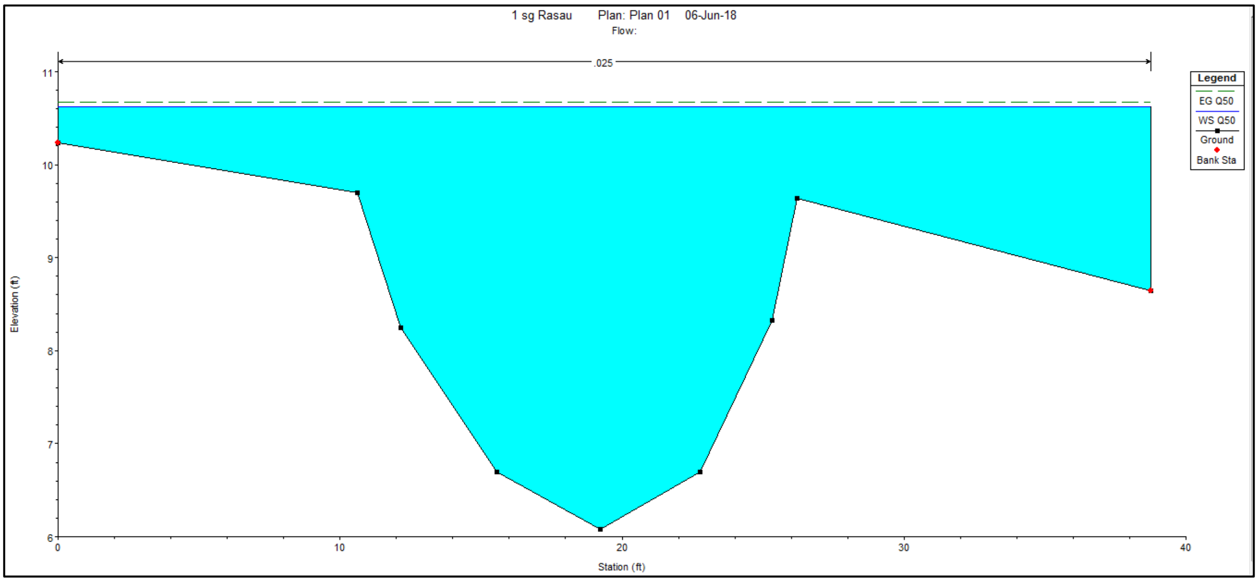

Figure 4.118 Water level at CH240 of Rasau River (without bridge piers) – Q_{50}

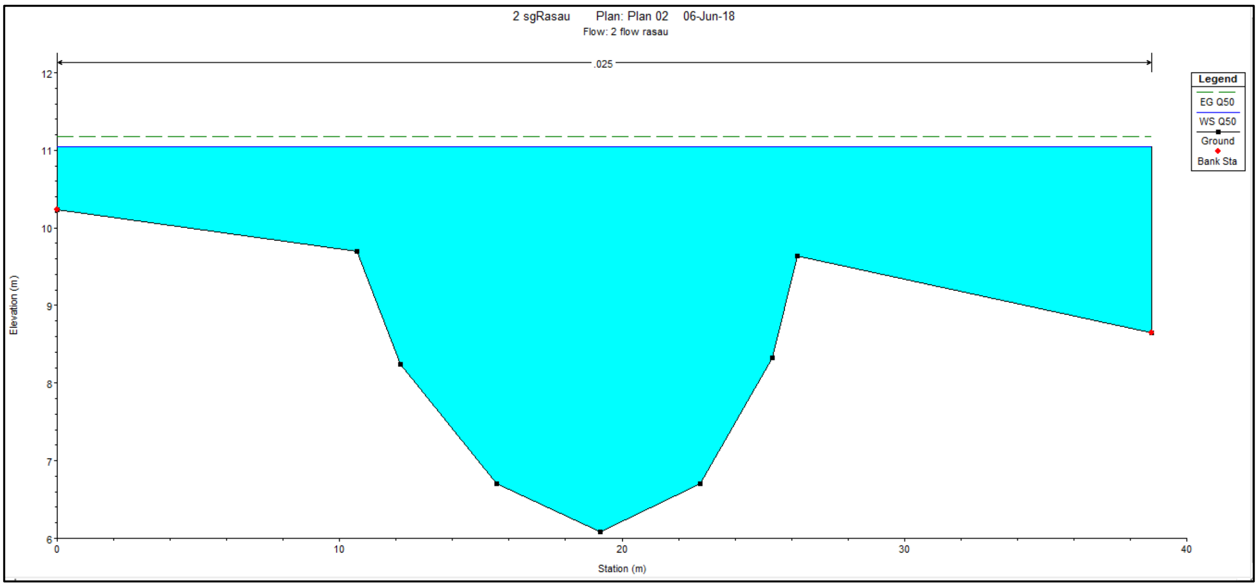

Figure 4.119 Water level at CH240 of Rasau River (with bridge piers) – Q_{50}

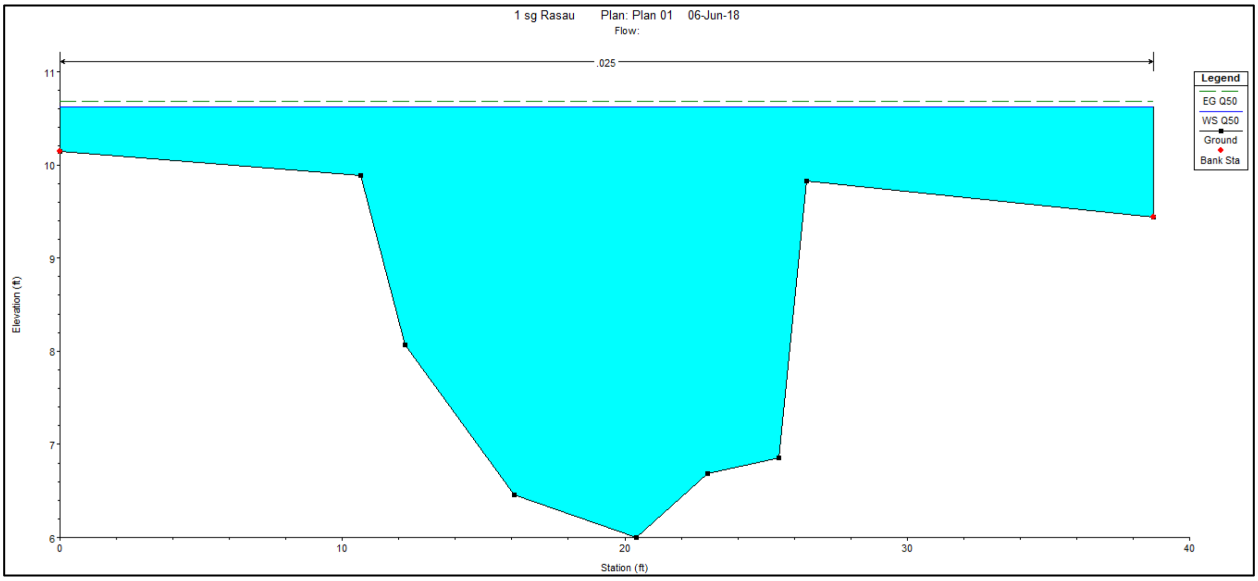

Figure 4.120 Water level at CH260 of Rasau River (without bridge piers) – Q_{50}

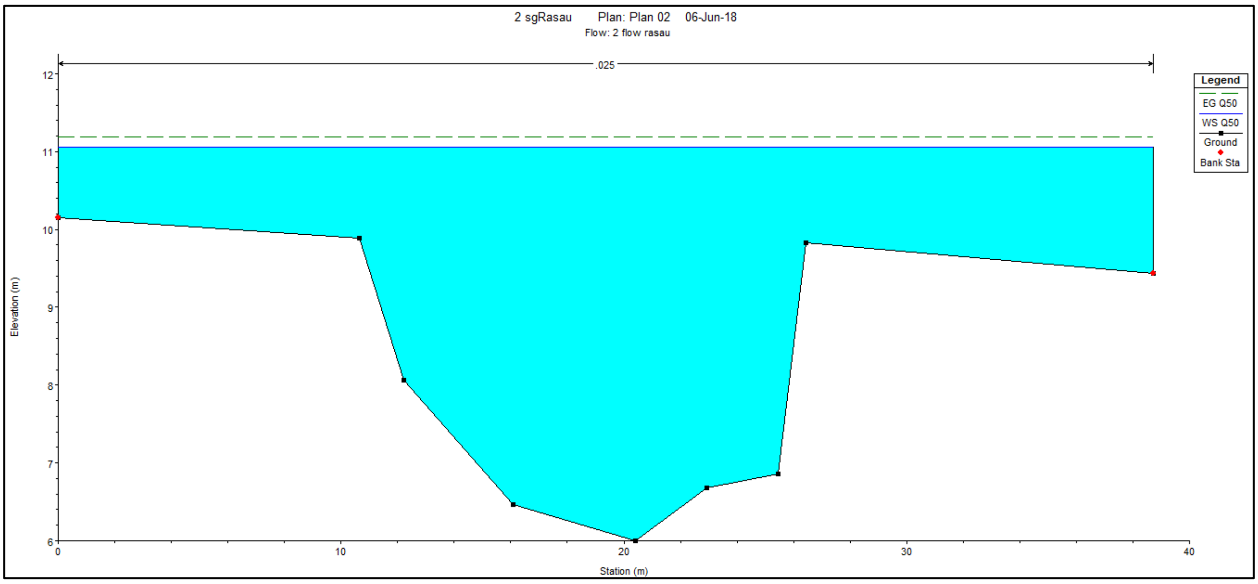

Figure 4.121 Water level at CH260 of Rasau River (with bridge piers) – Q_{50}

4.5.5 Water level for Q_{100} with and with and without bridge piers

The difference in water levels of Rasau River along the chainage with 100-year ARI is tabulated in Table 4.7 and water level profiles together with backwater effect were discussed.

Table 4.7 Difference of water level with Q_{100}

Q_{100} = 172.0 m^3/s						
Chainage	**Before bridge construction Water level (m)**		**After bridge construction Water level (m)**		**Difference of water level (m)**	
	Left bank	**Right bank**	**Left bank**	**Right bank**	**Left bank**	**Right bank**
CH 0	9.97	9.97	10.44	10.44	0.47	0.47
CH 20	10.38	10.38	10.86	10.86	0.48	0.48
CH 40	10.51	10.51	11.05	11.05	0.54	0.54
CH 60	10.52	10.52	11.15	11.15	0.63	0.63
CH 80	10.51	10.51	11.18	11.18	0.67	0.67
CH 100	10.45	10.45	11.18	11.18	0.73	0.73
CH 110 U	10.45	10.45	11.17	11.17	0.72	0.72
CH 110 D	10.44	10.44	11.18	11.18	0.74	0.74
CH 120	10.44	10.44	11.19	11.19	0.75	0.75
CH 140	10.64	10.64	11.20	11.20	0.56	0.56
CH 160	10.74	10.74	11.09	11.09	0.45	0.45
CH 180	10.75	10.75	11.18	11.18	0.43	0.43
CH 200	10.77	10.77	11.22	11.22	0.45	0.45
CH 220	10.78	10.78	11.23	11.23	0.45	0.45
CH 220	10.78	10.78	11.23	11.23	0.48	0.48
CH 240	10.79	10.79	11.24	11.24	0.45	0.45

4.5.5.1 Water level for Q_{100} at upstream the bridge

Based on Table 4.7 and Figure 4.122 to Figure 133, it can be said that the water levels at the upstream of the river were between 9.97m to 10.38m above sea level without the bridge piers condition and between 10.44 and 11.18m with the bridge piers presence. This shows that, during rainfall event of 100-year ARI, the water levels were surpassed the river bank along CH 0 to CH 100 and the water is overflown onto both the left bank and right bank at all chainage.

The differences in water levels along the upstream chainage which start from CH 0 to CH 100 were between 0.47m to 0.73m. This indicates that with the presence of

Bridge 3 at the Rasau River has affected the increase in water levels up to 0.73m at CH 100 where the bridge piers are located.

Moreover, this situation shows that the water levels had overflown onto the left and right bank higher than normal condition at the same chainage without the bridge piers presence. Even though along the studied cross section should be already flooded with water but the level was still at the safest level if compared with the water levels at each chainage or cross section with the presence of bridge piers.

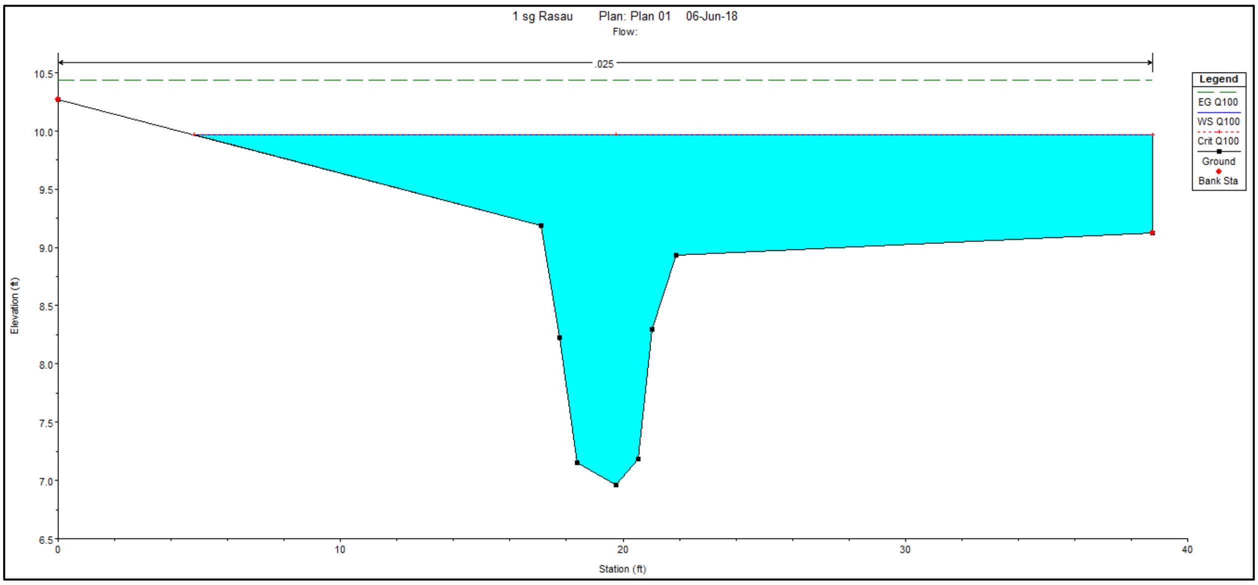

Figure 4.122 Water level at CH0 of the Rasau River (without bridge piers) – Q_{100}

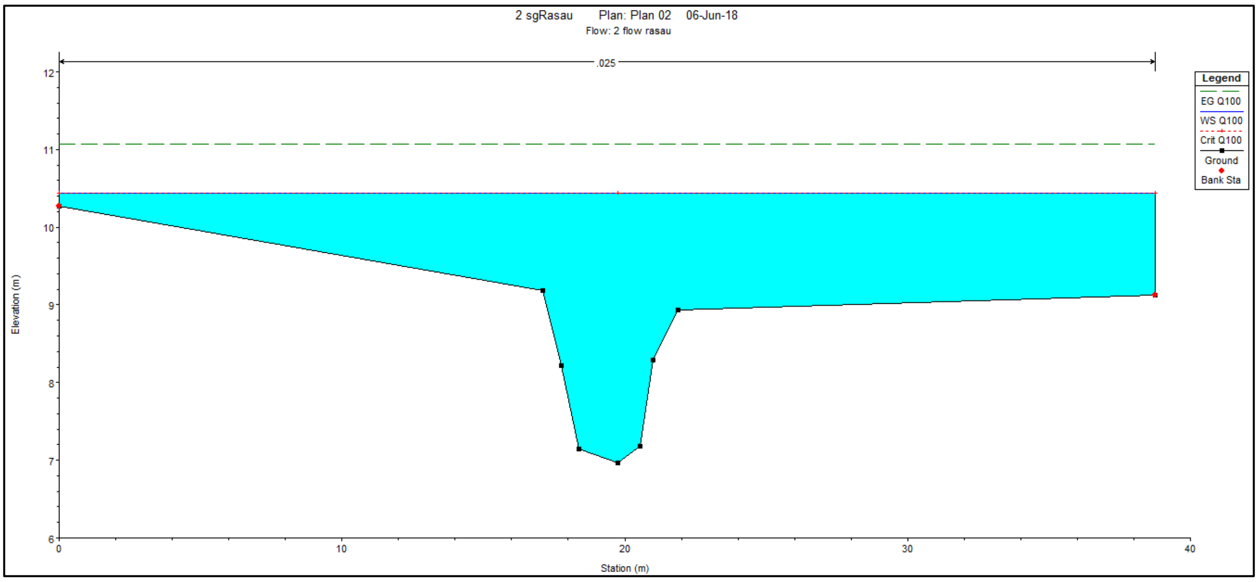

Figure 4.123 Water level at CH0 of the Rasau River (with bridge piers) – Q_{100}

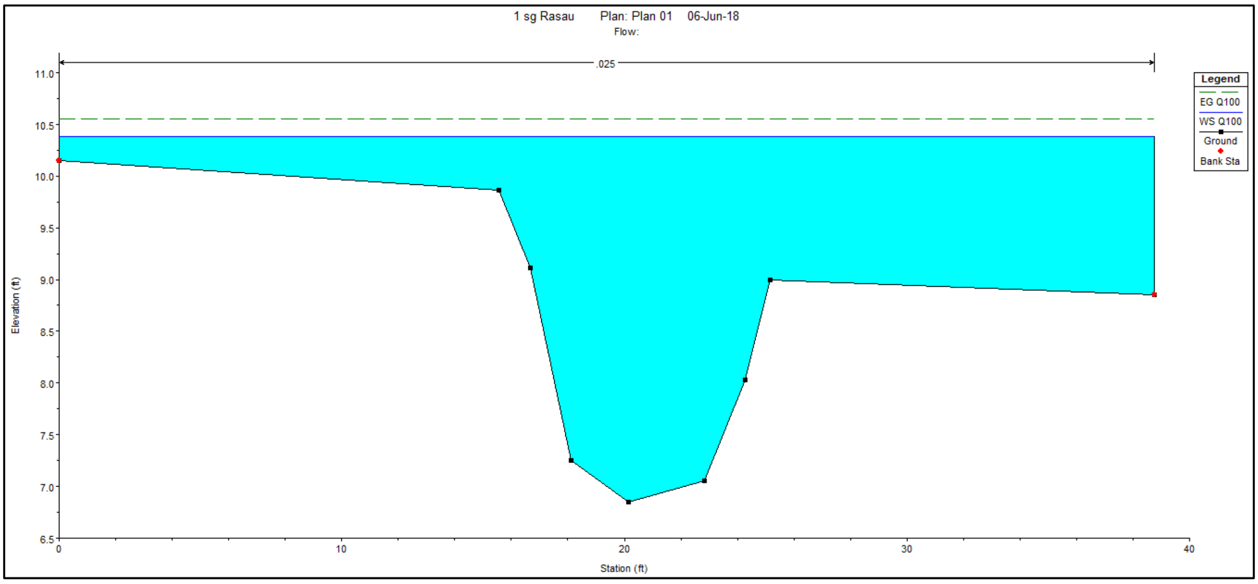

Figure 4.124 Water level at CH20 of Rasau River (without bridge piers) – Q_{100}

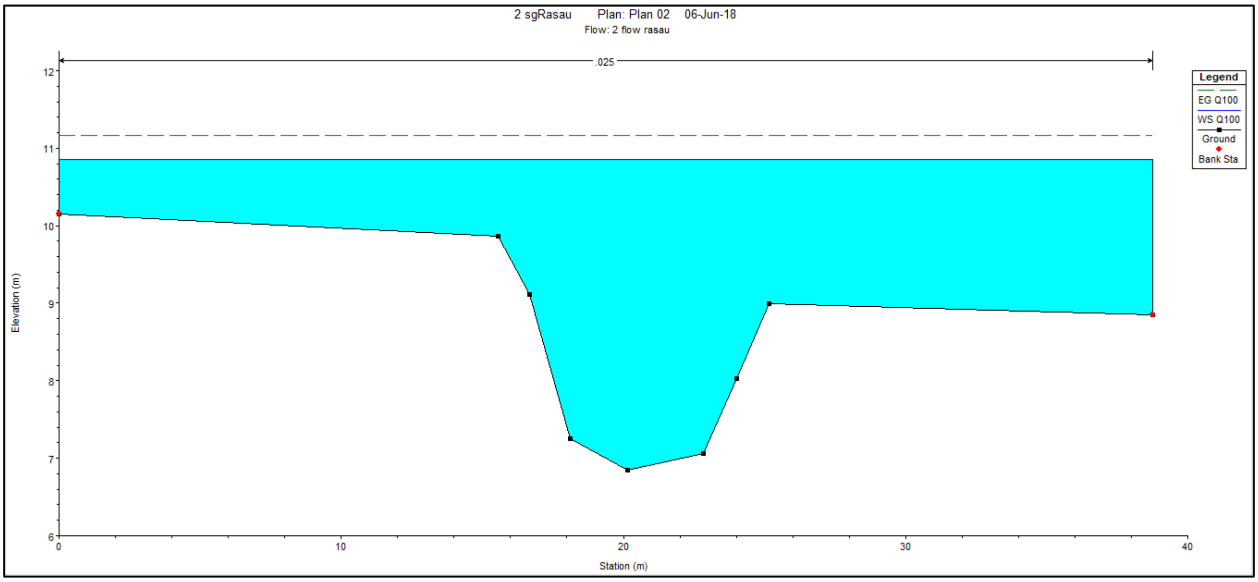

Figure 4.125 Water level at CH20 of Sg Rasau (with bridge piers) – Q_{100}

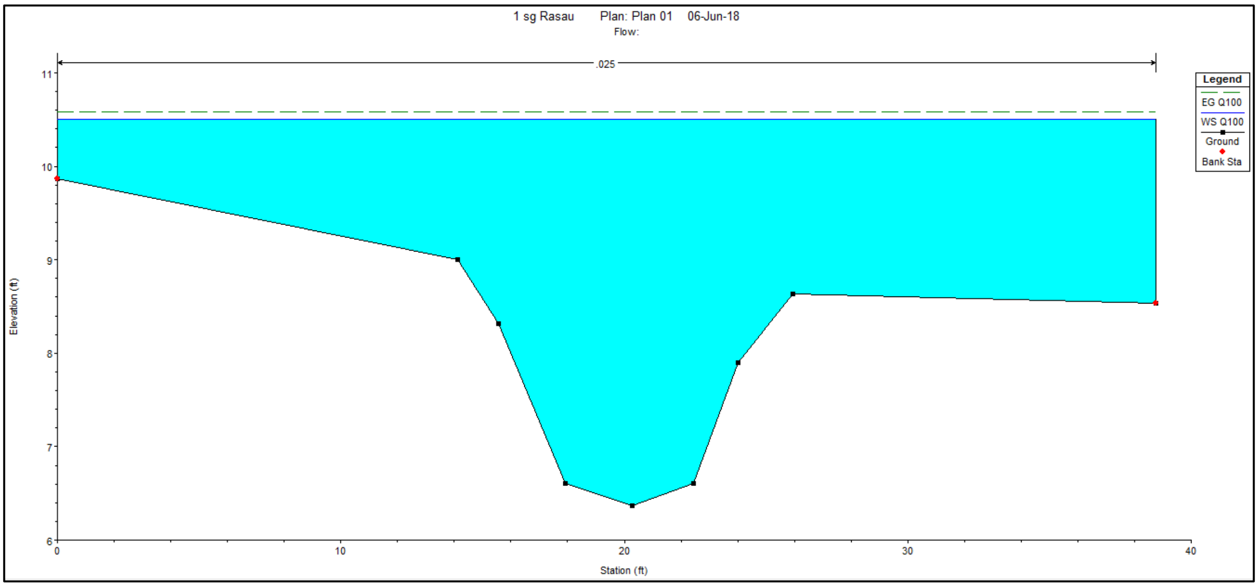

Figure 4.126 Water level at CH40 of Rasau River (without bridge piers) – Q_{100}

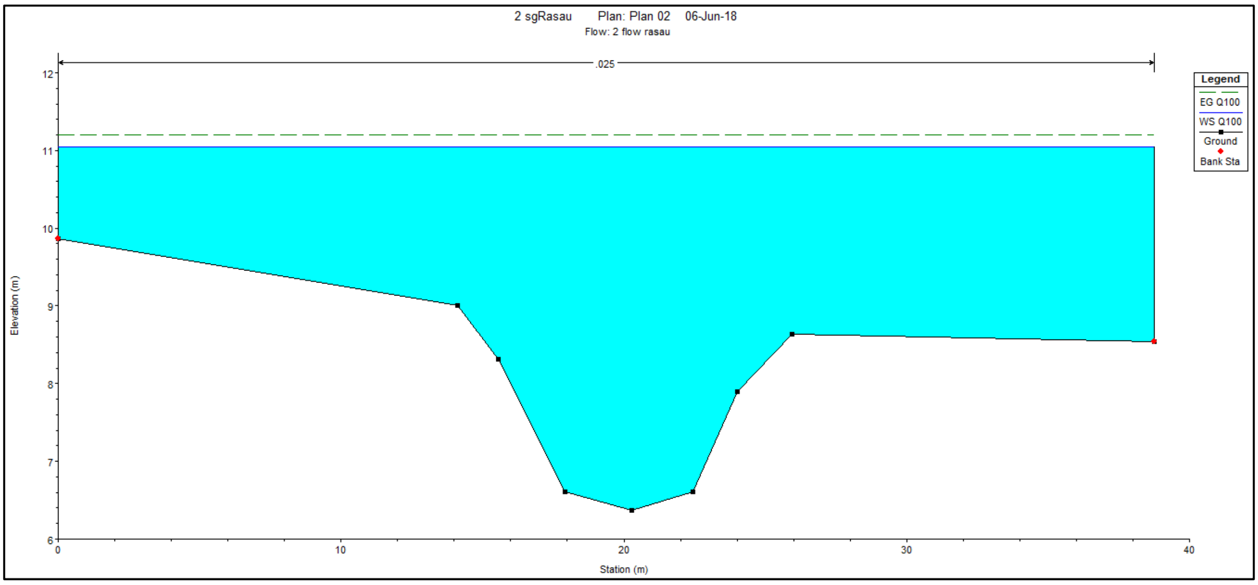

Figure 4.127 Water level at CH 40 of Rasau River (with bridge piers) – Q_{100}

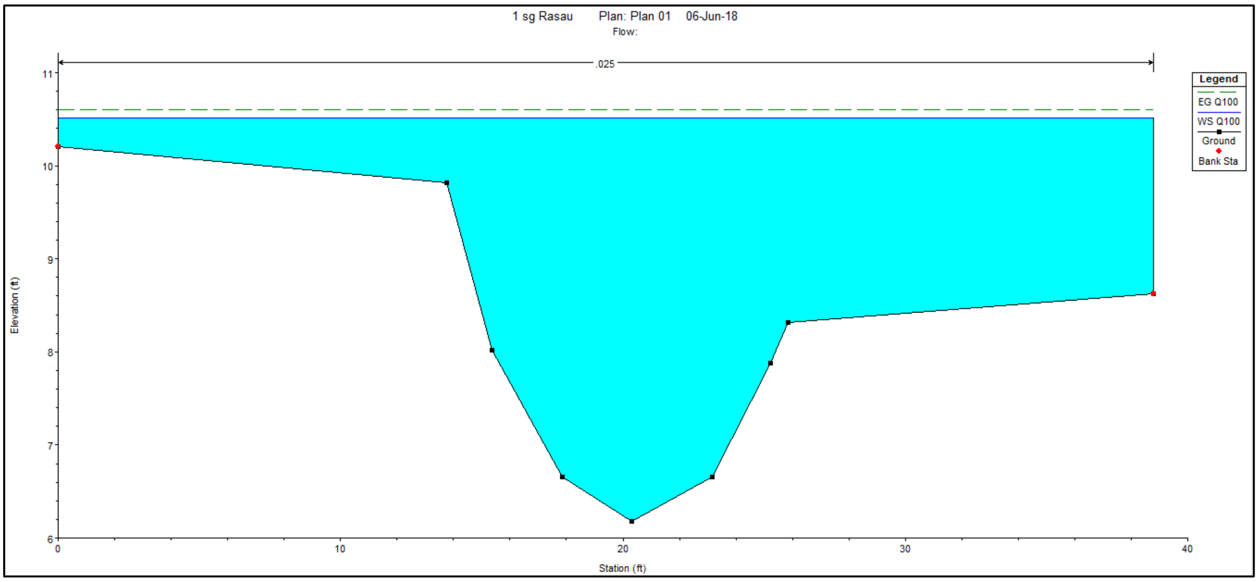

Figure 4.128 Water level at CH60 of Rasau River (without bridge piers) – Q_{100}

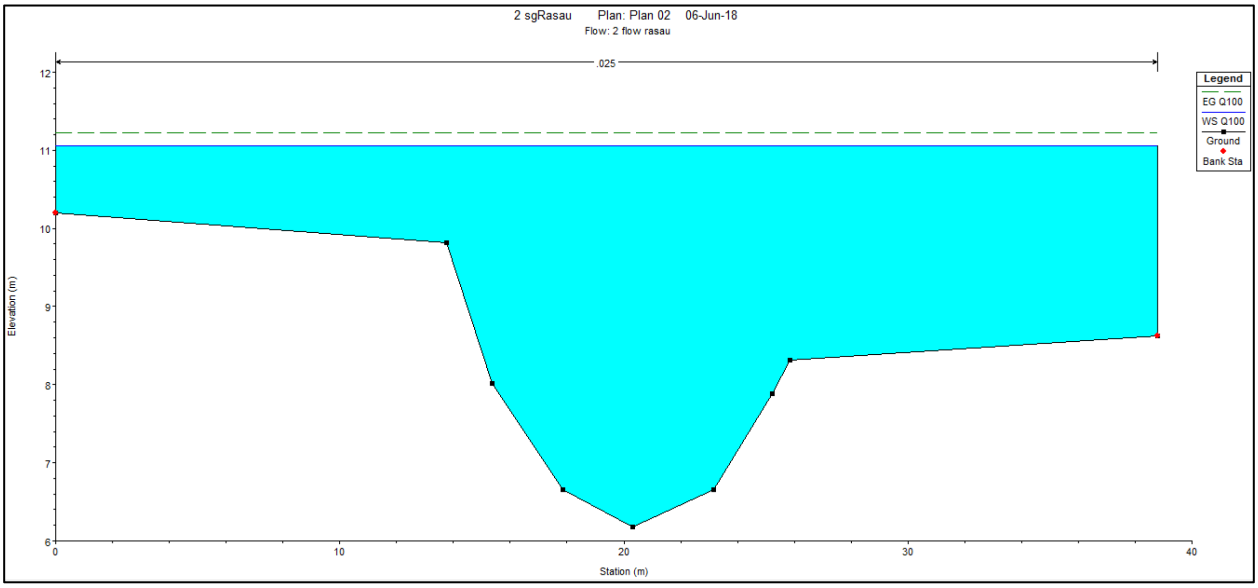

Figure 4.129 Water level at CH60 of Rasau River (with bridge piers) – Q_{100}

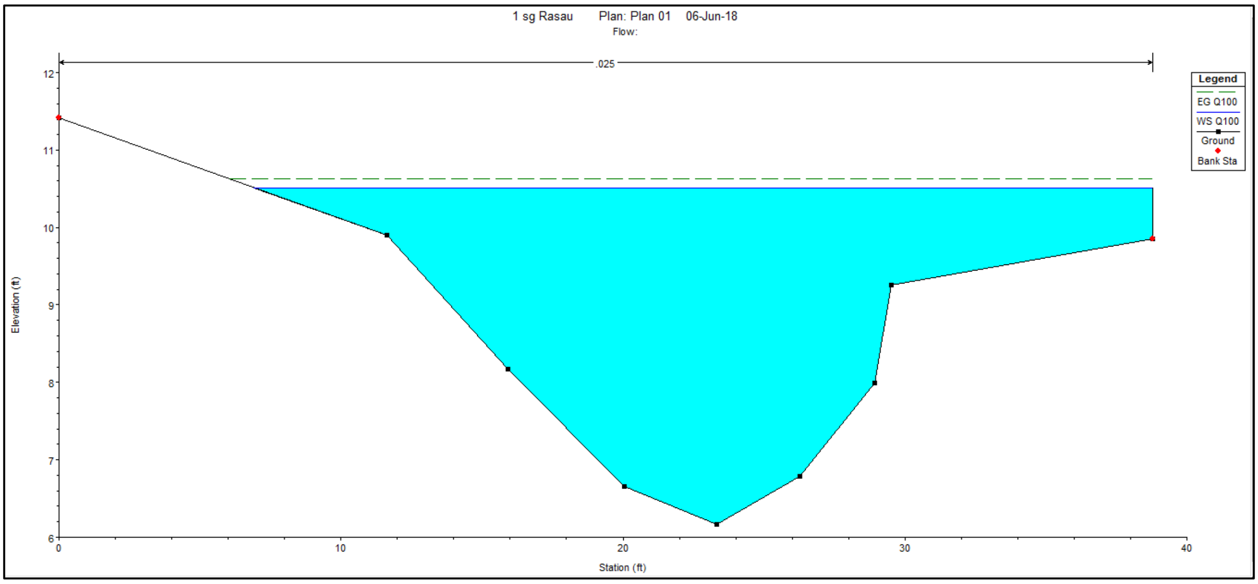

Figure 4.130 Water level at CH80 of Rasau River (without bridge piers) – Q_{100}

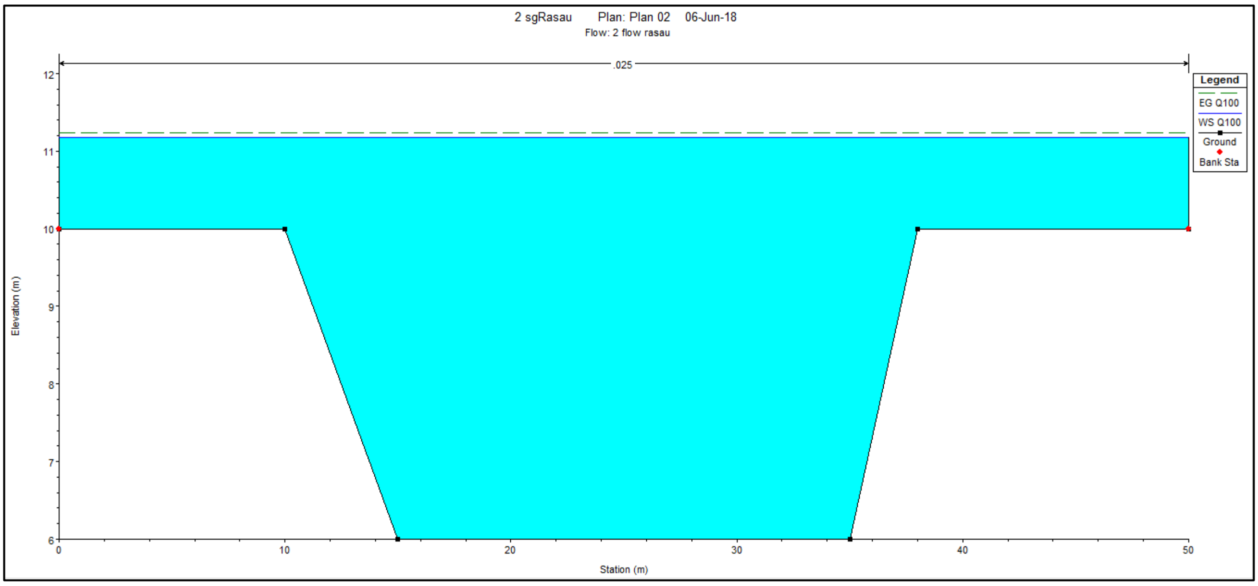

Figure 4.131 Water level at CH80 of Rasau River (with bridge piers) – Q_{100}

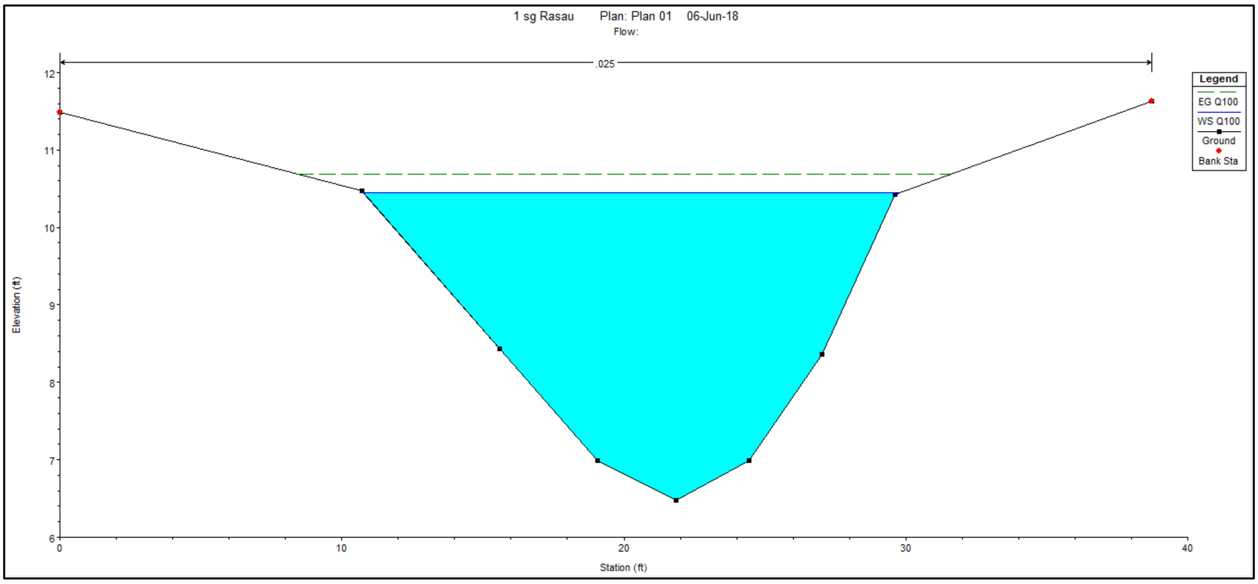

Figure 4.132 Water level at CH100 of Rasau River (without bridge piers) – Q_{100}

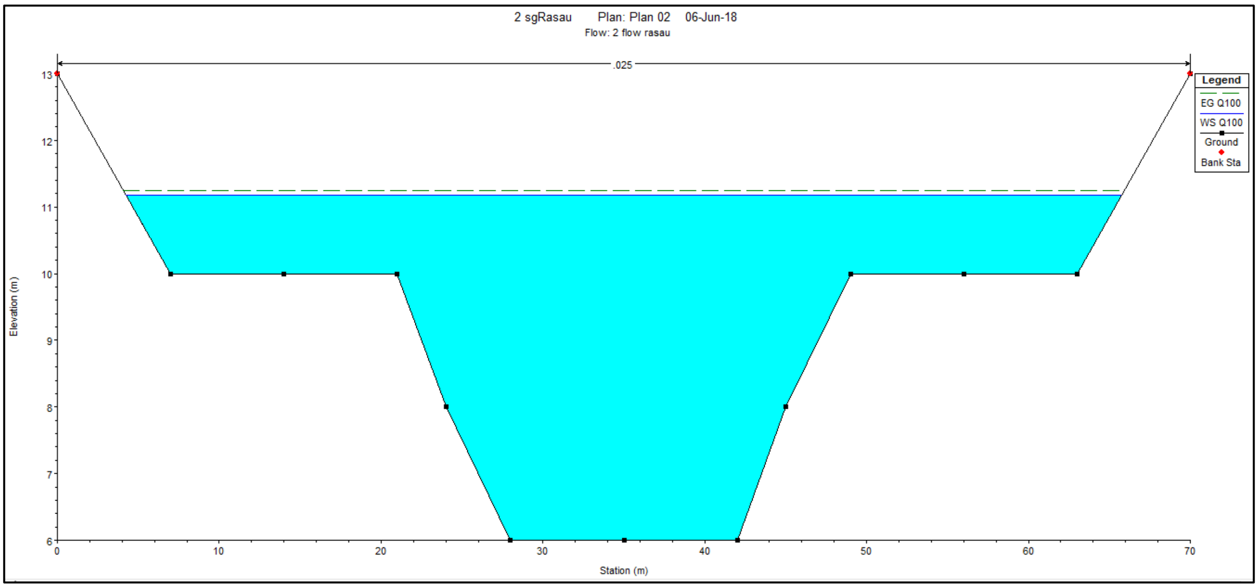

Figure 4.133 Water level at CH100 of Rasau River (with bridge piers) – Q_{100}

4.5.5.2 Water level for Q_{100} under the bridge

With reference to the Table 4.7 and Figure 4.144 to Figure 4.147, these simulations showed that the water levels at chainages where the bridge will be located were 10.44m to 10.55m above sea level and between 11.17m to 11.19m with the bridge piers presence. The water levels were contained in the river channel without overflown to the left and right bank.

Apart from that, during rainfall event of 100-year ARI, the water level under the bridge cross section which starts from CH 100 to CH 120 has shown that the differences in water levels between the two conditions are 0.72m to 0.74m. Water levels condition below the bridge indicated that the backwater effect increased the water levels up to 0.73m more but only at level of 11.18m above sea level which was still not overflow onto the road level which at 15.92m.

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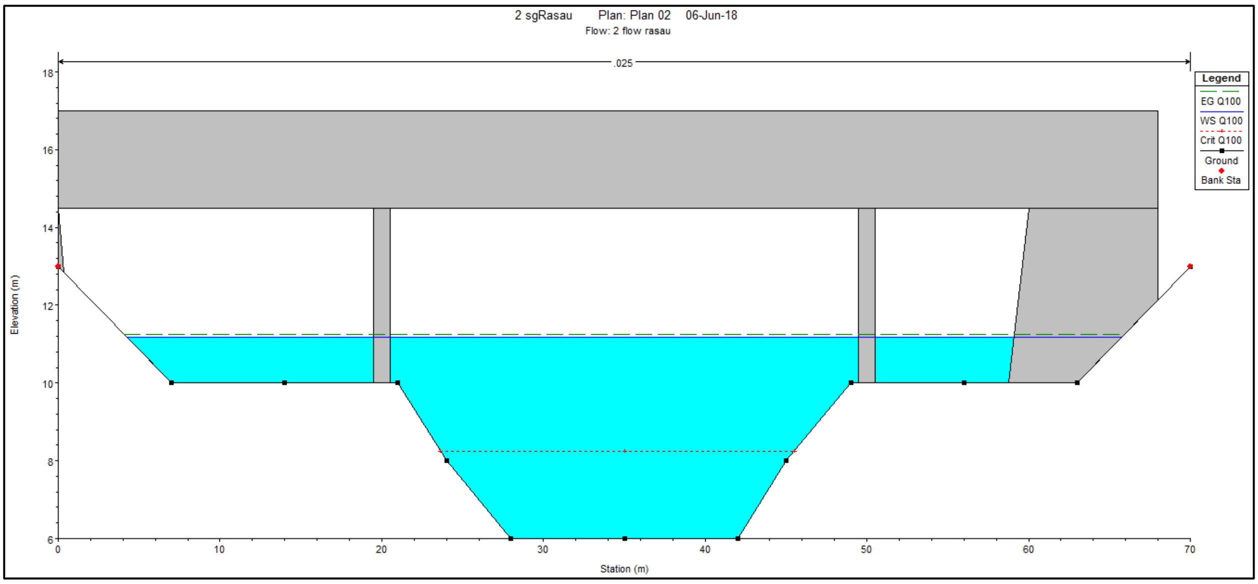

Figure 4.134 Water level at CH110 downstream of Rasau River (with bridge piers) – Q_{100}

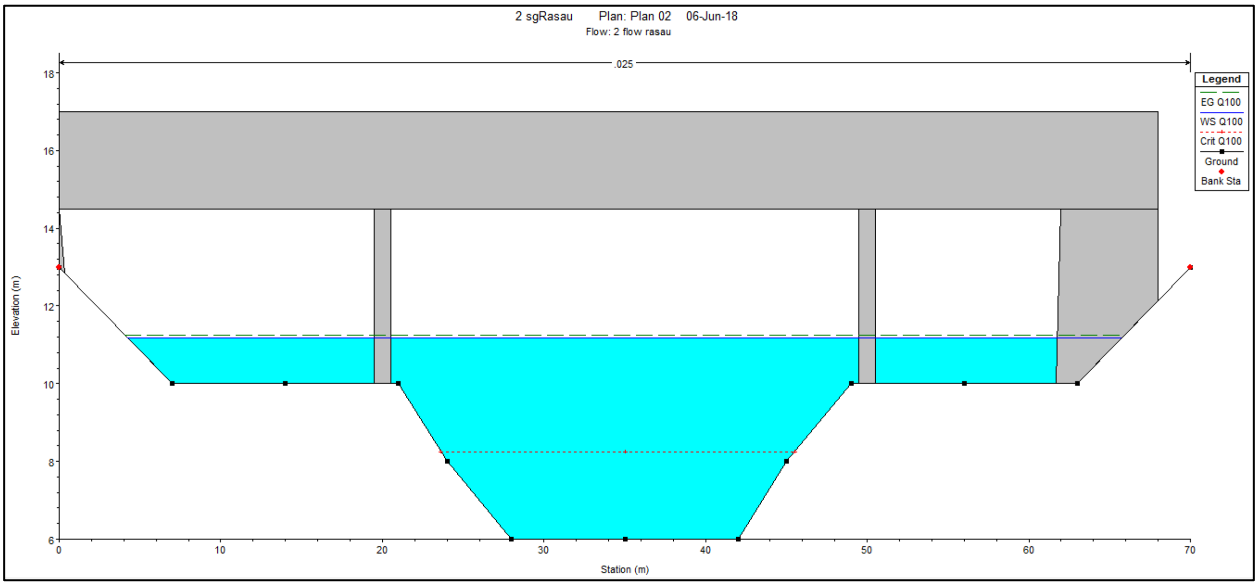

Figure 4.135 Water level at CH110 upstream of Rasau River (with bridge piers) – Q_{100}

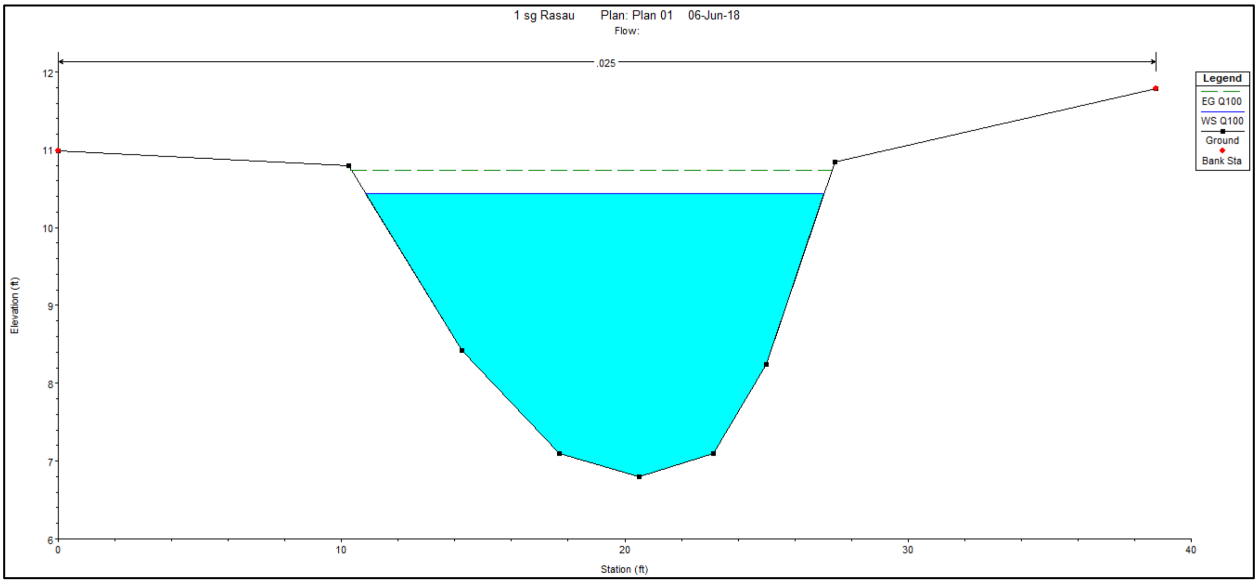

Figure 4.136 Water level at CH120 of Rasau River (without bridge piers) – Q_{100}

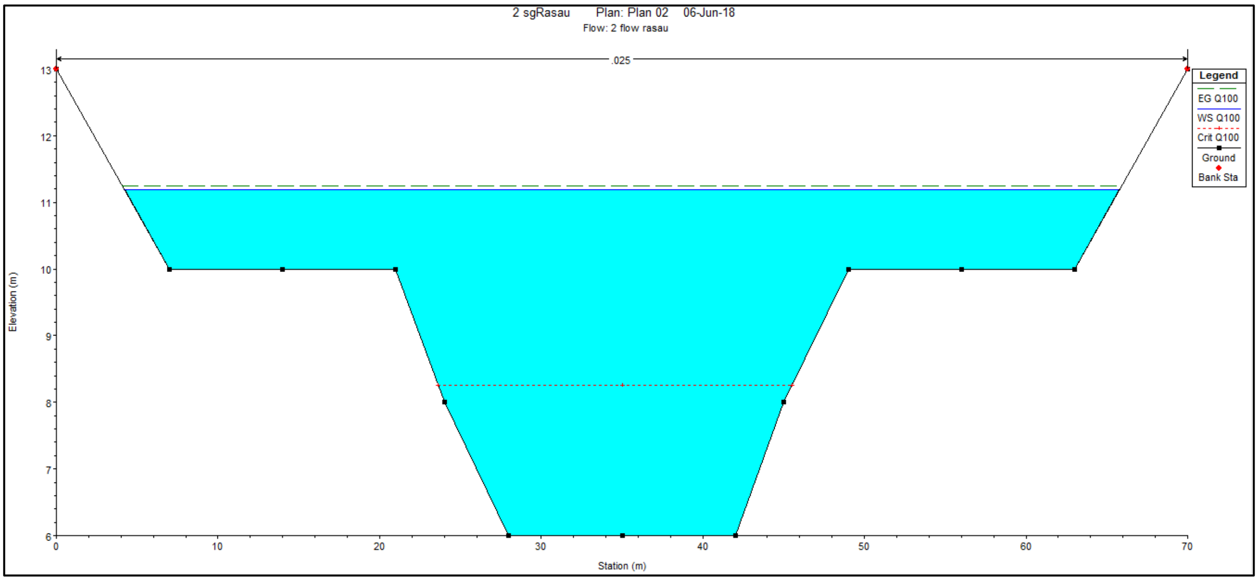

Figure 4.137 Water level at CH120 of Rasau River (with bridge piers) – Q_{100}

4.5.5.3 Water level for Q_{100} at downstream of the bridge

Based on Table 4.3 and Figure 4.138 to Figure 4.152, water levels were overflown along CH 140 to CH 260 except at CH 140(without bridge piers) and CH 160(without bridge piers) because of its higher level of left banks. Water levels along the downstream chainages were between 10.64m to 10.79m above sea level without the bridge piers presence and between 11.09m and 11.24m with the bridge piers presence.

In addition, the water levels along the downstream cross section of the river also can be discussed. This study showed that the increases in water levels from CH 120 to CH 240 had slightly fluctuated. The differences in water levels were from 0.43m to 0.56m different along the chainage with 100-year ARI condition.

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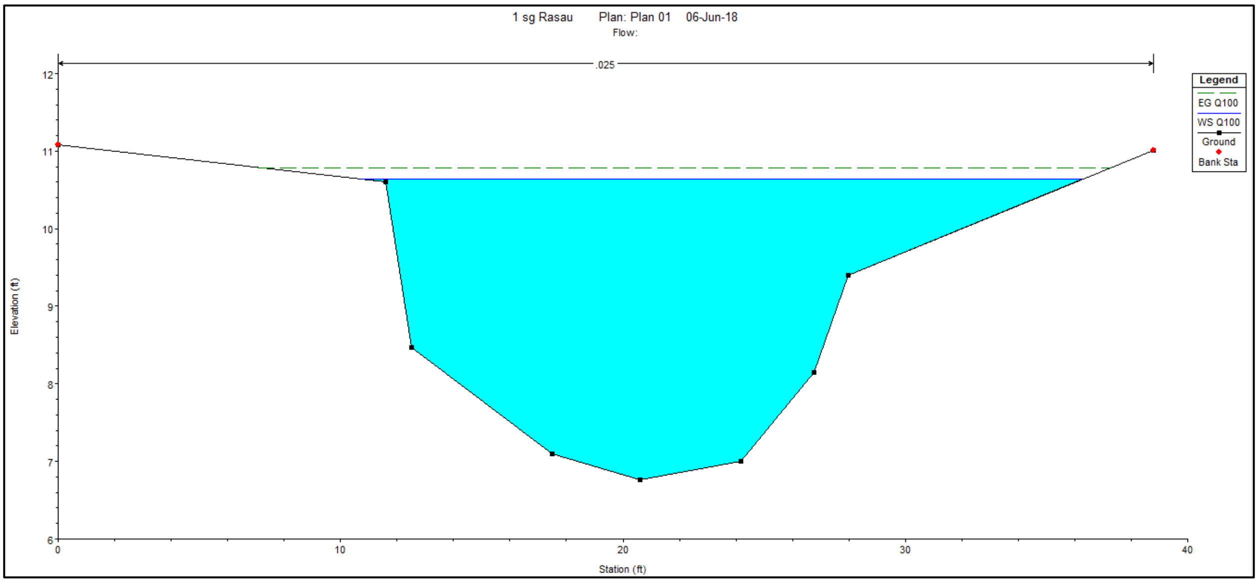

Figure 4.138 Water level at CH140 of Rasau River (without bridge piers) – Q_{100}

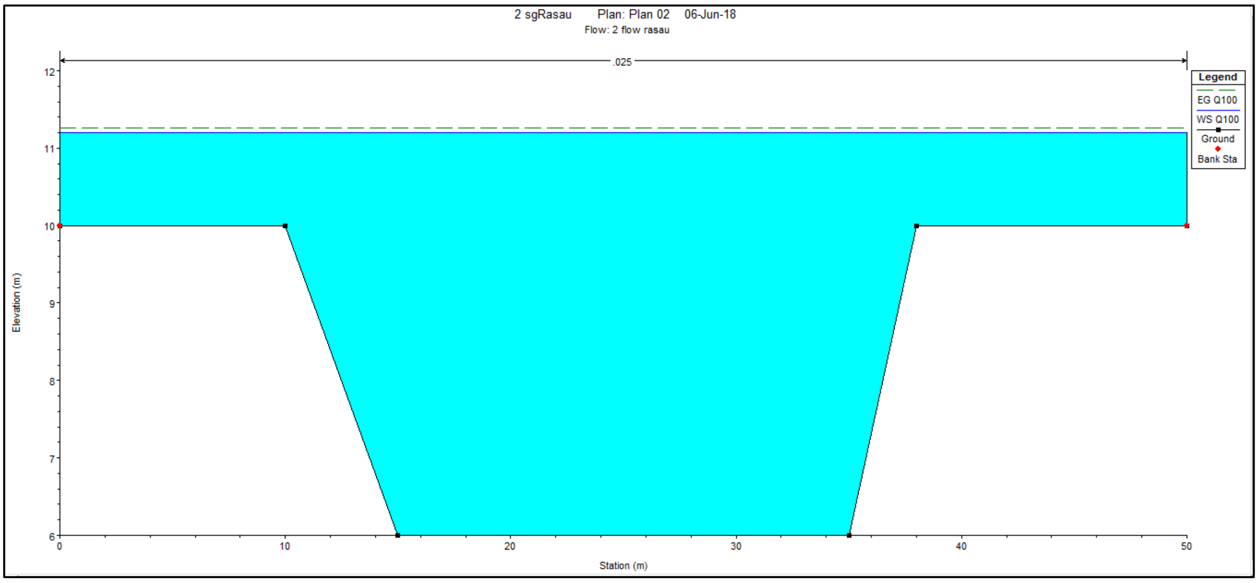

Figure 4.139 Water level at CH140 of Rasau River (with bridge piers) – Q_{100}

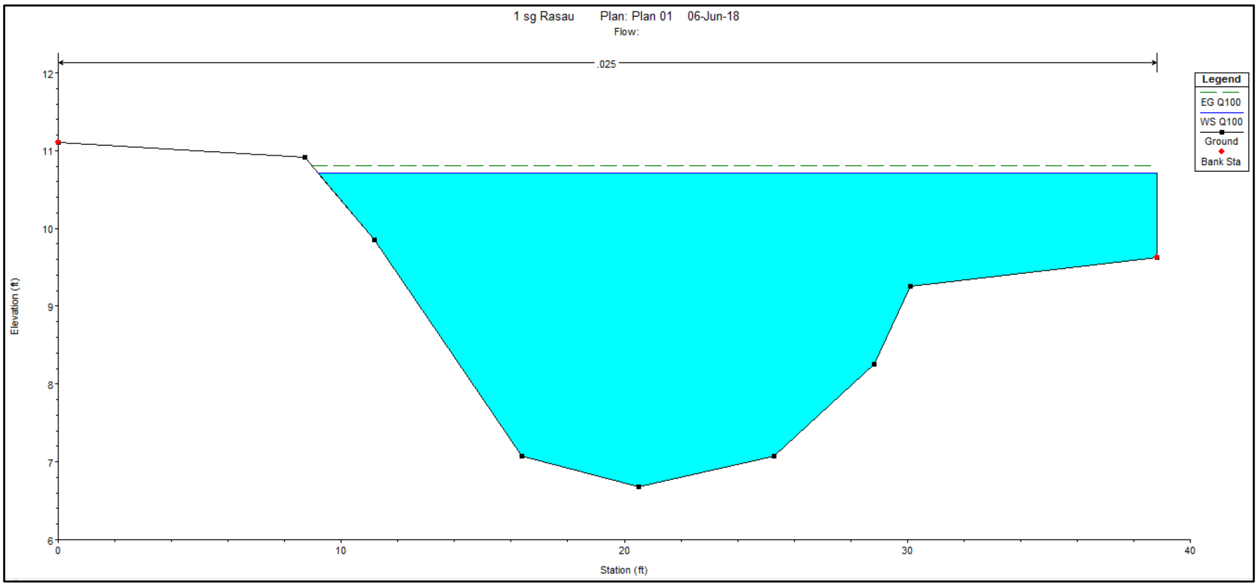

Figure 4.140 Water level at CH160 of Rasau River (without bridge piers) – Q_{100}

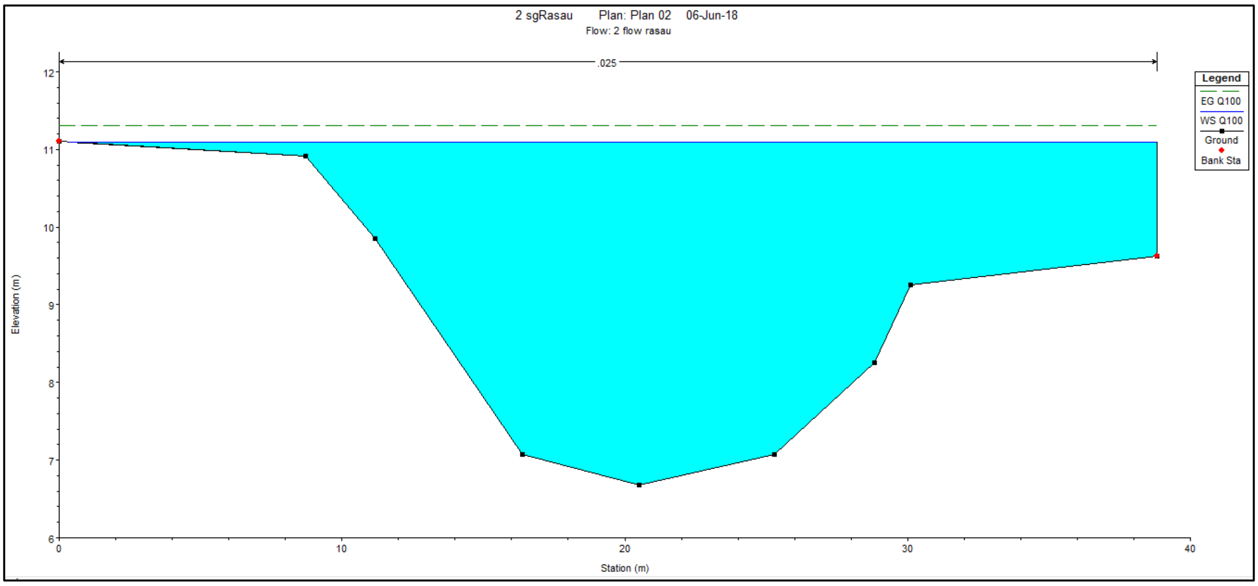

Figure 4.141 Water level at CH160 of Rasau River (with bridge piers) – Q_{100}

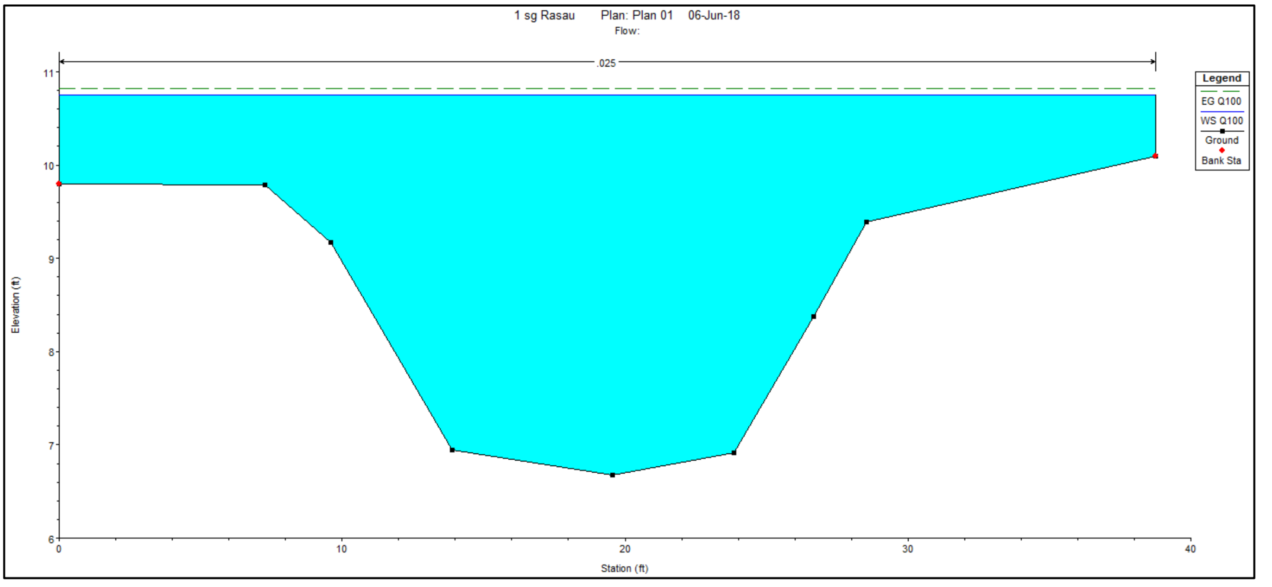

Figure 4.142 Water level at CH180 of Rasau River (without bridge piers) – Q_{100}

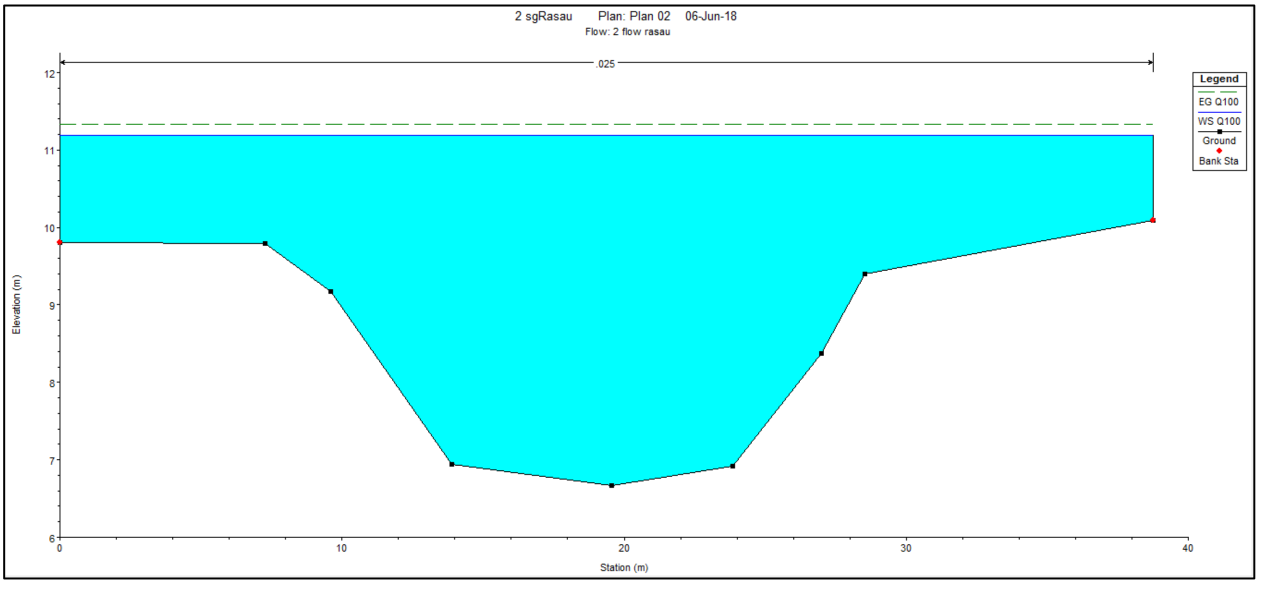

Figure 4.143 Water level at CH180 of Rasau River (with bridge piers) – Q_{100}

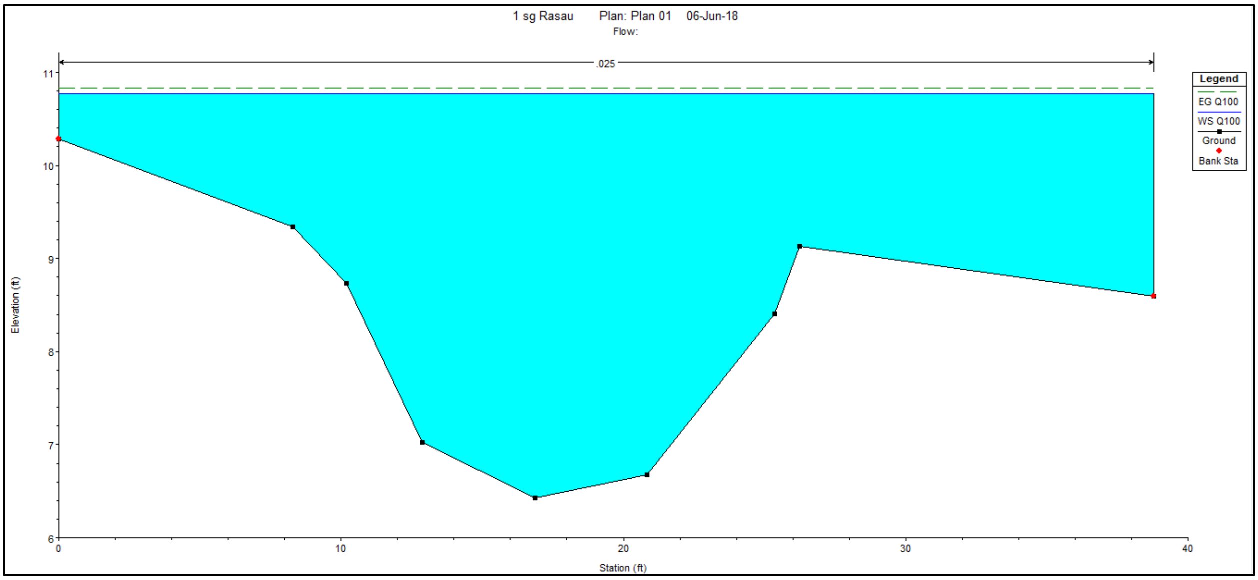

Figure 4.144 Water level at CH200 of Rasau River (without bridge piers) – Q_{100}

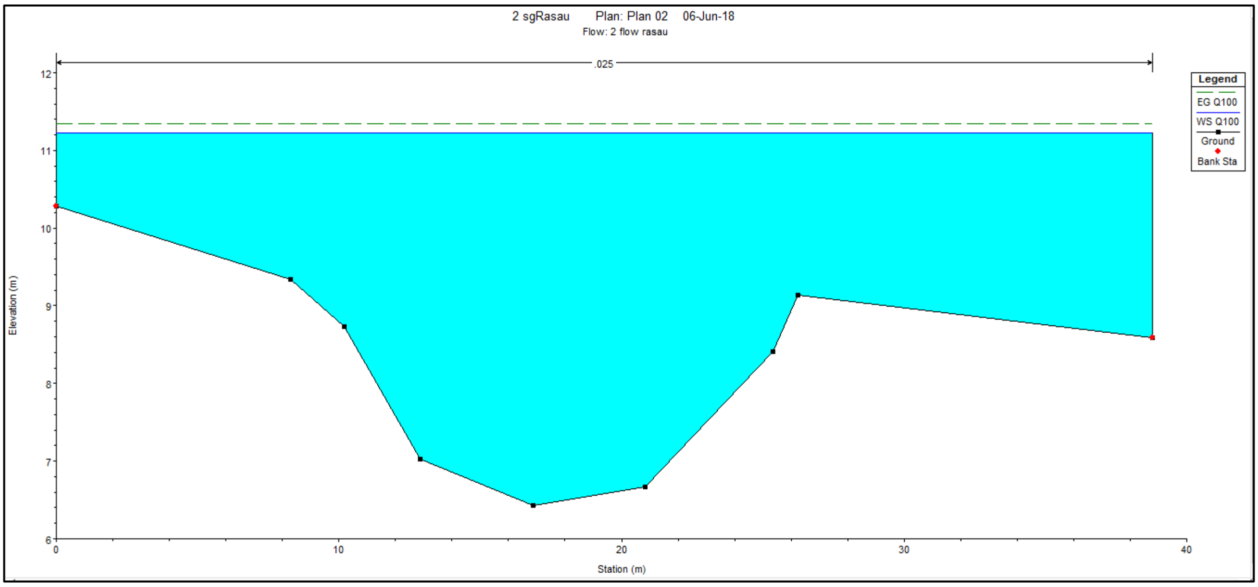

Figure 4.145 Water level at CH200 of Rasau River (with bridge piers) – Q_{100}

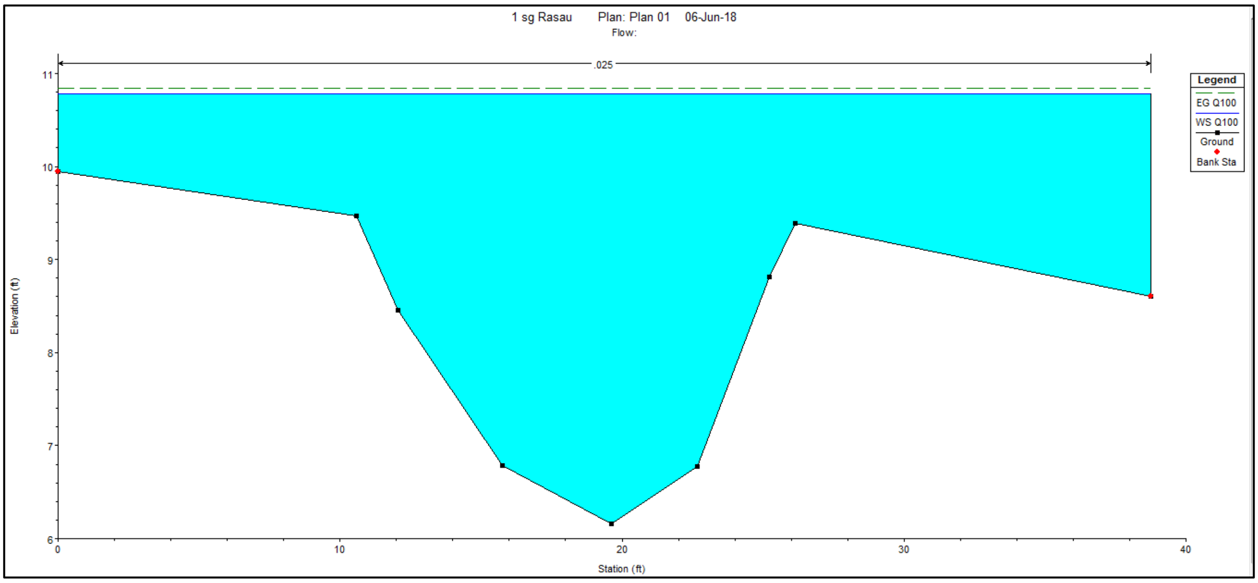

Figure 4.146 Water level at CH220 of Rasau River (without bridge piers) – Q_{100}

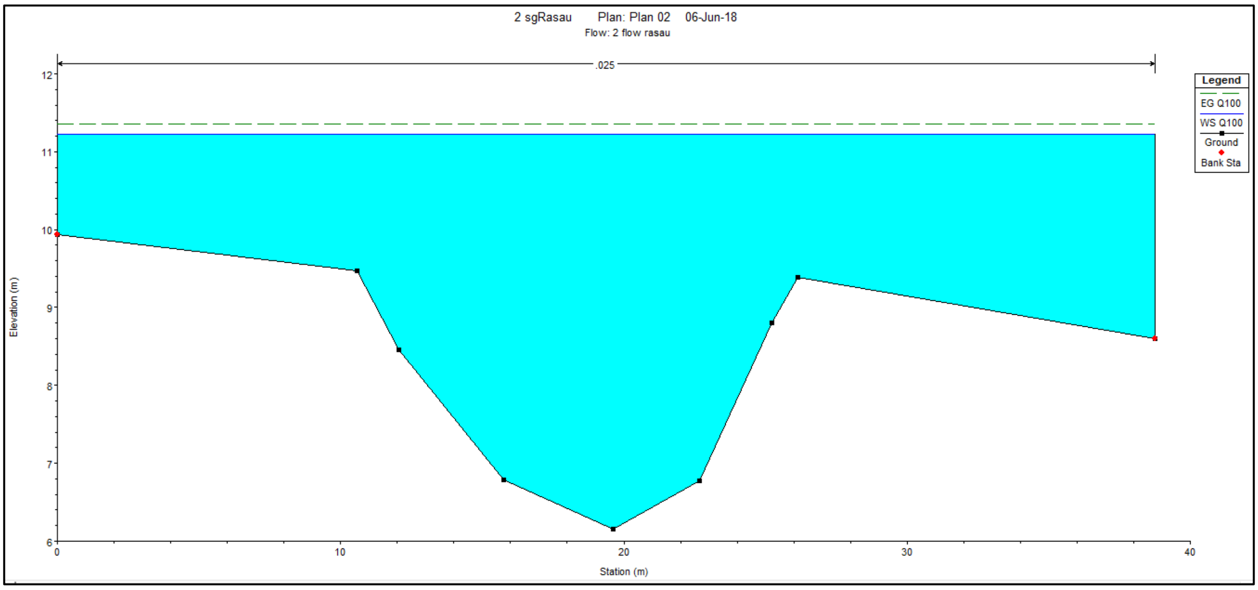

Figure 4.147 Water level at CH220 of Rasau River (with bridge piers) – Q_{100}

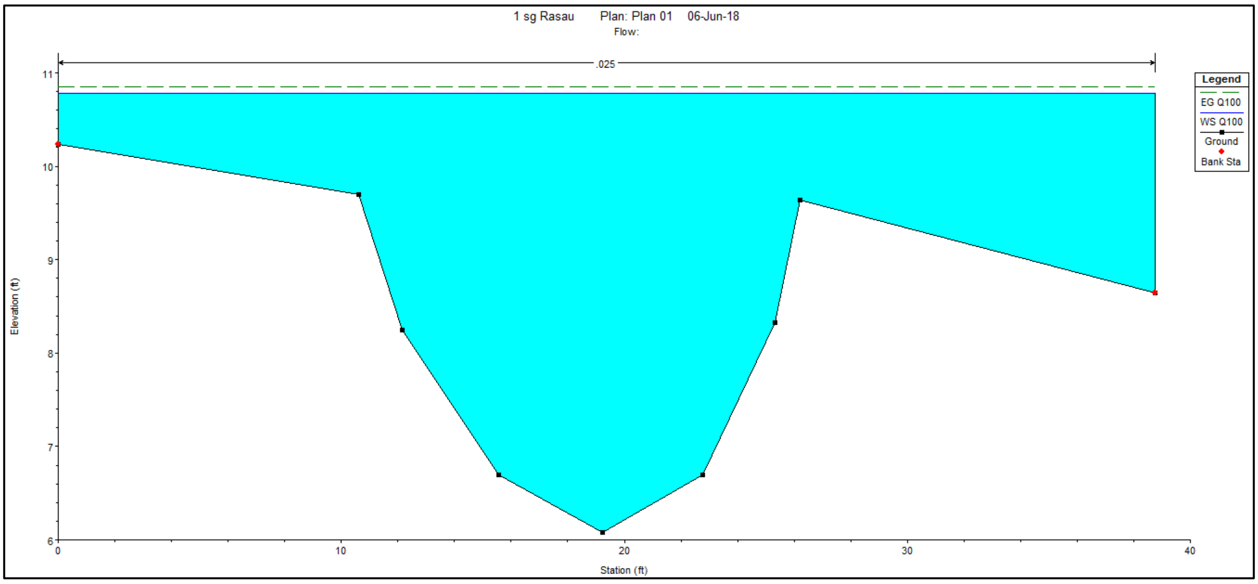

Figure 4.148 Water level at CH240 of Rasau River (without bridge piers) – Q_{100}

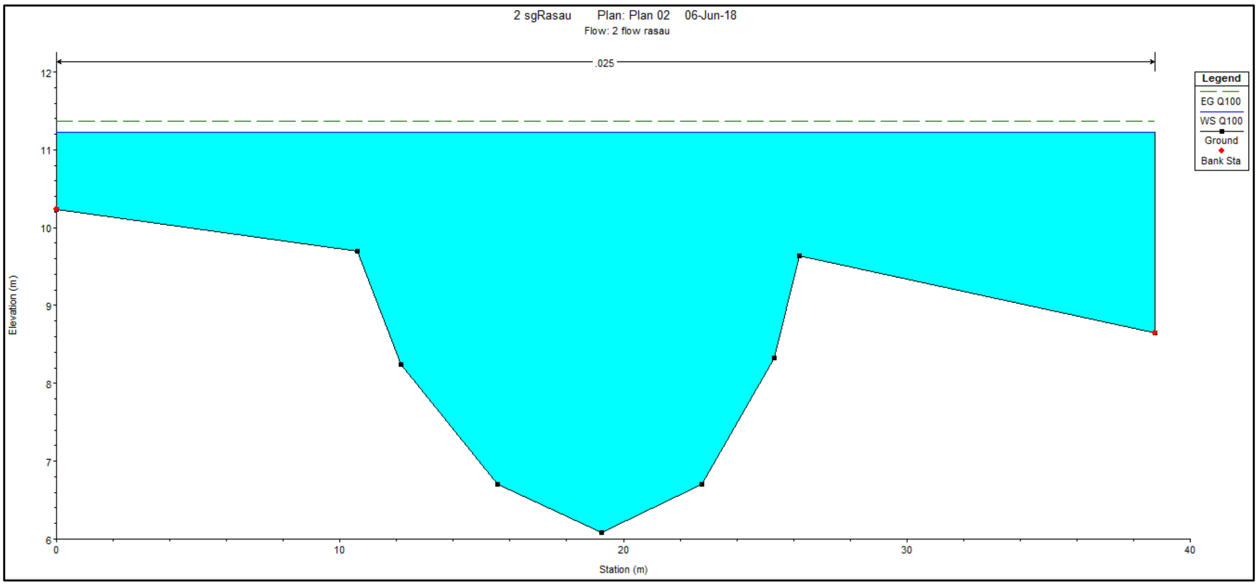

Figure 4.149 Water level at CH240 of Rasau River (with bridge piers) – Q_{100}

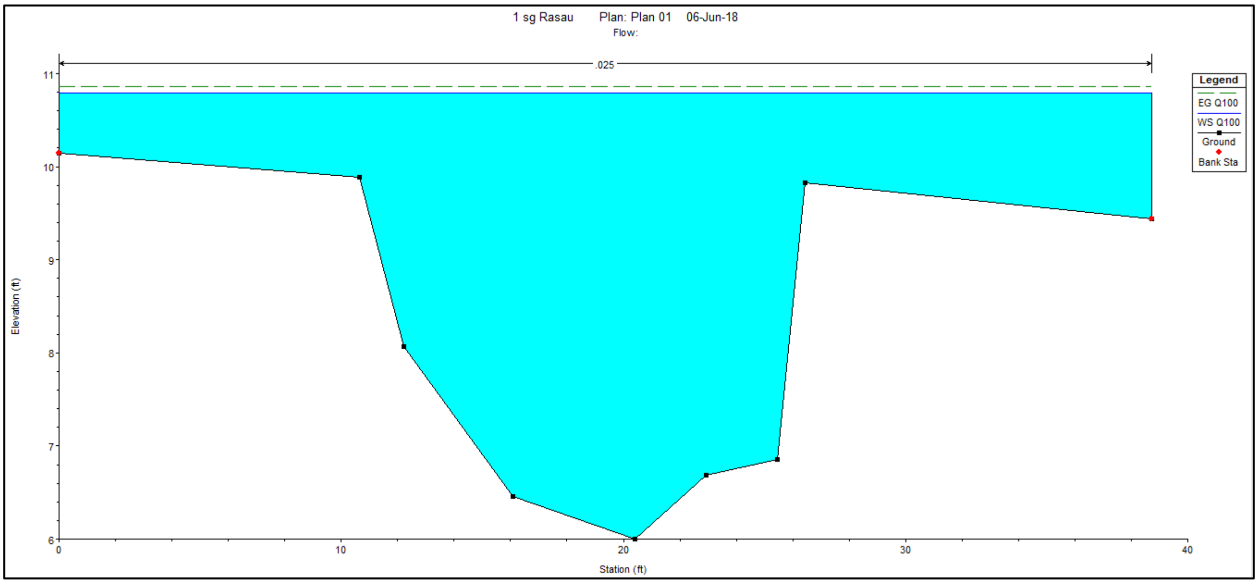

Figure 4.150 Water level at CH260 of Rasau River (without bridge piers) – Q_{100}

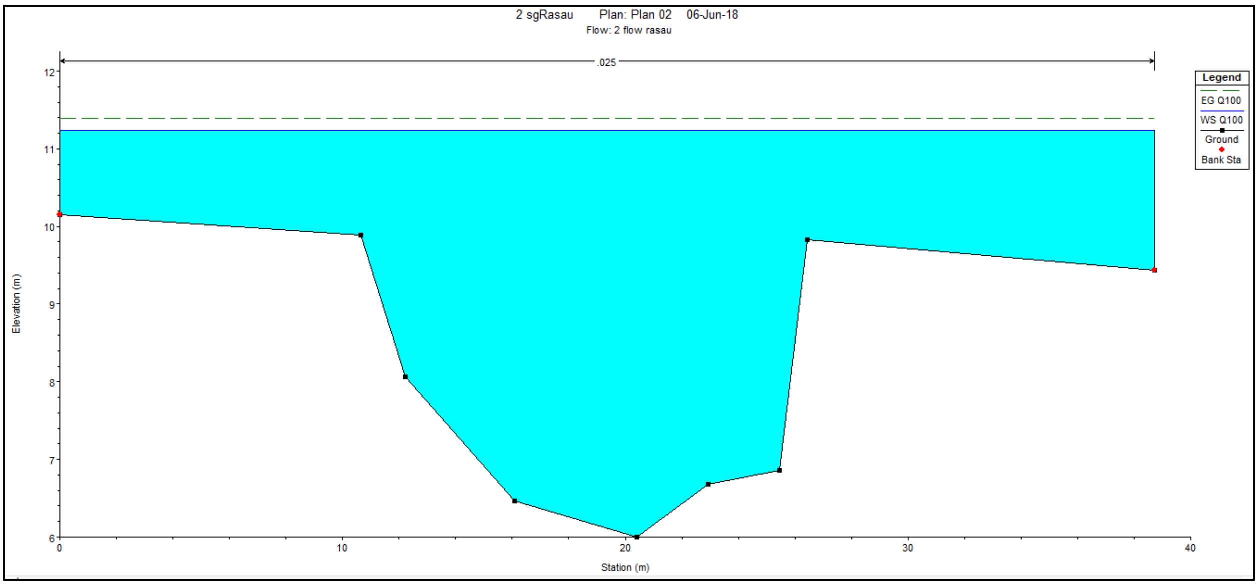

Figure 4.151 Water level at CH260 of Rasau River (with bridge piers) – Q_{100}

4.5.6 Water level along longitudinal cross section and 3D plot

With the applications of the HEC-RAS software, the water levels also can be viewed simultaneously from CH0 to CH260. Figure 4.152 shows the water levels with 5-year, 10-year, 20-year, 50-year, 100-year ARI along the longitudinal cross section of the river without the bridge piers presence. 3D plot of the water profiles including all the peak flows without the bridge piers is illustrated in the Figure 4.153.

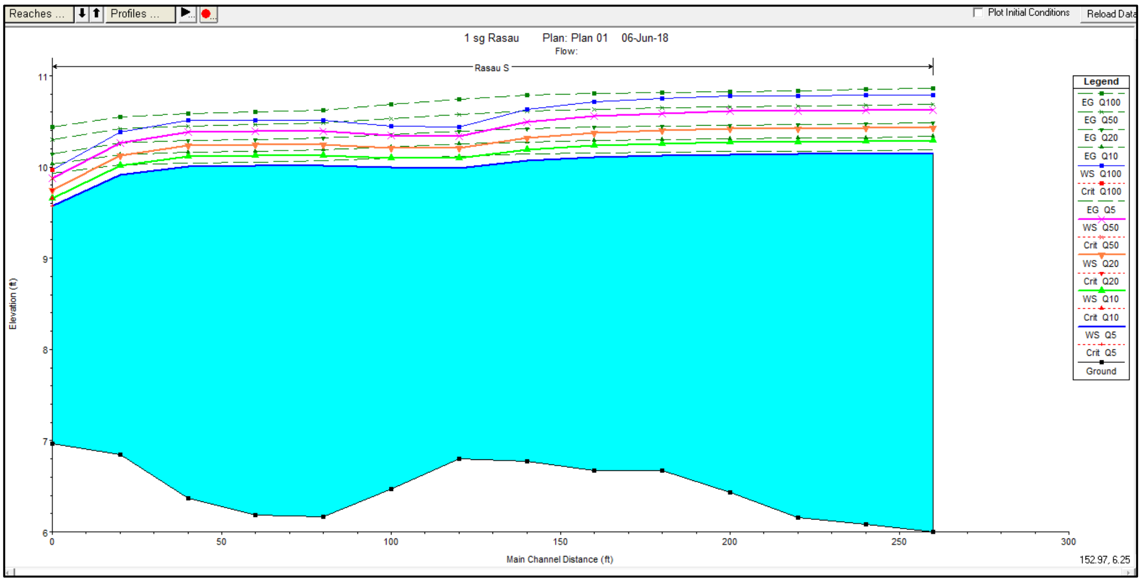

Figure 4.152 Water levels along the longitudinal cross section (without bridge piers)

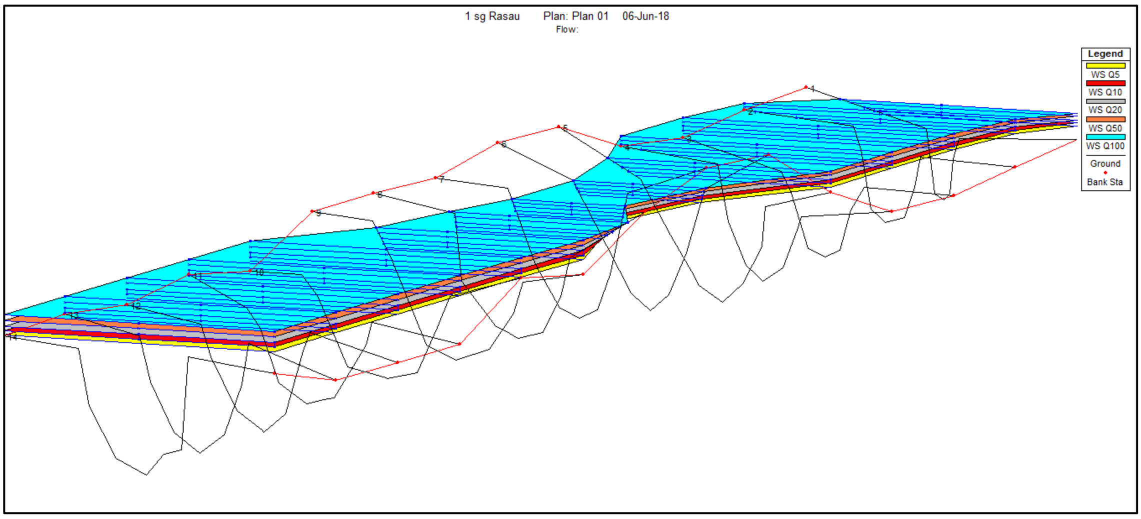

Figure 4.153 3D plot (without bridge piers)

Figure 4.154 shows the water level with 5-year, 10-year, 20-year, 50-year, 100-year ARI along the longitudinal cross section of the river with the bridge piers presence. 3D plot of the water profiles including all the peak flows with the bridge piers presence is illustrated in the Figure 4.155.

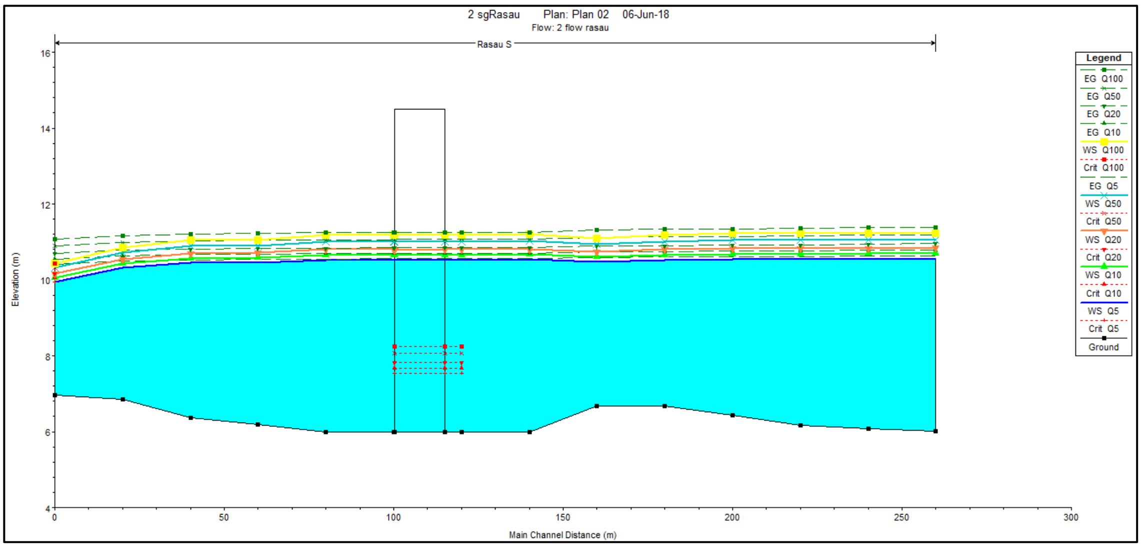

Figure 4.154 Water levels along the longitudinal cross section (with bridge piers)

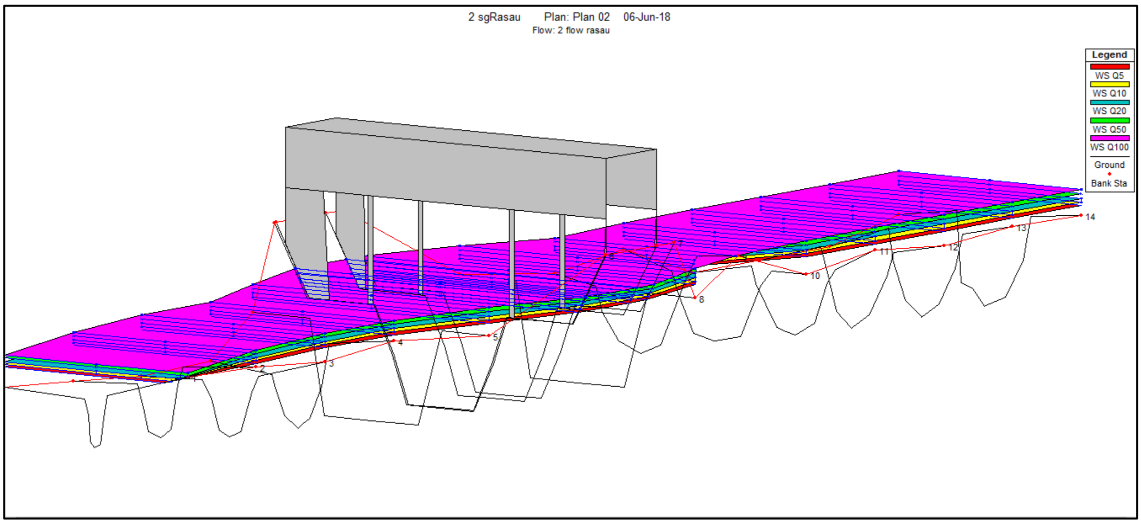

Figure 4.155 3D plot (with bridge piers)

From this analysis, the water level for Q_{100} was expected to be the highest while Q_5 was expected to be the lowest. The presence of the bridge piers does influence the water level. The level with Q_{100} before and after bridge piers were 11.18m and 11.09m respectively.

4.6 Discussion

There are many factors influence the flow of water and the water levels of the river. This overall chapter discussed analysis and simulations conducted using HEC-RAS software to determine the water level profiles and backwater effect due to presence of bridge piers in a river. As refer to the Guideline for River Development by DID 1973, with 100-year ARI, the minimum freeboard for river crossing structure must be 0.6m from soffit level. Thus this simulation result with 11.18m water level under the bridge had complied with Guideline for River Development with freeboard of 0.74m as shown in the Figure 4.134.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The main objective of the study was to determine the backwater effect due to presence of bridge piers by generate a simulation model. The outcome of the simulation provided an understanding on the behaviour of the river and the risk of flooding in river channel due to the presence of the bridge piers. The simulation was run under basic condition on rainfall catchment area due to the lack of several data.

At the early stage of the study, a 2 dimensional river network with a cross section and longitudinal section diagram was produced for 260 meter long river reach that contained fourteen chainages at which the distance between chainages is 20m. The surface ground levels of the river were known from the topographical map.

After conducting several simulations and study on the river network using HEC-RAS software developed by Hydrologic Engineering Center and by data analysis, a few conclusions were made from the study results.

The analysis showed that the water levels in the channel increased as the flow increased. It is also showed that the level of the water increased more with the presence of bridge piers as compared without the bridge piers condition. Along the downstream of the bridge the water levels rise higher when the bridge piers is presence and caused the water to over flow onto both left and right bank.

Moreover, water levels at the middle of the river where the bridge is located and at the downstream channel of the bridge were also surpassing the left and right bank for both with bridge piers and without the bridge piers condition. From the various analysis on the effect of backwater to the river condition, the results of the simulations showed that the water levels and flow of water of a river were affected by the presence of the bridge piers at the middle part of a river. Apart from that, higher value of peak flow from the upstream river due to higher precipitation level produced from heavy rainfall might also affect the water levels.

From the simulation, the proposed bridge along CH 0 to CH 260 can be accepted for the construction purposed since it is comply with the Guideline for River Development for having a water level of 11.18m from sea level and freeboard of 0.74m under the soffit of the bridge.

Furthermore, the simulation showed that the objective of the study was achieved successfully however, further improvement could be done for a better result. Further understanding can bring many advantages to the development of area near the river. This study can be the commencement for more precise and detailed investigation on the behaviour of the Rasau River.

5.2 Recommendations

It is recommended that further experiments shall be carried out to give more effective and beneficial results;

i. Set up a rainfall station near the study area to give a precise rainfall profile of catchment area.
ii. Conduct a real flood event study and compare the result of the simulation with the real flood event report from DID.
iii. Use AcrGIS software to get more accurate area of the catchment.
iv. Conduct a river survey to produce a precise cross section of the river basin.

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APPENDIX A

Construction Drawing of Bridge 3, Rasau River

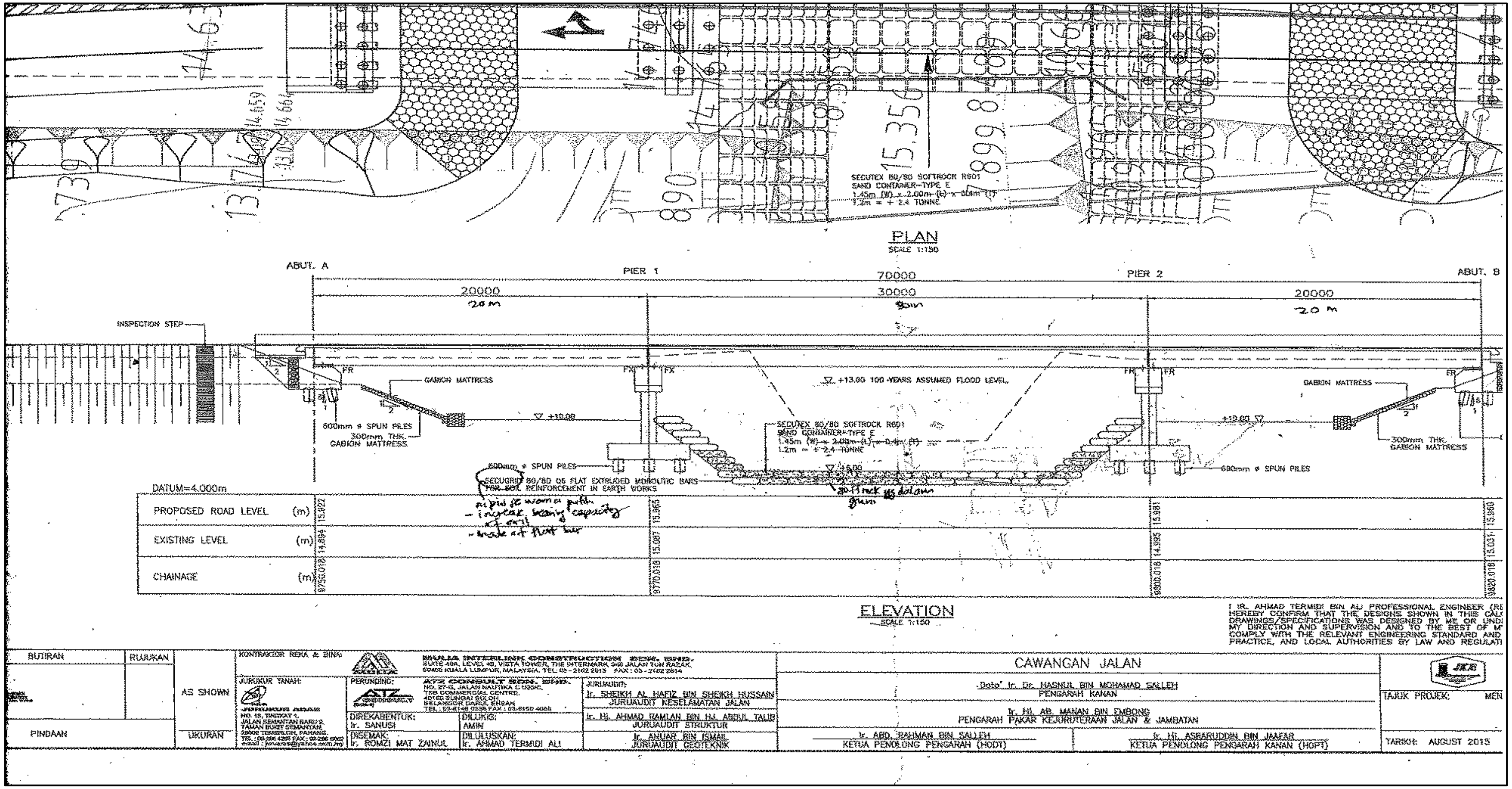

Figure A1 Cross section drawing of Rasau River under the bridge

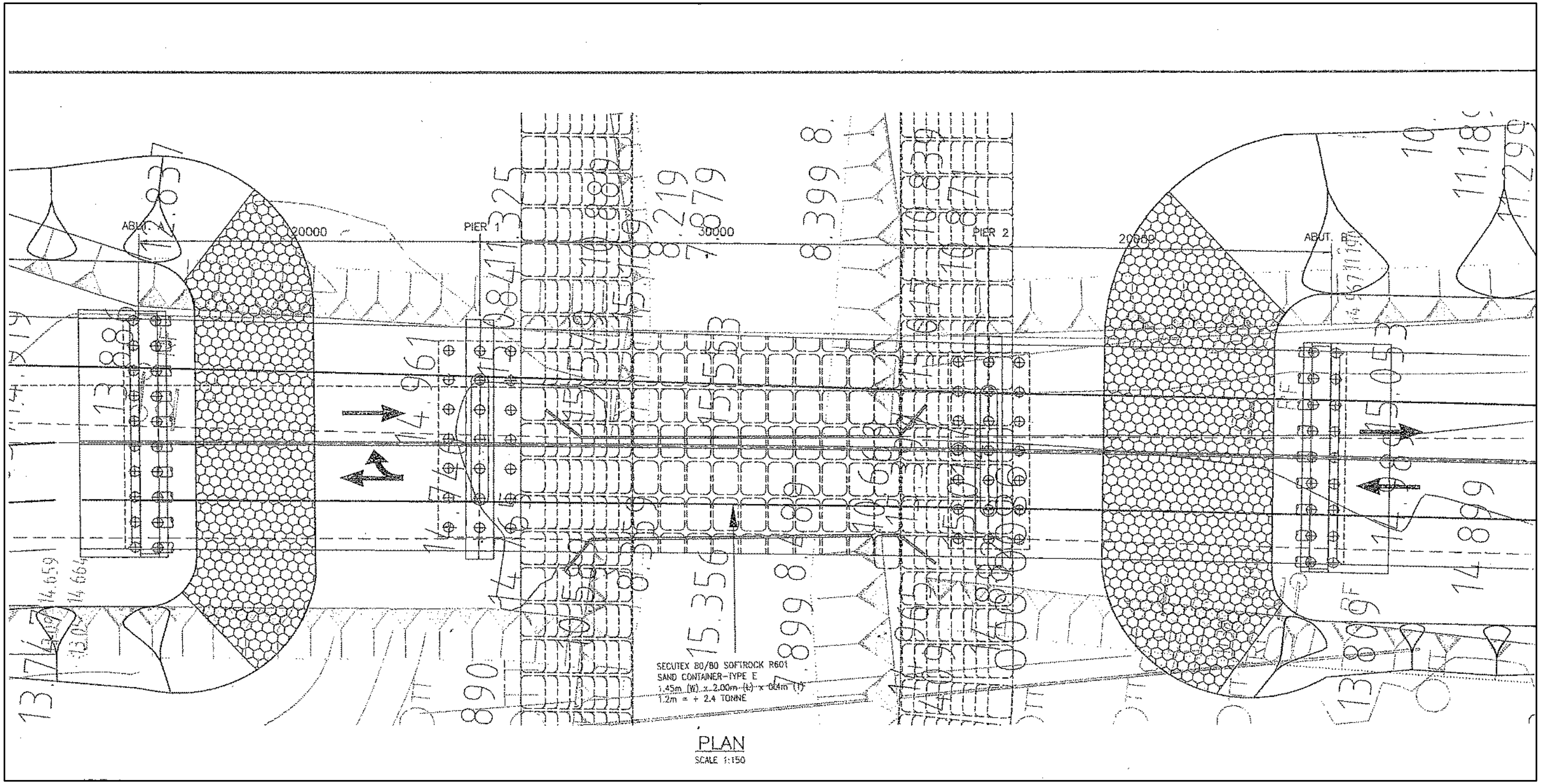

Figure A2 Plan view of Bridge 3

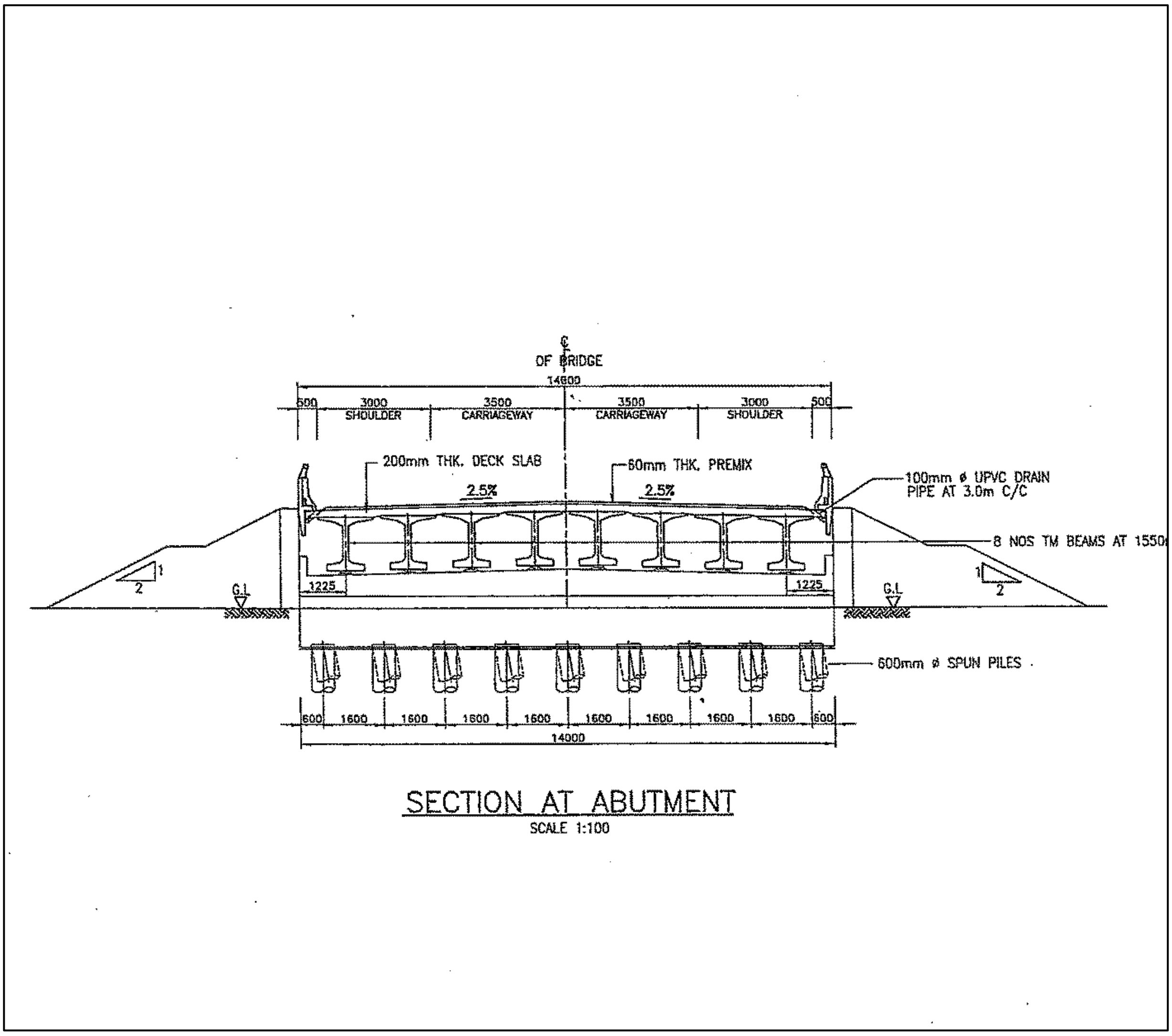

Figure A3 Drawing of section at abutment

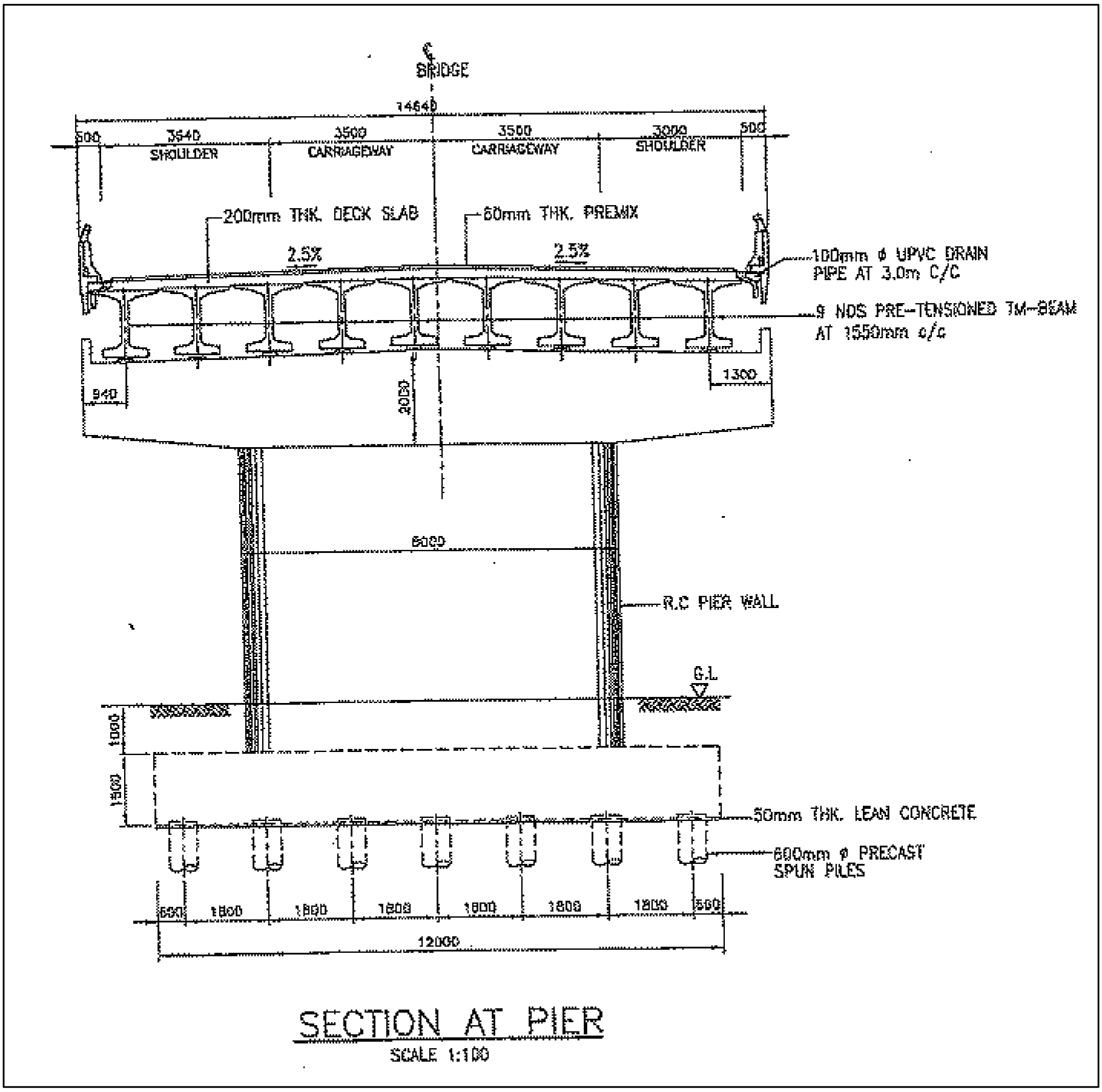

Figure A4 Drawing of section at pier

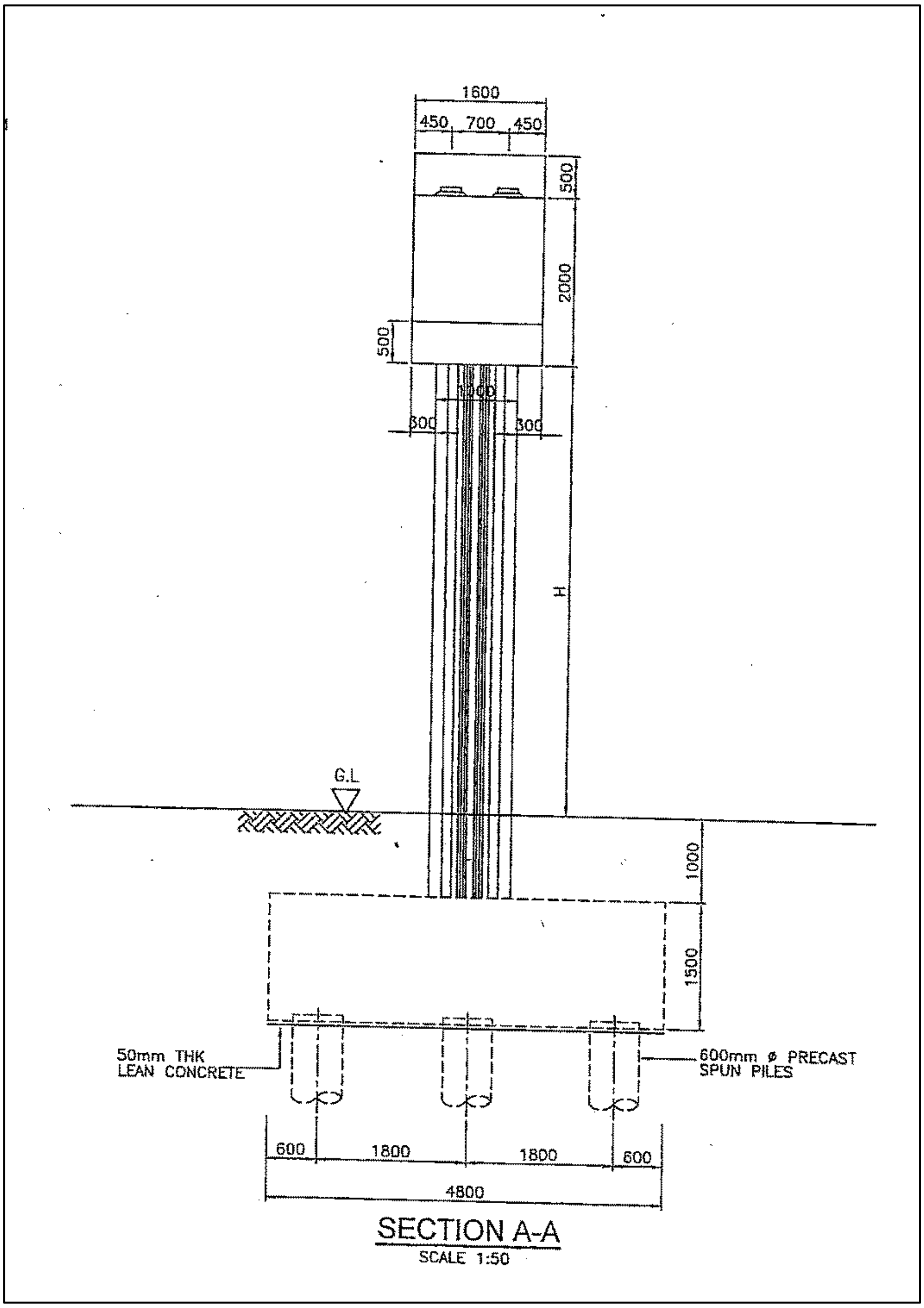

Figure A5 Drawing of section A-A at pier

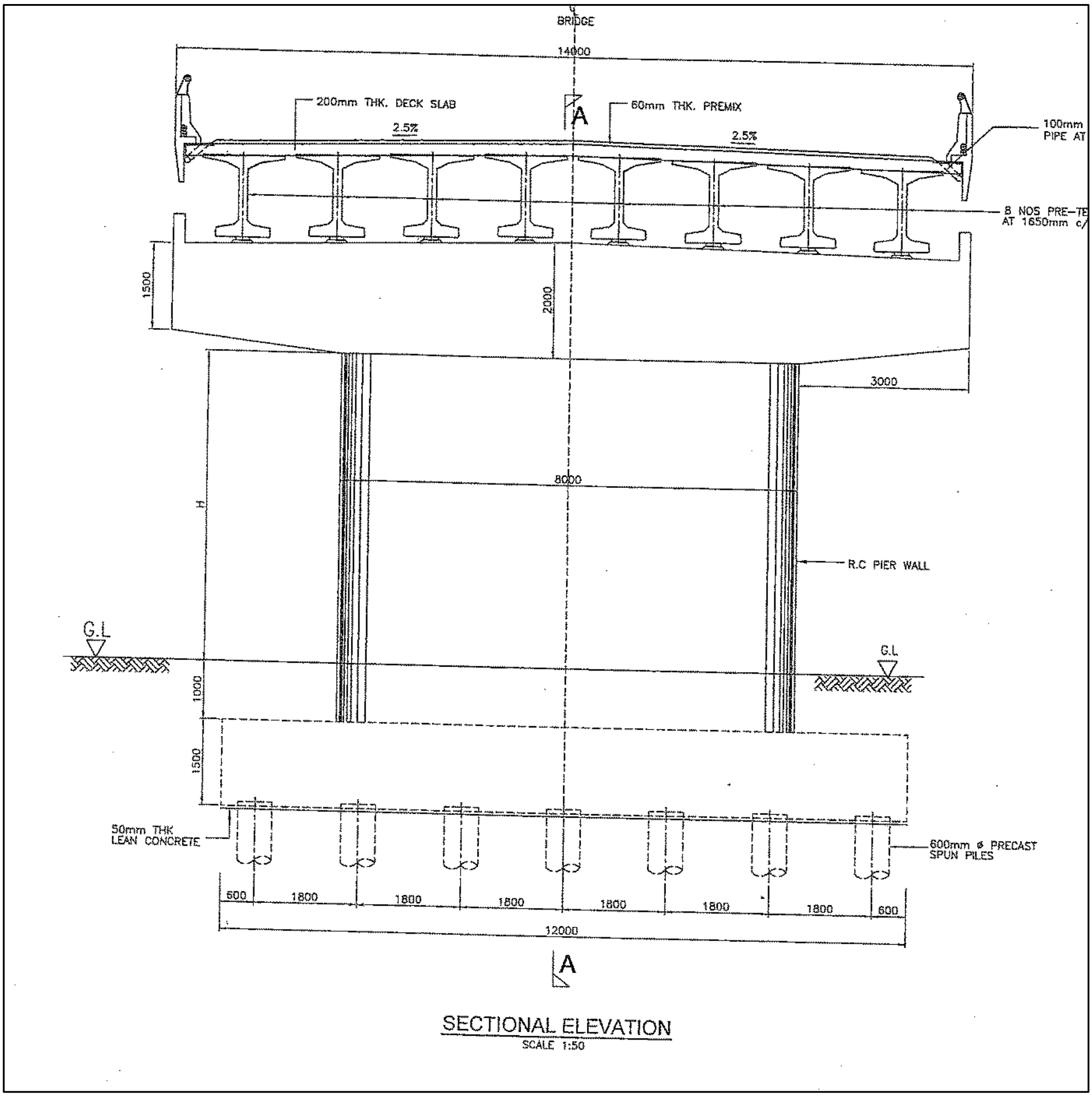

Figure A6 Drawing of sectional elevation

APPENDIX B

MSMA 2^{nd} Edition, 2012

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Malacca	1	2222001	Bukit Sebukor	95.823	0.169	0.660	0.947
	2	2224038	Chin Chin Tepi Jalan	54.241	0.161	0.114	0.846
	3	2321006	Ladang Lendu	72.163	0.184	0.376	0.900
Negeri Sembilan	1	2719001	Setor JPS Sikamat	52.823	0.167	0.159	0.811
	2	2722202	Kg Sawah Lebar K Pilah	44.811	0.181	0.137	0.811
	3	2723002	Sungai Kepis	54.400	0.176	0.134	0.842
	4	2725083	Ladang New Rompin	57.616	0.191	0.224	0.817
	5	2920012	Petaling K Kelawang	50.749	0.173	0.235	0.854
Pahang	1	2630001	Sungai Pukim	46.577	0.232	0.169	0.687
	2	2634193	Sungai Anak Endau	66.179	0.182	0.081	0.589
	3	2828173	Kg Gambir	47.701	0.182	0.096	0.715
	4	3026156	Pos Iskandar	47.452	0.184	0.071	0.780
	5	3121143	Simpang Pelangai	57.109	0.165	0.190	0.867
	6	3134165	Dispensari Nenasi	61.697	0.152	0.120	0.593
	7	3231163	Kg Unchang	55.568	0.179	0.096	0.649
	8	3424081	JPS Temerloh	73.141	0.173	0.577	0.896
	9	3533102	Rumah Pam Pahang Tua	58.483	0.212	0.197	0.586
	10	3628001	Pintu Kaw. Pulau Kertam	50.024	0.211	0.089	0.716
	11	3818054	Setor JPS Raub	53.115	0.168	0.191	0.833
	12	3924072	Rmh Pam Paya Kangsar	62.301	0.167	0.363	0.868
	13	3930012	Sungai Lembing PCC Mill	45.999	0.210	0.074	0.590
	14	4023001	Kg Sungai Yap	65.914	0.195	0.252	0.817
	15	4127001	Hulu Tekai Kwsn."B"	59.861	0.226	0.213	0.762
	16	4219001	Bukit Bentong	73.676	0.165	0.384	0.879
	17	4223115	Kg Merting	52.731	0.184	0.096	0.805
	18	4513033	Gunung Brinchang	42.004	0.164	0.046	0.802

Figure B1 Fitting constant for the IDF Empirical Equation

No. of Block	Storm Duration								
	15-min	30-min	60-min	180-min	6-hr	12-hr	24-hr	48-hr	72-hr
1	0.255	0.124	0.053	0.053	0.044	0.045	0.022	0.027	0.016
2	0.376	0.130	0.059	0.061	0.081	0.048	0.024	0.028	0.023
3	0.370	0.365	0.063	0.063	0.083	0.064	0.029	0.029	0.027
4		0.152	0.087	0.080	0.090	0.106	0.031	0.033	0.033
5		0.126	0.103	0.128	0.106	0.124	0.032	0.037	0.036
6		0.103	0.153	0.151	0.115	0.146	0.035	0.040	0.043
7			0.110	0.129	0.114	0.127	0.039	0.046	0.047
8			0.088	0.097	0.090	0.116	0.042	0.048	0.049
9			0.069	0.079	0.085	0.081	0.050	0.049	0.049
10			0.060	0.062	0.081	0.056	0.054	0.054	0.051
11			0.057	0.054	0.074	0.046	0.065	0.058	0.067
12			0.046	0.042	0.037	0.041	0.093	0.065	0.079
13							0.083	0.060	0.068
14							0.057	0.055	0.057
15							0.052	0.053	0.050
16							0.047	0.048	0.049
17							0.040	0.046	0.048
18							0.039	0.044	0.043
19							0.033	0.038	0.038
20							0.031	0.034	0.035
21							0.029	0.030	0.030
22							0.028	0.029	0.024
23							0.024	0.028	0.022
24							0.020	0.019	0.016

Figure B2 Normalised design rainfall temporal pattern