ANALYSIS OF WATER PROFILE AT THE BUKIT KUANG BRIDGE, KEMAMAN

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B. ENG (HONS.) CIVIL ENGINEERING

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ANALYSIS OF WATER PROFILE AT THE
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FARRA INTAN SYAMIRA BINTI ZULKIFLI

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ABSTRAK

Pembangunan yang pesat berlaku berhampiran kawasan sungai telah mengakibatkan peningkatan permintaan infrastruktur sungai seperti jambatan. Jambatan baru yang dikenali sebagai Jambatan Bukit Kuang telah dibina menyeberangi Sungai Chukai dan telah dibuka pada tahun 2014. Ia penting untuk memahami ciri hidrologi dan hidraulik sungai untuk memastikan kecekapan reka bentuk jambatan dan keselamatannya. Analisis profil air telah dijalankan untuk menentukan paras air di antara dua keadaan iaitu jambatan lama dan jambatan baru. Jambatan lama mempunyai sepuluh tiang manakala untuk jambatan baru mempunyai dua tiang. Simulasi menggunakan data hujan 2013 hingga 2018 sebagai input kepada HEC-HMS menghasilkan aliran puncak dan hidrograf untuk ARI 5 tahun, 10 tahun, 20 tahun, 50 tahun dan 100 tahun. Hasil simulasi HEC-HMS digunakan di HEC-RAS untuk menentukan profil air untuk 2,700 meter sebelum dan selepas lokasi jambatan jambatan. Dari analisis di jambatan baru, paras air di ARI 100 tahun tidak akan melimpah di sepanjang sungai kerana kedalaman sungai ini telah dikorek untuk membolehkan kapal-kapal lalu.

ABSTRACT

Massive development occurs near the river area which resulted in increase in demand of river infrastructure such as bridge. A new replacement bridge crossing the Chukai River called the Bukit Kuang Bridge, Kemaman was opened on 2014. 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. Analysis of water profiles were carried out to determine the water levels at the old bridge piers and the new bridge piers. The old bridge had ten piers while for the new bridge has two piers. The new replacement bridge was design to allow ships to pass through. Simulation using the 2013 to 2018 rainfall data as input to HEC-HMS produced peak flows and hydrographs for 5-year, 10-year, 20-year, 50-year and 100-year ARI. Output of HEC-HMS were used in HEC-RAS to determine the water profile for 2,700 meter before and after the bridge piers location. From the analysis at the new bridge piers, the water level at 100-year ARI shall not overflow along the river since the river was deepen to allow ships passing through.

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LIST OF SYMBOLS

A	Catchment area
B	Top width
C	Runoff coefficient
d	Depth
I	Rainfall intensity
L	Main stream length
N	Manning roughness coefficient
P	Wetted perimeter
Q	Flow rate
Q_b	Baseflow
R	Catchment storage coefficient
S	Weighted slope of main stream
T	Time
T_c	Time of concentration
V	Velocity

LIST OF ABBREVIATIONS

ARI	Annual recurrence interval
cfs	Cubic feet per second
CH	Chainage
CMSB	Cergas Murni Sdn. Bhd.
DID	Department of Irrigation and Drainage
JUPEM	Department of Survey and Mapping Malaysia
m/s	Meter per second
m^3/s	Cubic meter per second
MACRES	Malaysia Remote Sensing Agency
MSMA	Urban Stormwater Management Manual for Malaysia

CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

Floods are natural phenomena. Floods are temporary overflow of a normally dry area due to overflow of a body of water, unusual build up, runoff of surface waters, or abnormal erosion or undermining of shoreline (Ostria, 2018). It causes the most damage to infrastructure compared to any other natural hazards in the world. Flood loss prevention and mitigation includes structural flood control measures such as construction of dams or river dikes and non-structural measures such as flood forecasting and warning, flood hazard and risk management, public participation and institutional arrangement (Tingsanchali, 2012). Bridge structures located over waterways are prone to failure under flood events. Failure of a bridge can impact on the community significantly by reducing the evacuation capability and recovery operations during and after a disaster.

However, all floods are not alike. There are a few types of floods such as surge flood, river flood and surface flood (Maddox, 2014). Some floods develop slowly, sometimes over a period of days. But, flash floods can develop quickly, sometimes in just a few minutes and without any visible sign of rain (Maddox, 2014). The damage from a river flood can be widespread as the overflow affects smaller rivers downstream, often causing dams and dikes to break and swamp nearby areas. This research presents the results of an investigation into flood estimation on the Chukai River, Kemaman as shown in Figure 1.1.

Figure 1.1 Chukai River (LLC, 2005)

1.2 PROBLEM STATEMENT

Malaysia is experiencing two monsoonal seasons, which have induced heavy rainfall. The increasing of rainfall intensity and longer duration of rainfall has caused a flood. Southeast Asia has long experienced a monsoon climate with dry and wet seasons. With mean annual rainfall precipitation locally in excess of 5,000 mm, the very intense rainstorms in the steep mountains of Malaysia have caused frequent and devastating flash floods (Julien, 2018).

Floods often occur in a developed area. This is because the rain would be absorbed in areas that are not developed compared to the developed area. As a result, if a huge sum of rain within the created range, as it were a small is retained into the ground and the rest will be water run-off and stream to the lower zone. In order to prevent this flood from occurring, the study should be carry out to predict the flood. Flood warning systems should be made to predict the occurrence of flood.

Generally, a great majority of bridges are built across rivers and routinely the water flow force on the pier is calculated using the methods specified in the design codes (Yin-Hui Wang, 2014). The development or renovation of bridges may require placement of bridge piers within the channel or floodplain of natural waterways. These piers would obstruct the flow and cause an increase in water levels backwater upstream of the bridge for subcritical flows (Randall J. Charbeneau, 2001). The sum of backwater caused by piers depends mainly on their geometric shape, their position within the stream, the stream rate, and the sum of channel blockage.

Bukit Kuang Bridge is the replacement bridge on Chukai River near Chukai, Kemaman, Terengganu. It is one of the bridge construction projects with a new four-lane carriageway with new vertical clearance to allow the ships to pass through.

Figure 1.2 illustrates the Bukit Kuang Bridge located across Chukai River where bridge piers act as the structural element.

Figure 1.2 Bukit Kuang Bridge, Kemaman, Terengganu

1.3 OBJECTIVES

This research outlines the following objectives:

i. To analyse the hydrological data for the critical month using HEC-HMS and HEC-RAS softwares.
ii. To determine the difference of the water levels at the old and new bridge piers.

1.4 SCOPE OF WORK

This research is to analyse the water profile and hydraulic characteristics and focus on water flow of Chukai River, Kemaman, Terengganu particularly the Bukit Kuang Bridge. This bridge, crosses the Chukai River, has a span of 770 m and a height of 15 metres.

In order to achieve the objectives, the research focused on the analysis of the water level profile with collected data. The hydrological data, such as rainfall and streamflow data were retrieved from the Department of Irrigation and Drainage Malaysia (DID). This study was conducted through January 2013 to December 2018. While, for map preparation such as base map, land use map and river map were delineated based on the topography map. In addition to this, the cross section of Bukit Kuang Bridge was provided by the Malaysian Public Works Department (MPWD).

1.5 SIGNIFICANCE OF RESEARCH

From this analysis, the cause of overflow can be determined by the analysis of the Chukai River using real rainfall events. This analysis can be used for future stream development and mitigation programme. The result of the simulation would determine the location of overflow for Chukai River. Therefore, with the simulation results gained using appropriate software, the designation for flood mitigation can be determined in order to prevent future flooding.

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.

The hydrological analysis had become a must within the advancement area. The most reason of the hydrological analysis is to form beyond any doubt that the catchment area is safe from any bad impacts due to the advancement. For the purpose of the study, the Chukai River which is the main river in Kemaman was chosen. The velocity of the river flow depends to the river gradient where the shallower the slope, the lower the water velocity.

2.2 HYDROLOGICAL CYCLE

The hydrological cycle describes exchanges of water between the oceans, atmosphere, land surface, biosphere, soils, groundwater systems and the solid Earth (Marshall, 2014). It is basic for the support of most life and ecosystems on the planet. Hydrological cycle includes various processes that change water from solid to liquid to gas form and transport it to every corner of earth's surface.

As a large portion of the water are in the sea, it is best to assume that the cycle is begun with the sea. Then, water evaporate at the surface of the ocean. Water vapour that is blown inland and precipitates over the continents has a more circuitous path back to the oceans, and can get diverted and stored in snowpack, vegetation, soils, wetlands, lakes or groundwater systems (Marshall, 2014).

Evaporation, one of the major processes in the cycle, is the exchange of water from the surface of the Earth to the environment. By evaporation, water in the fluid state is exchanged to the vaporous or vapour state. This exchange happens when a few atoms in a water mass have achieved adequate kinetic energy to eject themselves from the water surface. The main factors affecting evaporation are temperature, humidity, wind speed, and solar radiation (Britannica, 2018). Transpiration is the vanishing of water through moment pores or stomata, in the leaves of plants. For viable purposes, transpiration and the evaporation from all water, soils, snow, ice, vegetation, and different surfaces are lumped together and called evapotranspiration or total evaporation.

The change procedure from the vapour state to the fluid state is called condensation. Condensation may occur when the air contains more water vapour than it can get from a free water surface through evaporation at the common temperature. By condensation, water vapour in the air is discharged to frame precipitation. Precipitation that falls to the earth is appropriated in four principle ways. Some is come back to the atmosphere by evaporation, some might be captured by vegetation and after that evaporated from the surface of leaves, a few permeates into the dirt by infiltration, and the rest flows directly as surface runoff into the sea (Britannica, 2018). A portion of the invaded precipitation may later permeate into streams as groundwater spill over. Direct measurement of runoff is made by stream gauges and plotted against time on hydrographs. Figure 2.1 shows the major processes of hydrological cycle.

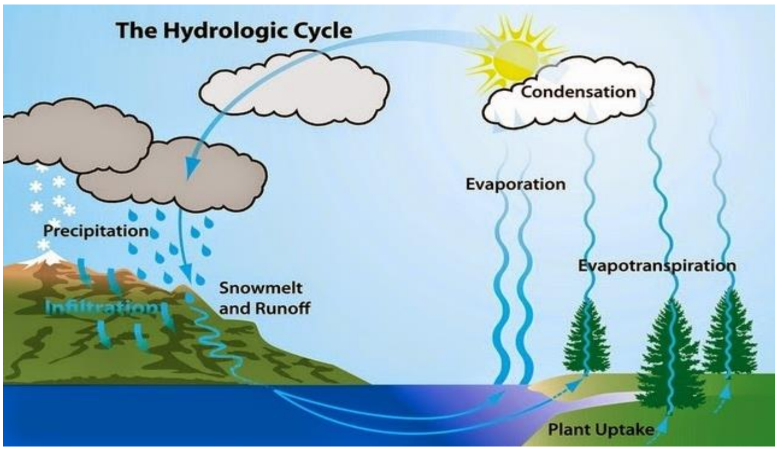


Figure 2.1 Hydrological Cycle (Antonio, 2015)

2.3 FLOOD

Flood impact is one of the most significant disasters in the world. More than half of global flood damages occur in Asia. Causes of floods are due to natural factors such as heavy rainfall, high floods and high tides and human factors such as blocking of channels or aggravation of drainage channels, improper land use, and deforestation in headwater regions. Floods result in losses of life and damage properties. Throughout Malaysia, including Sabah and Sarawak, there is total of 189 river basins (89 of the river basins are in peninsula Malaysia, 78 in Sabah and 22 in Sarawak), with the main channels flowing directly to the South China Sea and 85 of them are prone to become recurrent flooding (Garba, 2014).

Figure 2.2 shows the rainfall pattern in Malaysia and how is it influenced by the two monsoons, the south west and north east monsoons. The location of Malaysia itself consists of West Malaysia (Peninsula Malaysia) and East Malaysia (Sabah and Sarawak) and they are divided by the South China Sea (Garba, 2014).

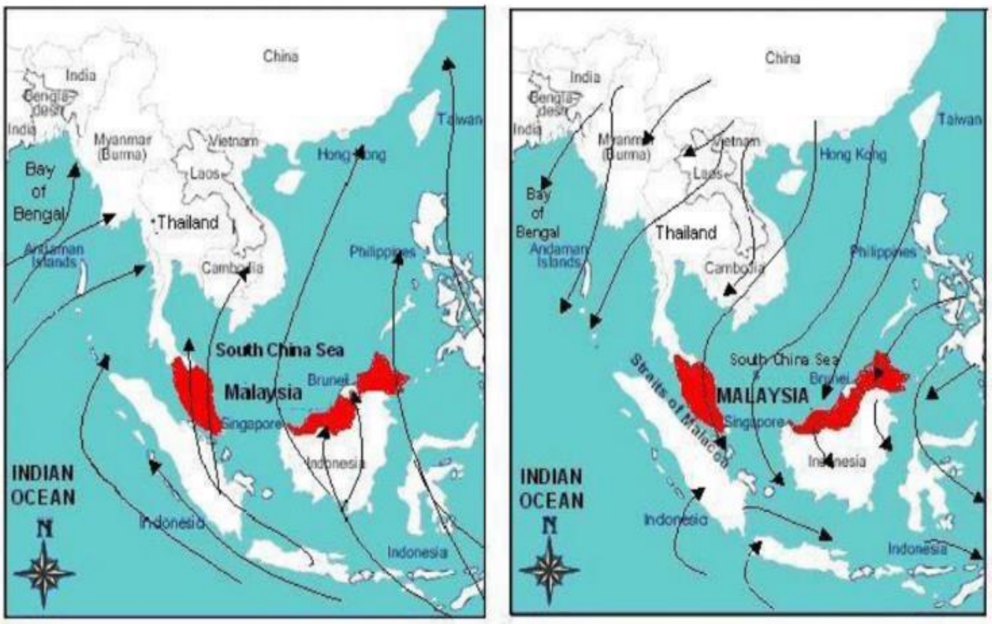


Figure 2.2 Southwest and Northeast Monsoons (Garba, 2014)

2.4 RAINFALL-RUNOFF RELATIONSHIP

Rainfall is liquid water in the form of droplets that have condensed from atmospheric water vapour and then become heavy enough to fall under gravity. It is a major component of the water cycle and is responsible for depositing most of the fresh water on the Earth. Rainfall is precipitation consisting of water drops larger than 0.5 mm. it can be classified as light rain when the intensity is smaller than 2.5 mm/h, moderate when it is between 2.5 and 7.5 mm/h and heavy when it exceeds 7.5 mm/h (Saxena, 2017).

After raining, the catchment surface area will be flowing with runoff. Runoff is described as a water flow over a surface, where then it will become stream flow when it reaches a defined channel. Rain falling on the watershed in an amount exceeding the soil or vegetation uptake becomes surface runoff. During rainfall, water is constantly being absorbed by the upper level of the soil after being intercept by vegetation within evaporate at the same time (G.Wetzel, 2001).

Runoff is the quantity of water discharged in surface streams. Runoff includes not only the waters that travel over the land surface and through channels to reach a stream but also interflow, the water that infiltrates the soil surface and travels by means of gravity toward a stream channel and eventually empties into the channel (Britannica, 2018).

Rainfall and runoff patterns affects man's activities in so many ways and as such design of agricultural, storm, water management, telecommunication, erosion, droughts and food security. 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, 2013).

$$Q = CiA \tag{2.1}$$

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)

Table 2.1 Runoff Factor for Rational Equation (DID)

Description of Area	C
Business	
Downtown	0.70 – 0.95
Neighbourhood	0.50 – 0.70
Residential	0.30 – 0.50
Single Family	0.40 – 0.60
Multi Units, Detached	0.60 – 0.75
Multi Units, Attached	
Residential Suburban	0.25 – 0.40
Apartment	0.50 – 0.70
Industrial	0.50 - 0.80
Light	0.60 – 0.90
Heavy	
Parks, Cemeteries	0.10 – 0.25
Playgrounds	0.20 – 0.35
Railroad Yard	0.20 – 0.35
Unimproved	0.10 – 0.30
Character of Surface	
Pavement	
Asphalt and Concrete	0.70 – 0.95
Bricks	0.70 – 0.85
Roof	0.75 – 0.95
Lawns, sandy soil	0.05 – 0.10
Flat, up to 2% grade	0.10 – 0.15
Average, 2% - 7 % grade	0.15 – 0.20
Steep, over 7 %	
Lawns, heavy soil	0.13 – 0.17
Flat, up to 2% grade	0.18 – 0.22
Average, 2% - 7 % grade	0.25 – 0.35
Steep, over 7 %	

Table 2.1 portrayed the list of C value for different rate of infiltration. For each type of land for the range, a range of the value of C is given. The lower number is used for storms of a low intensity, storms with greater intensity will have relatively more runoff, justifying the use of a higher C factor.

2.5 CATCHMENT AREA

Catchment area or watershed can be defined 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, 2013). Basically, for a major river basin watershed regions can be up to square miles and a couple of sections of land in an urban area. The loci focuses (the edge line) that separates two adjacent watersheds is known as watershed divide.

Bedient (2013) additionally characterize watershed with definition saying that a watershed is a contiguous area that channels to an outlet, such that precipitation that falls within the watershed runs off through that single outlet. For instance, direct runoff from the surface and stream 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 2.3 portrays the catchment region as characterized dependent on topographic or elevation data.

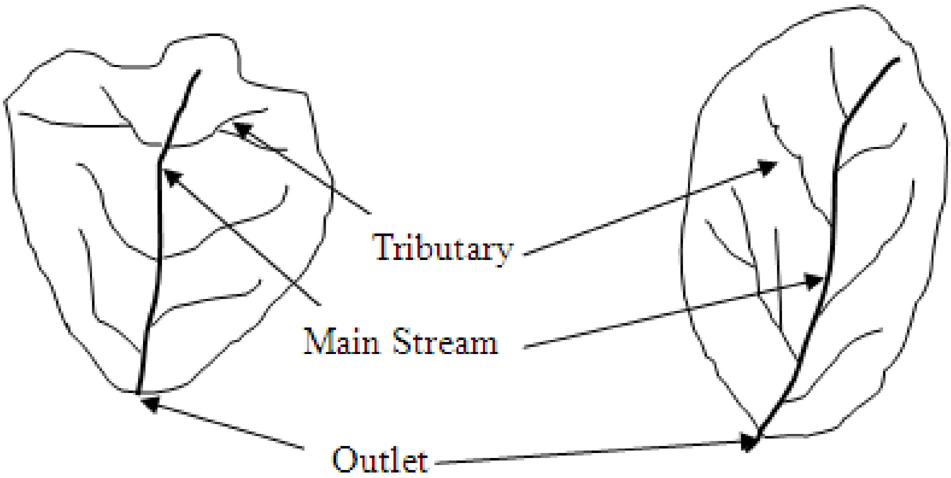


Figure 2.3 Watershed Area (Subramanya, 2008)

2.6 HYDROGRAPH

The hydrograph is a graphical representation of the temporal variation of flow is the main source of information to study flow behaviour (Ternynck, 2016). Figure 2.4 shows a typical hydrograph. Rainfall is ordinarily the most input to a watershed and the stream is considered the yield of the watershed. A hydrograph may be a representation of how a watershed reacts to rainfall. They are used in water and hydrological planning. These hydrographs are an easy and quick way to assess the hydrologic conditions in the hydrograph simulation analysis whether rivers are rising, falling or remaining steady, whether flows are high, low or near base flow; in the event that the streams have peaked or close peak (Szilagyi, 1997).

A Unit Hydrograph is by definition, a hydrograph having a volume of one inch of runoff which is associated with a precipitation event of specified duration and areal pattern (uniform over the basin). The Unit Hydrograph is a hypothetical hydrograph which is proposed to describe how a river at a specific point will respond to one inch of runoff and can be utilized to determine how the stream will respond to any amount of runoff. Based on current condition, the hydrologic models compute the sum of runoff expected at a point on a river. The computed runoff sum is then applied to the unit hydrograph at that specific point, to produce an estimate hydrograph which is particular to that area and occasion. The calculated runoff amount is then applied to the unit hydrograph at that specific point, to produce a forecast hydrograph which is specific to that area and event (Ratnayaka, 2009).

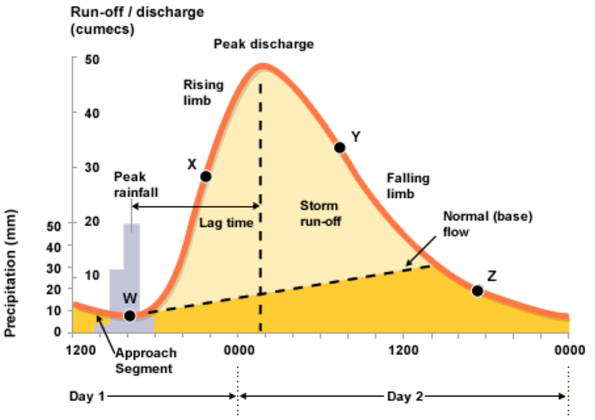


Figure 2.4 Hydrograph (BBC, 2014)

2.6.1 Clark Method

The Clark unit hydrograph is defined by two parameters, time of concentration Tc and storage coefficient R. It is a method for estimating the concentration time and storage coefficient of the Clark model using the rainfall-runoff measurements (Yoo, 2016). Based on the equation 2.2 and 2.3, the value of T_c and R are:

$$Tc = 2.32A^{-0.1188}\,L^{0.9573}\,S^{-0.5074} \qquad 2.2$$

$$R = 2.976A^{-0.1943}\,L^{0.9995}\,S^{-0.4588} \qquad 2.3$$

where:

T_c = Time of concentration

R = Catchment storage coefficient

A = Catchment area

L = Main stream length

S = Weighted slope of main stream

A base flow is required to derive the total design hydrograph. Base flow was taken for rather dry and moderate wet antecedent catchment conditions. Based on the equation 2.4, the value of Q_b is:

$$Q_b = 0.11\ A^{0.85889} \qquad 2.4$$

where:

Q_b = Baseflow (m^3 /s)

A = Catchment area ($km^{2)}$

Figure 2.5 shows the conceptual model of Clark's method.

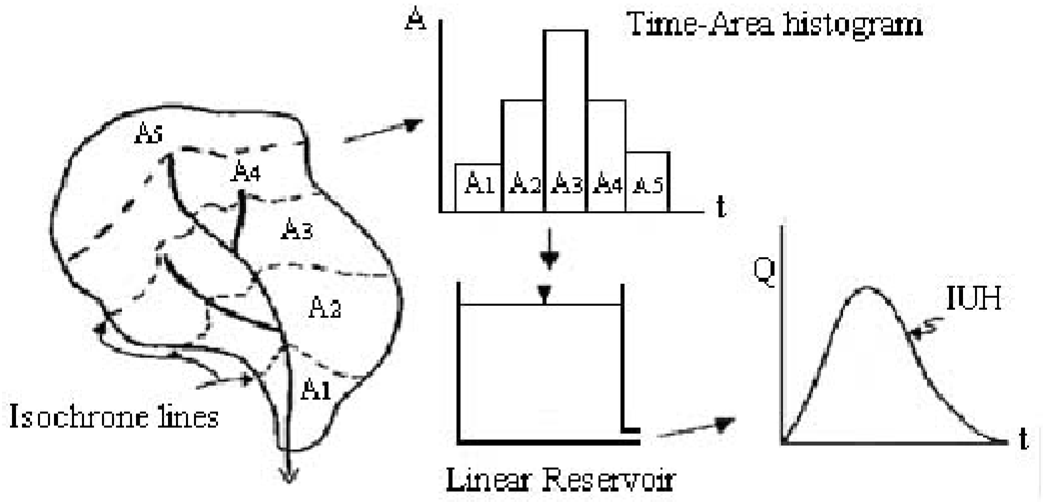


Figure 2.5 Conceptual Model of Clark's Method (Christidis, 2017)

2.7 FLOW OF WATER

Water flow is the continuous movement of water in runoff or open channels. This flow is often quantified as discharge, defined as the rate of flow or the volume of water that passes through a channel cross section in a specific period of time. The flow rate can be measured in meters cubed per second (m^3/s), or in litres per second (*L/s*). Litres are more common for measures of liquid volume, and $1\ m^3/s = 1000\ L/s$. Water flow measurement should be done when the flow is at its lowest, usually at the end of the dry season (Murdoch, 1996). For any flow, the discharge Q at a channel section is expressed by Equation 2.5.

$$Q = AV \tag{2.5}$$

where:

Q	= Liquid flow rate	(m^3/s or L/s)
A	= Area of the pipe or channel	(m^2)
V	= Velocity of the liquid	(m/s)

Discharge is commonly calculated as the product of velocity and cross-sectional area. Surface water velocity is the direction and speed with which the water is moving, measured in feet per second (ft/s) or meters per second (m/s). The cross-sectional area of an open channel is the area (ft^2 or m^2) of a slice in the water column made perpendicular to the flow direction.

2.7.1 Flow of Water in Open Channel

Open channel flow is defined as a flow of liquid that occurs in a sloped channel having solid bottom and side walls and it is open to the atmosphere at the top (Manjula, 2015). It is driven by the component of the gravitational force along the channel slope as shown in Figure 2.6. Channel slope will appear in all the open-channel flow equation, while the pipe flow equations include only the slope of the energy grade line (Houghtalen, 2000). 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, 2000). Open-channel flow must have a free surface, whereas pipe flow has none, since the water must fill the whole conduit (Chow, 1959).

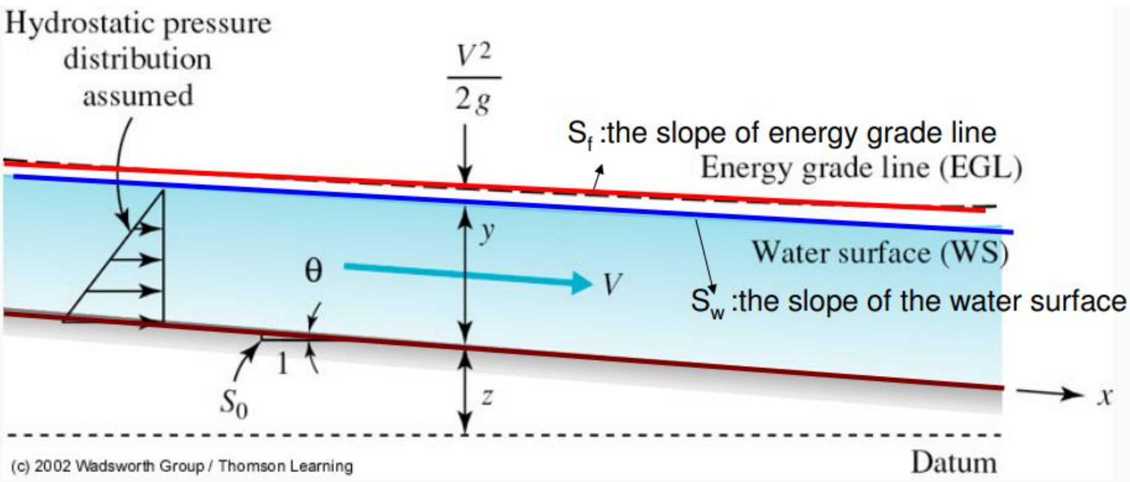


Figure 2.6 Open- Channel Flow (Chanson, 2002)

Open channel flow can be classified into numerous sorts and described in different ways as shown in Figure 2.7. The following classification is made according to the change in flow depth with respect to time and space. Firstly is classification of stream based on time as the model and the flow are steady and unsteady flow. Chow (1959) expressed 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. 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.

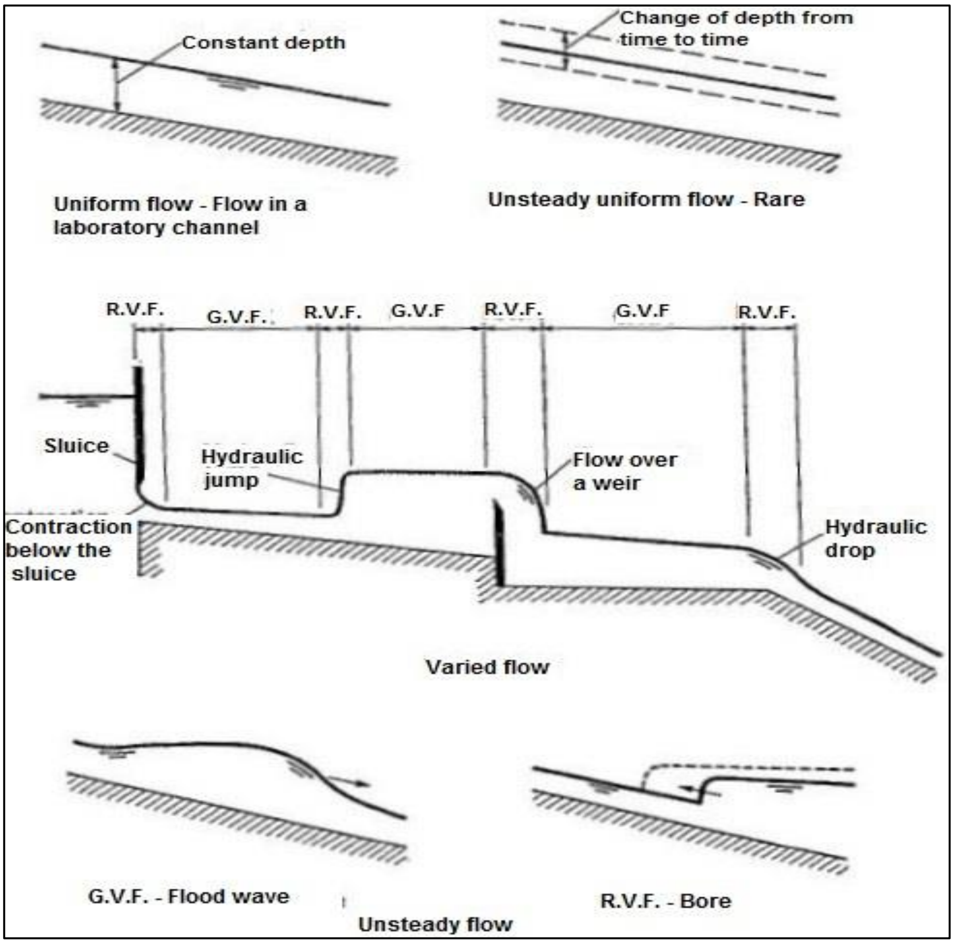


Figure 2.7 Various types of Open-Channel (Chow, 1959)

2.8 TYPES OF CHANNEL

Open channel are natural or manmade conveyance structure which has a free surface at atmospheric pressure. For example, flow in rivers, streams, flow in sanitary and storm sewers flowing partially full. 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 2.8).

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 2.6 and 2.7 respectively.

$$R = \frac{A}{P} \qquad 2.6$$

$$D = \frac{A}{B} \qquad 2.7$$

where

A = flow area

P = wetted perimeter

B = top width

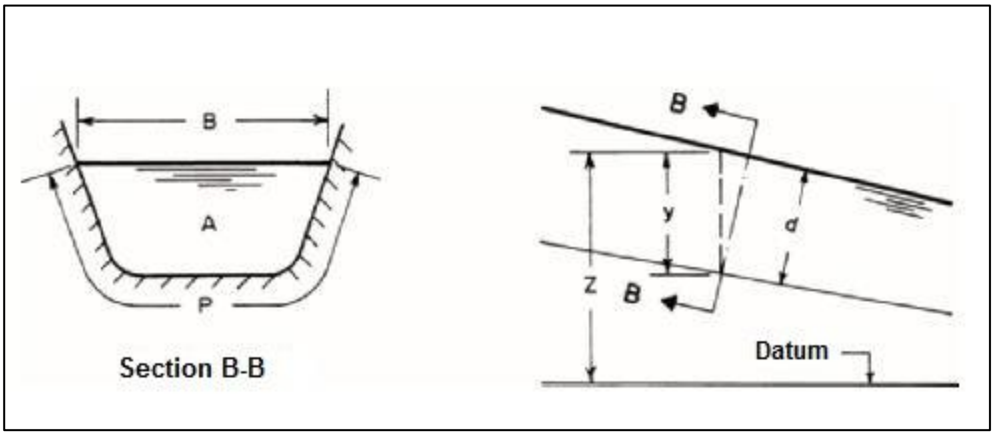


Figure 2.8 Definition sketch (Chaudhry, 2008)

Expression for A, P, D and R for typical channel cross sections are presented in Figure 2.9. It can be noted, in this research the type of channel is analysed as a rectangular channel.

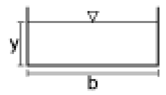
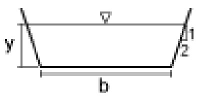
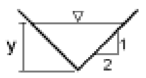
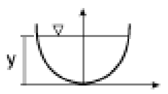
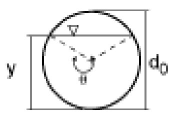
Channel type	Area A	Wetted permiter P	Hydraulic radius R	Top width T	Hydraulic depth D
	by	$\frac{by}{b+2y}$	$\frac{by}{b+2y}$	b	y
	$b+2y$	$b+2y\sqrt{1+z^2}$	$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$	$b+2zy$	$\frac{(b+zy)y}{b+2zy}$
	zy^2	$2y\sqrt{1+z^2}$	$\frac{zy}{2\sqrt{1+z^2}}$	$2zy$	$\frac{1}{2}y$
	$\frac{2}{3}Ty$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

Figure 2.9 Characteristics of typical channel cross section (Braca, 2008)

2.9 TYPES OF SOFTWARES

2.9.1 HEC-RAS

HEC-RAS is designed to perform one and two-dimensional hydraulic calculations for a full network of natural and constructed channels. It is a one dimensional hydraulic modelling program based on 4 types of analysis in rivers which are steady flow models, unsteady flow models, sediment transport models and water quality analysis (Polo, 2015). It allows simulating flow in natural riverbeds or artificial channels to determine the water level being its main goal develop flood studies and determine floodable areas.

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.

For steady flow models components, it is intended for calculating water surface profiles for steady gradually varied flow. The system can handle a full network of channels, a dendritic system, or a single river reach. The steady flow component is capable of modelling subcritical, supercritical, and mixed flow regimes water surface profiles. Figure 2.10 shows an example on how a steady flow water surface profile look like.

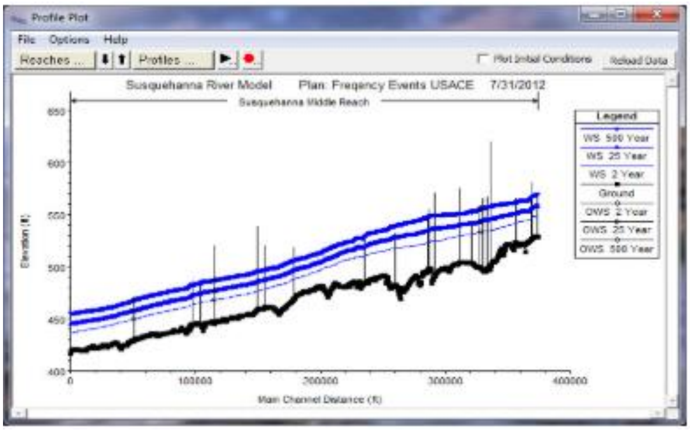


Figure 2.10 Steady Flow Water Surface Profiles (Engineers U. A., 2016)

Meanwhile, for unsteady flow component can be used to perform subcritical, supercritical and mixed flow regime (subcritical, supercritical, hydraulics jumps and drawdowns) calculations in the unsteady flow computations module. The hydraulics calculations for cross sections, bridges, culverts and other hydraulic structure that were developed for the steady flow component were incorporated into the unsteady flow module. Figure 2.11 shows one and two dimensional unsteady flow simulation.

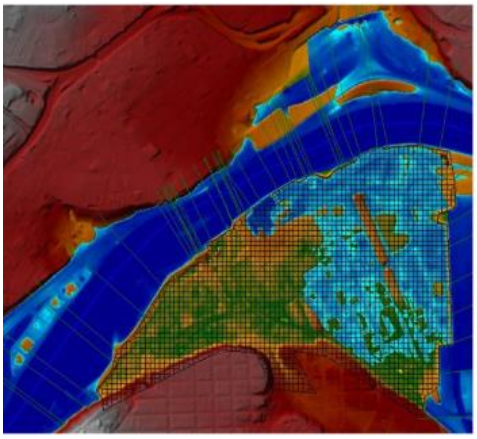


Figure 2.11 One and Two-Dimensional Unsteady Flow Simulation

(Engineers U. A., 2016)

For sediment transport models component, it is intended for the simulation of one-dimensional transport boundary calculations resulting from scour and deposition over moderate time periods. The model is designed to simulate long-term trends of scour and deposition in a stream channel that might result from modifying the frequency and duration of the water discharge and stage, or modifying the channel geometry. This system can be used to evaluate deposition in reservoirs, design channel contractions required to maintain navigation depths, predict the influence of dredging on the rate of deposition, estimate maximum possible scour during large flood events, and evaluate sedimentation in fixed channels (Engineers U. A., 2016).

While for water quality analysis, this component is intended to allow the user to perform riverine water quality analyses. An advection-dispersion module is included with this version of HEC–RAS, adding the capability to model water temperature (Engineers U. A., 2016).

2.9.2 HEC-HMS

The Hydrologic Modelling System (HEC-HMS). Figure 2.12 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 (Engineers U. A., 2018).

The software features a completely integrated work environment including a database, information passage utilities, computation engine, and detailing devices. Simulation results are stored in HEC-DSS (Data Storage System) and can be used in conjunction with other computer program for studies of water availability, urban drainage, flow estimating, future urbanization affect, reservoir spillway design, flood damage reduction, floodplain control and frameworks operation.

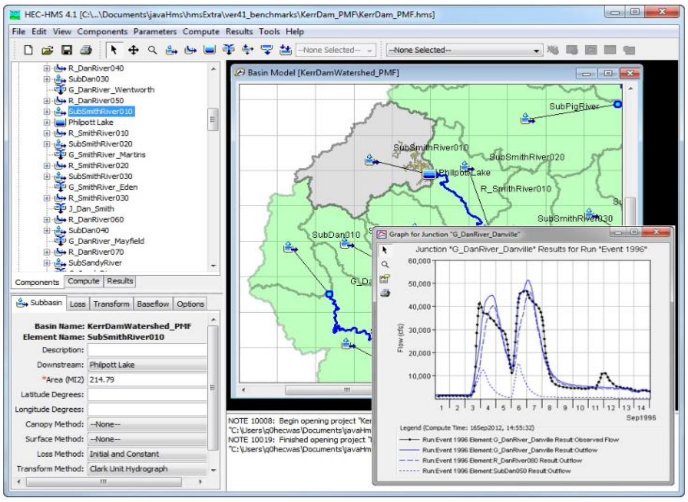


Figure 2.12 The Hydrologic Modelling System (HEC-HMS)

(Engineers U. A., 2018)

2.9.3 AGIS Software

AGIS Software can assist user with plotting their geographical information or download and display information from a variety of sources on the web. A section from that, the software has a multi-document interface and can be utilized 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, ARCInfo and Garmin GPS. Furthermore, a built-in scripting language can be used to create animation and automate map displays using data from other sources, such as 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).

2.9.4 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, 2018). InfoWorks RS is one of the modelling programs that combine the advanced flow simulation engine, both hydrological and hydraulic models, Geographic Information Systems (GIS) functionality and database storage within one single environment. InfoWorks RS allows planners and engineers to carry out fast and accurate modelling of the key elements of river and channel systems.

InfoWorks RS includes full arrangement modelling of open channels, floodplains, embankments and hydraulic structures. Rainfall-runoff simulation is available using both event based and conceptual hydrological methods. Full interactive views of data are available using geographical plan views, sectional view, long sections, spreadsheet and time varying graphical data. The fundamental information can be accessed from any graphical or geographical view.

Animated presentation of results in topographical plan, long section and cross section views is standard, together with results reporting and analysis using tables and graphs (Figure 2.13). Full flood-mapping capability is provided based on a sophisticated flood-interpolation model overlaid onto an imported ground model. An optional fully-dynamic, 2D surface flood simulation, integrated with the surface-channel hydraulic simulation also available (Infoworks, 2018).

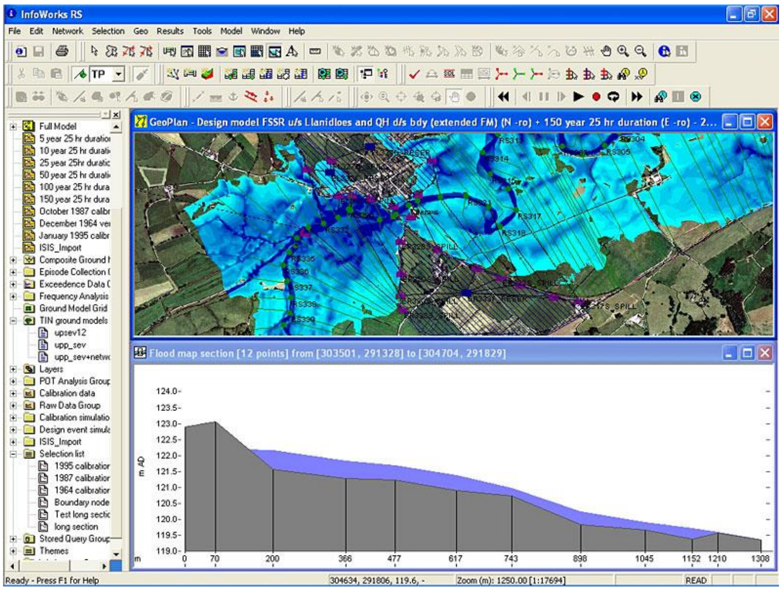


Figure 2.13 InfoWorks RS (Infoworks, 2018)

2.9.5 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 MIKE 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 show a variety of tasks related to river hydraulics, water quality, flooding, determining, navigation as well as catchment flow and runoff (Figure 2.14). It gives the big differences in calculation features and add-on module enables river engineer to conduct all required demonstrating activities within one displaying package.

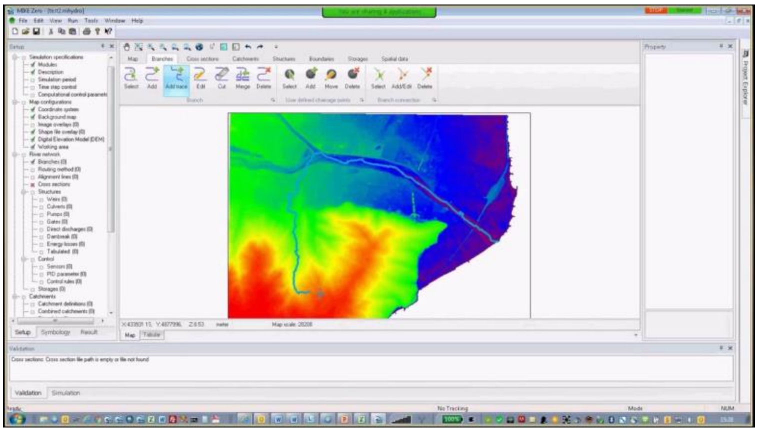


Figure 2.14 Mike HYDRO River (MIKE, 2018)

CHAPTER 3

METHODOLOGY

3.1 INTRODUCTION

To conduct this study, several methods of analysis need to be taken in order to achieve the objectives and for a good result. The objectives includes to determine the difference of the water levels at the old and new bridge piers. In addition, the simulation was also carried out to analyse the hydrological data for the critical month using HEC-RAS software. The method conducted includes data collection, data analysis, simulation of the river and bridge 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 precisely follow the specification of the software in order to give an exact simulation. There are numerous 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 outlined the methodology for this study.

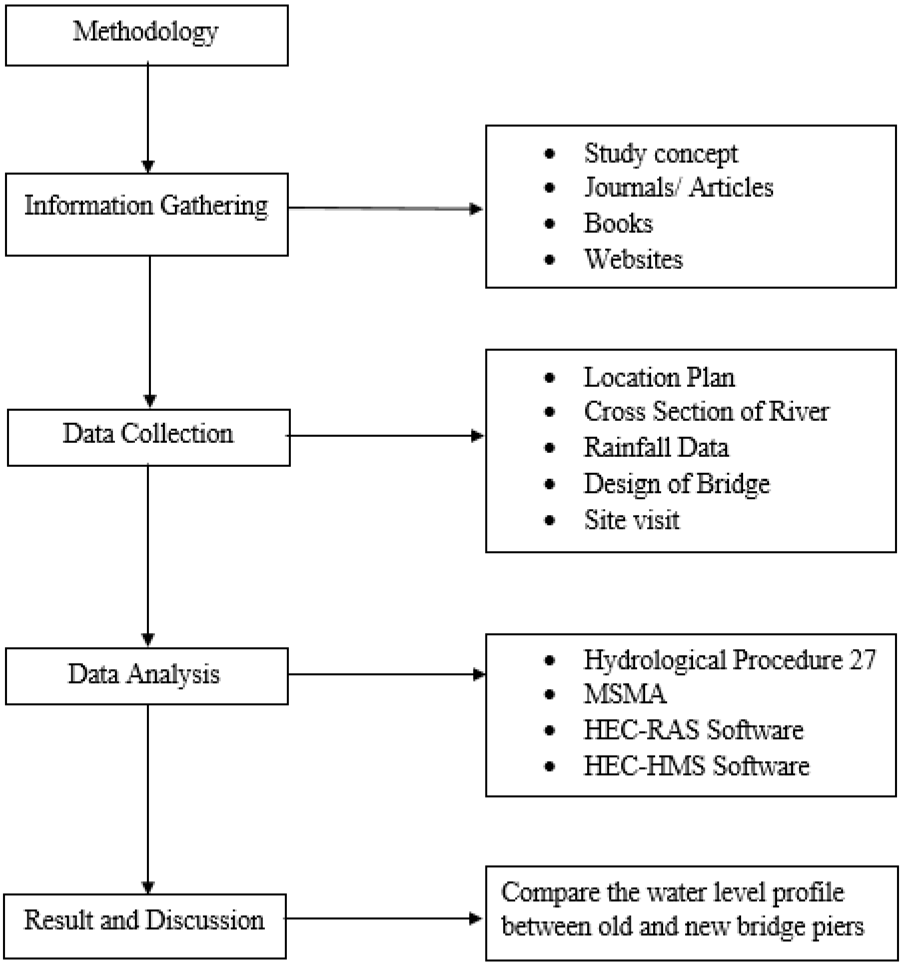


Figure 3.1 Methodology of the Study

3.2 REFERENCES 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, information such as river cross section, hydraulics and hydrological data related to the Chukai River was collected from the corresponding official departments and contractor of the bridge project at the Chukai 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 Chukai River were obtained in order to run the simulation successfully. The primary data required are listed in the Table 3.1:

Table 3.1 The necessary information and data required

No	Type of information	Format	Data properties	Sources
1	Topographical map	Digital	Identification of catchment area and river channel	JUPEM
2	Location Plan	Digital or hard copy	Reference	JUPEM
3	River Cross Section	Digital	Main input	DID
4	Rainfall data	Digital	Hydrological analysis	DID
5	Satellite image	Digital	Catchment activities	MACRES
6	Bridge Design	Digital	Reference	CMSB

NOTE:

JUPEM : Surveying and Mapping Department Malaysia

DID : Department of Irrigation and Drainage Malaysia

MACRES : Malaysia Remote Sensing Centre

CMSB : Cergas Murni Sdn. Bhd.

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 collected in the form of digital from the Department of Irrigation and Drainage (DID). In this software application the digital cross section is used which are presented in Microsoft Excel form and it is 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 Department of Irrigation and Drainage Malaysia (DID) from 1st January 2013 to 31st December 2018. The data showed the rainfall depth for daily total. 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 JPS Kemaman rainfall station from 1st January to 7th January 2018.

Table 3.2 Rainfall Data for Sungai Chukai

No.	Station name	Date	RF Daily (mm)
1	JPS Kemaman	01/01/2018	59.00
2	JPS Kemaman	02/01/2018	75.00
3	JPS Kemaman	03/01/2018	88.00
4	JPS Kemaman	04/01/2018	0.00
5	JPS Kemaman	05/01/2018	0.00
6	JPS Kemaman	06/01/2018	0.00
7	JPS Kemaman	07/01/2018	12.00

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 equation 3.1 and 3.2, the value of Tc and R are:

$$Tc = 2.32A^{-0.1188}\,L^{0.9573}\,S^{-0.5074} \qquad 3.1$$

$$R = 2.976A^{-0.1943}\,L^{0.9995}\,S^{-0.4588} \qquad 3.2$$

where:

Tc = Time of concentration

R = Catchment storage coefficient

A = Catchment area (20.40 km²)

L = Main stream length (23.70 km)

S = Weighted slope of main stream (80.17)

$$Tc = 2.32\,(20.4)^{-0.1188}\,(23.70)^{0.9573}\,(80.17)^{-0.5074}$$

$$= 3.63 \text{ hours}$$

$$R = 2.976\,(20.4)^{-0.1943}\,(23.70)^{0.995}\,(80.17)^{-0.4588}$$

$$= 5.09$$

A base flow is required to derive the total design hydrograph. Based on the equation 3.3, the value of Q_b is:

$$Q_b = 0.11\ A^{0.85889} \tag{3.3}$$

where:

Q_b = Baseflow (m^3 /s)

A = Catchment area (km^2)

$Q_b = 0.11\ (20.4)^{0.85889}$

$= 1.47\ m^3/s$

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. The formula use is based on Equation 3.4:

$$i = \frac{\lambda T^K}{(d + \theta)^\eta} \tag{3.4}$$

where:

λ = 55.899

K = 0.201

θ = 0.000

η = 0.580

According to MSMA, the location of the nearest rainfall station with the study area is JPS Kemaman station. Thus, the fitting constant value for the IDF Empirical equation for JPS Kemaman station can be used in this study. The value of the rainfall intensity is as listed in Table 3.3:

Table 3.3 Rainfall intensity, *i* for various ARI

ARI	Rainfall Intensity, *i* (mm/hr)
5-year ARI	36.57
10-year ARI	42.04
20-year ARI	48.32
50-year ARI	58.10
100-year ARI	66.78

Storm profile also known as temporal pattern with durations of 180 minutes is used to calculate for station JPS Kemaman. The required input parameters are storm duration and total storm depth where the mass curves of selected duration were constructed and temporal smoothing has been carried out by means of mass curve average. For the Chukai River, it is categorised as Region 1. The rainfall depth for this station is shown in Table 3.4:

Table 3.4 Rainfall depth for various ARI

No. of block	5-year ARI	10-year ARI	20-year ARI	50-year ARI	100-year ARI
1	9.43	10.83	12.45	14.97	17.21
2	7.97	9.15	10.52	12.65	14.54
3	7.83	9.00	10.35	12.44	14.30
4	7.97	9.15	10.52	12.65	14.54
5	8.10	9.30	10.70	12.86	14.79
6	10.62	12.20	14.03	16.87	19.39
7	10.35	11.90	13.68	16.45	18.91
8	13.28	15.25	17.54	21.09	24.24
9	15.93	18.30	21.05	25.31	29.09
10	14.60	16.78	19.29	23.20	26.67
11	17.52	20.13	23.15	27.84	32.00
12	9.16	10.52	12.10	14.55	16.73

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 procedures listed below needed to be carried out in order to achieve 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 Figure 3.2

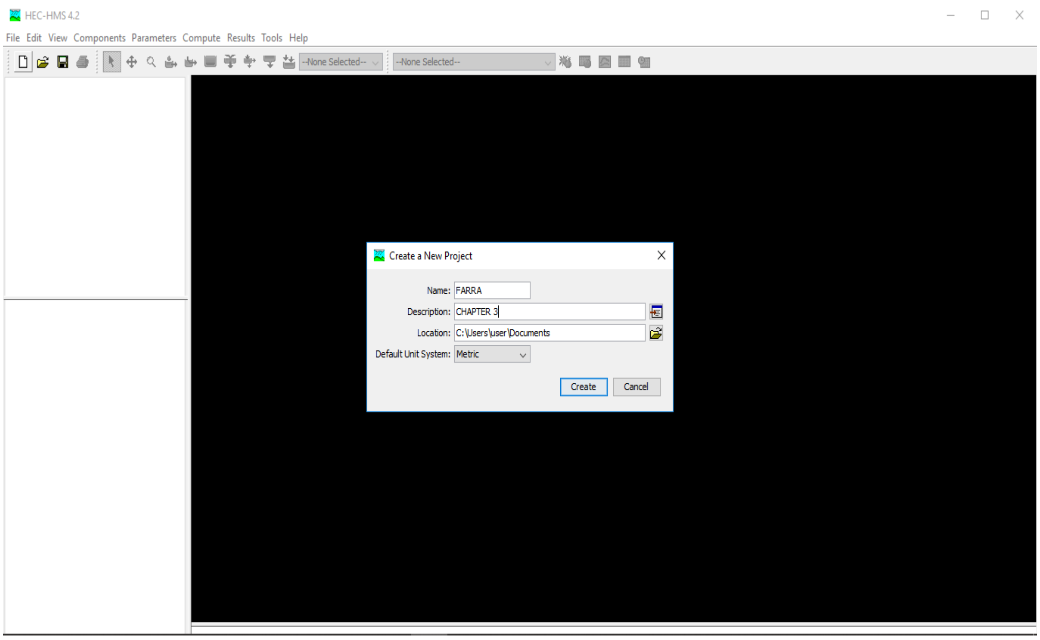


Figure 3.2 Create a new project

2. Next, create a new basin model by go to Components and click on Basin Model Manager as shown in the Figure 3.3.

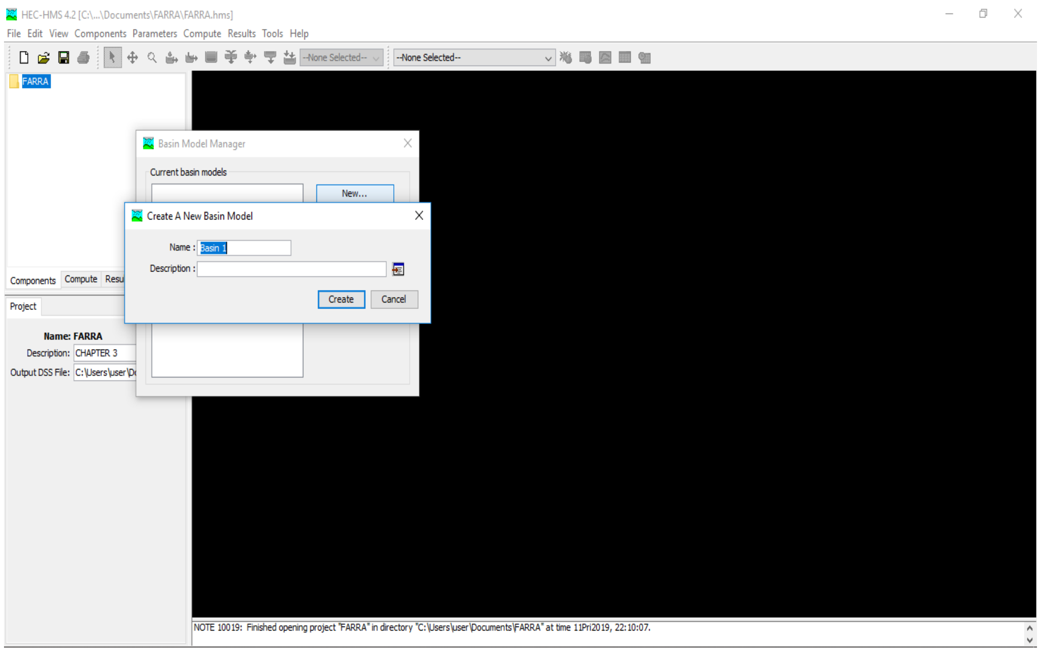


Figure 3.3 Create a new basin model

3. After that, click on **Sub-basin Creation Tool** before a new sub-basin can be create as shown in the Figure 3.4.

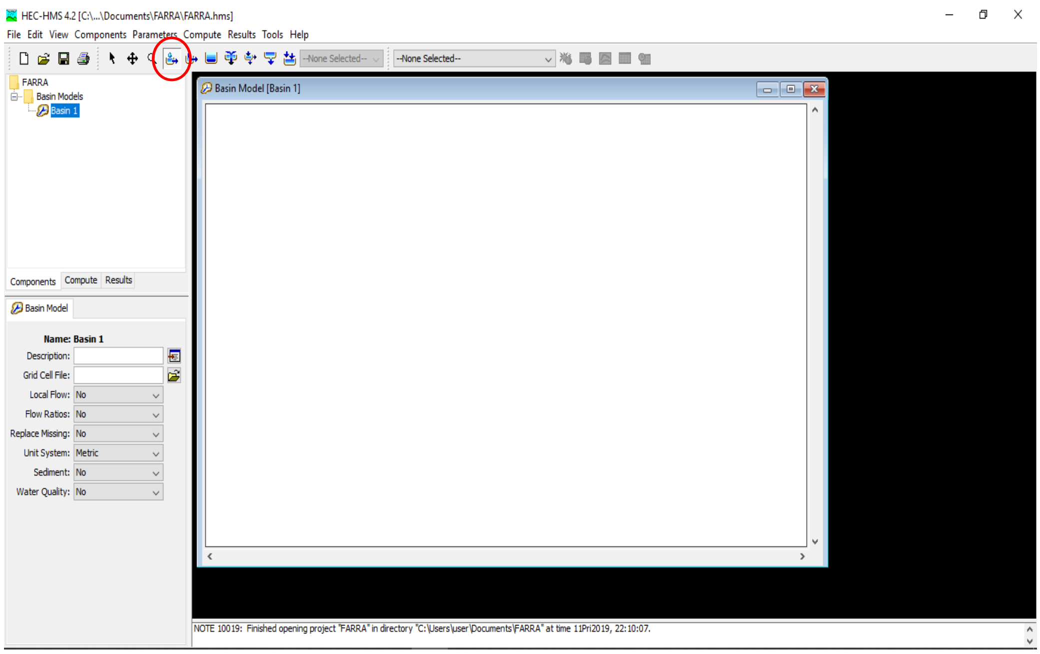


Figure 3.4 Click on Sub-basin Creation Tool to start create a new sub-basin

4. Next, click on the **Basin Model** commend to make one sub-basin for the catchment area as shown in the Figure 3.5.

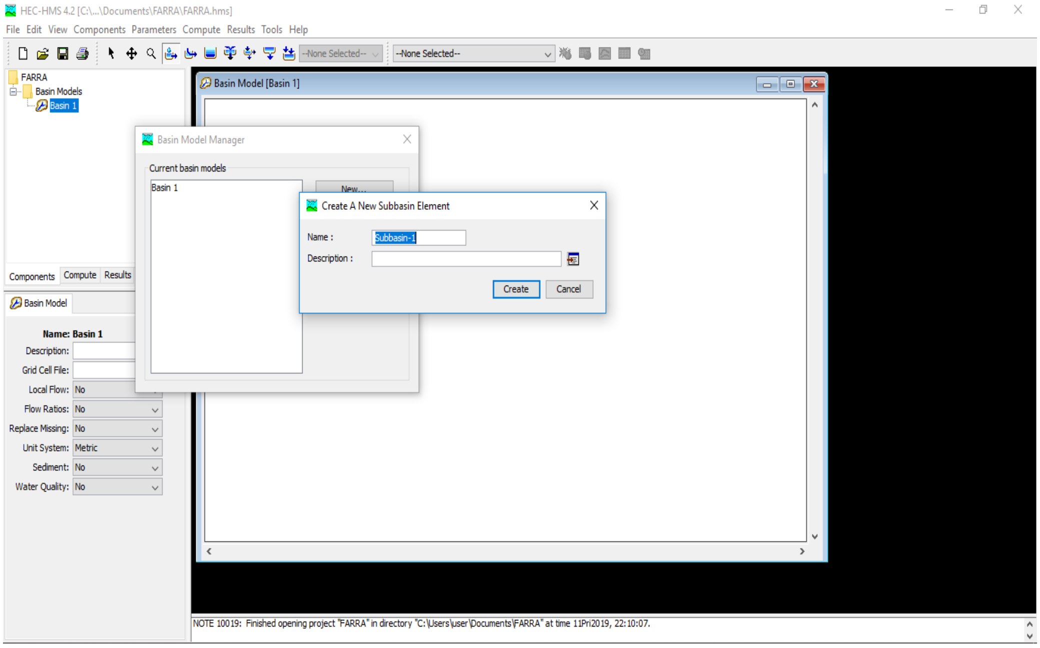


Figure 3.5 Create a new sub-basin

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 3.6.

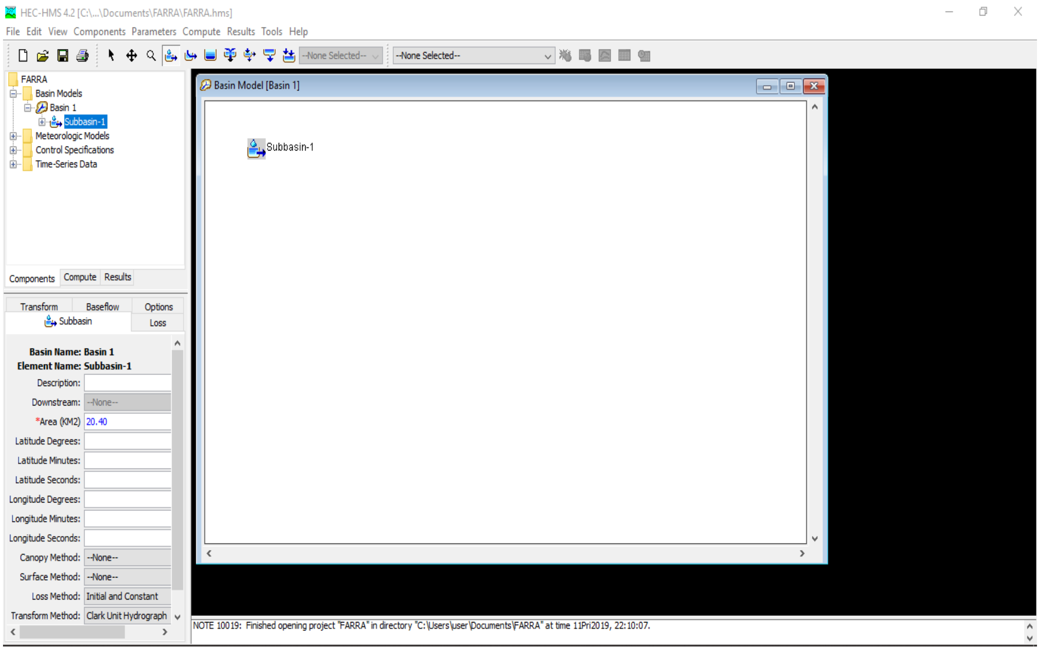


Figure 3.6 Fill in the component for the sub-basin

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 3.7.

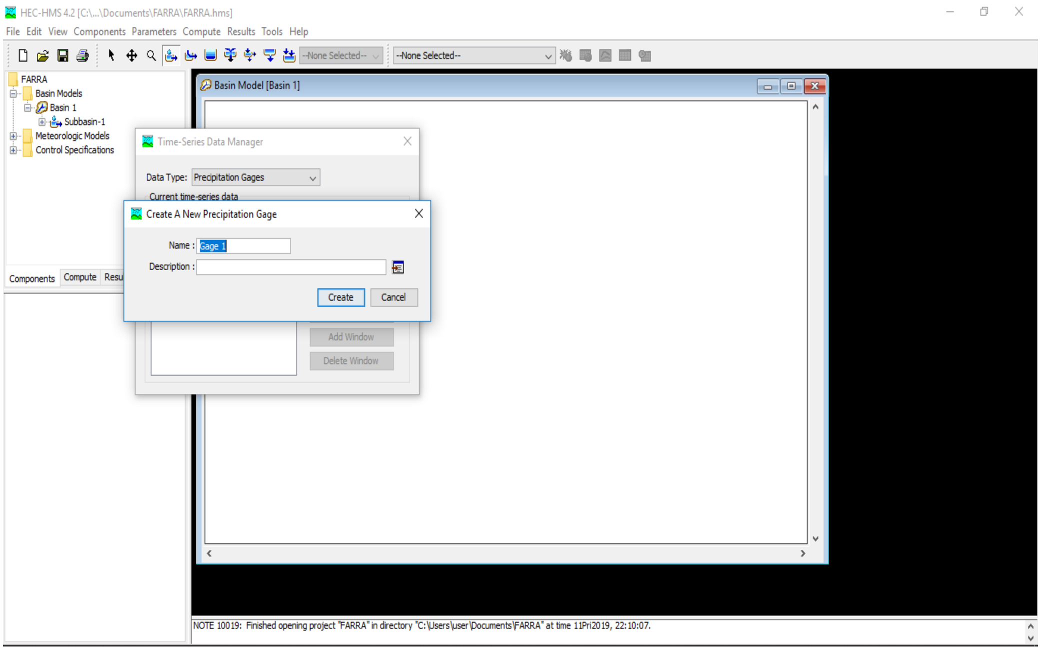


Figure 3.7 Click Time Series Data Manager to create a new time Series data Manager

7. At **Time Series Gage** field, choose 15 minutes for the time interval on the chosen date. Fill in the precipitation value as shown in the Figure 3.8 and Figure 3.9.

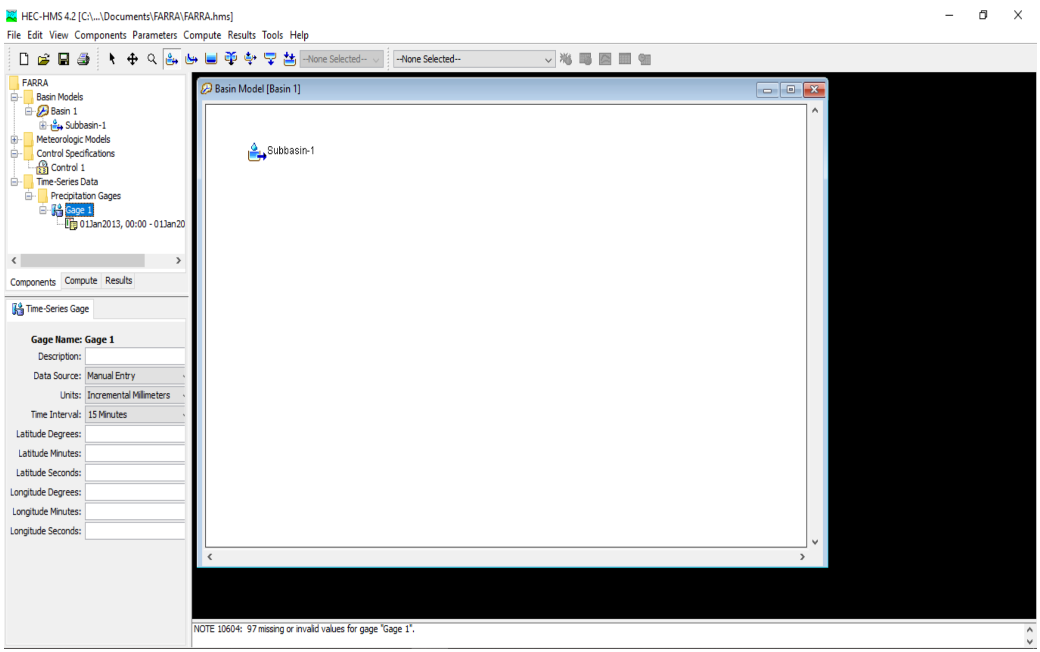


Figure 3.8 Fill in the requirement data in Time Series Gage field

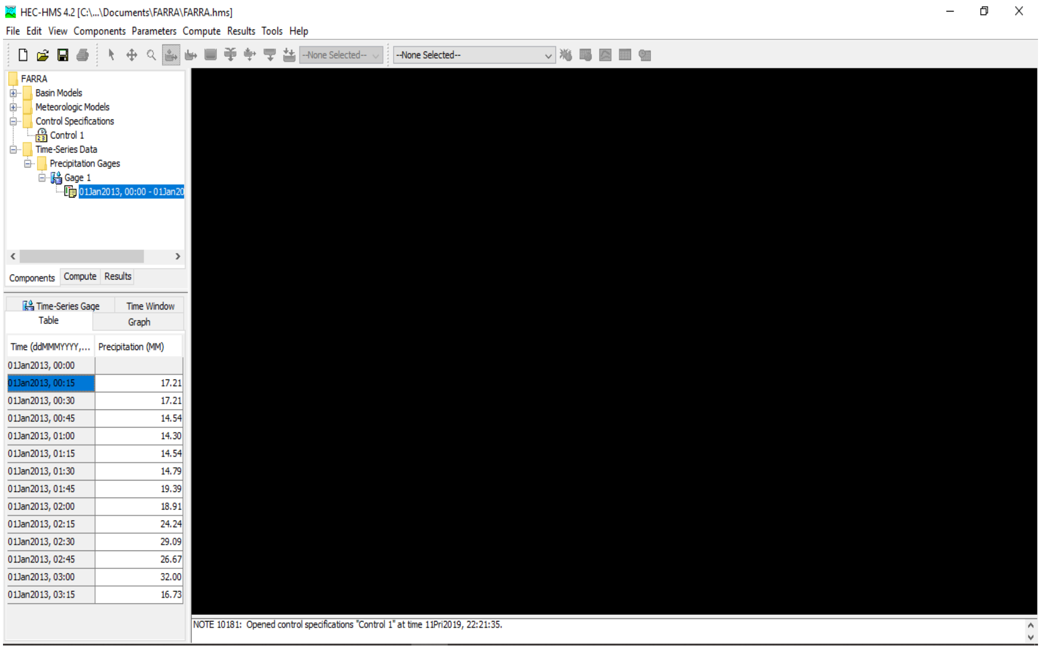


Figure 3.9 Insert the precipitation value

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 3.10, Figure 3.11 and Figure 3.12.

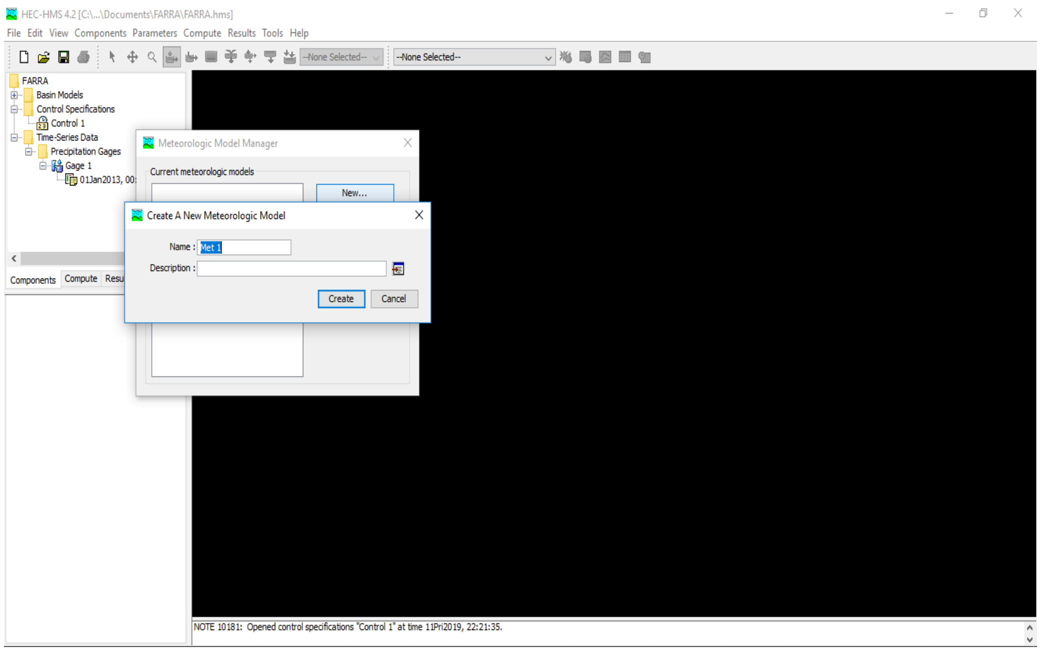


Figure 3.10 Create a new Met

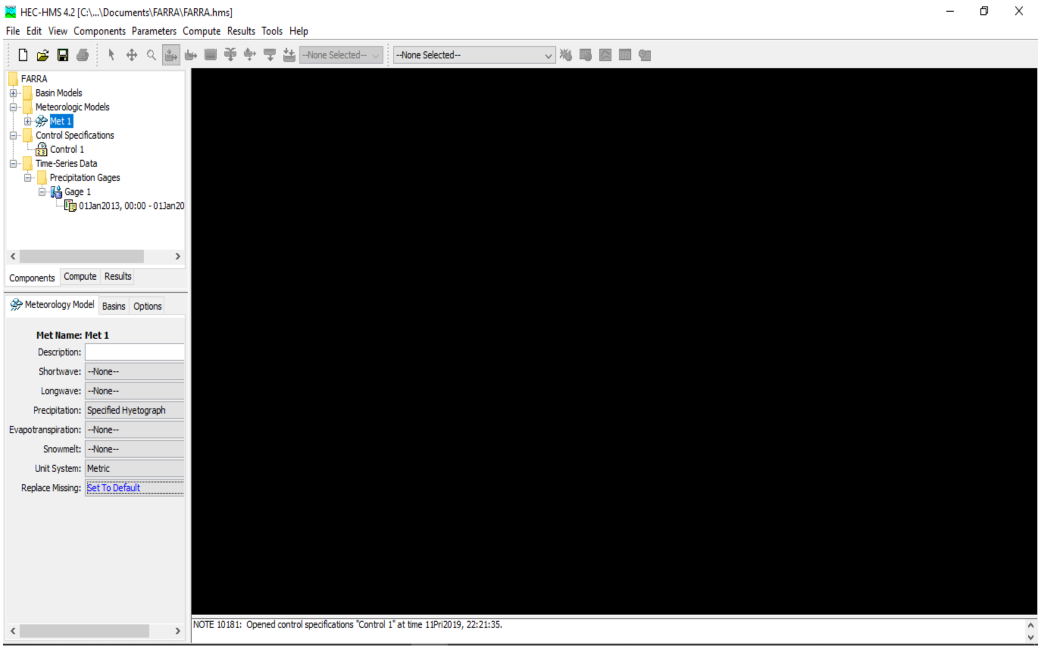


Figure 3.11 Change the Replace Missing to Set to Default

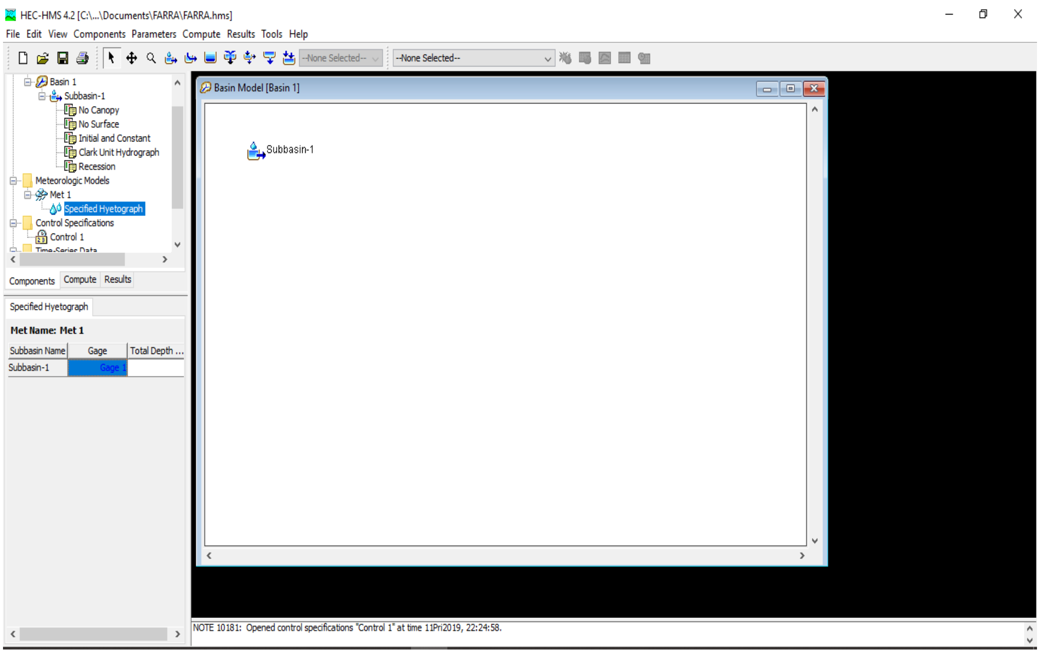


Figure 3.12 Change every column to Gage 1

9. Next, choose **Control Specification Manager** in the Components to create a new Control as shown in the Figure 3.13. Then, put a same date and time as before with time interval 15 minutes as shown in the Figure 3.14.

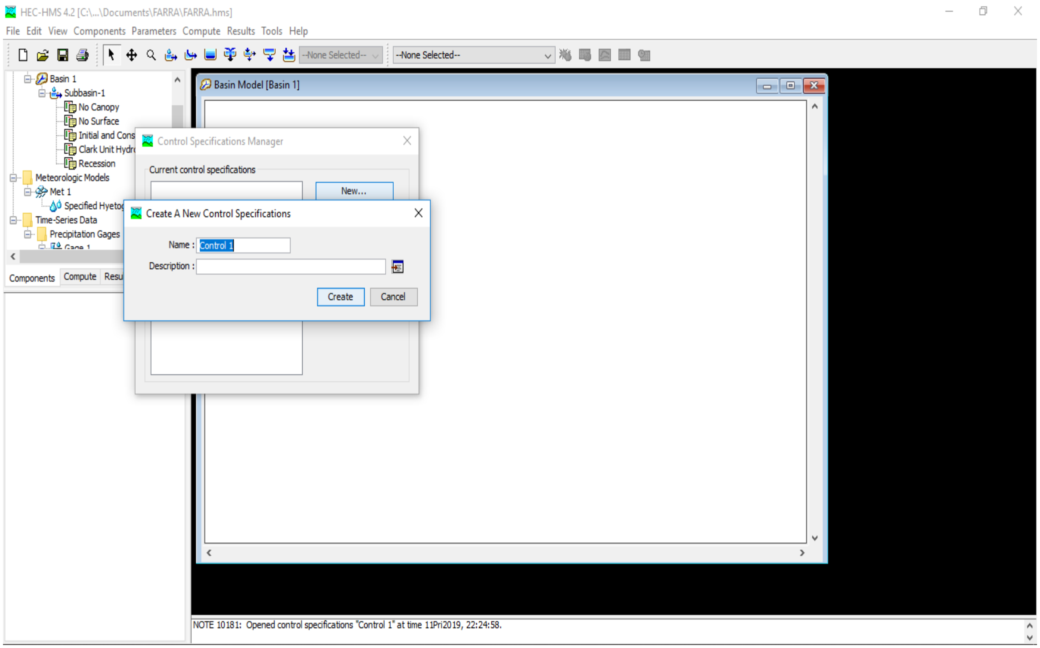


Figure 3.13 Choose Control Specification Manager to start create a new Control

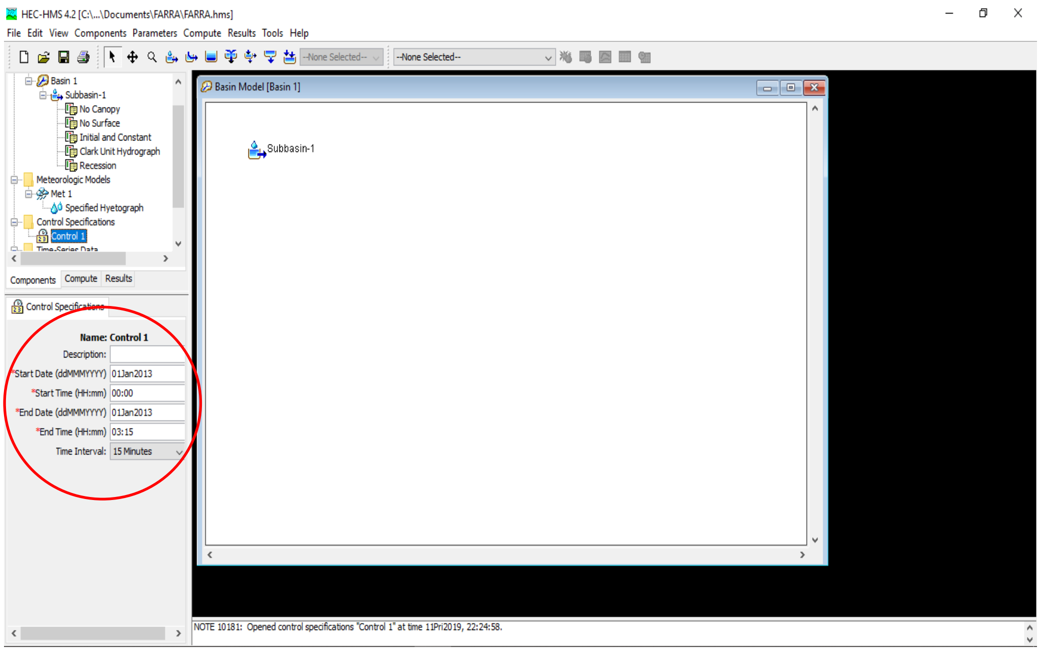


Figure 3.14 Put same date and time in the Control field

10. Next, create a **Compute** and run the program. A **Compute** can be create by go to **Compute** and click **Compute**>**Create Compute**>**Simulation Run**. A window will pop up as shown in Figure 3.15. The click **Next** to finish creates a **Compute.**

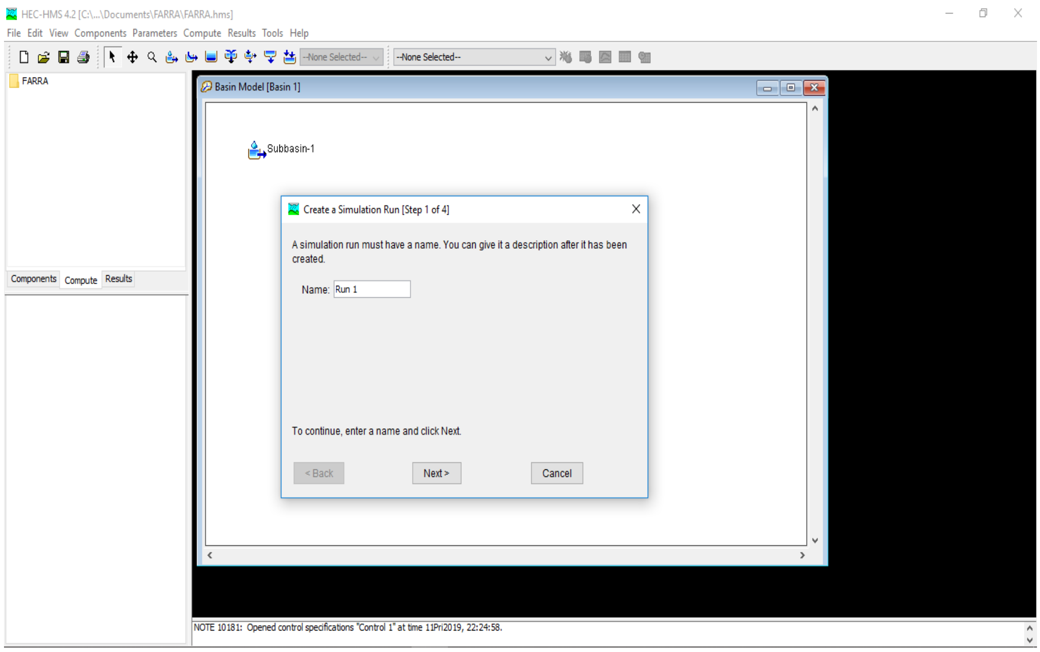


Figure 3.15 Create a Compute

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 3.16.

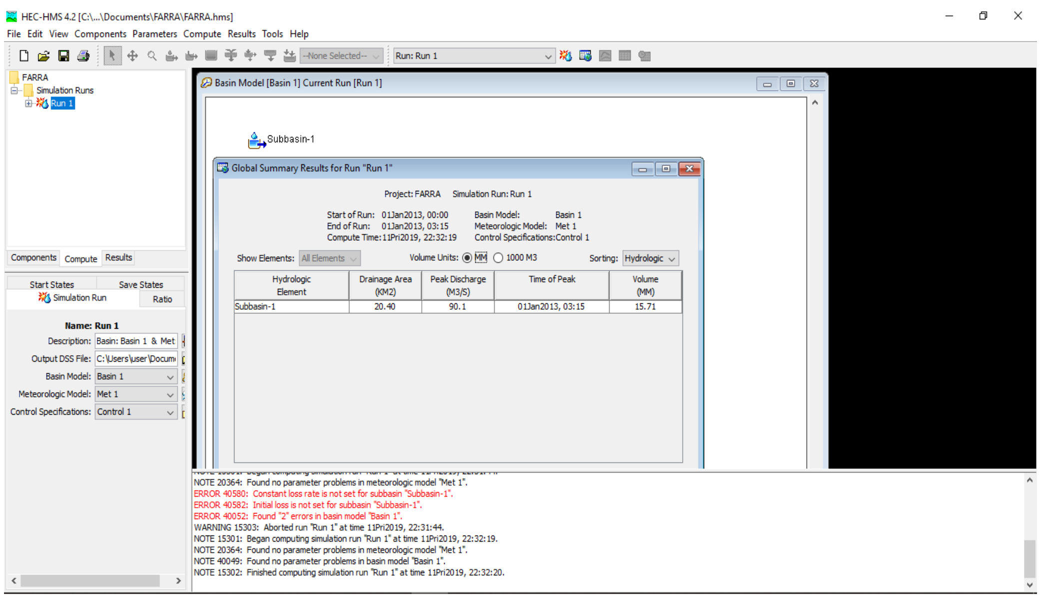


Figure 3.16 Peak discharge in shown in the Global Summary

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 inputs.

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 reach.

iii. Next, insert the cross section. 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 3.17 below shows an example of a complete cross section data input window.

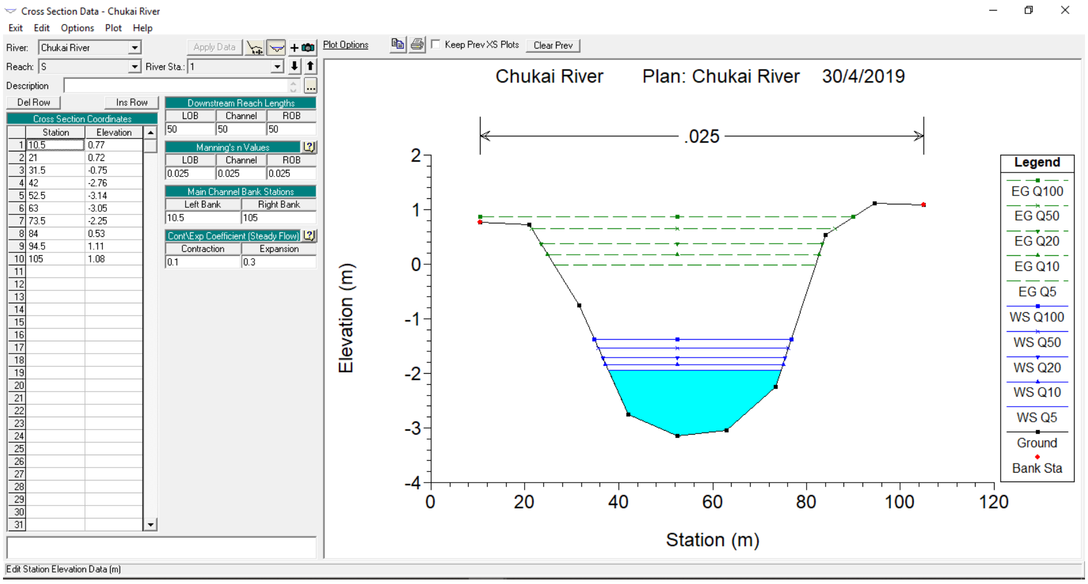


Figure 3.17 Input window for cross section

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, click on **Plot Cross Section** in the Plot menu brings up a plot of the cross section as shown in the Figure 3.18.

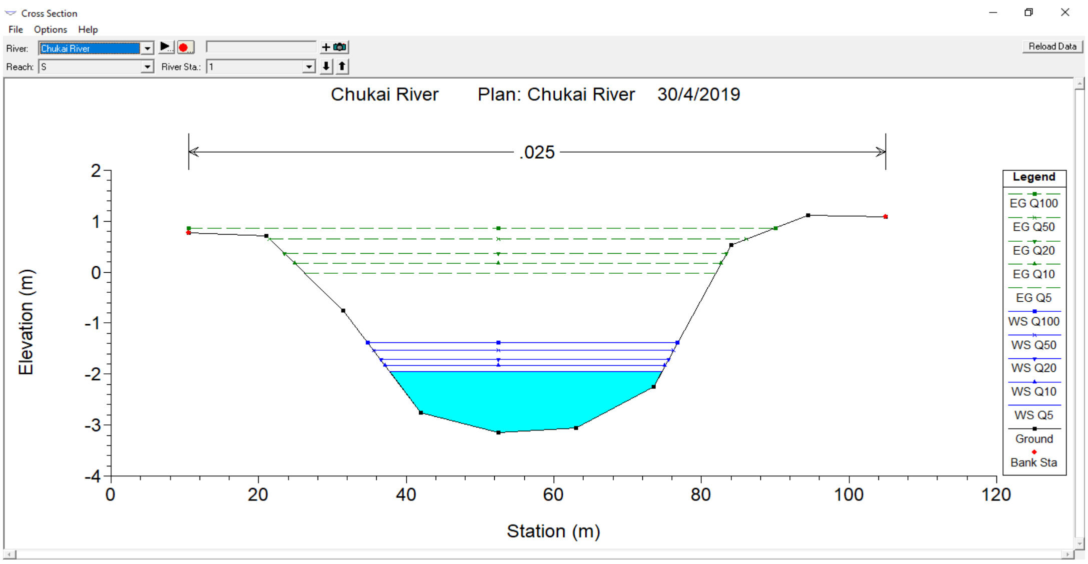


Figure 3.18 Cross section with water surface profiles

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

xi. Bridges in HEC-RAS require 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 3.19.

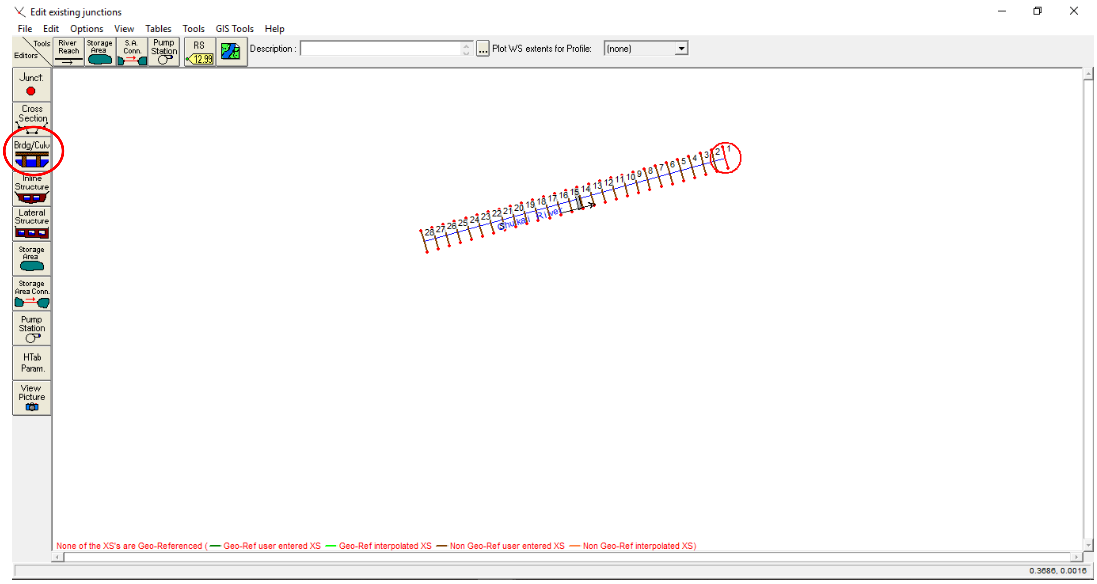


Figure 3.19 Brdg/Culv button

xiii. Click on Options and select **Add a Bridge and/or Culvert** to start the process of creating the bridge as shown in Figure 3.20.

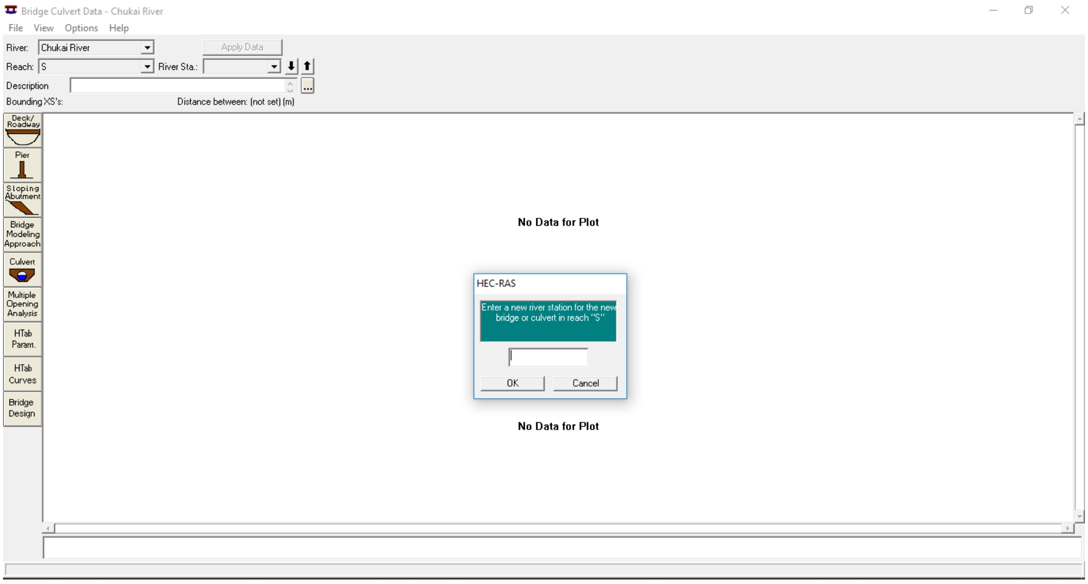


Figure 3.20 Process of creating a bridge by click Options >Add a bridge and/or culvert

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 3.21 and Figure 3.22.

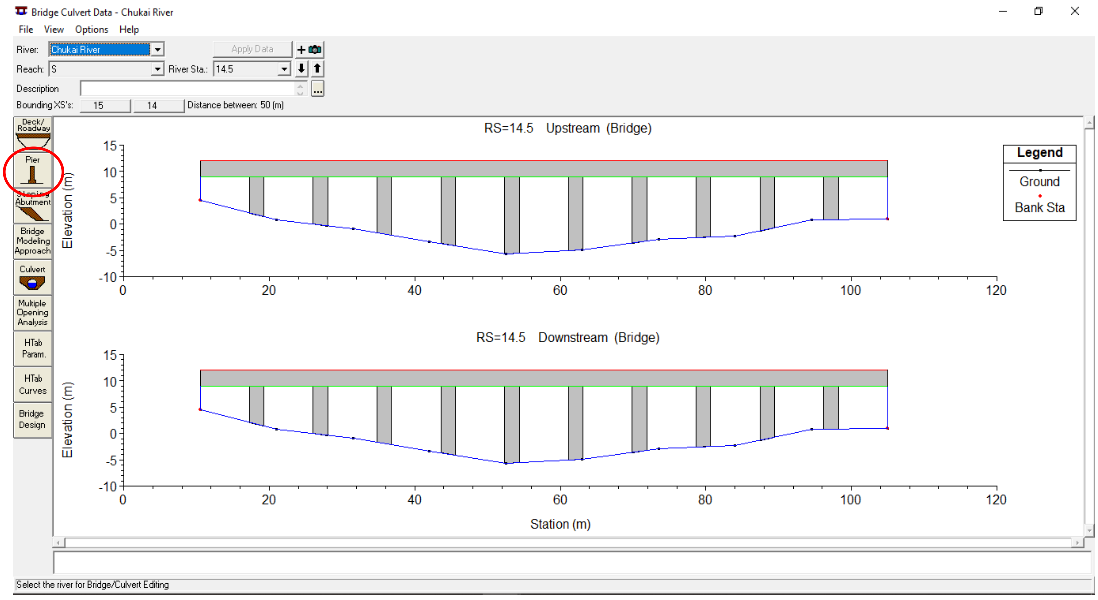


Figure 3.21 Pier window is used for defining the size and location of the piers

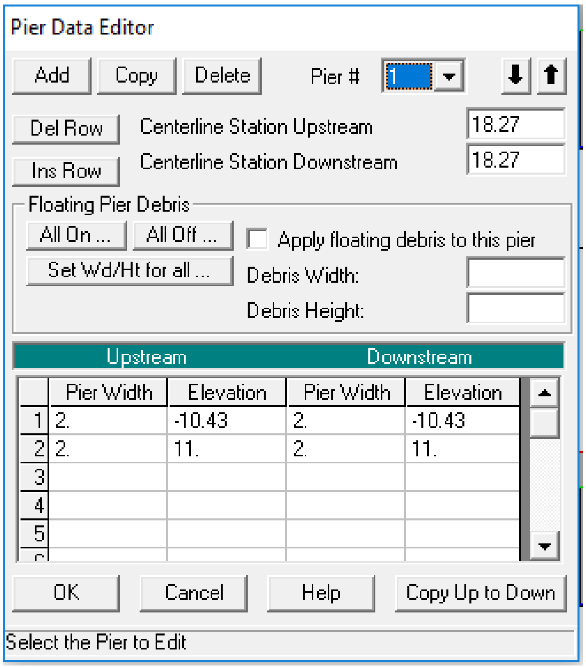


Figure 3.22 Insert required information in Pier Data Editor

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 3.23 and Figure 3.24.

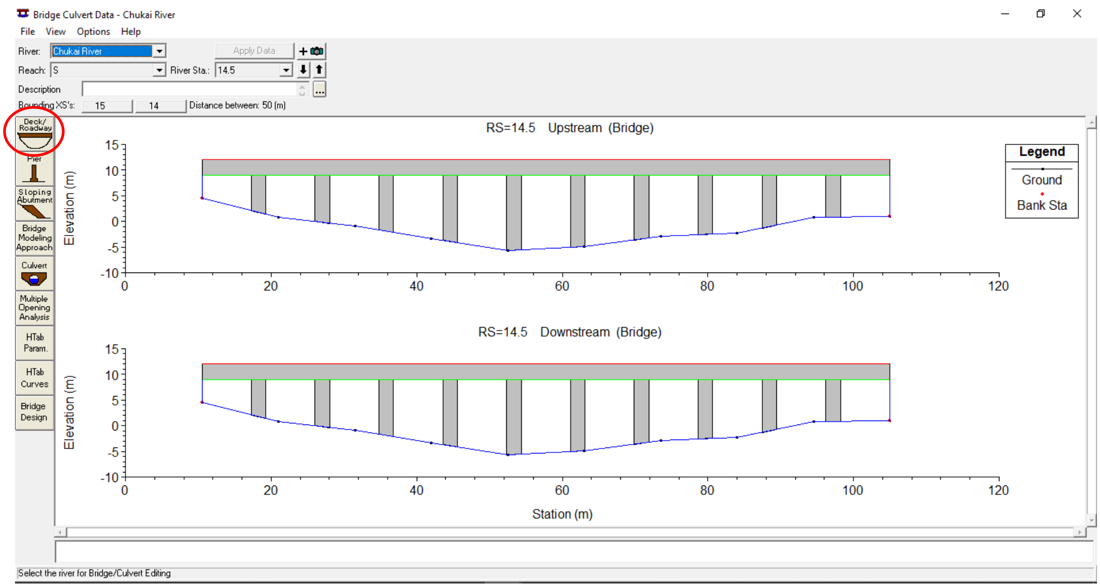


Figure 3.23 Click Deck/Roadway window to start insert required data

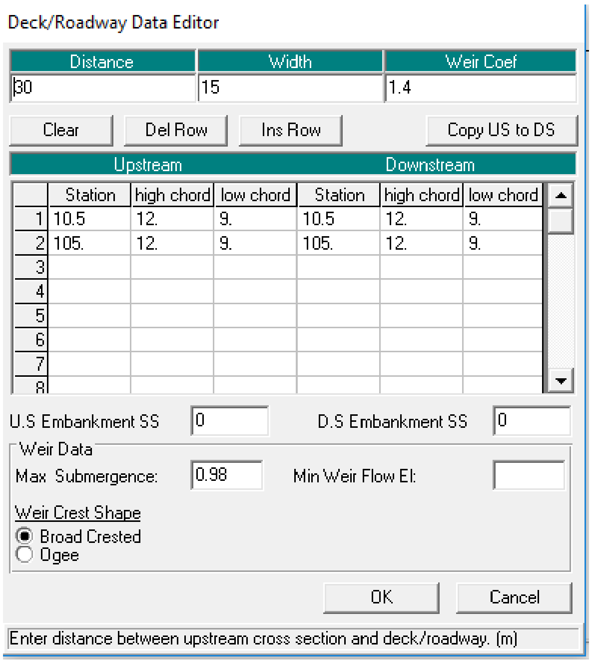


Figure 3.24 Insert all required data in Deck/Roadway Data Editor

xvi. Next is inserting the require information for the sloping abutment by clicks the **Sloping Abutment** editor as shown in the Figure 3.25.

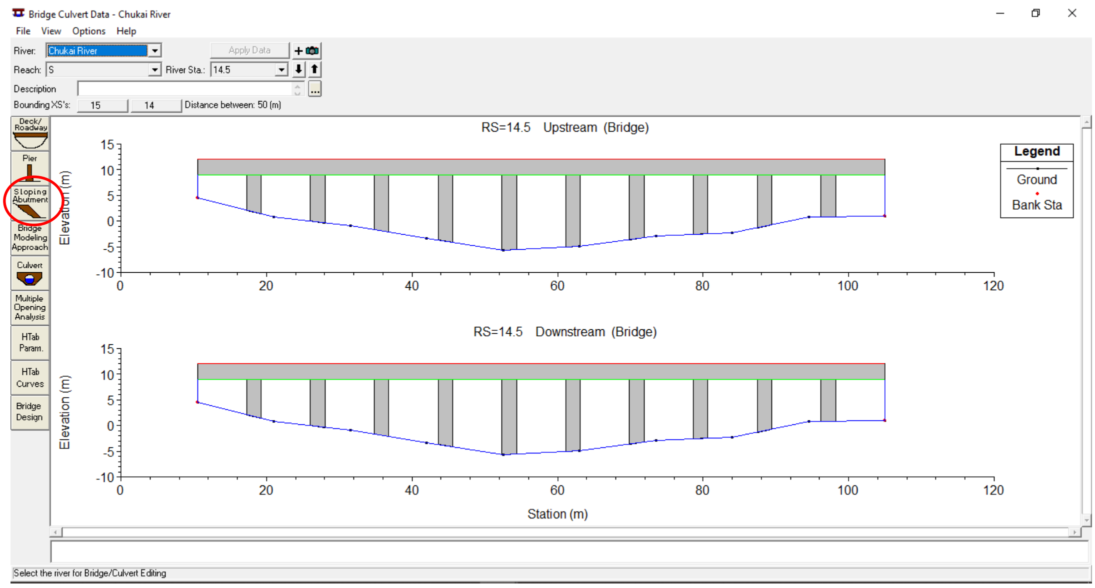


Figure 3.25 Click Sloping Abutment to start insert required information

xvii. Window sloping abutment editor will appear as shown as Figure 3.26. Then, insert all the known information about the sloping abutment.

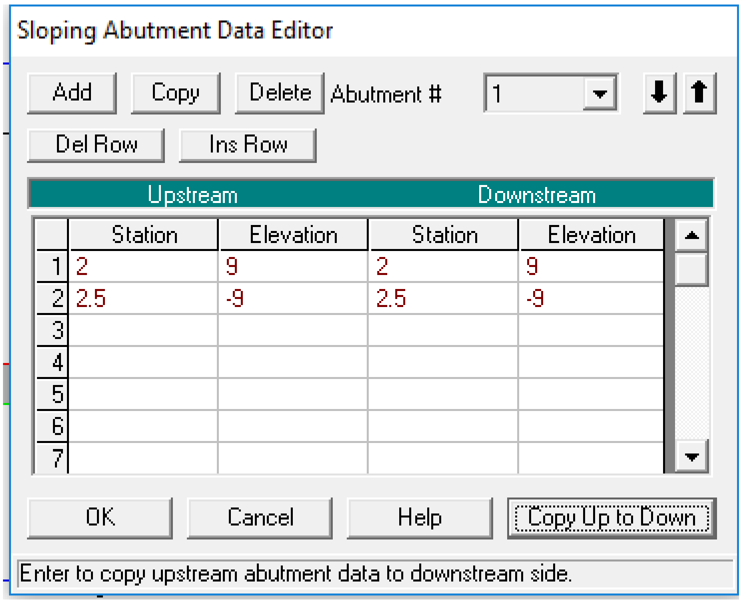


Figure 3.26 Insert the known information about sloping abutment

3.5.2.1 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 3.27.

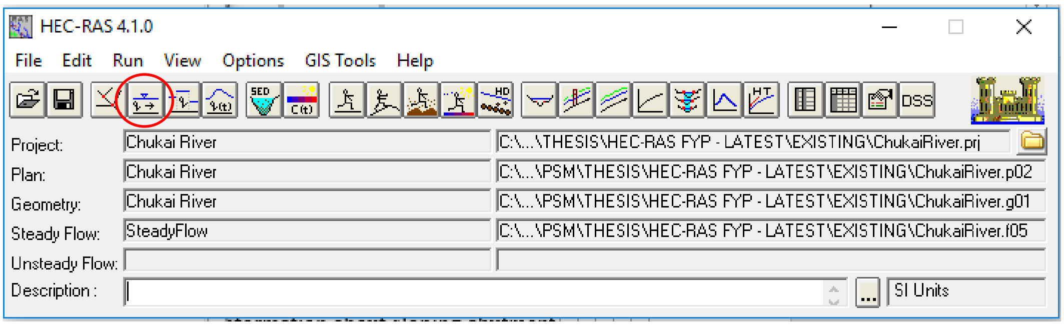


Figure 3.27 Click steady flow icon to start insert steady flow data

iv. **Steady Flow** window will appear as shown in the Figure 3.28. Then insert the value of the flow rate(s).

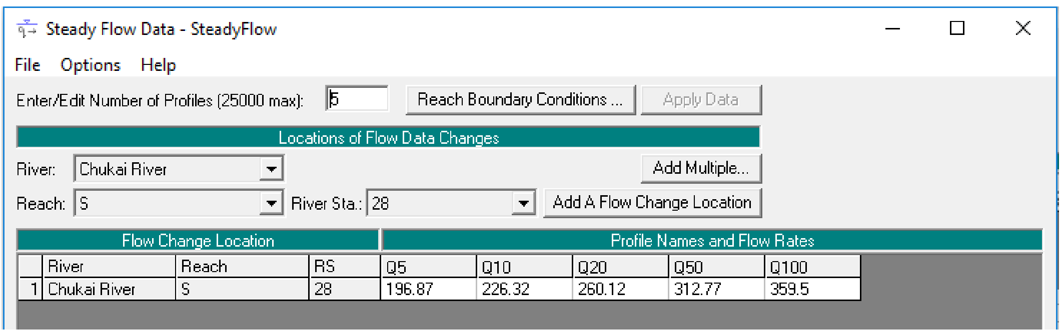


Figure 3.28 Insert the flow rate,Q data in the steady flow window

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 3.29. Then tick the **Mixed** condition before click **Compute** button as shown in the Figure 3.30.

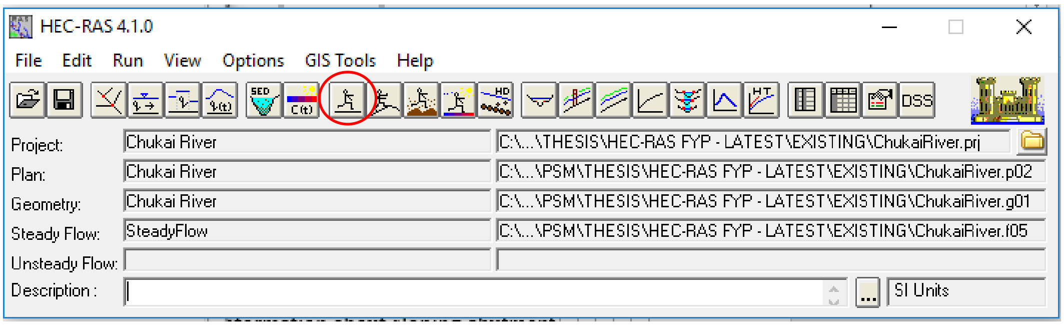


Figure 3.29 Click 'perform steady flow simulation' to start the simulation

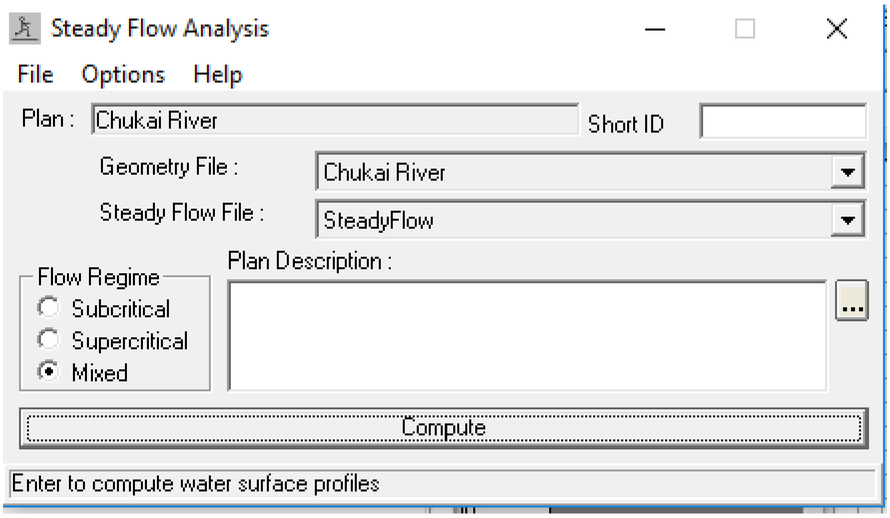


Figure 3.30 Click compute button to compute steady flow simulation

3.5.2.2 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 boundaries 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 3.31.

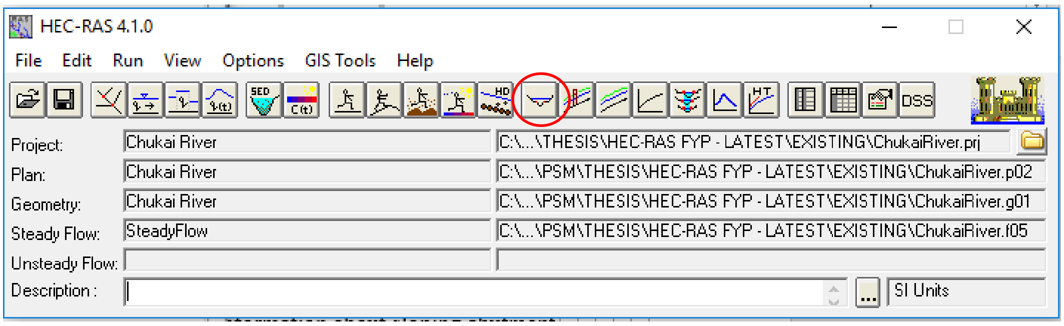


Figure 3.31 ‘View cross section’ icon which use to view water profile at specific cross section

iv. Then, the result of water level profile will appear as shown in Figure 3.32.

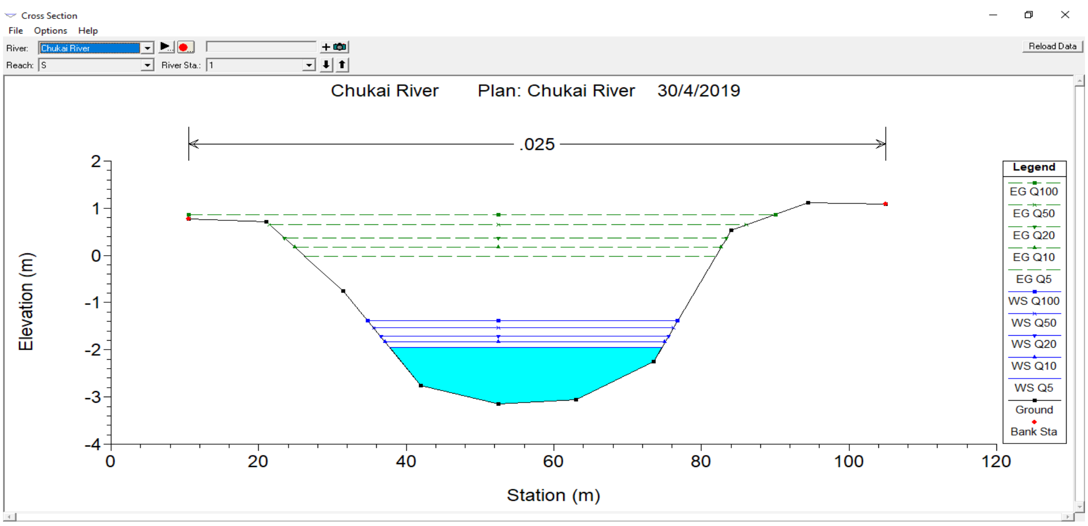


Figure 3.32 Result of water level profile at specific cross section

v. To view result in the form of longitudinal cross section, click ‘**view profile’** icon as shown in the Figure 3.33.

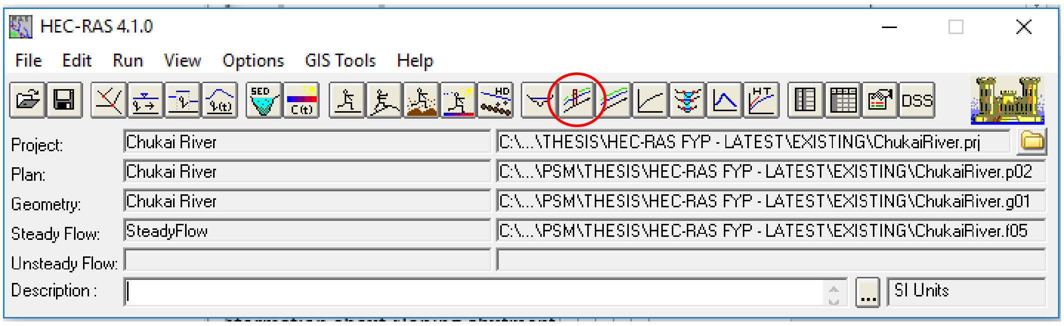


Figure 3.33 ‘View profile’ icon

vi. The water level profile with the longitudinal cross section will appear as shown in the Figure 3.34.

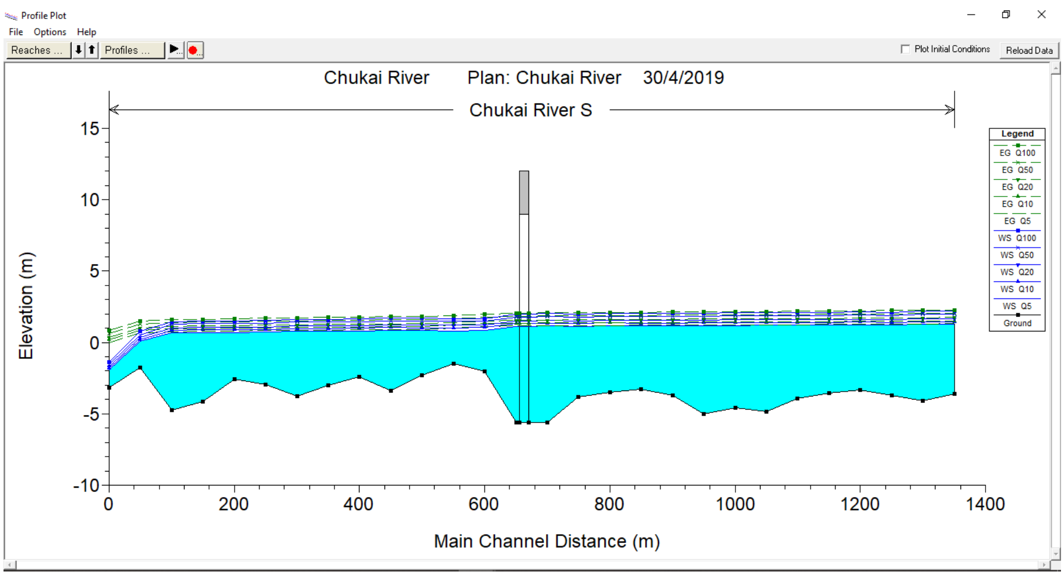


Figure 3.34 Longitudinal cross section with water level profile

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 3.35.

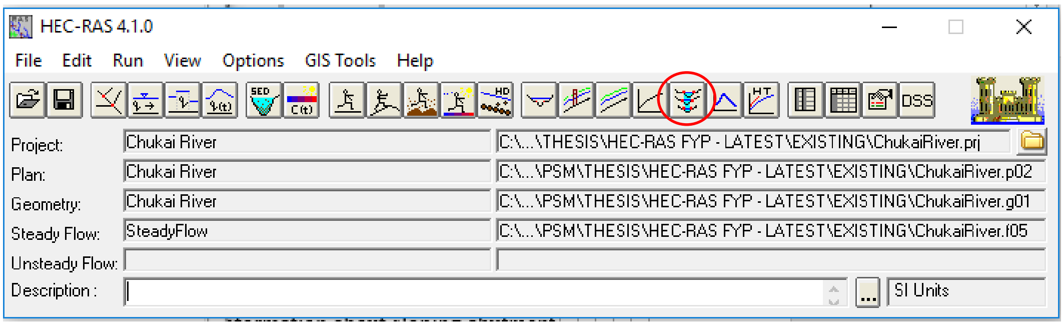


Figure 3.35 ‘View 3D multiple cross section plot” is to view the 3D plot of water profile

viii. The result of the 3D XYZ perspective plot will appear as shown in the Figure 3.36.

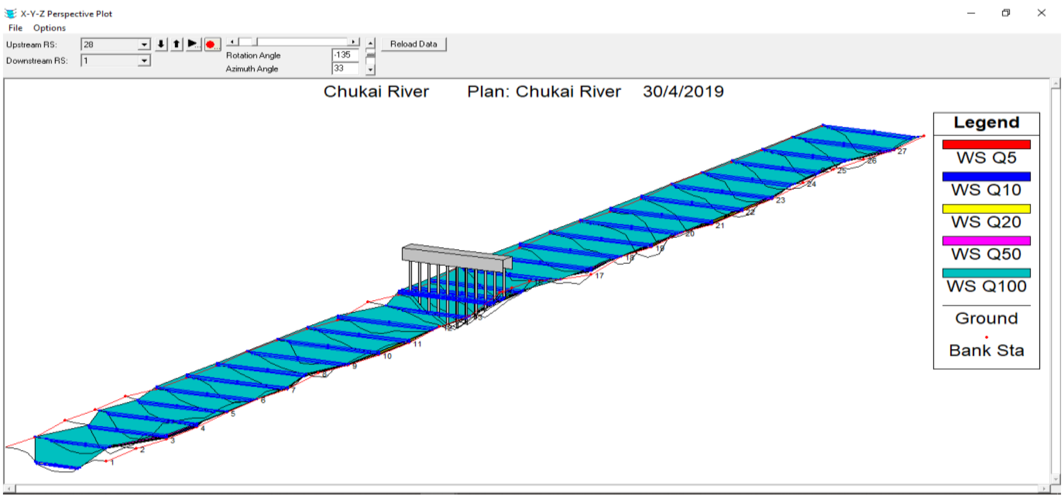
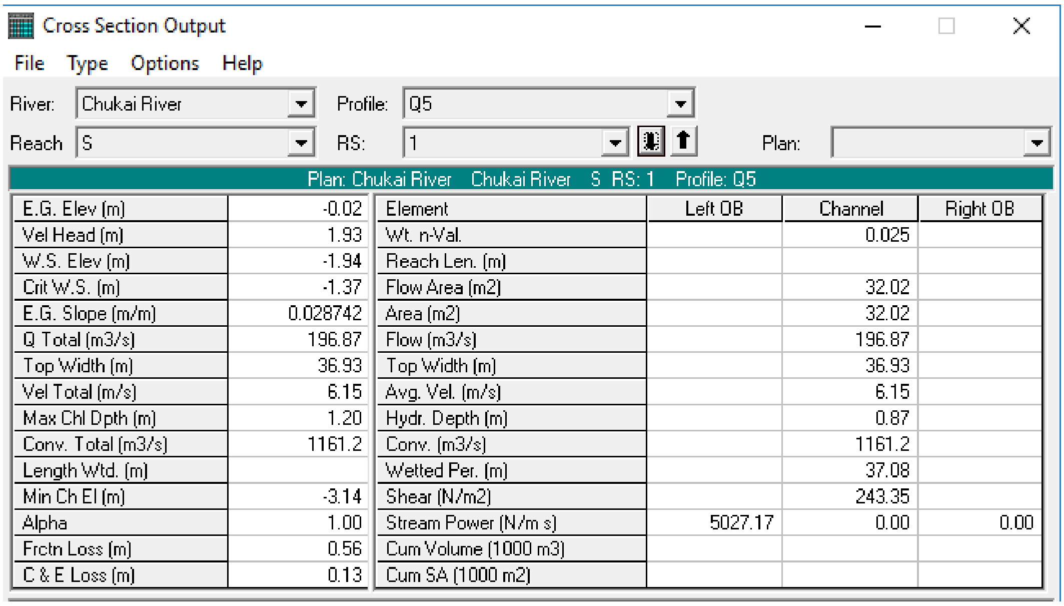


Figure 3.36 3D XYZ perspective plot of reach

ix. Another output or result is cross section output which is in the form of tables as shown in the Figure 3.37. All results can be found by clicking on the View menu in the main program window.



		Element	Left OB	Channel	Right OB
E.G. Elev (m)	-0.02	Wt. n-Val.		0.025	
Vel Head (m)	1.93	Reach Len. (m)			
W.S. Elev (m)	-1.94	Flow Area (m2)		32.02	
Crit W.S. (m)	-1.37	Area (m2)		32.02	
E.G. Slope (m/m)	0.028742	Flow (m3/s)		196.87	
Q Total (m3/s)	196.87	Top Width (m)		36.93	
Top Width (m)	36.93	Avg. Vel. (m/s)		6.15	
Vel Total (m/s)	6.15	Hydr. Depth (m)		0.87	
Max Chl Dpth (m)	1.20	Conv. (m3/s)		1161.2	
Conv. Total (m3/s)	1161.2	Wetted Per. (m)		37.08	
Length Wtd. (m)		Shear (N/m2)		243.35	
Min Ch El (m)	-3.14	Stream Power (N/m s)	5027.17	0.00	0.00
Alpha	1.00	Cum Volume (1000 m3)			
Frctn Loss (m)	0.56	Cum SA (1000 m2)			
C & E Loss (m)	0.13				

Figure 3.37 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 effect of flow at the bridge piers which resulted in overflow to the catchment area, the difference between the water levels at old and new bridge piers is determined.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 INTRODUCTION

The results and analysis for this study were focused on the flow of the river channel and water profiles at upstream of the bridge, under the bridge and at the downstream of the bridge. Simulations were based on 5-year, 10-year, 20-year, 50-year and 100-year ARI. Flow rates and water levels of the river were then simulated and recorded.

In order to determine the 100-year flood level, rainfall data from 1st January 2013 to 31st December 2018 were used. These data were the input for the HEC-HMS software to provide the peak flow and the hydrographs. From the peak flow computed, the water level profile of the river can be determined using the HEC-RAS software.

4.2 SIMULATION ANALYSIS

The scenarios of the simulations were carried out by using the HEC-RAS software. The outcomes of the simulation were then analysed and discussed an outlined in Table 4.1

Table 4.1 Types of analysis

No. of Analysis	Case	Event	Analysis Type
1	Old bridge piers	Flow rate, Q with 5-year, 10-year, 20-year, 50-year and 100-year ARI	Elevation vs. Distance (5,400 m)
2	New bridge piers	Flow rate, Q with 5-year, 10-year, 20-year, 50-year and 100-year ARI	Elevation vs. Distance (5,400 m)

4.3 PEAK FLOWS AND HYDROGRAPH

The result of HEC-HMS simulation is shown as Figure 4.1. It is clearly stated that the critical rainfall of Chukai River was at the end of year 2013. The flash flood of the Chukai River occurred on 3rd December 2013.

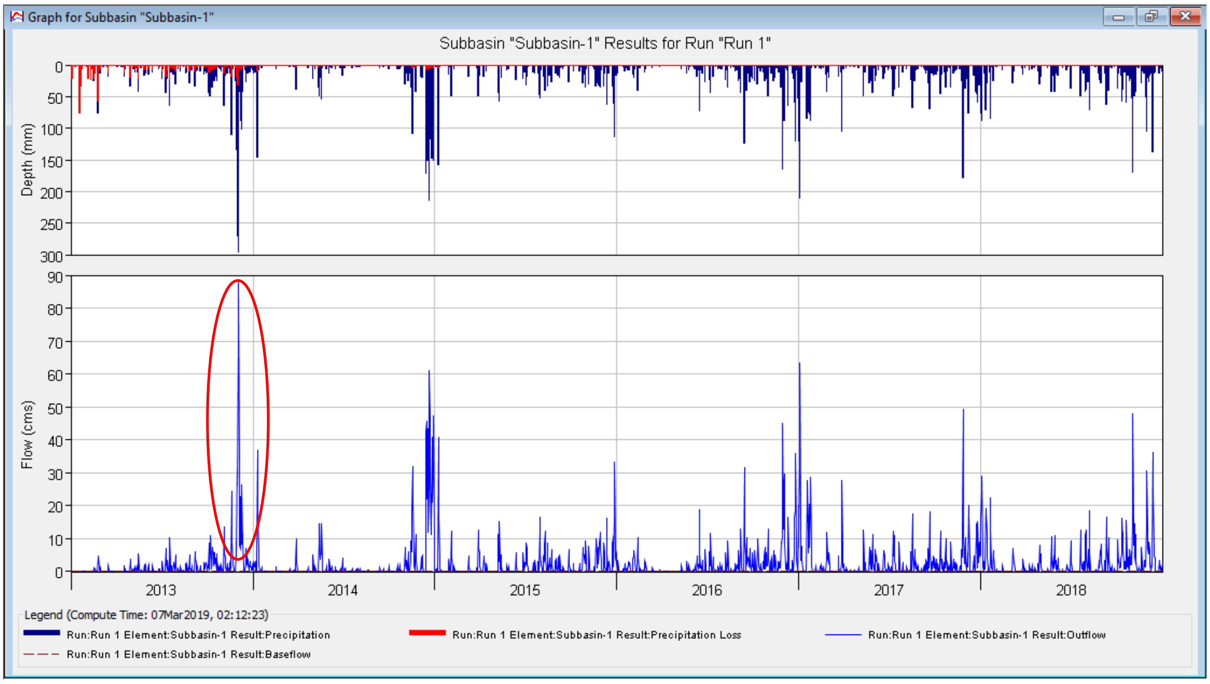


Figure 4.1 Critical rainfall of the Chukai River

From the simulations, the highest rainfall depth per year occurred towards the end of the year. It is because during the end of year, it is the monsoon season. Based on the analysis, 2013 recorded the highest rainfall among the years analysed. Kemaman recorded highest flood on 3rd December 2013.

In this study, the hydrographs were produced from HEC-HMS software. The hydrograph for 5-year, 10-year, 20-year, 50-year and 100-year ARI are shown in Figure 4.2 to Figure 4.6.

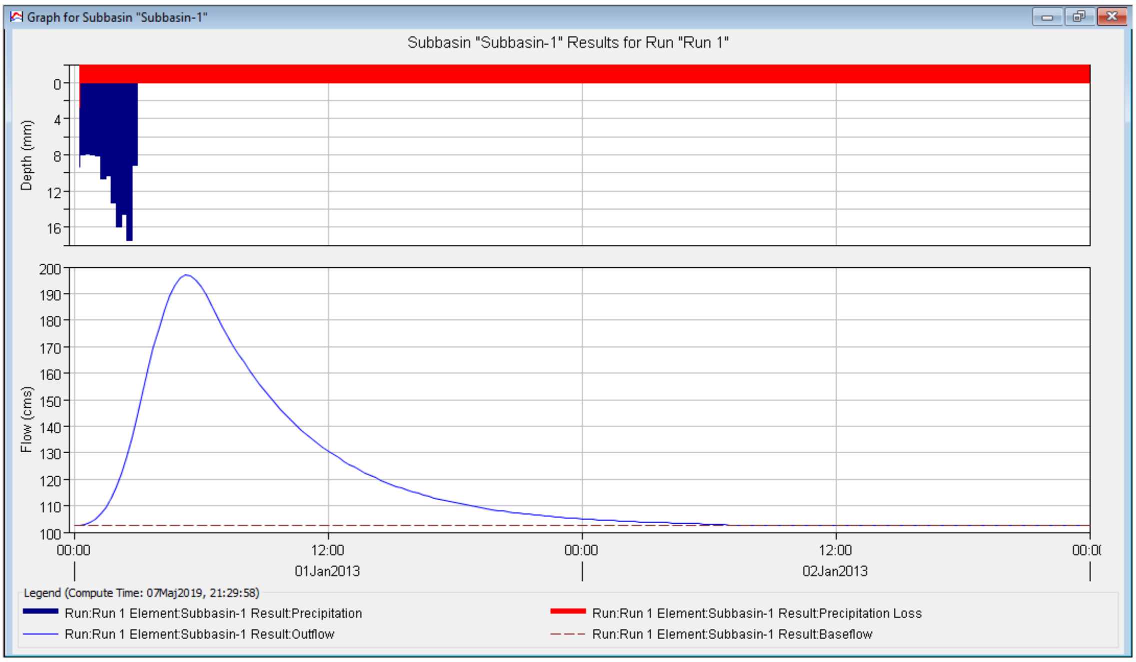


Figure 4.2 Hydrograph for 5-year ARI

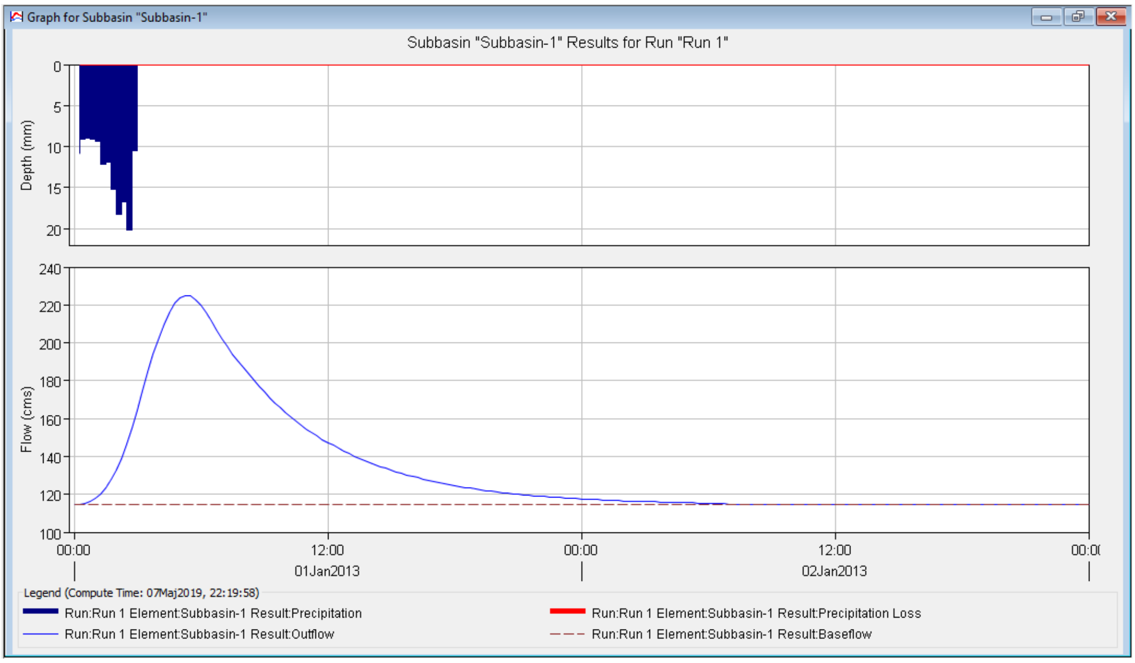


Figure 4.3 Hydrograph for 10-year ARI

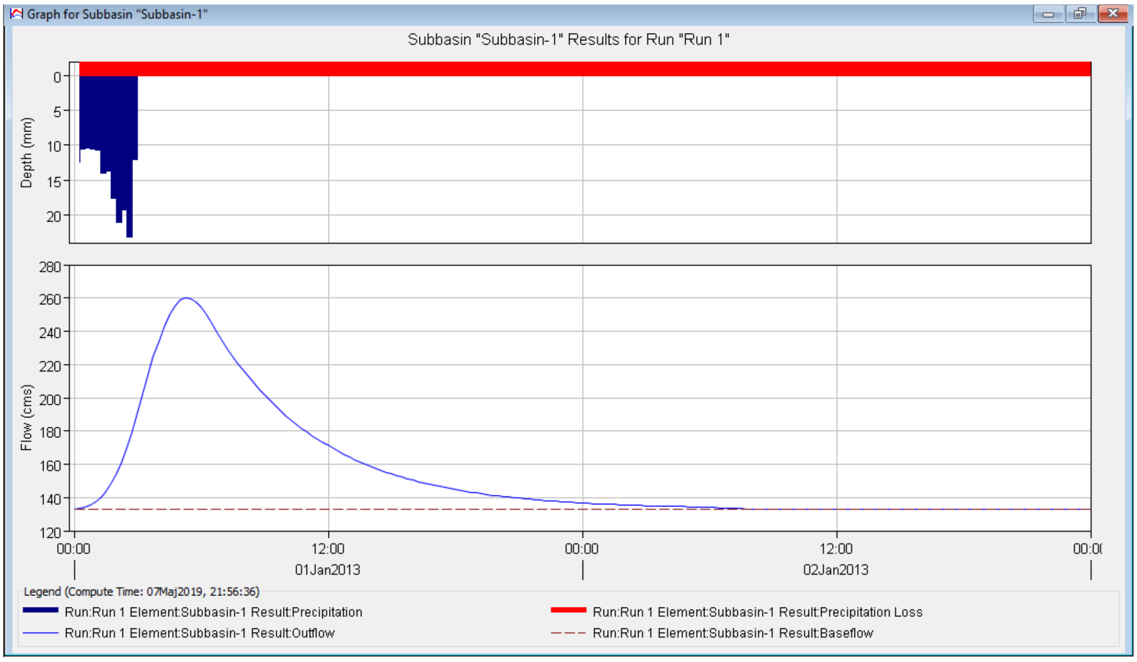


Figure 4.4 Hydrograph for 20-year ARI

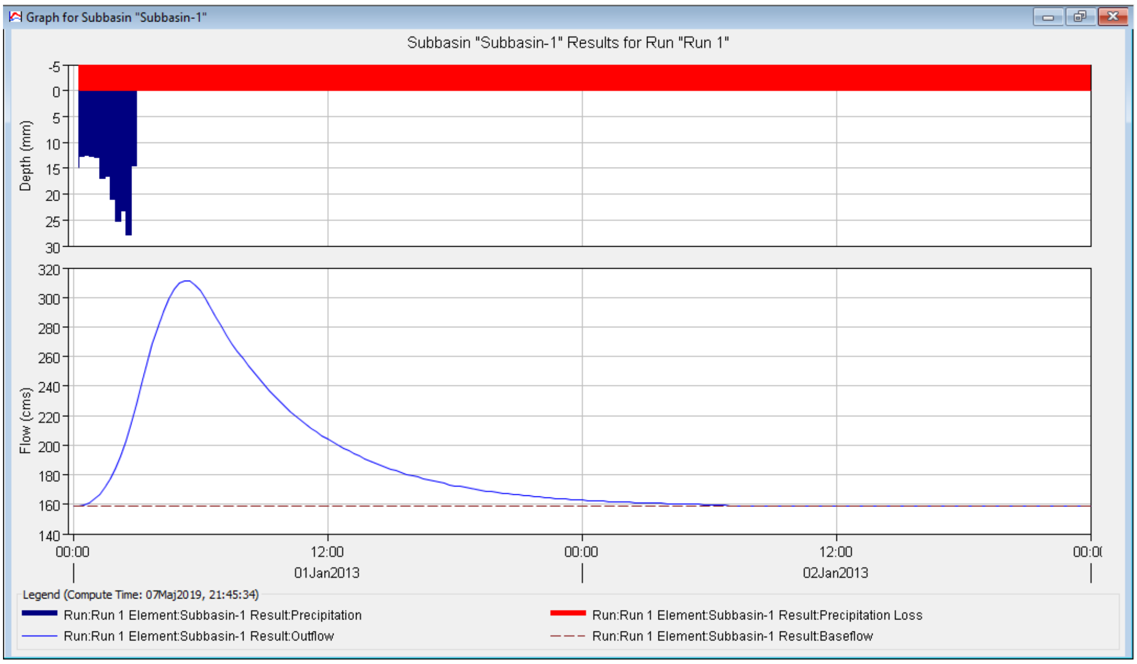


Figure 4.5 Hydrograph for 50-year ARI

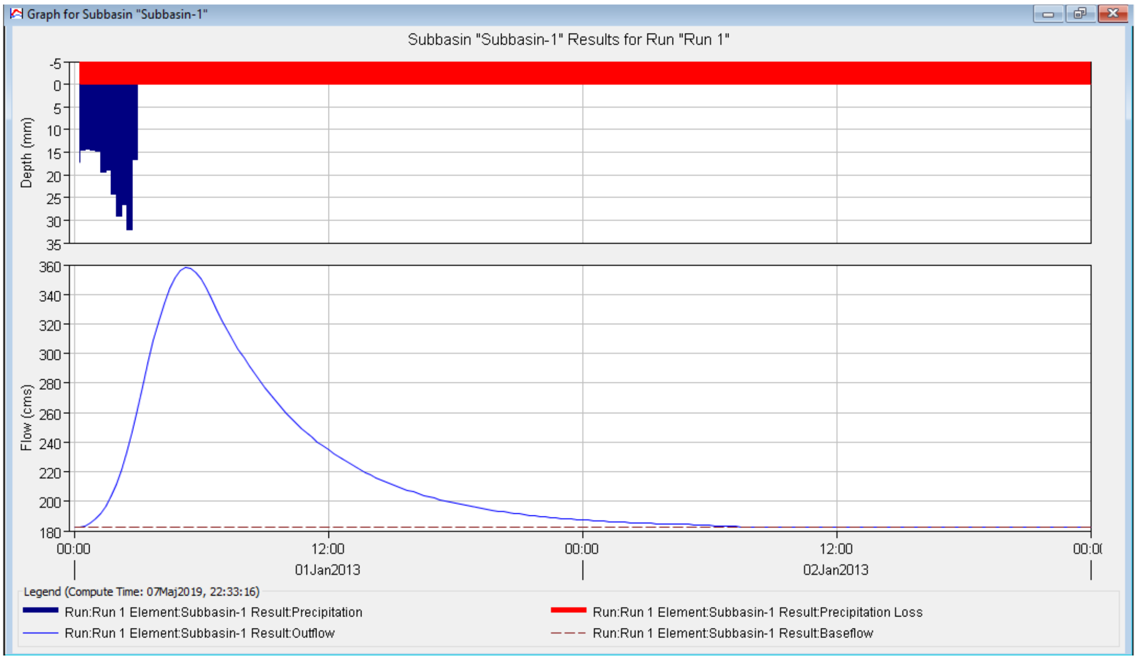


Figure 4.6 Hydrograph for 100-year ARI

From Figure 4.2 to Figure 4.6, we can see the peak flow as shown in the hydrographs. The peak flows were increased as the ARI increased. It is because the flow of water increased depends on the ARI. The hydrograph is needed because it is used to show how the water flow in a drainage basin or river runoff responds to a period of rain.

Rainfall data from 1st January 2013 to 31st December 2018 were incorporated in the HEC-HMS in order to estimate the peak flow also for 5-year, 10-year, 20-year, 50-year and 100-year ARI. Table 4.2 summarized the peak flow for each ARI.

Table 4.2 Results from HEC-HMS

Event (ARI)	**Rainfall Intensity, *i* (mm/hr)**	**Rainfall depth (mm)**	**Q_{peak} (m^3/s)**
5-year	36.57	132.76	196.87
10-year	42.04	152.51	226.32
20-year	48.32	175.38	260.12
50-year	58.10	210.80	312.77
100-year	66.78	242.41	359.50

4.4 ANALYSIS OF SIMULATION RESULTS (HEC-RAS)

In this study, cross section of the river was the main input for the simulation process. This study involved 28 chainages along the Chukai River inclusive of the new bridge called the Bukit Kuang Bridge. The direction of flow is from CH 10,350 to CH 15,750. Figure 4.7 illustrates the cross section plan with respect to chainages along the Chukai River.

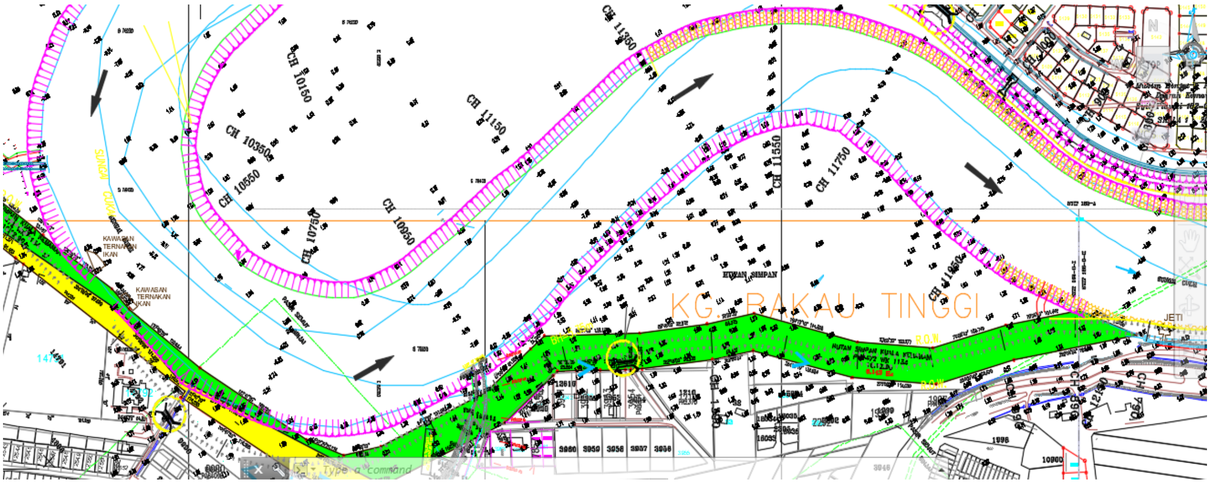


Figure 4.7 Cross section plan of Chukai River

4.5 WATER LEVEL FOR Q_5 FOR OLD AND NEW BRIDGE PIERS

The water level profiles were determined as the output from HEC-RAS software. Other than that, the backwater effect at the bridge was determined by having the difference in water levels at each cross section or chainage. The difference in water levels of the Chukai River along the chainage with 5-year ARI is tabulated in Table 4.3. For old bridge, CH 10,750 and CH 10,950 were overflown on the left bank. While, for CH 12,550, CH 12,950, CH 13,050 (D), CH 13,050 (U), CH 13,150, CH 13,750 and CH 14,350 were overflown on the right bank. For CH 13550, it was recorded overflown on both banks. These chainages were located at the upstream, under the bridge and the downstream of the river. For the new bridge, there is no overflown on both banks.

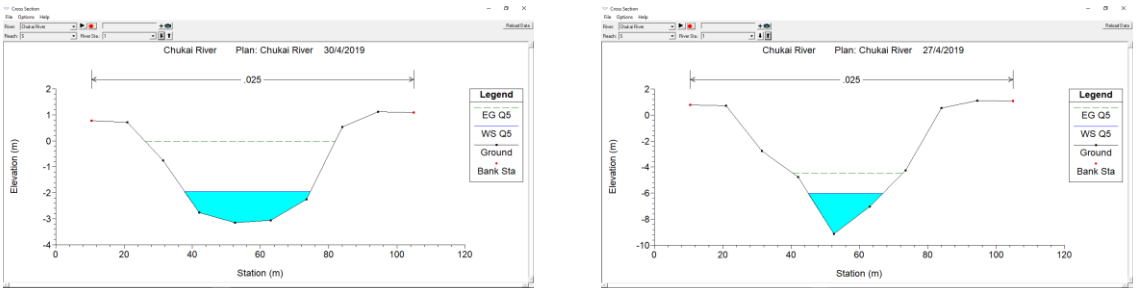
Table 4.3 Difference of water levels with Q_5

$Q_5 = 196.87\ m^3/s$							
Chainage	Old Bridge Water levels (m)		New Bridge Water levels (m)		Difference of water levels (m)		Overflow
	Left bank	Right bank	Left bank	Right bank	Left bank	Right bank	
CH 10,350 (1)	-1.94	-1.94	-6.01	-6.01	-4.07	-4.07	
CH 10,550 (2)	0.06	0.06	-4.84	-4.84	-4.90	-4.90	
CH 10,750 (3)	0.69	0.69	-4.16	-4.16	-4.85	-4.85	(Left bank-Old)
CH 10,950 (4)	0.71	0.71	-3.90	-3.90	-4.61	-4.61	(Left bank-Old)
CH 11,150 (5)	0.70	0.70	-3.96	-3.96	-4.66	-4.66	
CH 11,350 (6)	0.72	0.72	-3.98	-3.98	-4.70	-4.70	
CH 11,550 (7)	0.78	0.78	-3.30	-3.30	-4.08	-4.08	
CH 11,750 (8)	0.78	0.78	-3.31	-3.31	-4.09	-4.09	
CH 11,950 (9)	0.81	0.81	-3.34	-3.34	-4.15	-4.15	
CH 121,50 (10)	0.85	0.85	-3.29	-3.29	-4.14	-4.14	
CH 12,350 (11)	0.82	0.82	-3.27	-3.27	-4.09	-4.09	
CH 12,550 (12)	0.79	0.79	-3.26	-3.26	-4.05	-4.05	(Right bank-Old)
CH 12,750 (13)	0.86	0.86	-2.40	-2.40	-3.26	-3.26	
CH 12,950 (14)	1.13	1.13	-1.68	-1.68	-2.81	-2.81	(Right bank-Old)
CH 13,050 (D) (14.5)	1.13	1.13	-1.68	-1.68	-2.81	-2.81	(Right bank-Old)
CH 13,050 (U) (14.5)	1.13	1.13	-1.68	-1.68	-2.81	-2.81	(Right bank-Old)
CH 13,150 (15)	1.15	1.15	-1.67	-1.67	-2.82	-2.82	(Right bank-Old)
CH 13,350 (16)	1.13	1.13	-1.71	-1.71	-2.84	-2.84	
CH 13,550 (17)	1.17	1.17	-1.67	-1.67	-2.84	-2.84	(Both banks-Old)
CH 13,750 (18)	1.16	1.16	-1.67	-1.67	-2.83	-2.83	(Right bank-Old)
CH 13,950 (19)	1.16	1.16	-1.68	-1.68	-2.84	-2.84	
CH 14,150 (20)	1.17	1.17	-1.64	-1.64	-2.81	-2.81	
CH 14,350 (21)	1.18	1.18	-1.62	-1.62	-2.80	-2.80	(Right bank-Old)
CH 14,550 (22)	1.20	1.20	-1.62	-1.62	-2.82	-2.82	
CH 14,750 (23)	1.21	1.21	-1.62	-1.62	-2.83	-2.83	
CH 14,950 (24)	1.21	1.21	-1.62	-1.62	-2.83	-2.83	
CH 15,150 (25)	1.22	1.22	-1.62	-1.62	-2.84	-2.84	
CH 15,350 (26)	1.20	1.20	-1.62	-1.62	-2.82	-2.82	
CH 15,550 (27)	1.25	1.25	-1.61	-1.61	-2.87	-2.87	
CH 15,750 (28)	1.25	1.25	-1.60	-1.60	-2.85	-2.85	

4.5.1 Water level for Q_5 at upstream of the bridge

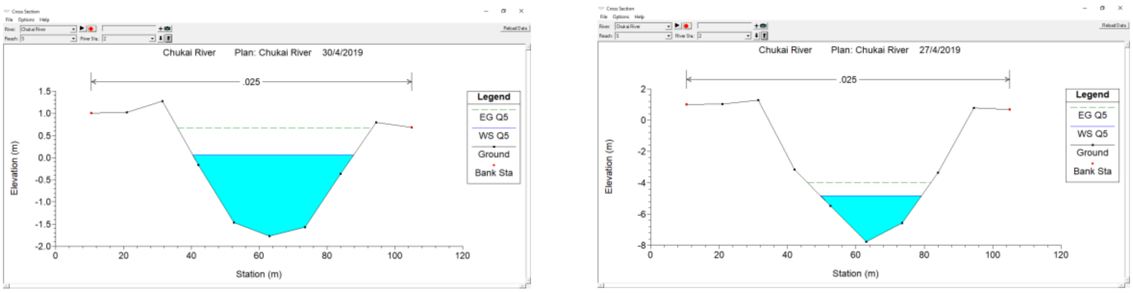
Based on Table 4.3 and Figure 4.8 to Figure 4.20 the water levels were in between -1.94 m to 0.86 m above sea level at the upstream of the bridge with the old bridge piers while with the new bridge piers, the water levels were between -6.01 m to -2.40 m above sea level at the upstream of the bridge. The water levels surpassed the river bank along CH 10,350 to CH 15,750 during rainfall event of 5-year ARI. In other words, the water was overflown onto the several chainages due to its incline bank surface as stated in Table 4.3.

Besides, the differences in water levels along the upstream chainage which start from CH 10,350 to CH 12,750 (Figure 4.8 to Figure 4.20) were between -4.07 m to -3.26 m. These indicated that with the presence of new bridge piers, flooding will not occur.



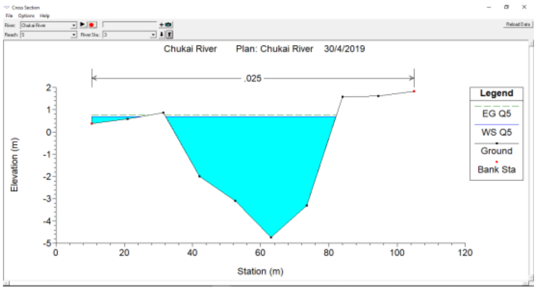
(a) Old bridge piers (b) New bridge piers

Figure 4.8 Water level at CH 10,350 of Chukai River – 5-year ARI

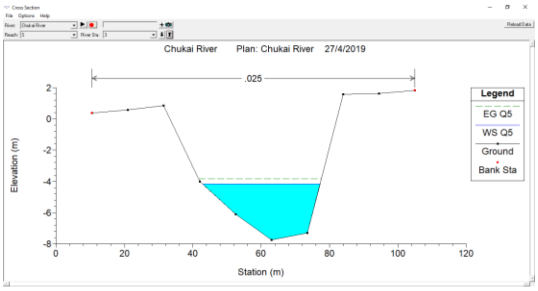


(a) Old bridge piers (b) New bridge piers

Figure 4.9 Water level at CH 10,550 of Chukai River –5-year ARI

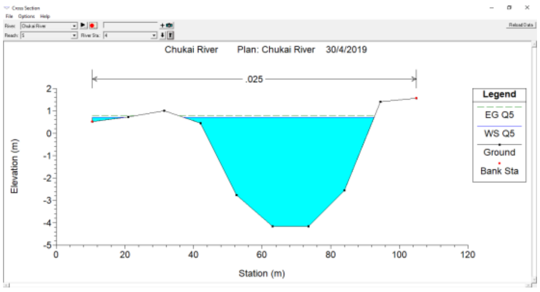


(a) Old bridge piers

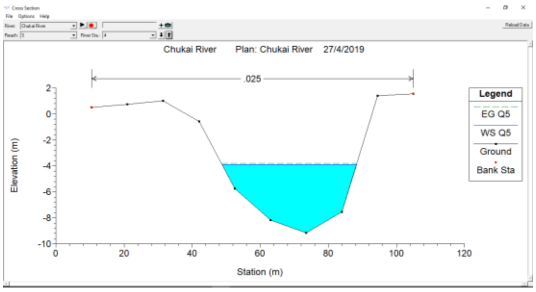


(b) New bridge piers

Figure 4.10 Water level at CH 10,750 of Chukai River –5-year ARI

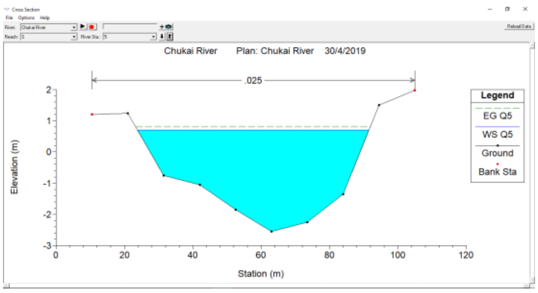


(a) Old bridge piers

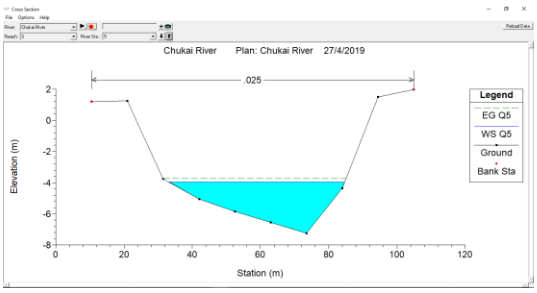


(b) New bridge piers

Figure 4.11 Water level at CH 10,950 of Chukai River –5-year ARI

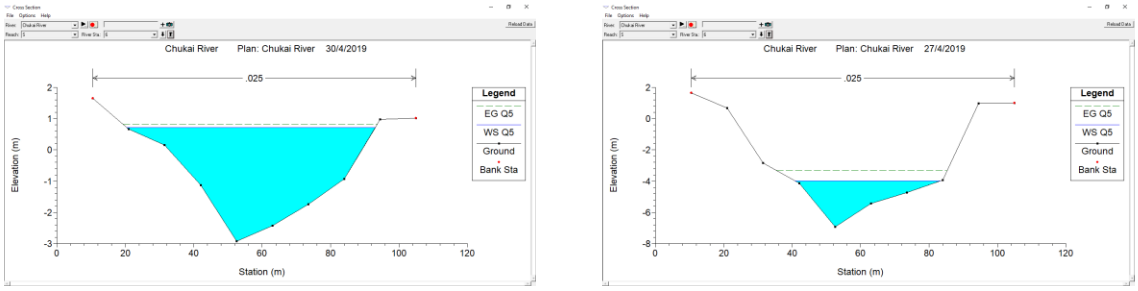


(a) Old bridge piers



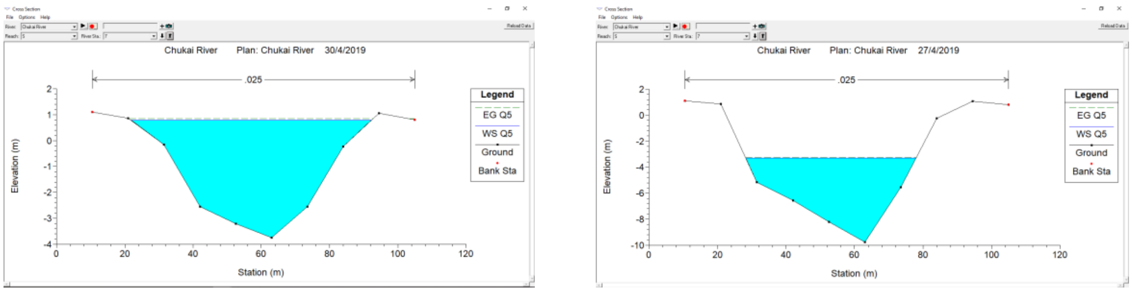
(b) New bridge piers

Figure 4.12 Water level at CH 11,150 of Chukai River –5-year ARI



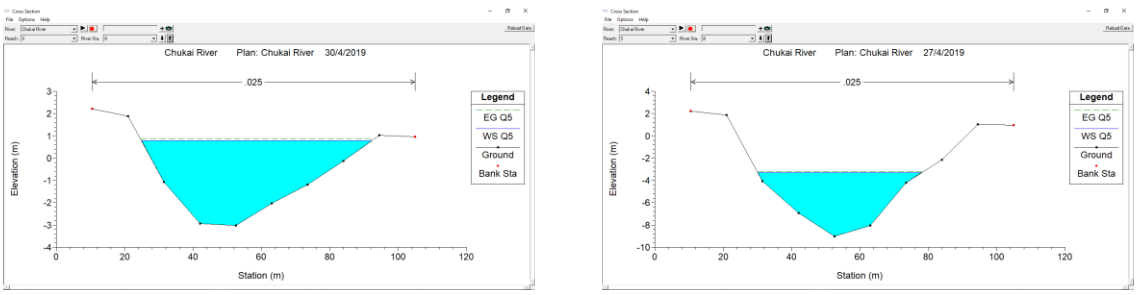
(a) Old bridge piers (b) New bridge piers

Figure 4.13 Water level at CH 11,350 of Chukai River –5-year ARI



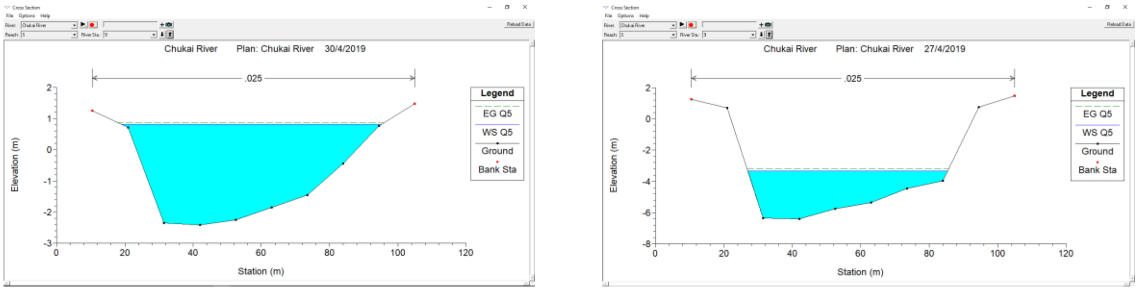
(a) Old bridge piers (b) New bridge piers

Figure 4.14 Water level at CH 11,550 of Chukai River –5-year ARI



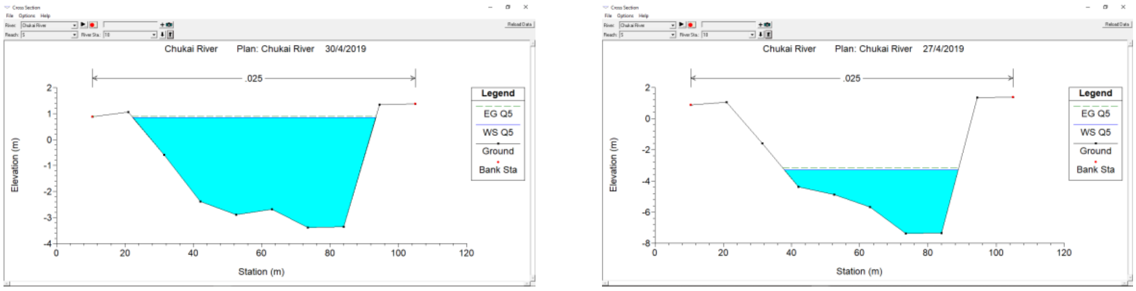
(a) Old bridge piers (b) New bridge piers

Figure 4.15 Water level at CH 11,750 of Chukai River –5-year ARI



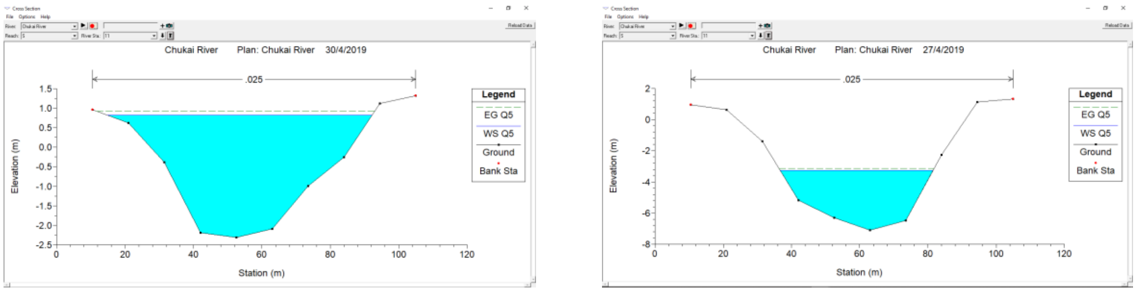
(a) Old bridge piers (b) New bridge piers

Figure 4.16 Water level at CH 11,950 of Chukai River –5-year ARI



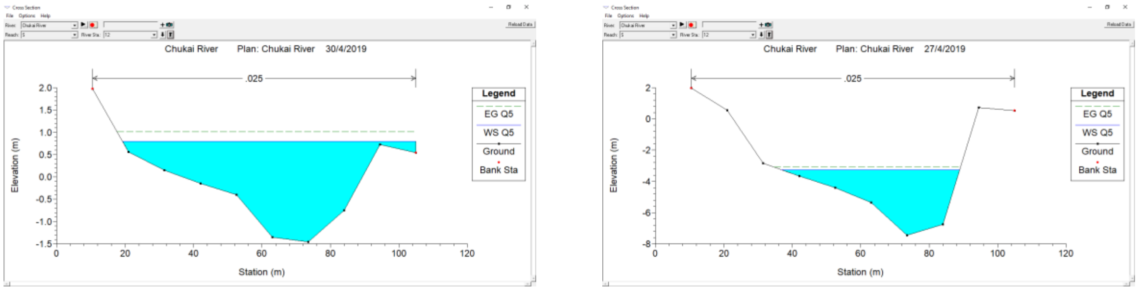
(a) Old bridge piers (b) New bridge piers

Figure 4.17 Water level at CH 12,150 of Chukai River –5-year ARI



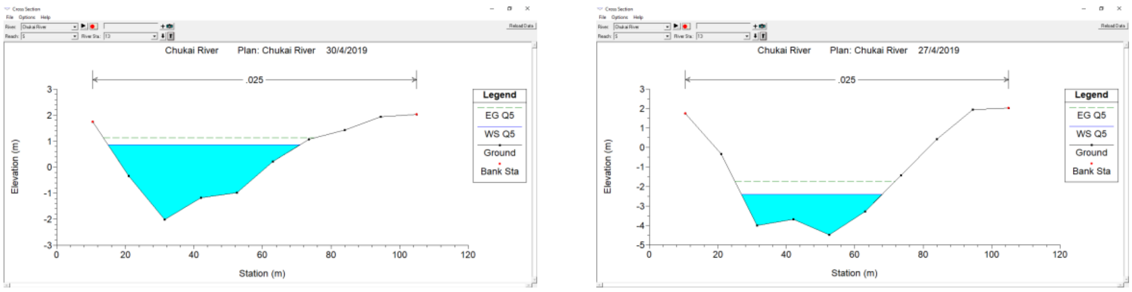
(a) Old bridge piers (b) New bridge piers

Figure 4.18 Water level at CH 12,350 of Chukai River –5-year ARI



(a) Old bridge piers (b) New bridge piers

Figure 4.19 Water level at CH 12,550 of Chukai River –5-year ARI

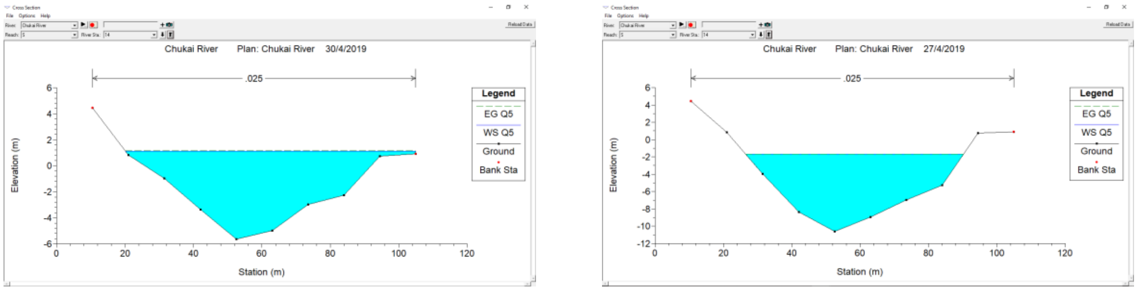


(a) Old bridge piers (b) New bridge piers

Figure 4.20 Water level at CH 12,750 of Chukai River –5-year ARI

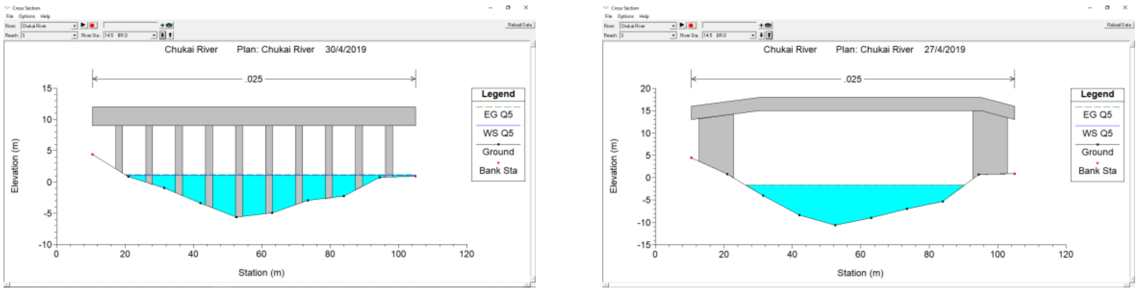
4.5.2 Water level for Q_5 under the bridge

The Bukit Kuang Bridge is located between CH 12,950 and CH 13,150. Table 4.3 and Figure 4.21 to Figure 4.24 presented the water profiles at the bridge areas. These simulations showed that the water level at this chainage with 5-year ARI was 1.13 m above sea level with the old bridge piers and -1.68 m with the new bridge piers. Apart from that, during rainfall event of 5-year ARI, the water level under the bridge cross section which starts from CH 12,950 to CH 13,150 had the difference of -2.81 m.



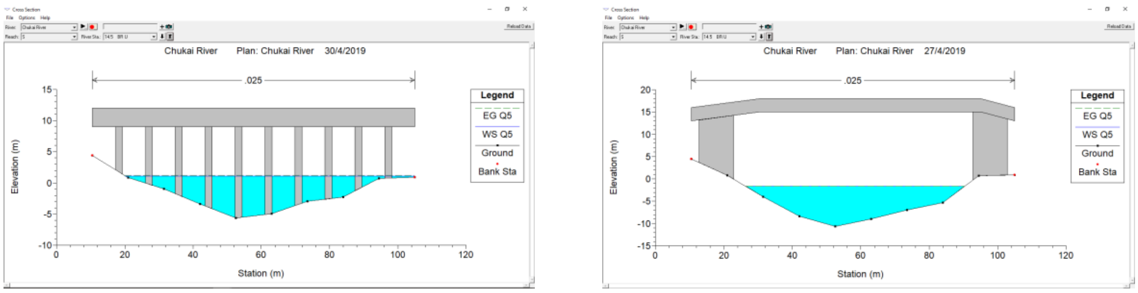
(a) Old bridge piers (b) New bridge piers

Figure 4.21 Water level at CH 12,950 of Chukai River –5-year ARI



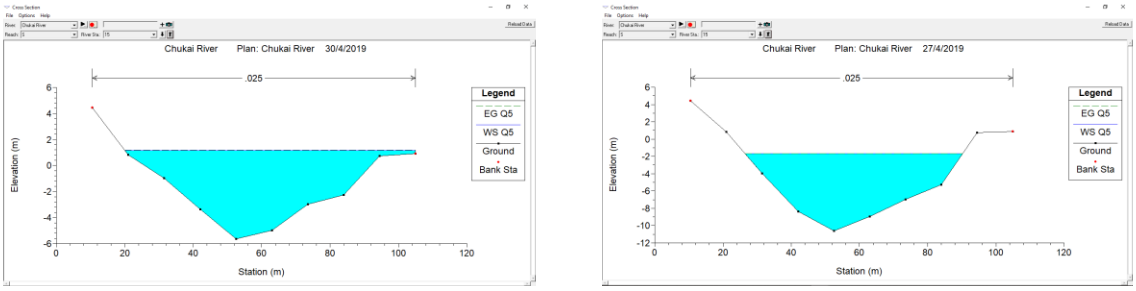
(a) Downstream old bridge piers (b) Downstream new bridge piers

Figure 4.22 Water level at CH 13,050 of Chukai River –5-year ARI



(a) Upstream old bridge piers (b) Upstream new bridge piers

Figure 4.23 Water level at CH 13,050 of Chukai River –5-year ARI



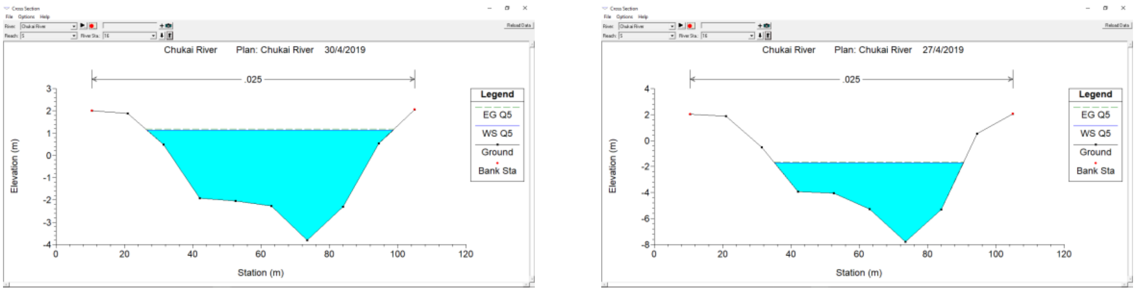
(a) Old bridge piers (b) New bridge piers

Figure 4.24 Water level at CH 13,150 of Chukai River –5-year ARI

4.5.3 Water level for Q_5 at downstream of the bridge

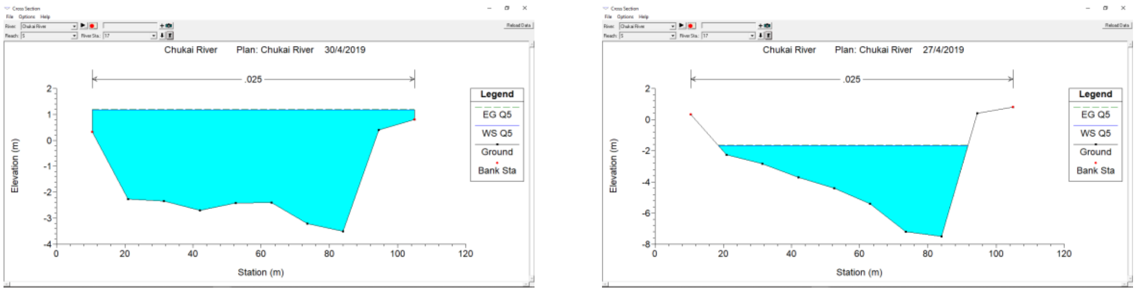
Based on Table 4.3 and Figure 4.25 to Figure 4.37, the water levels were in between 1.13 m to 1.25 m above sea level at the upstream of the bridge with the old bridge piers while with the new bridge piers, the water levels were between -1.67 m to -1.60 m above sea level at the downstream of the bridge.

Another part of concerned is the water levels due to backwater effect along the downstream cross section of the river. This study showed that the increases in water levels from CH 13,350 to CH 15,750 had slightly fluctuated. The water levels were from -2.84 m to -2.85 m different along the chainage with 5-year ARI condition.



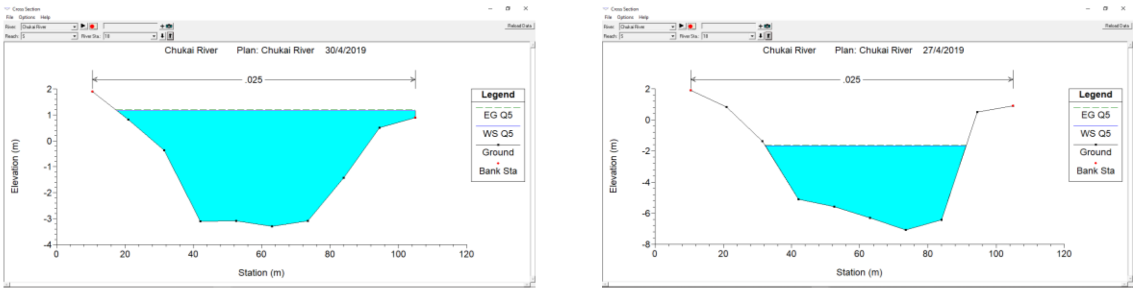
(a) Old bridge piers (b) New bridge piers

Figure 4.25 Water level at CH 13,350 of Chukai River –5-year ARI



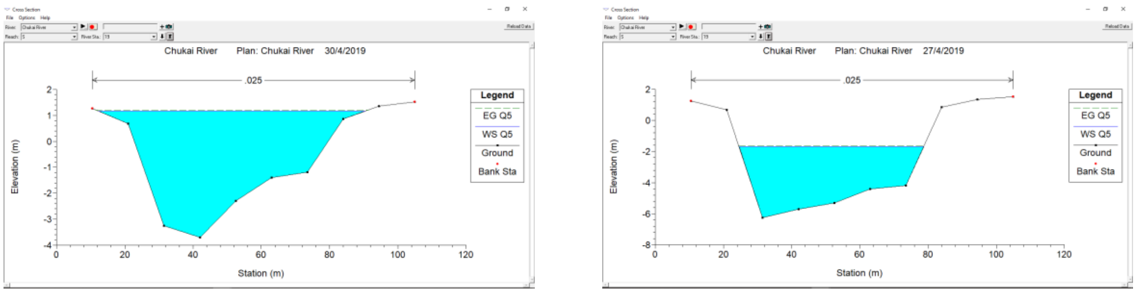
(a) Old bridge piers (b) New bridge piers

Figure 4.26 Water level at CH 13,550 of Chukai River –5-year ARI



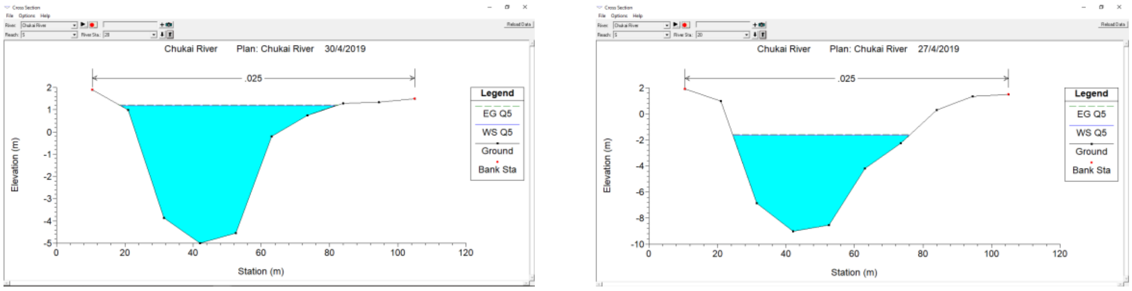
(a) Old bridge piers (b) New bridge piers

Figure 4.27 Water level at CH 13,750 of Chukai River –5-year ARI



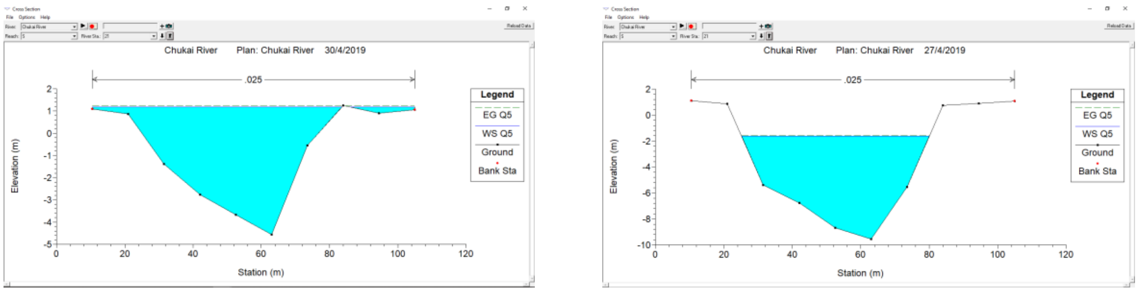
(a) Old bridge piers (b) New bridge piers

Figure 4.28 Water level at CH 13,950 of Chukai River –5-year ARI



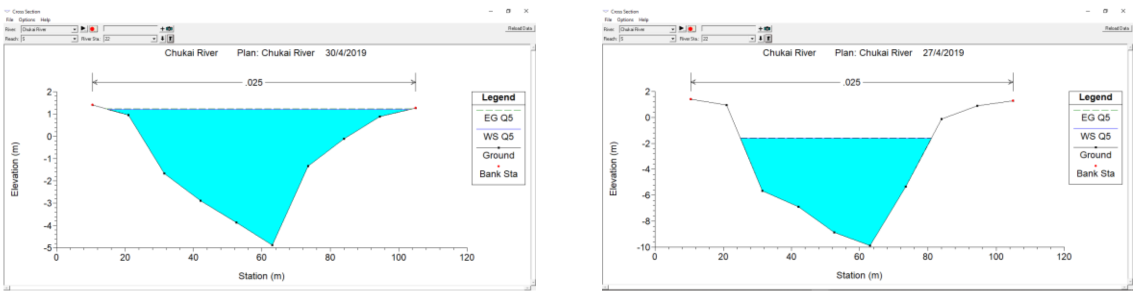
(a) Old bridge piers (b) New bridge piers

Figure 4.29 Water level at CH 14,150 of Chukai River –5-year ARI



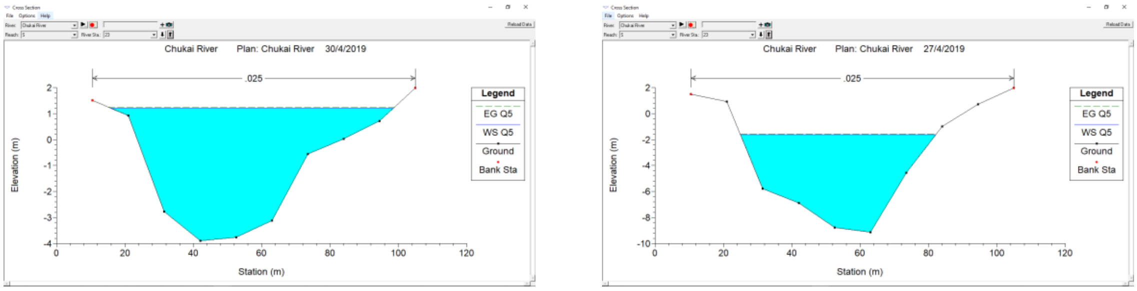
(a) Old bridge piers (b) New bridge piers

Figure 4.30 Water level at CH 14,350 of Chukai River –5-year ARI



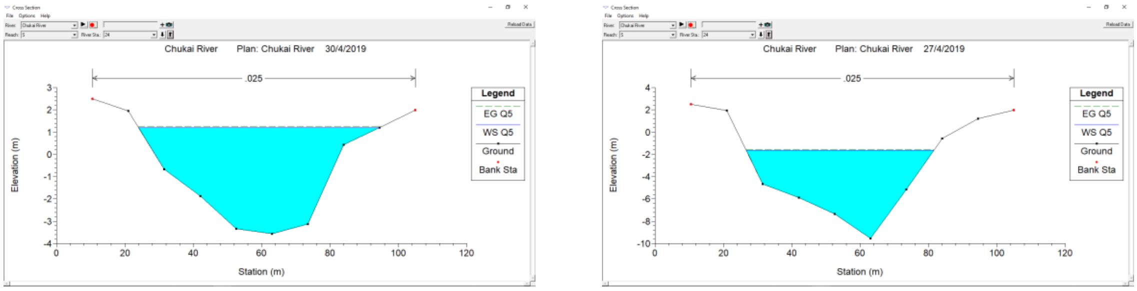
(a) Old bridge piers (b) New bridge piers

Figure 4.31 Water level at CH 14,550 of Chukai River –5-year ARI



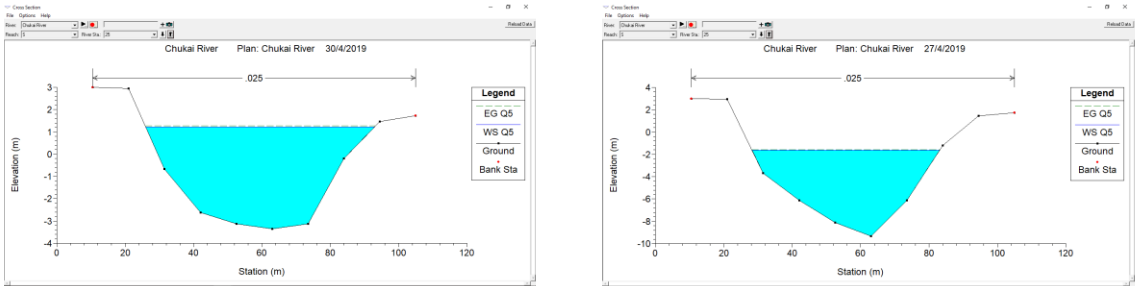
(a) Old bridge piers (b) New bridge piers

Figure 4.32 Water level at CH 14,750 of Chukai River –5-year ARI



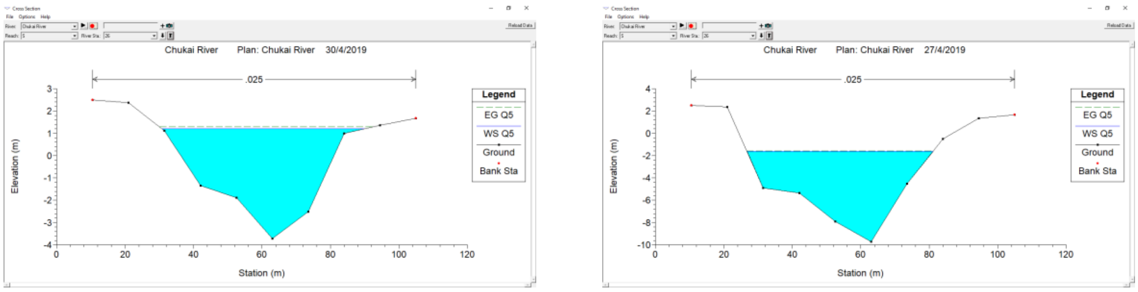
(a) Old bridge piers (b) New bridge piers

Figure 4.33 Water level at CH 14,950 of Chukai River –5-year ARI



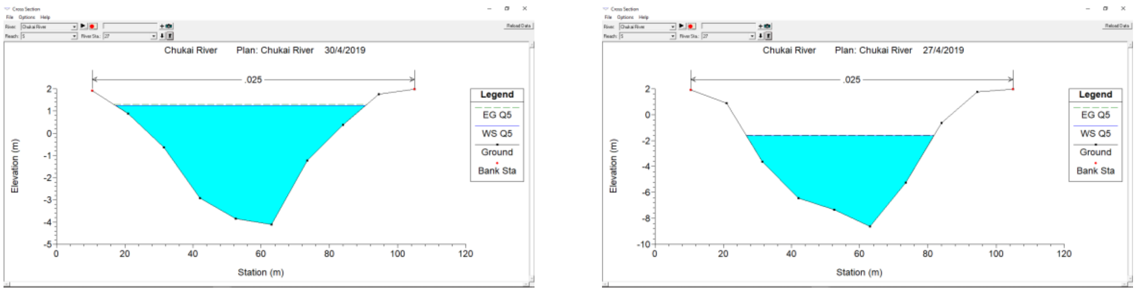
(a) Old bridge piers (b) New bridge piers

Figure 4.34 Water level at CH 15,150 of Chukai River –5-year ARI



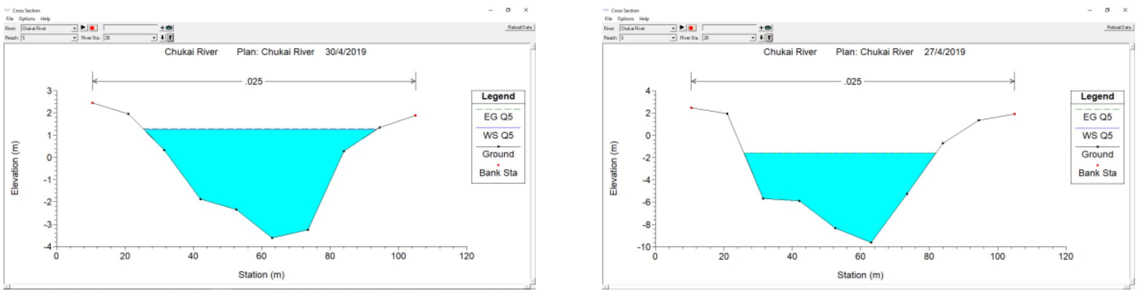
(a) Old bridge piers (b) New bridge piers

Figure 4.35 Water level at CH 15,350 of Chukai River –5-year ARI



(a) Old bridge piers (b) New bridge piers

Figure 4.36 Water level at CH 15,550 of Chukai River –5-year ARI



(a) Old bridge piers (b) New bridge piers

Figure 4.37 Water level at CH 15,750 of Chukai River –5-year ARI

4.6 WATER LEVEL FOR Q_{10} FOR OLD AND NEW BRIDGE PIERS

The difference in water levels of the Chukai River along the chainage with 10-year ARI is tabulated in Table 4.4. For old bridge, CH 10,750, CH 10,950, CH 12,150, CH 12,350 and CH 13,950 were overflown on the left bank. While, for CH 11,550, CH 12,550, CH 12,950, CH 13,050 (D), CH 13,050 (U), CH 13,150 and CH 13,750 were overflown on the right bank. For CH 13,550, CH 14,350 and CH 14,550, it were recorded overflown on both banks. These chainages were located at the upstream, under the bridge and the downstream of the river. For the new bridge, there is no overflown on both banks.

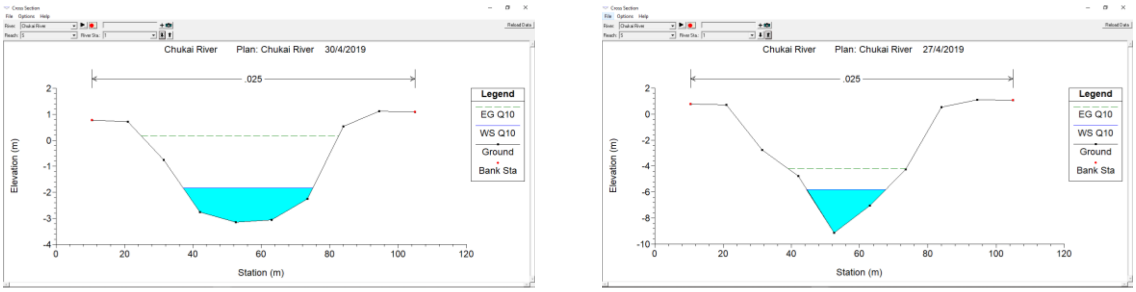
Table 4.4 Difference of water levels with Q_{10}

$Q_{10} = 226.32\ m^3/s$							
Chainage	Old Bridge Water levels (m)		New Bridge Water levels (m)		Difference of water levels (m)		Overflow
	Left bank	Right bank	Left bank	Right bank	Left bank	Right bank	
CH 10,350 (1)	-1.84	-1.84	-5.80	-5.80	-3.96	-3.96	
CH 10,550 (2)	0.19	0.19	-4.66	-4.66	-4.85	-4.85	
CH 10,750 (3)	0.86	0.86	-3.95	-3.95	-4.81	-4.81	(Left bank-Old)
CH 10,950 (4)	0.88	0.88	-3.67	-3.67	-4.55	-4.55	(Left bank-Old)
CH 11,150 (5)	0.88	0.88	-3.73	-3.73	-4.61	-4.61	(Right bank-Old)
CH 11,350 (6)	0.90	0.90	-3.83	-3.83	-4.73	-4.73	
CH 11,550 (7)	0.97	0.97	-3.12	-3.12	-4.09	-4.09	
CH 11,750 (8)	0.97	0.97	-3.13	-3.13	-4.10	-4.10	
CH 11,950 (9)	1.00	1.00	-3.16	-3.16	-4.16	-4.16	
CH 121,50 (10)	1.04	1.04	-3.11	-3.11	-4.15	-4.15	(Left bank-Old)
CH 12,350 (11)	1.02	1.02	-3.10	-3.10	-4.12	-4.12	(Left bank-Old)
CH 12,550 (12)	1.00	1.00	-3.08	-3.08	-4.08	-4.08	(Right bank-Old)
CH 12,750 (13)	1.04	1.04	-2.26	-2.26	-3.30	-3.30	
CH 12,950 (14)	1.32	1.32	-1.48	-1.48	-2.80	-2.80	(Right bank-Old)
CH 13,050 (D) (14.5)	1.32	1.32	-1.48	-1.48	-2.80	-2.80	(Right bank-Old)
CH 13,050 (U) (14.5)	1.32	1.32	-1.48	-1.48	-2.80	-2.80	(Right bank-Old)
CH 13,150 (15)	1.34	1.34	-1.48	-1.48	-2.80	-2.80	(Right bank-Old)
CH 13,350 (16)	1.32	1.32	-1.52	-1.52	-2.84	-2.84	
CH 13,550 (17)	1.36	1.36	-1.48	-1.48	-2.84	-2.84	(Both banks-Old)
CH 13,750 (18)	1.36	1.36	-1.47	-1.47	-2.83	-2.83	(Right bank-Old)
CH 13,950 (19)	1.35	1.35	-1.49	-1.49	-2.84	-2.84	(Left bank-Old)
CH 14,150 (20)	1.36	1.36	-1.44	-1.44	-2.80	-2.80	
CH 14,350 (21)	1.38	1.38	-1.42	-1.42	-2.80	-2.80	(Both banks-Old)
CH 14,550 (22)	1.40	1.40	-1.42	-1.42	-2.82	-2.82	(Both banks-Old)
CH 14,750 (23)	1.41	1.41	-1.42	-1.42	-2.83	-2.83	
CH 14,950 (24)	1.41	1.41	-1.42	-1.42	-2.83	-2.83	
CH 15,150 (25)	1.42	1.42	-1.41	-1.41	-2.83	-2.83	
CH 15,350 (26)	1.40	1.40	-1.41	-1.41	-2.81	-2.81	
CH 15,550 (27)	1.46	1.46	-1.41	-1.41	-2.87	-2.87	
CH 15,750 (28)	1.46	1.46	-1.40	-1.40	-2.86	-2.86	

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

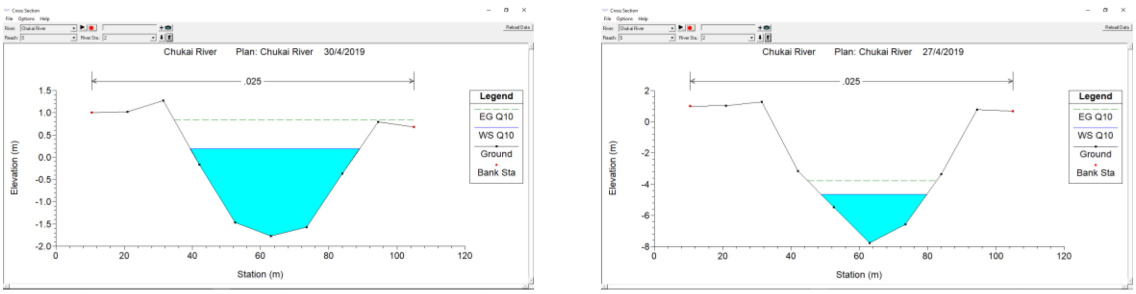
Based on Table 4.4 and Figure 4.38 to Figure 4.50 the water levels were in between -1.84 m to 1.04 m above sea level at the upstream of the bridge with the old bridge piers while with the new bridge piers, the water levels were between –5.80 m to -2.26 m above sea level at the upstream of the bridge. The water levels surpassed the river bank along CH 10,350 to CH 15,750 during rainfall event of 10-year ARI. In other words, the water was overflown onto the several chainages due to its incline bank surface as stated in Table 4.4.

Besides, the differences in water levels along the upstream chainage which start from CH 10,350 to CH 12,750 (Figure 4.38 to Figure 4.50) were between -3.96 m to -3.30 m. These indicated that with the presence of new bridge piers, flooding would not occur.



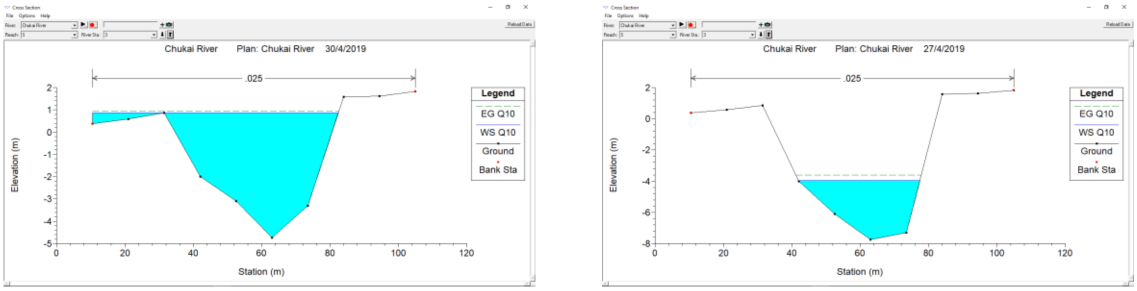
(a) Old bridge piers (b) New bridge piers

Figure 4.38 Water level at CH 10,350 of Chukai River – 10-year ARI



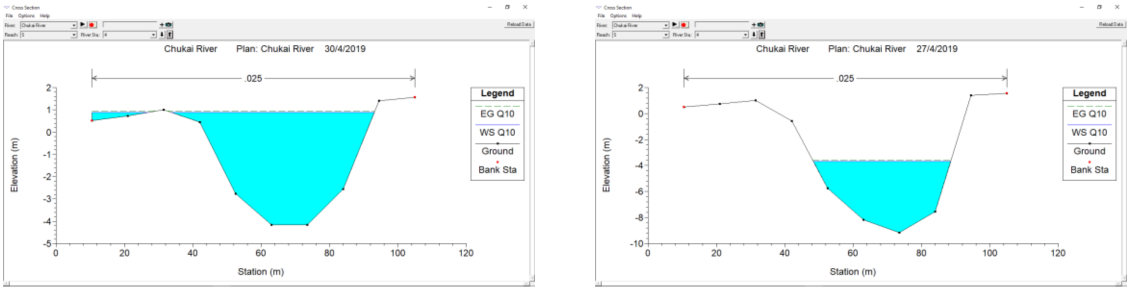
(a) Old bridge piers (b) New bridge piers

Figure 4.39 Water level at CH 10,550 of Chukai River –10-year ARI



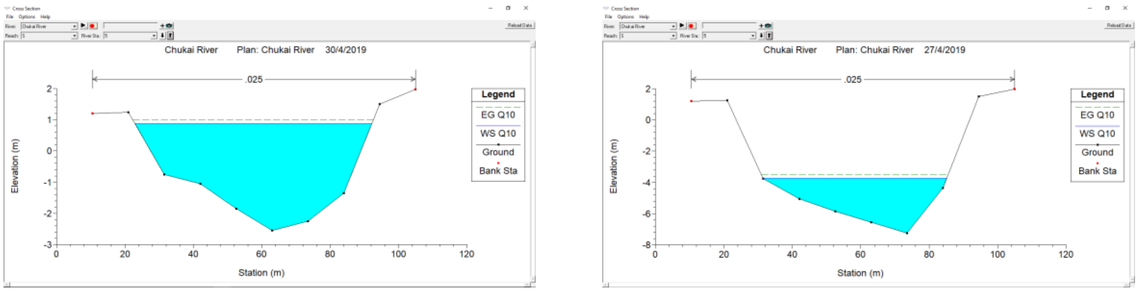
(a) Old bridge piers (b) New bridge piers

Figure 4.40 Water level at CH 10,750 of Chukai River –10-year ARI



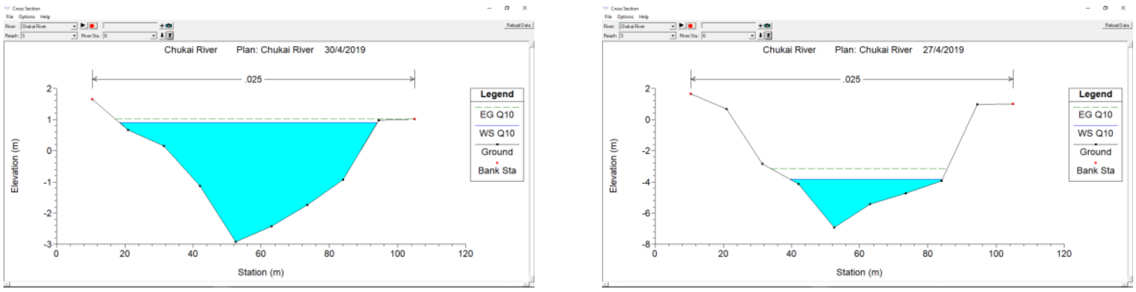
(a) Old bridge piers (b) New bridge piers

Figure 4.41 Water level at CH 10,950 of Chukai River –10-year ARI



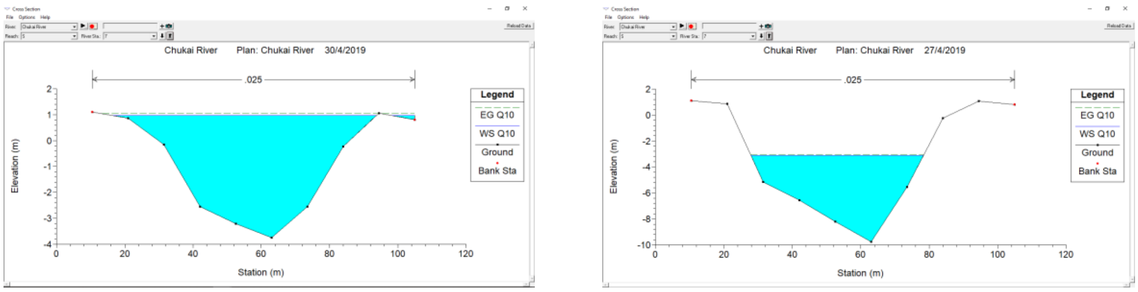
(a) Old bridge piers (b) New bridge piers

Figure 4.42 Water level at CH 11,150 of Chukai River –10-year ARI



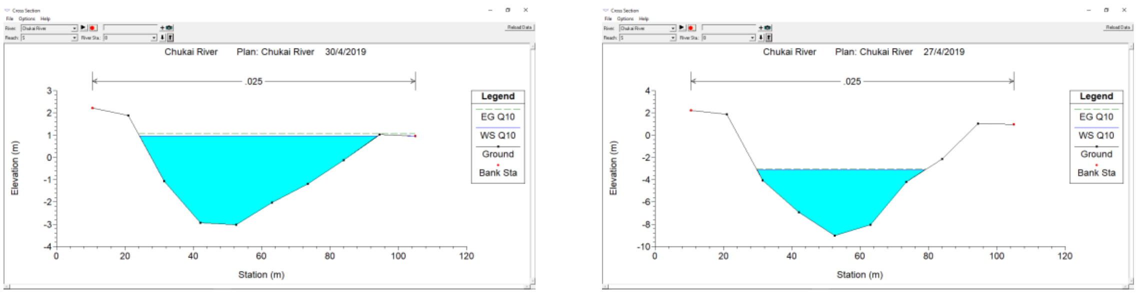
(a) Old bridge piers (b) New bridge piers

Figure 4.43 Water level at CH 11,350 of Chukai River –10-year ARI



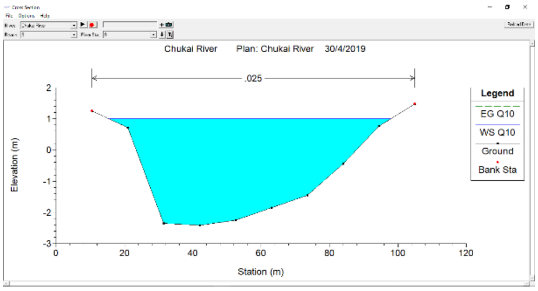
(a) Old bridge piers (b) New bridge piers

Figure 4.44 Water level at CH 11,550 of Chukai River –10-year ARI

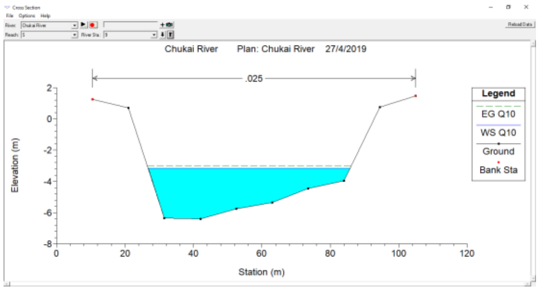


(a) Old bridge piers (b) New bridge piers

Figure 4.45 Water level at CH 11,750 of Chukai River –10-year ARI

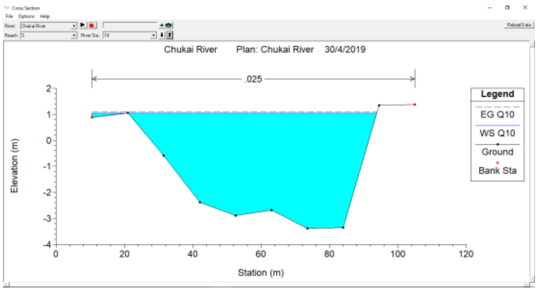


(a) Old bridge piers

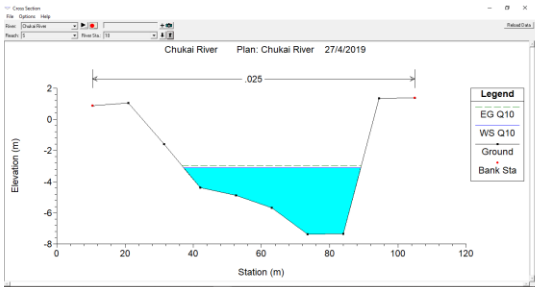


(b) New bridge piers

Figure 4.46 Water level at CH 11,950 of Chukai River –10-year ARI

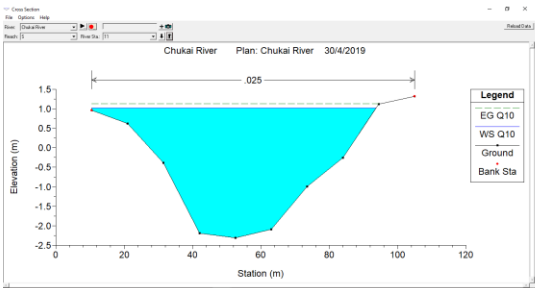


(a) Old bridge piers

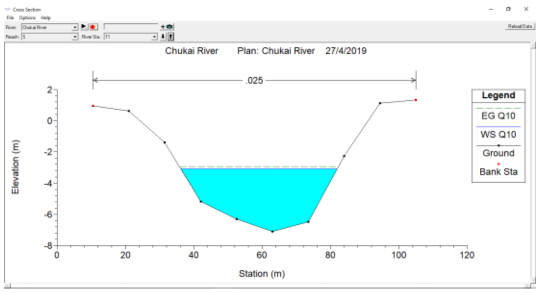


(b) New bridge piers

Figure 4.47 Water level at CH 12,150 of Chukai River –10-year ARI

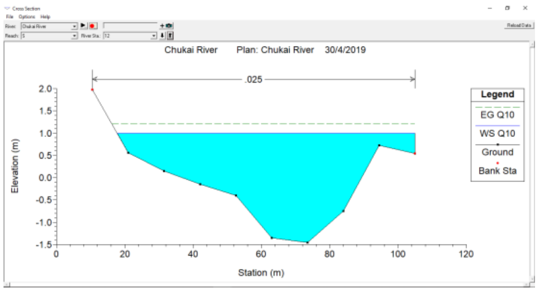


(a) Old bridge piers

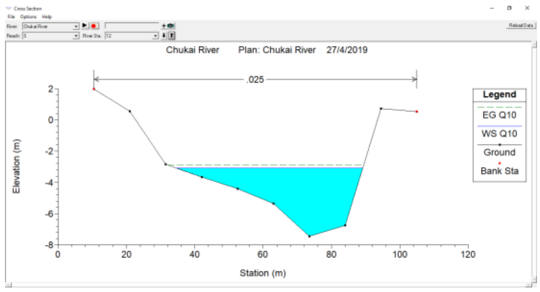


(b) New bridge piers

Figure 4.48 Water level at CH 12,350 of Chukai River –10-year ARI

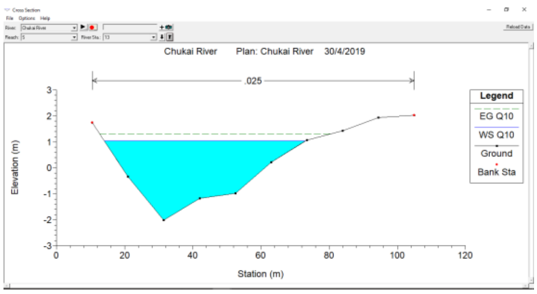


(a) Old bridge piers

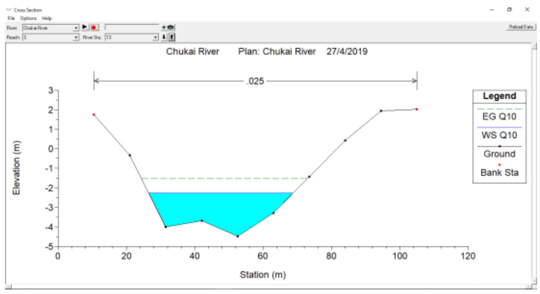


(b) New bridge piers

Figure 4.49 Water level at CH 12,550 of Chukai River –10-year ARI



(a) Old bridge piers

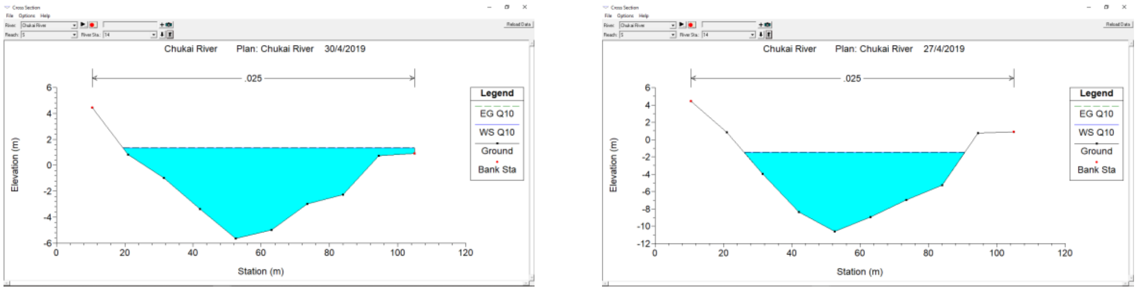


(b) New bridge piers

Figure 4.50 Water level at CH 12,750 of Chukai River –10-year ARI

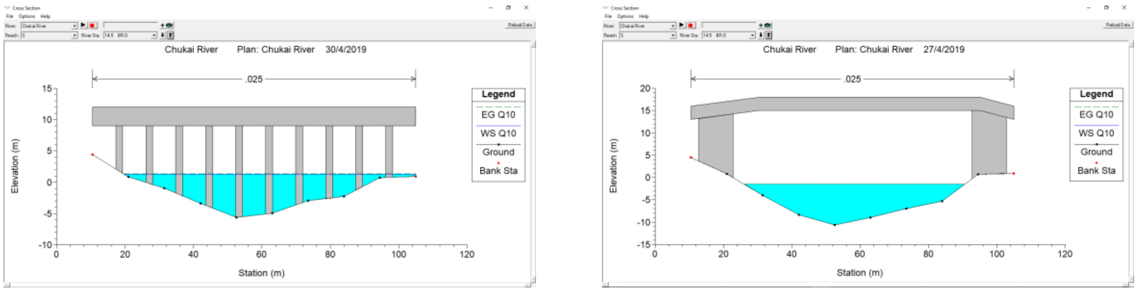
4.6.2 Water level for Q_{10} under the bridge

Table 4.4 and Figure 4.51 to Figure 4.54 presented the water profiles at the bridge areas. These simulations showed that the water level at this chainage with 10-year ARI is 1.32 m above sea level with the old bridge piers and -1.48 m with the new bridge piers. Apart from that, during rainfall event of 10-year ARI, the water level under the bridge cross section which starts from CH 12,950 to CH 13,150 had the difference of -2.80 m.



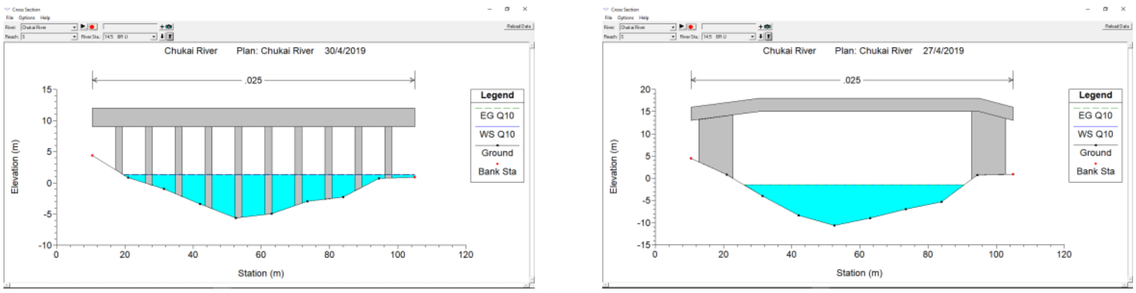
(a) Old bridge piers (b) New bridge piers

Figure 4.51 Water level at CH 12,950 of Chukai River –10-year ARI



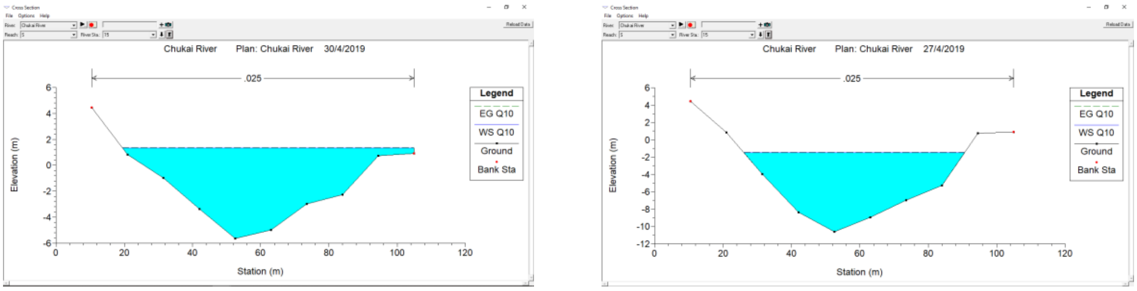
(a) Downstream old bridge piers (b) Downstream new bridge piers

Figure 4.52 Water level at CH 13,050 of Chukai River –10-year ARI



(a) Upstream old bridge piers (b) Upstream new bridge piers

Figure 4.53 Water level at CH 13,050 of Chukai River –10-year ARI



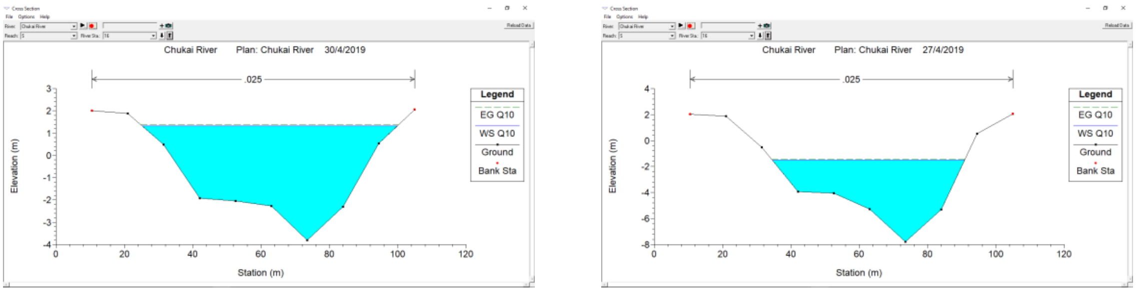
(a) Old bridge piers (b) New bridge piers

Figure 4.54 Water level at CH 13,150 of Chukai River –10-year ARI

4.6.3 Water level for Q_{10} at downstream of the bridge

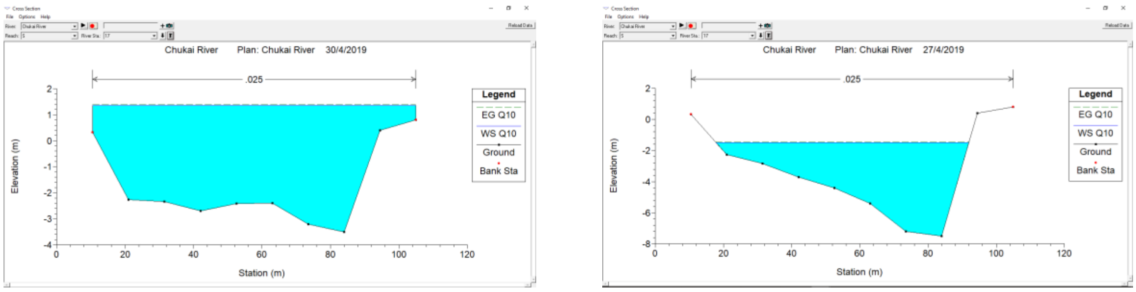
Based on Table 4.4 and Figure 4.55 to Figure 4.67, the water levels were in between 1.32 m to 1.46 m above sea level at the upstream of the bridge with the old bridge piers while with the new bridge piers, the water levels were between -1.52 m to -1.40 m above sea level at the downstream of the bridge.

This study showed that the increases in water levels from CH 13,350 to CH 15,750 had slightly fluctuated. The water levels were from -2.84 m to -2.86 m different along the chainage with 10-year ARI condition.



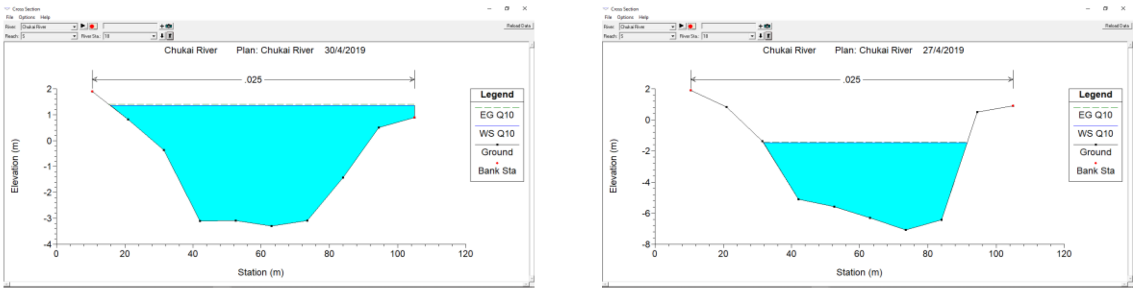
(a) Old bridge piers (b) New bridge piers

Figure 4.55 Water level at CH 13,350 of Chukai River –10-year ARI



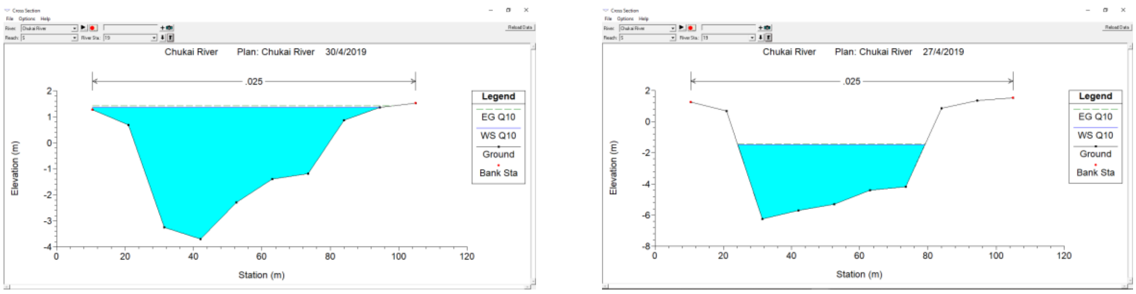
(a) Old bridge piers (b) New bridge piers

Figure 4.56 Water level at CH 13,550 of Chukai River –10-year ARI



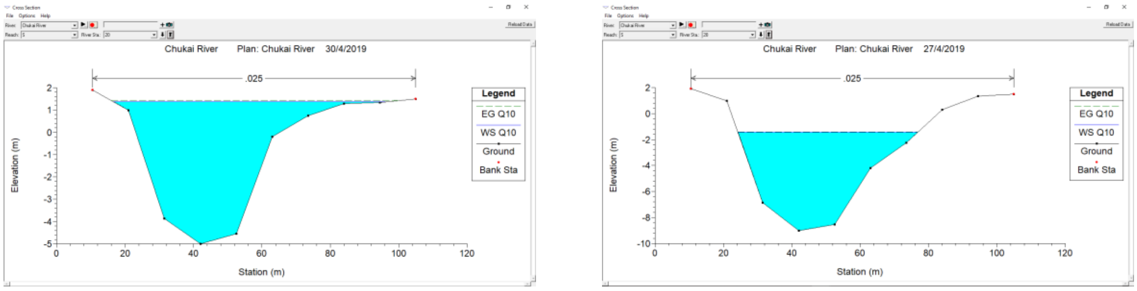
(a) Old bridge piers (b) New bridge piers

Figure 4.57 Water level at CH 13,750 of Chukai River –10-year ARI



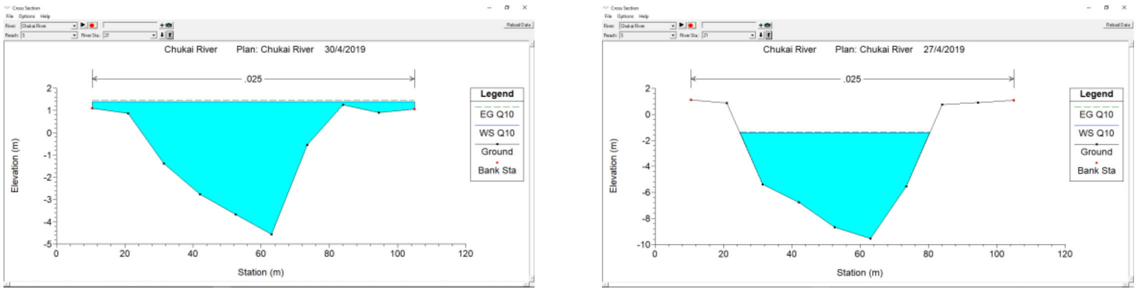
(a) Old bridge piers (b) New bridge piers

Figure 4.58 Water level at CH 13,950 of Chukai River –10-year ARI



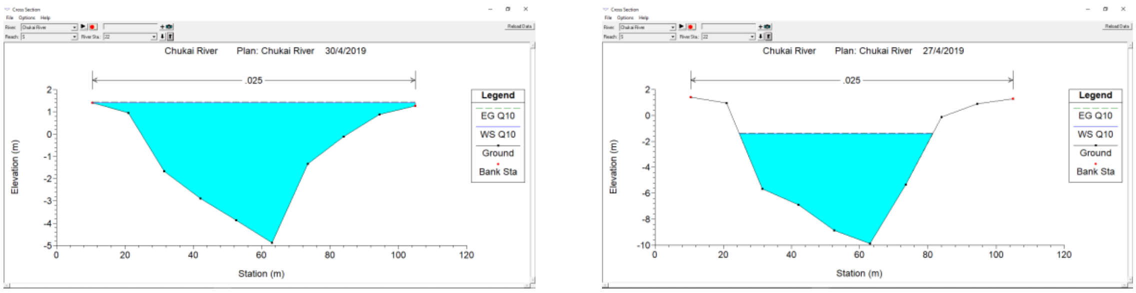
(a) Old bridge piers (b) New bridge piers

Figure 4.59 Water level at CH 14,150 of Chukai River –10-year ARI



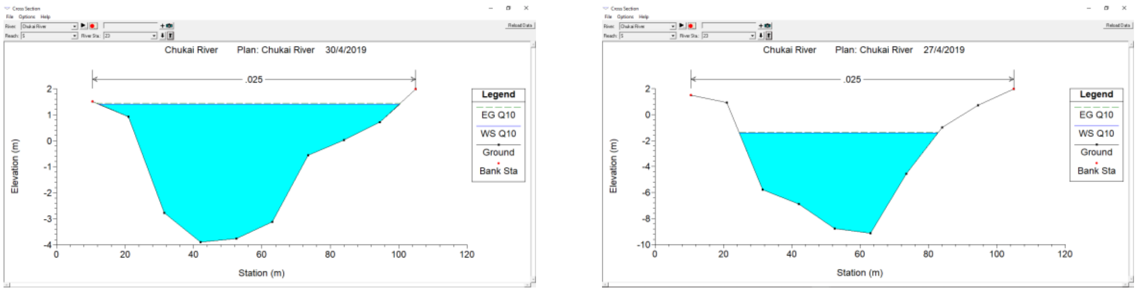
(a) Old bridge piers (b) New bridge piers

Figure 4.60 Water level at CH 14,350 of Chukai River –10-year ARI



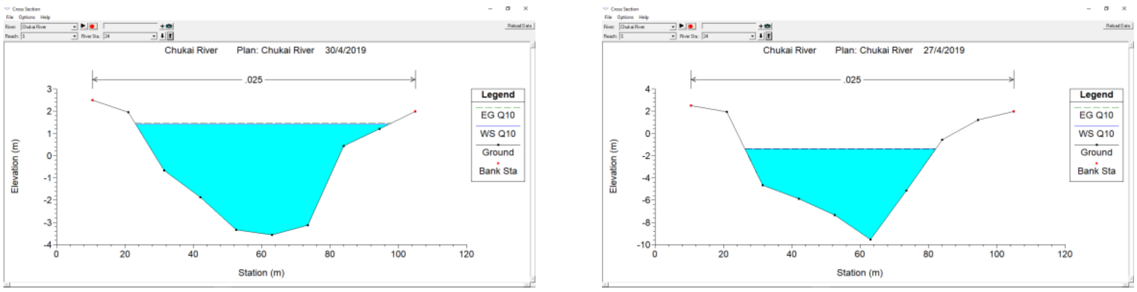
(a) Old bridge piers (b) New bridge piers

Figure 4.61 Water level at CH 14,550 of Chukai River –10-year ARI



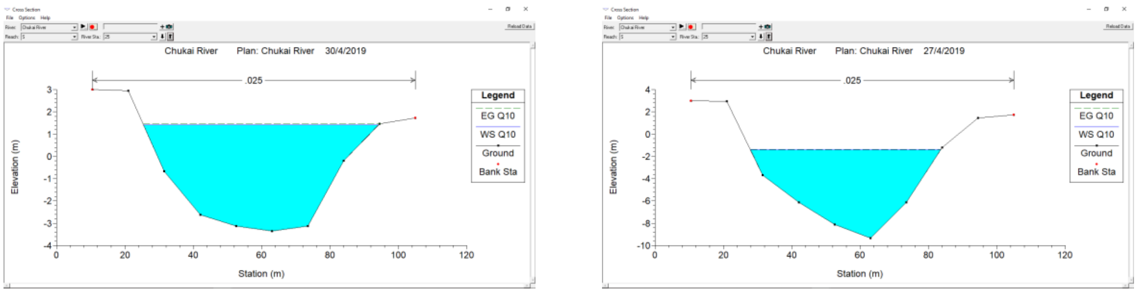
(a) Old bridge piers (b) New bridge piers

Figure 4.62 Water level at CH 14,750 of Chukai River –10-year ARI



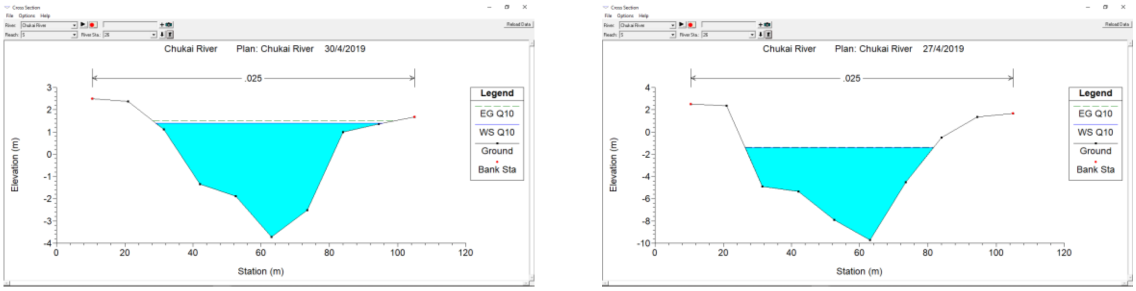
(a) Old bridge piers (b) New bridge piers

Figure 4.63 Water level at CH 14,950 of Chukai River –10-year ARI



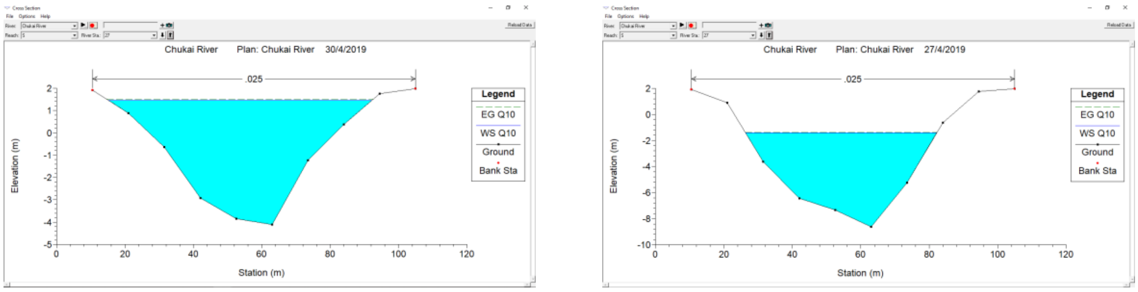
(a) Old bridge piers (b) New bridge piers

Figure 4.64 Water level at CH 15,150 of Chukai River –10-year ARI



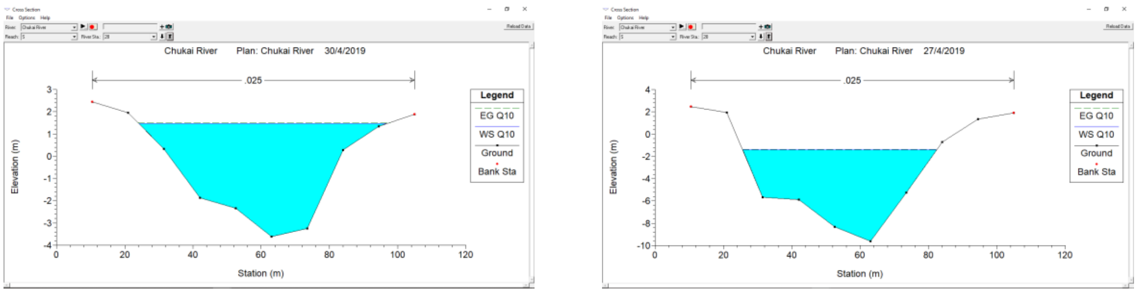
(a) Old bridge piers (b) New bridge piers

Figure 4.65 Water level at CH 15,350 of Chukai River –10-year ARI



(a) Old bridge piers (b) New bridge piers

Figure 4.66 Water level at CH 15,550 of Chukai River –10-year ARI



(a) Old bridge piers (b) New bridge piers

Figure 4.67 Water level at CH 15,750 of Chukai River –10-year ARI

4.7 WATER LEVEL FOR Q_{20} FOR OLD AND NEW BRIDGE PIERS

The difference in water levels of the Chukai River along the chainage with 20-year ARI are tabulated in Table 4.5. For old bridge, CH 10,750, CH 10,950, CH 12,150, CH 12,350 and CH 13,950 were overflown on the left bank. While, for CH 11,350, CH 11,550, CH 11,750, CH 12,550, CH 12,950, CH 13,050 (D), CH 13,050 (U), CH 13,150, CH 13,750 and CH 14,150 were overflown on the right bank. For CH 13,550, CH 14,350 and CH 14,550, it were recorded overflown on both banks. These chainages were located at the upstream, under the bridge and the downstream of the river. For the new bridge, there is no overflown on both banks.

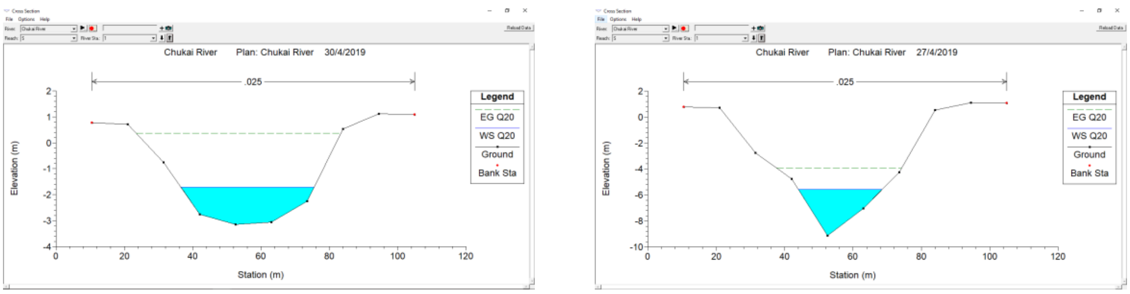
Table 4.5 Difference of water levels with Q_{20}

$Q_{20} = 260.12\ m^3/s$							
Chainage	Old Bridge Water levels (m)		New Bridge Water levels (m)		Difference of water levels (m)		Overflow
	Left bank	Right bank	Left bank	Right bank	Left bank	Right bank	
CH 10,350 (1)	-1.72	-1.72	-5.57	-5.57	-3.85	-3.85	
CH 10,550 (2)	0.34	0.34	-4.46	-4.46	-4.80	-4.80	
CH 10,750 (3)	1.03	1.03	-3.74	-3.74	-4.77	-4.77	(Left bank-Old)
CH 10,950 (4)	1.06	1.06	-3.43	-3.43	-4.49	-4.49	(Left bank-Old)
CH 11,150 (5)	1.06	1.06	-3.48	-3.48	-4.54	-4.54	
CH 11,350 (6)	1.08	1.08	-3.70	-3.70	-4.78	-4.78	(Right bank-Old)
CH 11,550 (7)	1.16	1.16	-2.93	-2.93	-4.09	-4.09	(Right bank-Old)
CH 11,750 (8)	1.16	1.16	-2.94	-2.94	-4.10	-4.10	(Right bank-Old)
CH 11,950 (9)	1.20	1.20	-2.96	-2.96	-4.16	-4.16	
CH 121,50 (10)	1.24	1.24	-2.92	-2.92	-4.16	-4.16	(Left bank-Old)
CH 12,350 (11)	1.22	1.22	-2.91	-2.91	-4.13	-4.13	(Left bank-Old)
CH 12,550 (12)	1.20	1.20	-2.88	-2.88	-4.08	-4.08	(Right bank-Old)
CH 12,750 (13)	1.22	1.22	-2.11	-2.11	-3.33	-3.33	
CH 12,950 (14)	1.52	1.52	-1.28	-1.28	-2.80	-2.80	(Right bank-Old)
CH 13,050 (D) (14.5)	1.52	1.52	-1.28	-1.28	-2.80	-2.80	(Right bank-Old)
CH 13,050 (U) (14.5)	1.52	1.52	-1.28	-1.28	-2.80	-2.80	(Right bank-Old)
CH 13,150 (15)	1.54	1.54	-1.27	-1.27	-2.81	-2.81	(Right bank-Old)
CH 13,350 (16)	1.52	1.52	-1.31	-1.31	-2.83	-2.83	
CH 13,550 (17)	1.57	1.57	-1.27	-1.27	-2.84	-2.84	(Both banks-Old)
CH 13,750 (18)	1.56	1.56	-1.26	-1.26	-2.82	-2.82	(Right bank-Old)
CH 13,950 (19)	1.56	1.56	-1.28	-1.28	-2.84	-2.84	(Left bank-Old)
CH 14,150 (20)	1.57	1.57	-1.23	-1.23	-2.80	-2.80	(Right bank-Old)
CH 14,350 (21)	1.59	1.59	-1.21	-1.21	-2.80	-2.80	(Both banks-Old)
CH 14,550 (22)	1.61	1.61	-1.20	-1.20	-2.81	-2.81	(Both banks-Old)
CH 14,750 (23)	1.62	1.62	-1.20	-1.20	-2.82	-2.82	
CH 14,950 (24)	1.61	1.61	-1.21	-1.21	-2.82	-2.82	
CH 15,150 (25)	1.63	1.63	-1.20	-1.20	-2.83	-2.83	
CH 15,350 (26)	1.61	1.61	-1.20	-1.20	-2.81	-2.81	
CH 15,550 (27)	1.67	1.67	-1.20	-1.20	-2.87	-2.87	
CH 15,750 (28)	1.67	1.67	-1.18	-1.18	-2.85	-2.85	

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

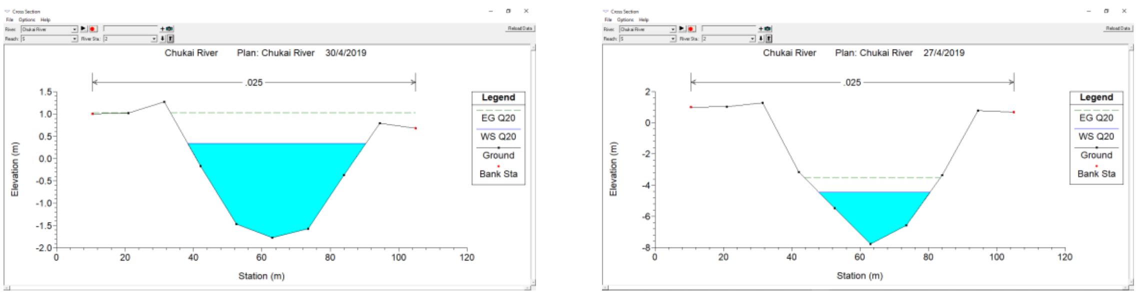
Based on Table 4.5 and Figure 4.68 to Figure 4.80 the water levels were in between -1.72 m to 1.22 m above sea level at the upstream of the bridge with the old bridge piers while with the new bridge piers, the water levels were between -5.57 m to -2.11 m above sea level at the upstream of the bridge. The water levels surpassed the river bank along CH 10,350 to CH 15,750 during rainfall event of 20-year ARI. In other words, the water was overflown onto the several chainages due to its incline bank surface as stated in Table 4.5.

Besides, the differences in water levels along the upstream chainage which start from CH 10,350 to CH 12,750 (Figure 4.68 to Figure 4.80) were between -3.85 m to -3.33 m. These indicated that with the presence of new bridge piers, flooding will not occur.



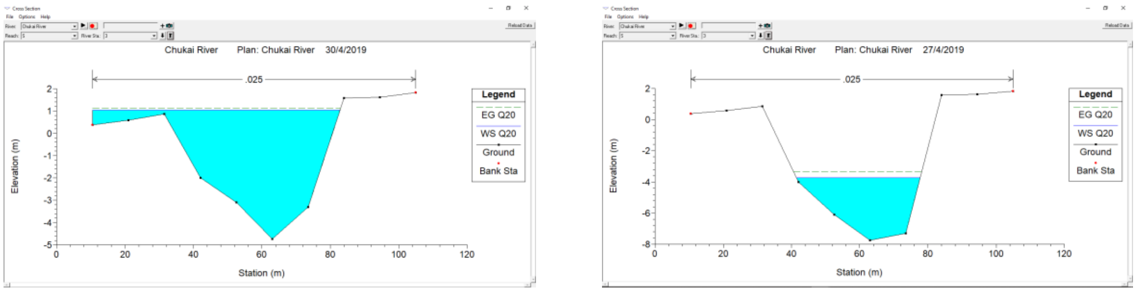
(a) Old bridge piers (b) New bridge piers

Figure 4.68 Water level at CH 10,350 of Chukai River – 20-year ARI



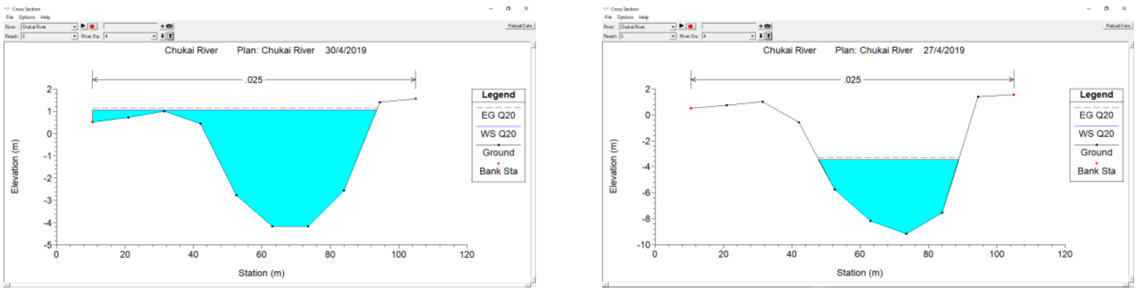
(a) Old bridge piers (b) New bridge piers

Figure 4.69 Water level at CH 10,550 of Chukai River –20-year ARI



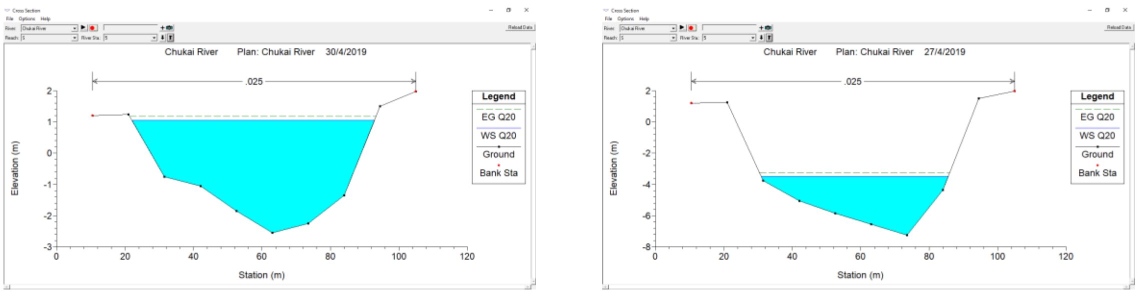
(a) Old bridge piers (b) New bridge piers

Figure 4.70 Water level at CH 10,750 of Chukai River –20-year ARI



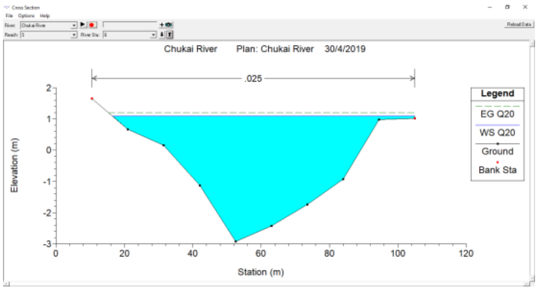
(a) Old bridge piers (b) New bridge piers

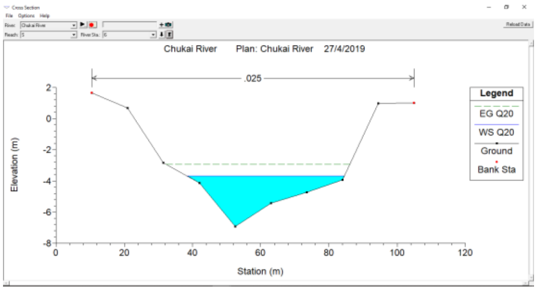
Figure 4.71 Water level at CH 10,950 of Chukai River –20-year ARI



(a) Old bridge piers (b) New bridge piers

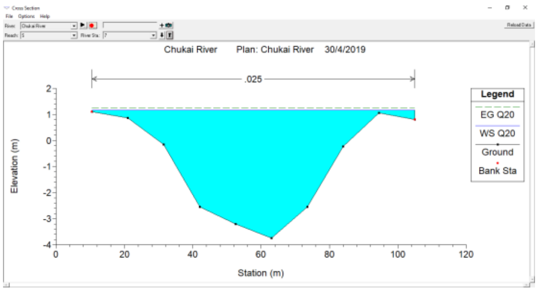
Figure 4.72 Water level at CH 11,150 of Chukai River –20-year ARI

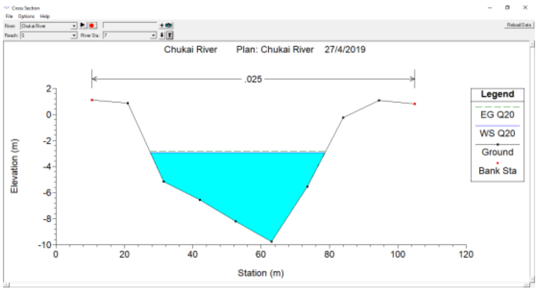




(a) Old bridge piers (b) New bridge piers

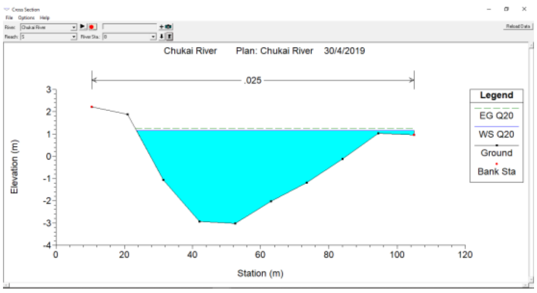
Figure 4.73 Water level at CH 11,350 of Chukai River –20-year ARI

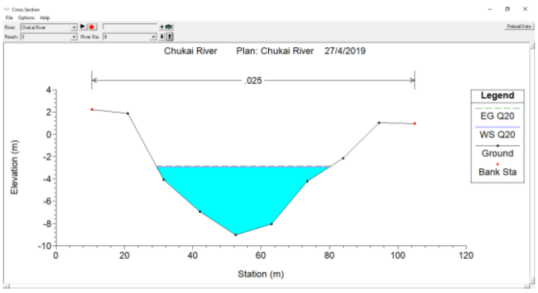




(a) Old bridge piers (b) New bridge piers

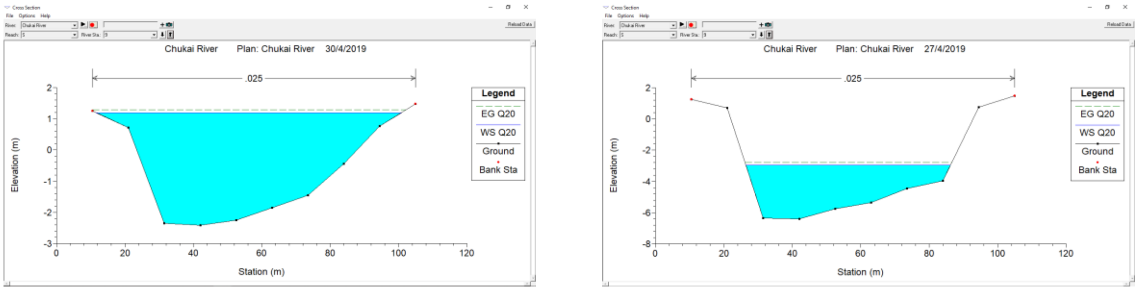
Figure 4.74 Water level at CH 11,550 of Chukai River –20-year ARI





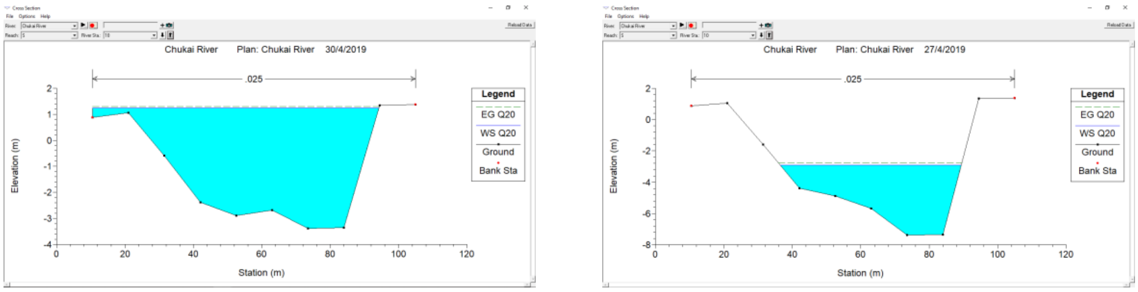
(a) Old bridge piers (b) New bridge piers

Figure 4.75 Water level at CH 11,750 of Chukai River –20-year ARI



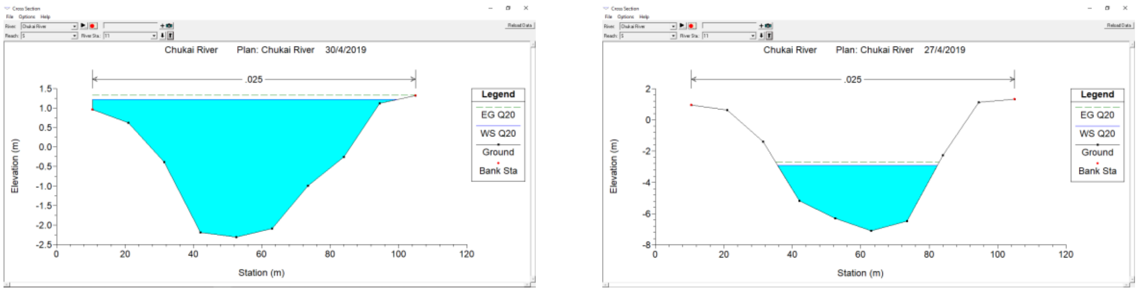
(a) Old bridge piers (b) New bridge piers

Figure 4.76 Water level at CH 11,950 of Chukai River –20-year ARI



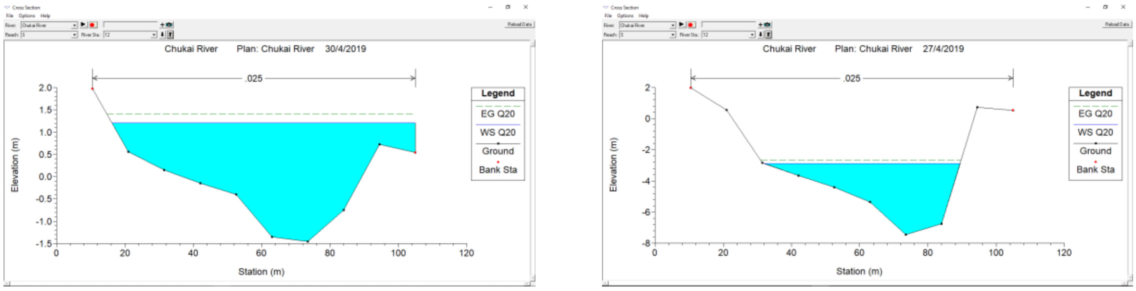
(a) Old bridge piers (b) New bridge piers

Figure 4.77 Water level at CH 12,150 of Chukai River –20-year ARI



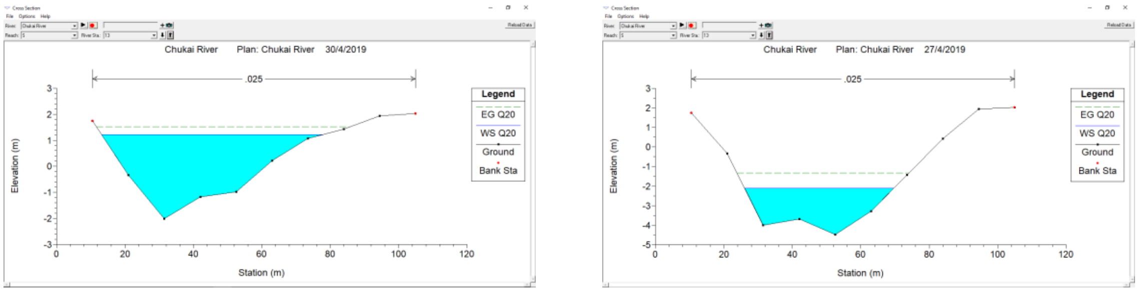
(a) Old bridge piers (b) New bridge piers

Figure 4.78 Water level at CH 12,350 of Chukai River –20-year ARI



(a) Old bridge piers (b) New bridge piers

Figure 4.79 Water level at CH 12,550 of Chukai River –20-year ARI

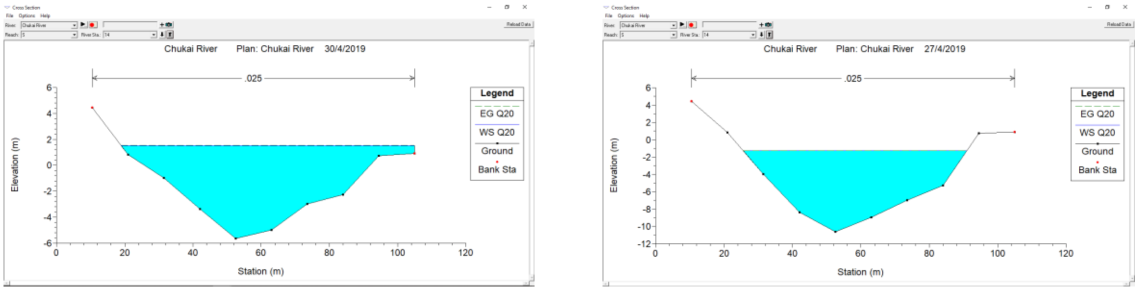


(a) Old bridge piers (b) New bridge piers

Figure 4.80 Water level at CH 12,750 of Chukai River –20-year ARI

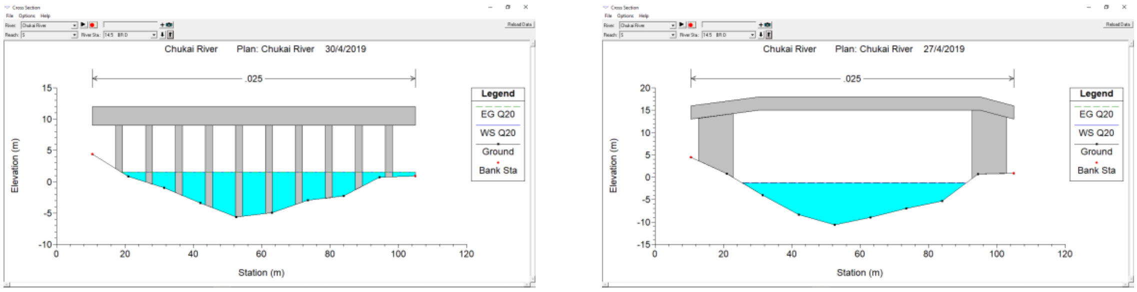
4.7.2 Water level for Q_{20} under the bridge

Table 4.5 and Figure 4.81 to Figure 4.84 presented the water profiles at the bridge areas. These simulations showed that the water level at this chainage with 20-year ARI is 1.52 m above sea level with the old bridge piers and -1.28 m with the new bridge piers. Apart from that, during rainfall event of 20-year ARI, the water level under the bridge cross section which starts from CH 12,950 to CH 13,150 had the difference of -2.80 m.



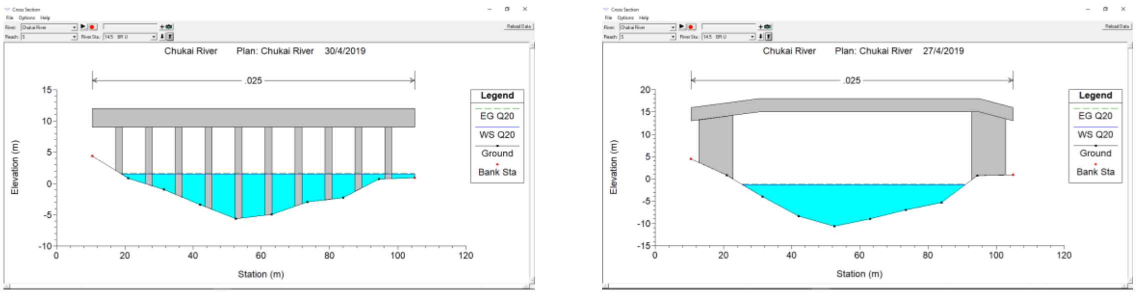
(a) Old bridge piers (b) New bridge piers

Figure 4.81 Water level at CH 12,950 of Chukai River –20-year ARI



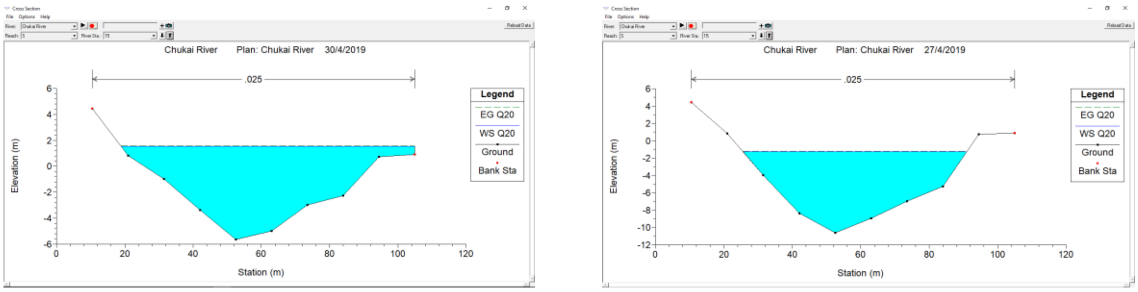
(a) Downstream old bridge piers (b) Downstream new bridge piers

Figure 4.82 Water level at CH 13,050 of Chukai River –20-year ARI



(a) Upstream old bridge piers (b) Upstream new bridge piers

Figure 4.83 Water level at CH 13,050 of Chukai River –20-year ARI



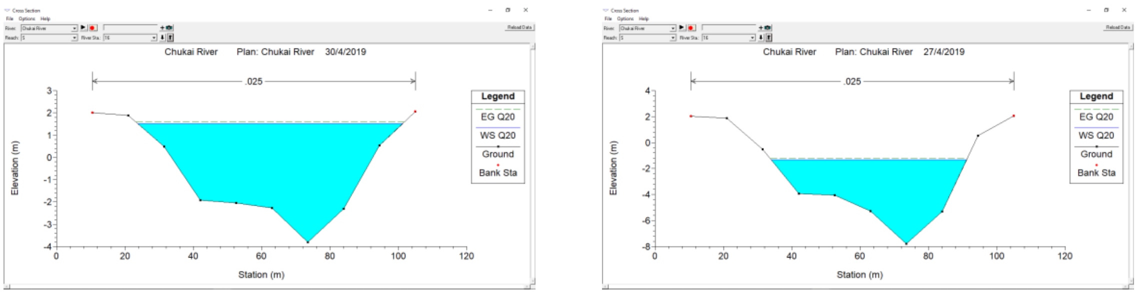
(a) Old bridge piers (b) New bridge piers

Figure 4.84 Water level at CH 13,150 of Chukai River –20-year ARI

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

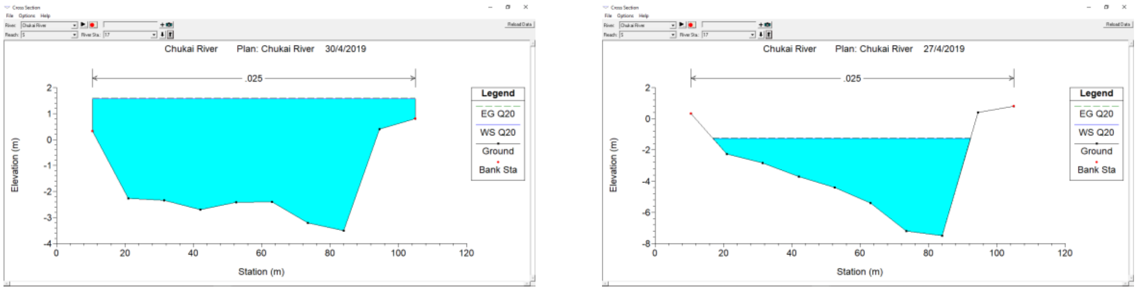
Based on Table 4.5 and Figure 4.85 to Figure 4.97, the water levels were in between 1.52 m to 1.67 m above sea level at the upstream of the bridge with the old bridge piers while with the new bridge piers, the water levels were between -1.31 m to -1.18 m above sea level at the downstream of the bridge.

This study showed that the increases in water levels from CH 13,350 to CH 15,750 had slightly fluctuated. The water levels were from -2.83 m to -2.85 m different along the chainage with 20-year ARI condition.



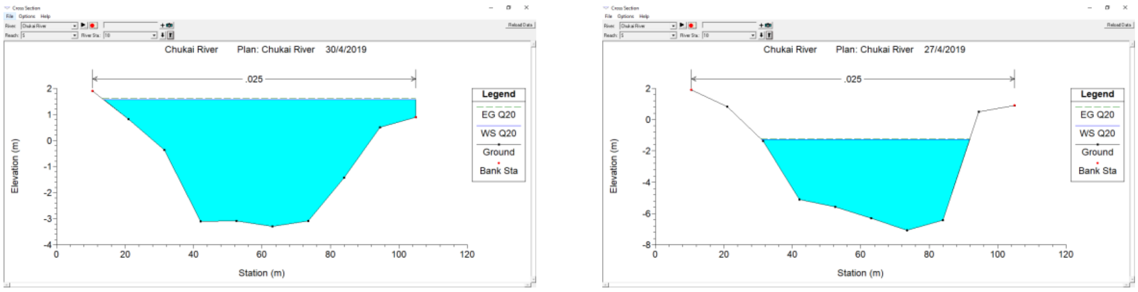
(a) Old bridge piers (b) New bridge piers

Figure 4.85 Water level at CH 13,350 of Chukai River –20-year ARI



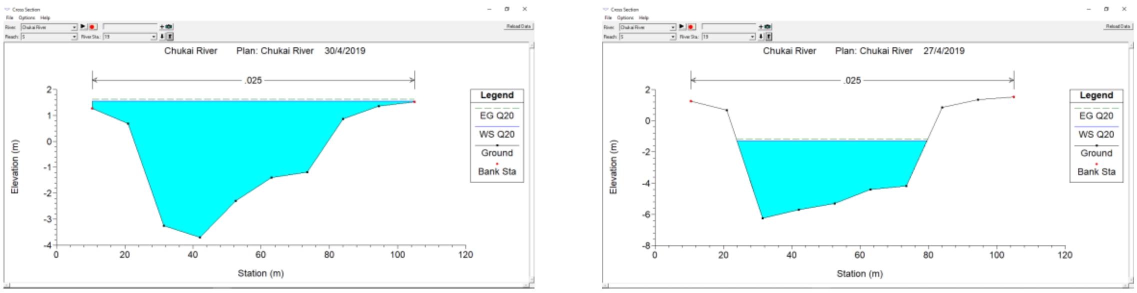
(a) Old bridge piers (b) New bridge piers

Figure 4.86 Water level at CH 13,550 of Chukai River –20-year ARI



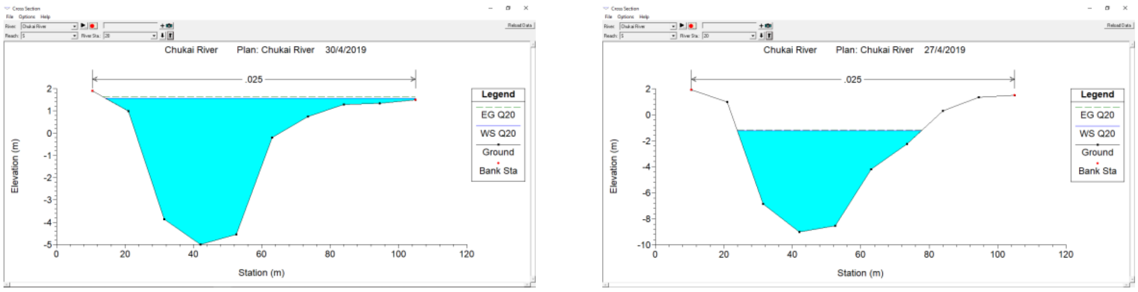
(a) Old bridge piers (b) New bridge piers

Figure 4.87 Water level at CH 13,750 of Chukai River –20-year ARI



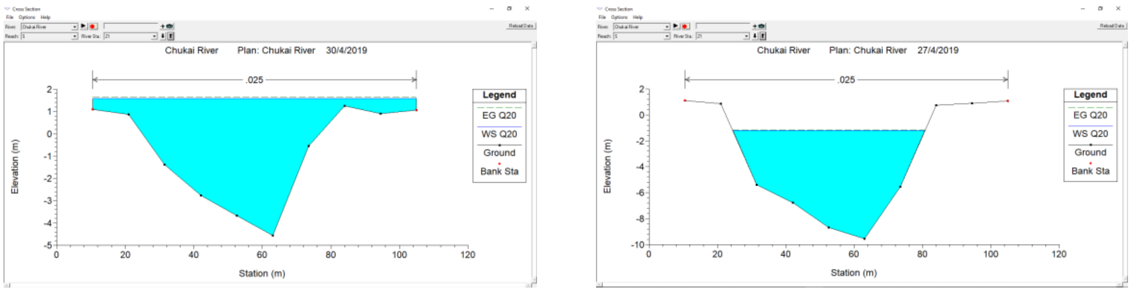
(a) Old bridge piers (b) New bridge piers

Figure 4.88 Water level at CH 13,950 of Chukai River –20-year ARI



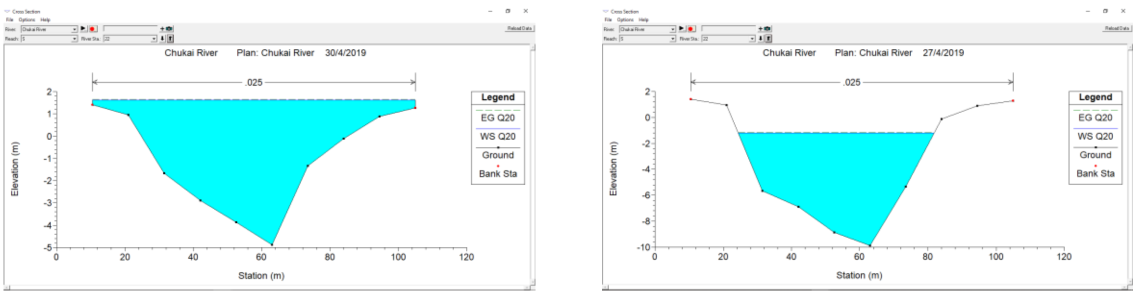
(a) Old bridge piers (b) New bridge piers

Figure 4.89 Water level at CH 14,150 of Chukai River –20-year ARI



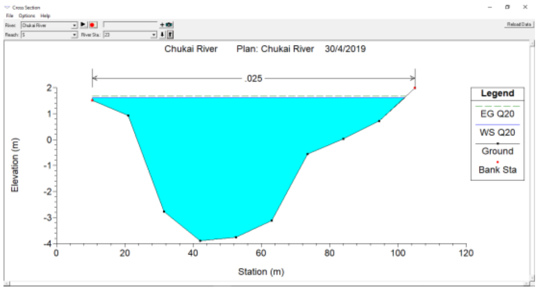
(a) Old bridge piers (b) New bridge piers

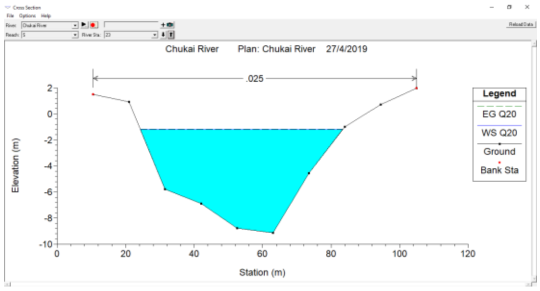
Figure 4.90 Water level at CH 14,350 of Chukai River –20-year ARI



(a) Old bridge piers (b) New bridge piers

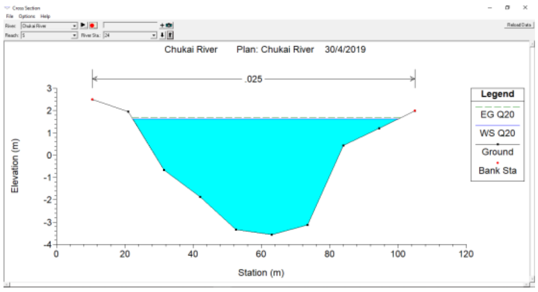
Figure 4.91 Water level at CH 14,550 of Chukai River –20-year ARI

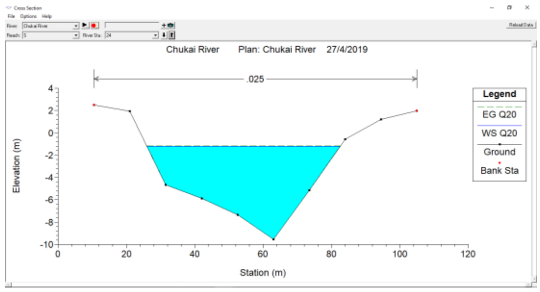




(a) Old bridge piers (b) New bridge piers

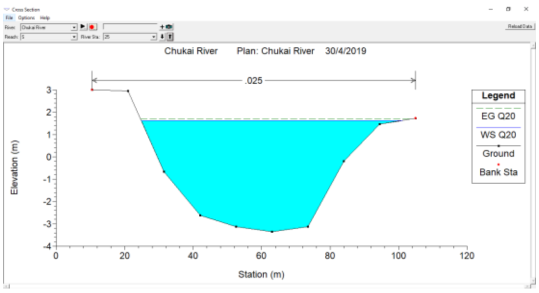
Figure 4.92 Water level at CH 14,750 of Chukai River –20-year ARI

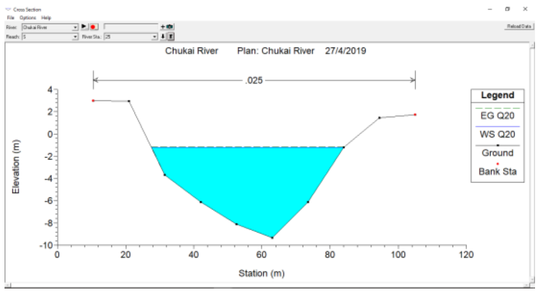




(a) Old bridge piers (b) New bridge piers

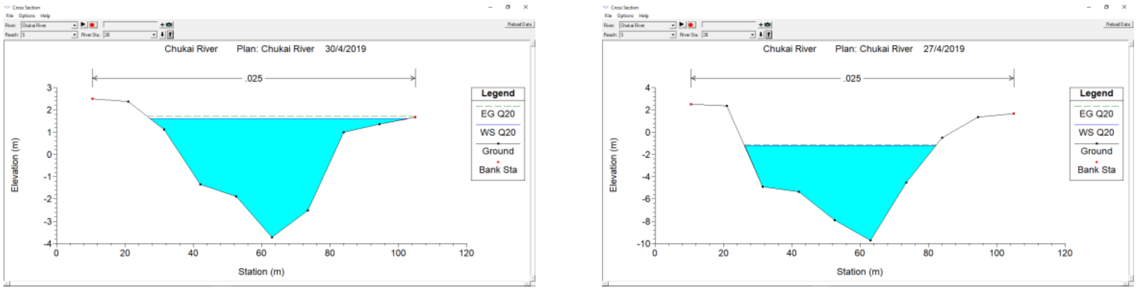
Figure 4.93 Water level at CH 14,950 of Chukai River –20-year ARI





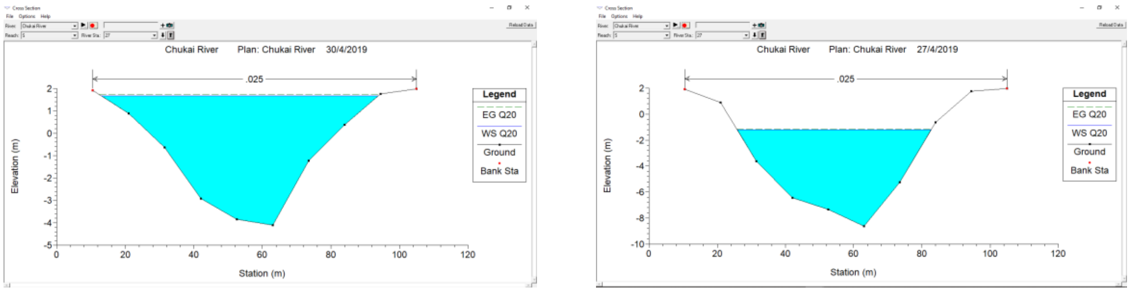
(a) Old bridge piers (b) New bridge piers

Figure 4.94 Water level at CH 15,150 of Chukai River –20-year ARI



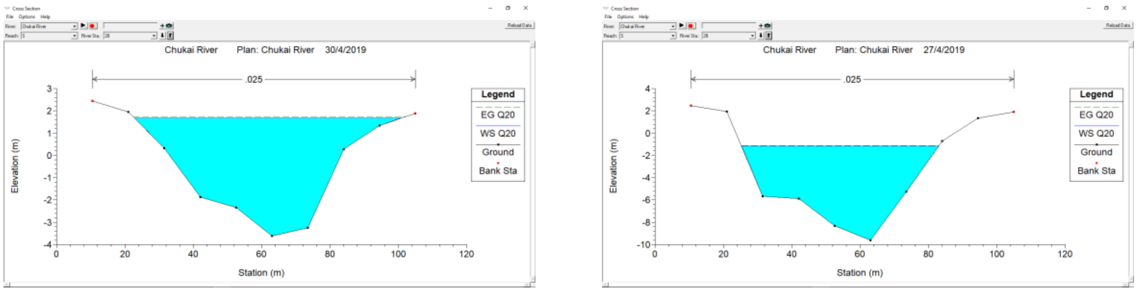
(a) Old bridge piers (b) New bridge piers

Figure 4.95 Water level at CH 15,350 of Chukai River –20-year ARI



(a) Old bridge piers (b) New bridge piers

Figure 4.96 Water level at CH 15,550 of Chukai River –20-year ARI



(a) Old bridge piers (b) New bridge piers

Figure 4.97 Water level at CH 15,750 of Chukai River –20-year ARI

4.8 WATER LEVEL FOR Q_{50} FOR OLD AND NEW BRIDGE PIERS

The difference in water levels of the Chukai River along the chainage with 50-year ARI is tabulated in Table 4.6. For old bridge, CH 10,750, CH 10,950, CH 11,150, CH 11,950 and CH 14,750 were overflown on the left bank. While, for CH 11,350, CH 11,550, CH 11,750, CH 12,150, CH 12,550, CH 13,050 (D), CH 13,050 (U), CH 13,150, CH 14,150, CH 15,150 and CH 15,350 were overflown on the right bank. For CH 12,350, CH 13,550, CH 13,750, CH 13,950 and CH 14,350 were recorded overflown on both banks. These chainages were located at the upstream, under the bridge and the downstream of the river. For the new bridge, there is no overflown on both banks.

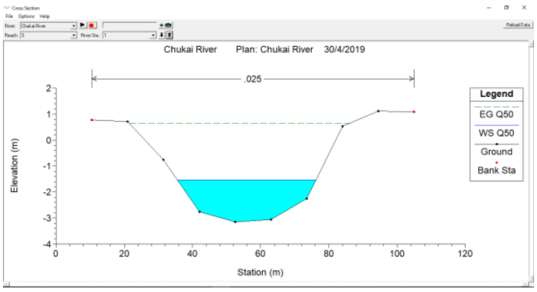
Table 4.6 Difference of water levels with Q_{50}

$Q_{50} = 312.77\ m^3/s$							
Chainage	Old Bridge Water levels (m)		New Bridge Water levels (m)		Difference of water levels (m)		Overflow
	Left bank	Right bank	Left bank	Right bank	Left bank	Right bank	
CH 10,350 (1)	-1.54	-1.54	-5.24	-5.24	-3.70	-3.70	
CH 10,550 (2)	0.54	0.54	-4.18	-4.18	-4.72	-4.72	
CH 10,750 (3)	1.28	1.28	-3.44	-3.44	-4.72	-4.72	(Left bank-Old)
CH 10,950 (4)	1.32	1.32	-3.10	-3.10	-4.42	-4.42	(Left bank-Old)
CH 11,150 (5)	1.31	1.31	-3.13	-3.13	-4.44	-4.44	(Left bank-Old)
CH 11,350 (6)	1.35	1.35	-3.36	-3.36	-4.71	-4.71	(Right bank-Old)
CH 11,550 (7)	1.42	1.42	-2.66	-2.66	-4.08	-4.08	(Right bank-Old)
CH 11,750 (8)	1.42	1.42	-2.67	-2.67	-4.09	-4.09	(Right bank-Old)
CH 11,950 (9)	1.46	1.46	-2.70	-2.70	-4.16	-4.16	(Left bank-Old)
CH 121,50 (10)	1.51	1.51	-2.66	-2.66	-4.17	-4.17	(Right bank-Old)
CH 12,350 (11)	1.49	1.49	-2.65	-2.65	-4.14	-4.14	(Both banks-Old)
CH 12,550 (12)	1.48	1.48	-2.61	-2.61	-4.09	-4.09	(Right bank-Old)
CH 12,750 (13)	1.48	1.48	-1.89	-1.89	-3.37	-3.37	
CH 12,950 (14)	1.79	1.79	-0.98	-0.98	-2.77	-2.77	
CH 13,050 (D) (14.5)	1.79	1.79	-0.98	-0.98	-2.77	-2.77	(Right bank-Old)
CH 13,050 (U) (14.5)	1.79	1.79	-0.98	-0.98	-2.77	-2.77	(Right bank-Old)
CH 13,150 (15)	1.81	1.81	-0.98	-0.98	-2.79	-2.79	(Right bank-Old)
CH 13,350 (16)	1.79	1.79	-1.02	-1.02	-2.81	-2.81	
CH 13,550 (17)	1.84	1.84	-0.97	-0.97	-2.81	-2.81	(Both banks-Old)
CH 13,750 (18)	1.84	1.84	-0.96	-0.96	-2.80	-2.80	(Both banks-Old)
CH 13,950 (19)	1.83	1.83	-0.99	-0.99	-2.82	-2.82	(Both banks-Old)
CH 14,150 (20)	1.85	1.85	-0.93	-0.93	-2.78	-2.78	(Right bank-Old)
CH 14,350 (21)	1.87	1.87	-0.90	-0.90	-2.77	-2.77	(Both banks-Old)
CH 14,550 (22)	1.89	1.89	-0.89	-0.89	-2.78	-2.78	
CH 14,750 (23)	1.90	1.90	-0.89	-0.89	-2.79	-2.79	(Left bank-Old)
CH 14,950 (24)	1.89	1.89	-0.90	-0.90	-2.79	-2.79	
CH 15,150 (25)	1.91	1.91	-0.89	-0.89	-2.80	-2.80	(Right bank-Old)
CH 15,350 (26)	1.89	1.89	-0.89	-0.89	-2.78	-2.78	(Right bank-Old)
CH 15,550 (27)	1.96	1.96	-0.88	-0.88	-2.84	-2.84	
CH 15,750 (28)	1.96	1.96	-0.86	-0.86	-2.82	-2.82	

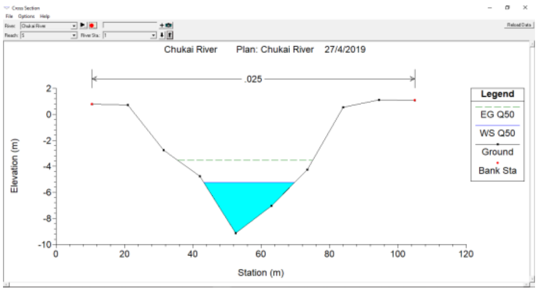
4.8.1 Water level for Q_{50} at upstream of the bridge

Based on Table 4.6 and Figure 4.98 to Figure 4.110 the water levels were in between -1.54 m to 1.48 m above sea level at the upstream of the bridge with the old bridge piers while with the new bridge piers, the water levels were between -5.24 m to -1.89 m above sea level at the upstream of the bridge. The water levels surpassed the river bank along CH 10,350 to CH 15,750 during rainfall event of 50-year ARI. In other words, the water was overflown onto the several chainages due to its incline bank surface as stated in Table 4.6.

Besides, the differences in water levels along the upstream chainage which start from CH 10,350 to CH 12,750 (Figure 4.98 to Figure 4.110) were between -3.70 m to -3.37 m. These indicated that with the presence of new bridge piers, flooding will not occur.

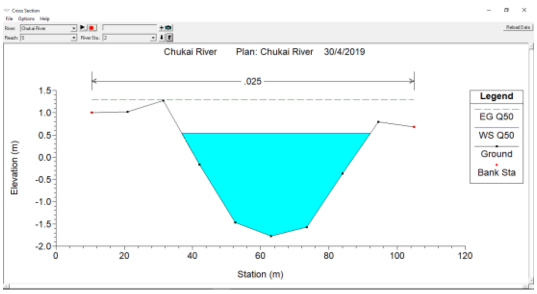


(a) Old bridge piers

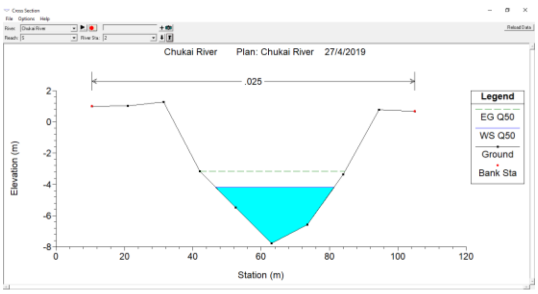


(b) New bridge piers

Figure 4.98 Water level at CH 10,350 of Chukai River – 50-year ARI

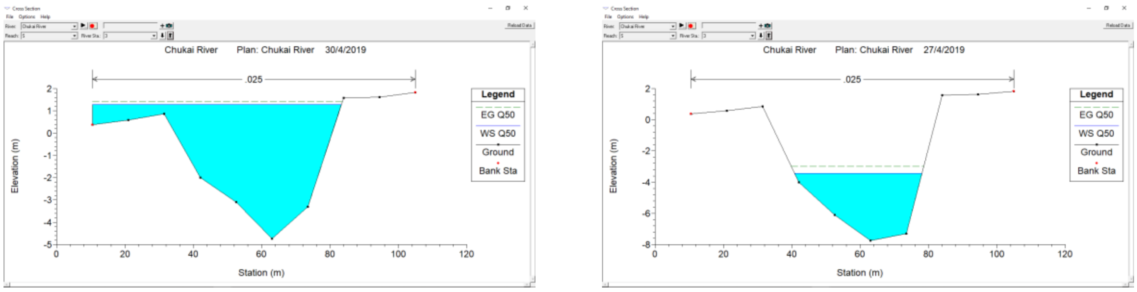


(a) Old bridge piers



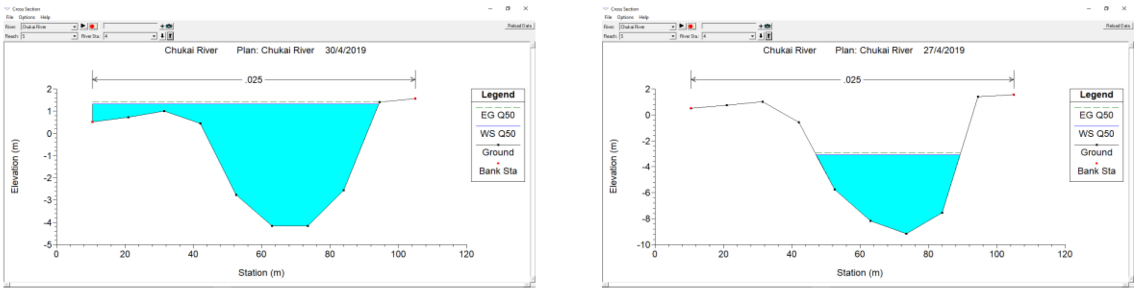
(b) New bridge piers

Figure 4.99 Water level at CH 10,550 of Chukai River –50-year ARI



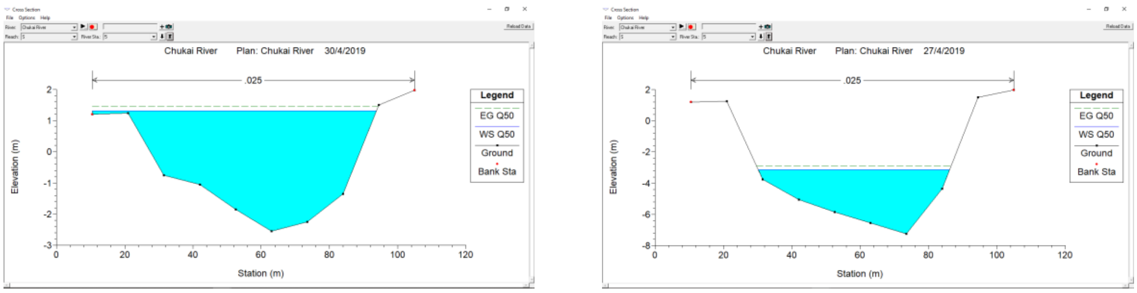
(a) Old bridge piers (b) New bridge piers

Figure 4.100 Water level at CH 10,750 of Chukai River –50-year ARI



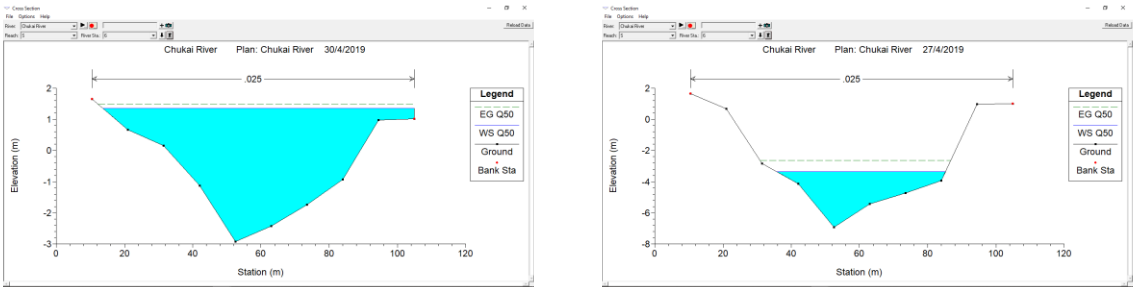
(a) Old bridge piers (b) New bridge piers

Figure 4.101 Water level at CH 10,950 of Chukai River –50-year ARI



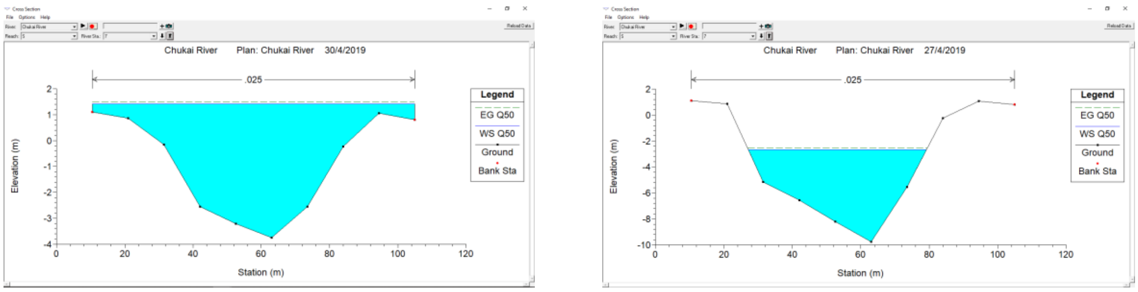
(a) Old bridge piers (b) New bridge piers

Figure 4.102 Water level at CH 11,150 of Chukai River –50-year ARI



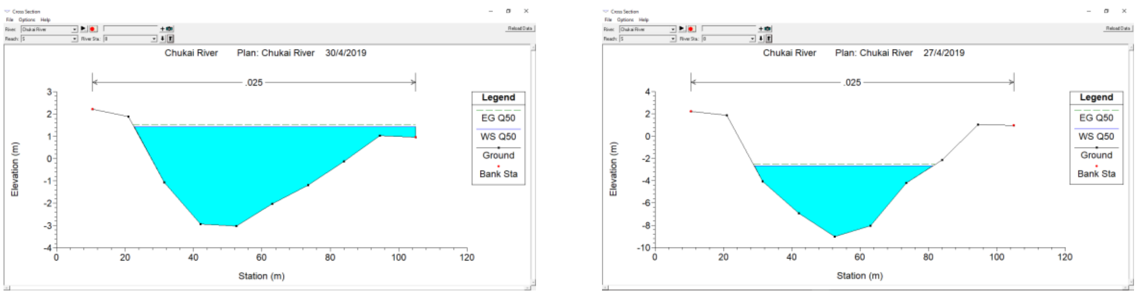
(a) Old bridge piers (b) New bridge piers

Figure 4.103 Water level at CH 11,350 of Chukai River –50-year ARI



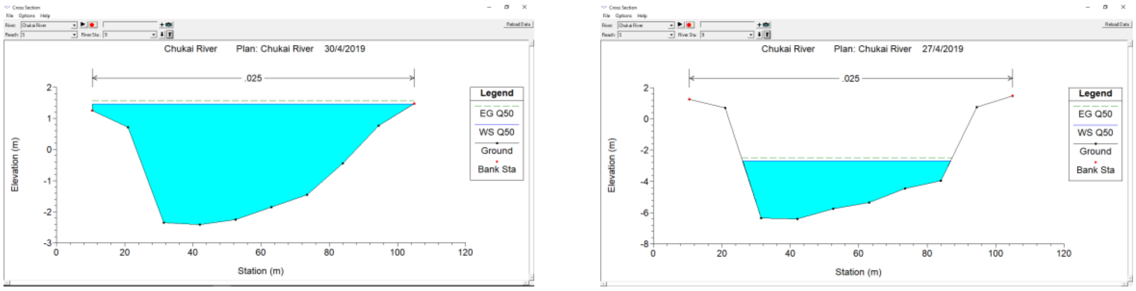
(a) Old bridge piers (b) New bridge piers

Figure 4.104 Water level at CH 11,550 of Chukai River –50-year ARI



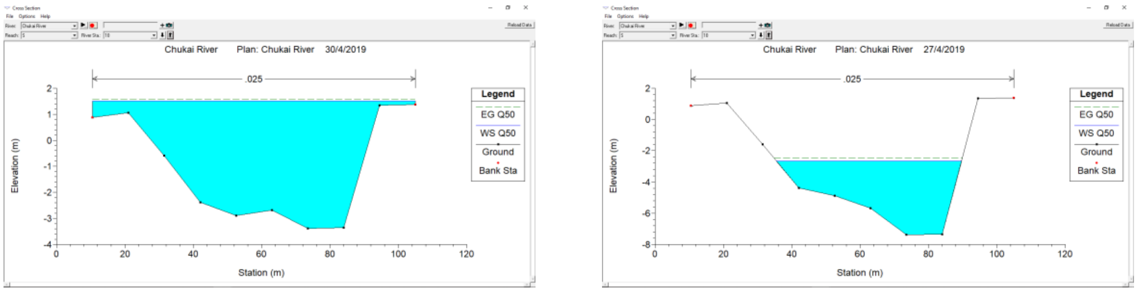
(a) Old bridge piers (b) New bridge piers

Figure 4.105 Water level at CH 11,750 of Chukai River –50-year ARI



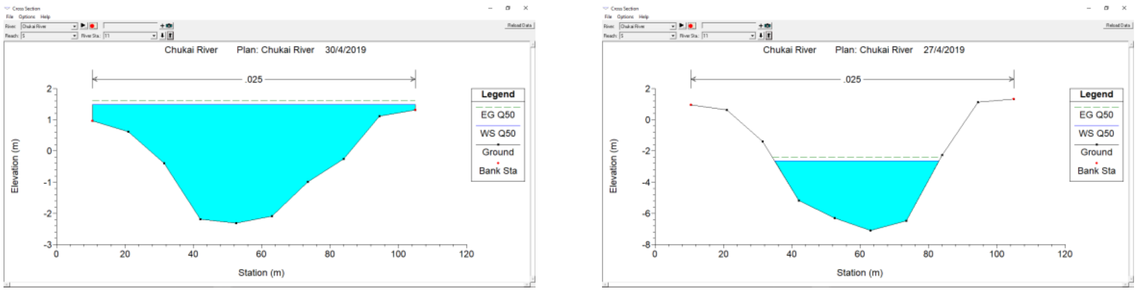
(a) Old bridge piers (b) New bridge piers

Figure 4.106 Water level at CH 11,950 of Chukai River –50-year ARI



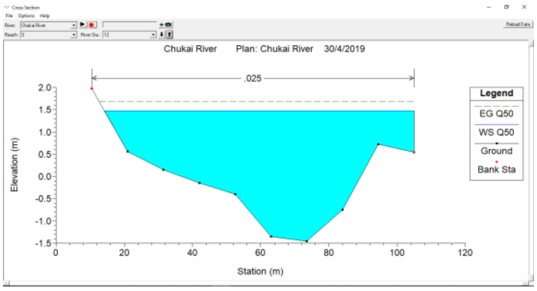
(a) Old bridge piers (b) New bridge piers

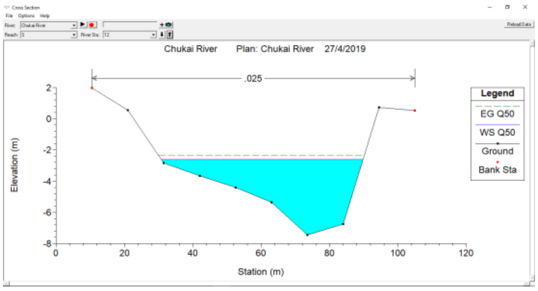
Figure 4.107 Water level at CH 12,150 of Chukai River –50-year ARI



(a) Old bridge piers (b) New bridge piers

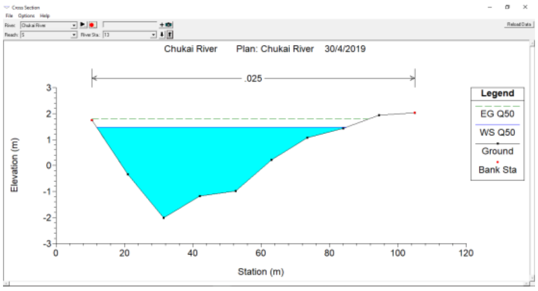
Figure 4.108 Water level at CH 12,350 of Chukai River –50-year ARI

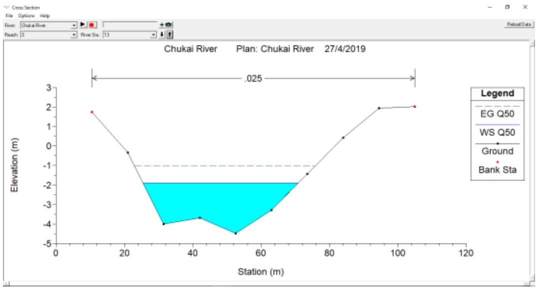




(a) Old bridge piers (b) New bridge piers

Figure 4.109 Water level at CH 12,550 of Chukai River –50-year ARI



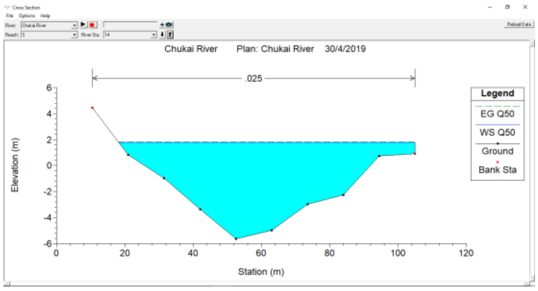


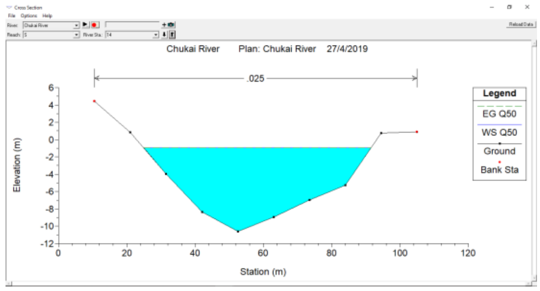
(a) Old bridge piers (b) New bridge piers

Figure 4.110 Water level at CH 12,750 of Chukai River –50-year ARI

4.8.2 Water level for Q_{50} under the bridge

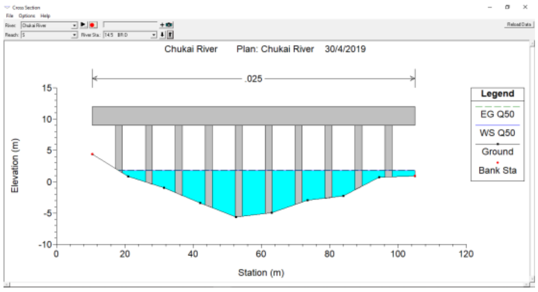
Table 4.6 and Figure 4.111 to Figure 4.114 presented the water profiles at the bridge areas. These simulations showed that the water level at this chainage with 50-year ARI is 1.79 m above sea level with the old bridge piers and -0.98 m with the new bridge piers. Apart from that, during rainfall event of 50-year ARI, the water level under the bridge cross section which starts from CH 12,950 to CH 13,150 had the difference of -2.77 m.

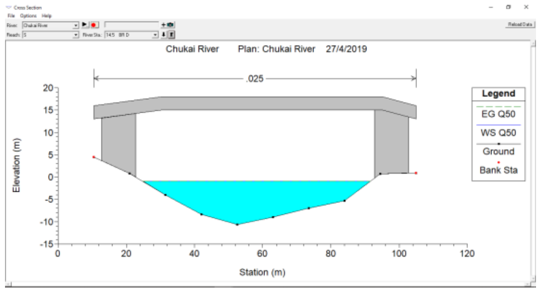




(a) Old bridge piers (b) New bridge piers

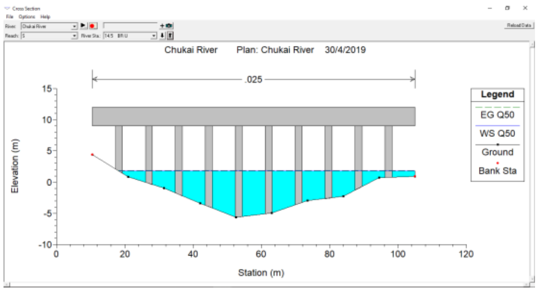
Figure 4.111 Water level at CH 12,950 of Chukai River –50-year ARI

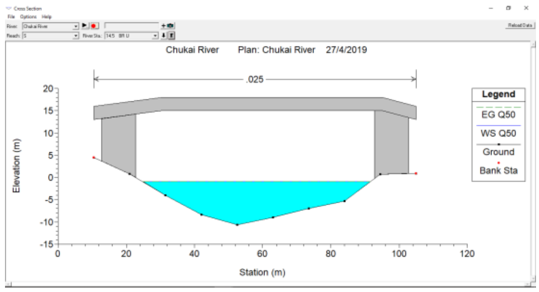




(a) Downstream old bridge piers (b) Downstream new bridge piers

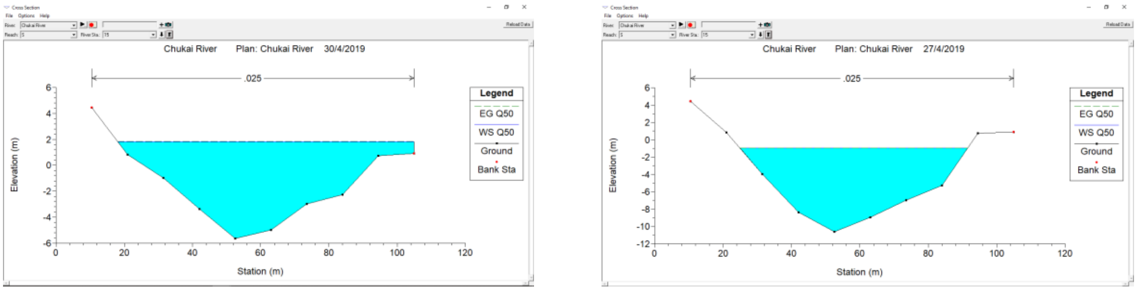
Figure 4.112 Water level at CH 13,050 of Chukai River –50-year ARI





(a) Upstream old bridge piers (b) Upstream new bridge piers

Figure 4.113 Water level at CH 13,050 of Chukai River –50-year ARI



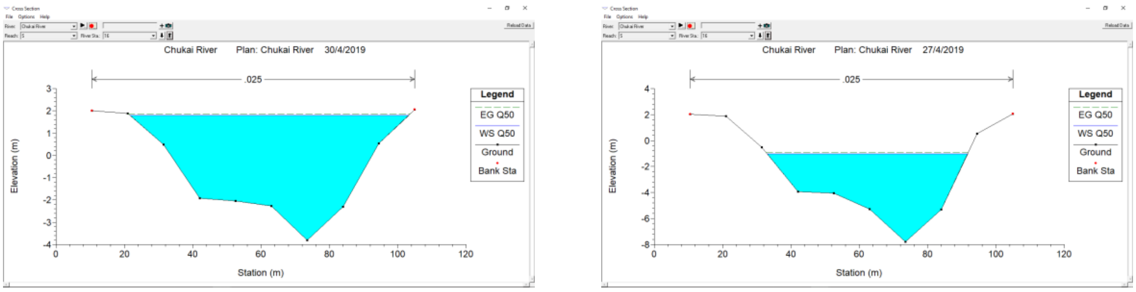
(a) Old bridge piers (b) New bridge piers

Figure 4.114 Water level at CH 13,150 of Chukai River –50-year ARI

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

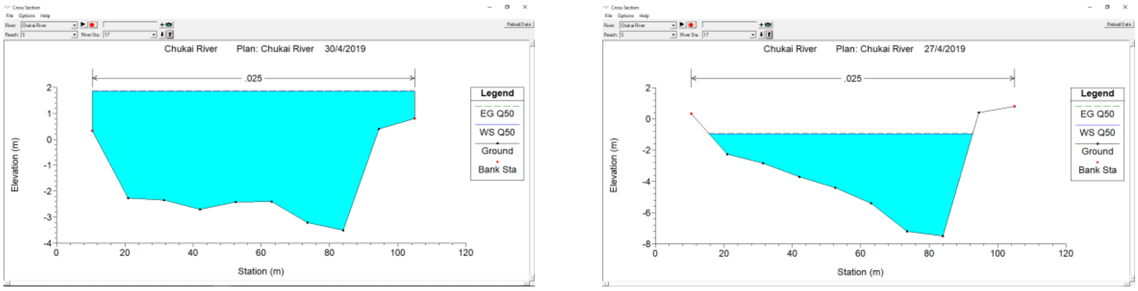
Based on Table 4.6 and Figure 4.115 to Figure 4.127, the water levels were in between 1.79 m to 1.96 m above sea level at the upstream of the bridge with the old bridge piers while with the new bridge piers, the water levels were between -1.02 m to -0.86 m above sea level at the downstream of the bridge.

This study showed that the increases in water levels from CH 13,350 to CH 15,750 had slightly fluctuated. The water levels were from -2.81 m to -2.82 m different along the chainage with 50-year ARI condition.



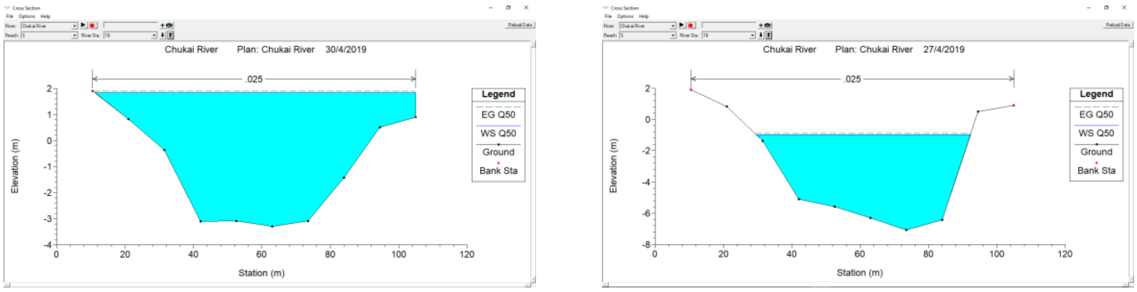
(a) Old bridge piers (b) New bridge piers

Figure 4.115 Water level at CH 13,350 of Chukai River –50-year ARI



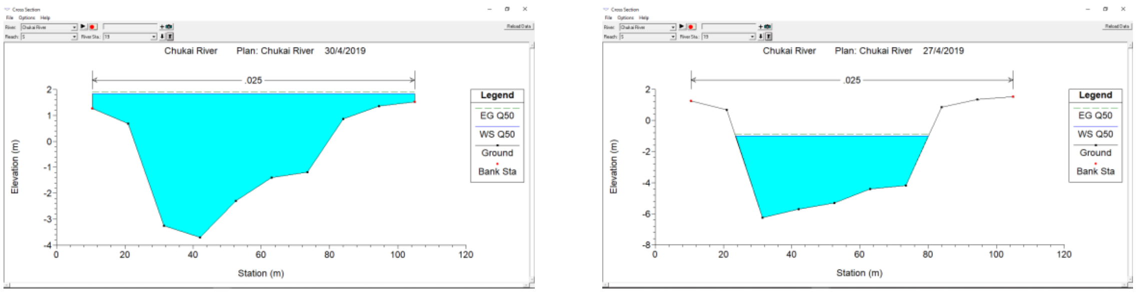
(a) Old bridge piers (b) New bridge piers

Figure 4.116 Water level at CH 13,550 of Chukai River –50-year ARI



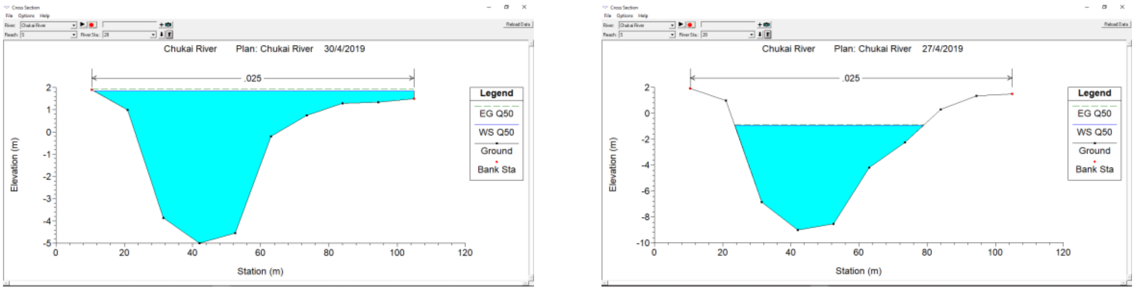
(a) Old bridge piers (b) New bridge piers

Figure 4.117 Water level at CH 13,750 of Chukai River –50-year ARI



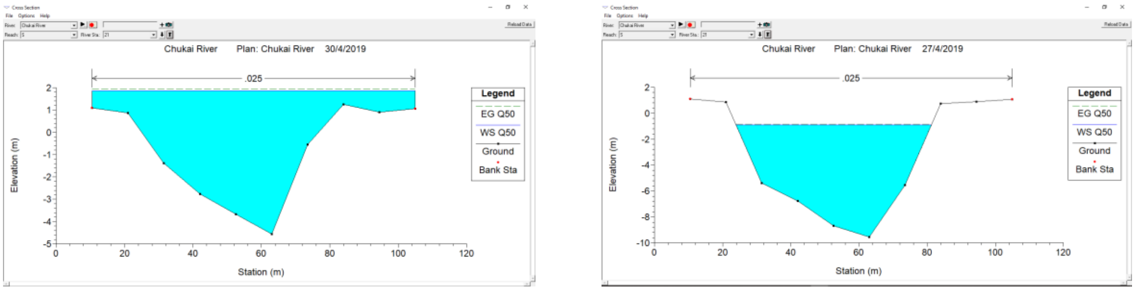
(a) Old bridge piers (b) New bridge piers

Figure 4.118 Water level at CH 13,950 of Chukai River –50-year ARI



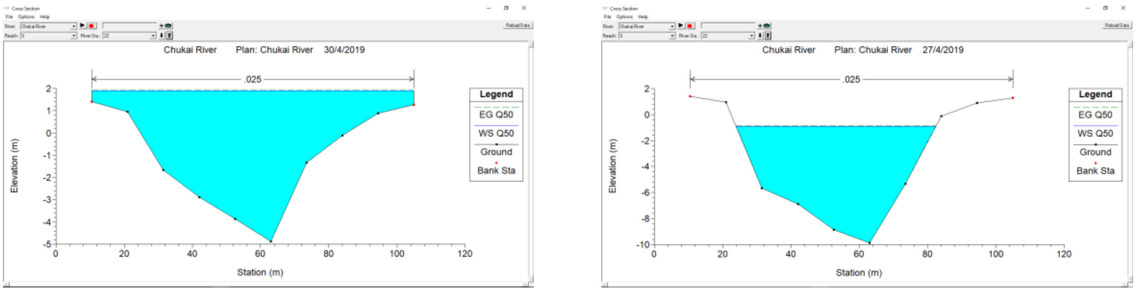
(a) Old bridge piers (b) New bridge piers

Figure 4.119 Water level at CH 14,150 of Chukai River –50-year ARI



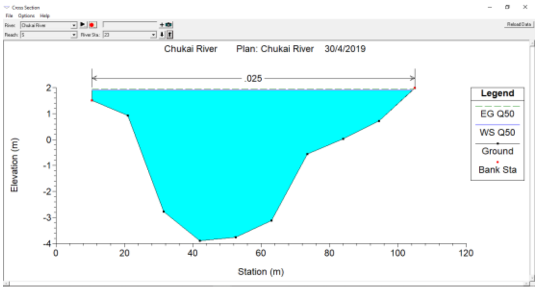
(a) Old bridge piers (b) New bridge piers

Figure 4.120 Water level at CH 14,350 of Chukai River –50-year ARI

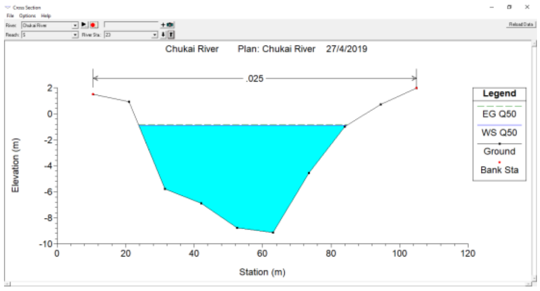


(a) Old bridge piers (b) New bridge piers

Figure 4.121 Water level at CH 14,550 of Chukai River –50-year ARI

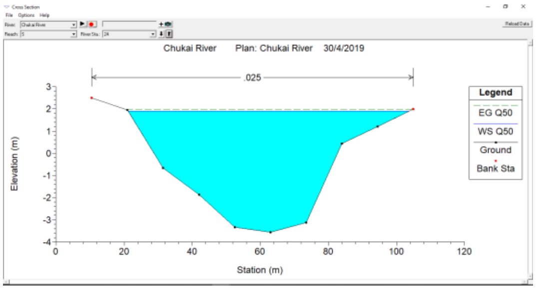


(a) Old bridge piers

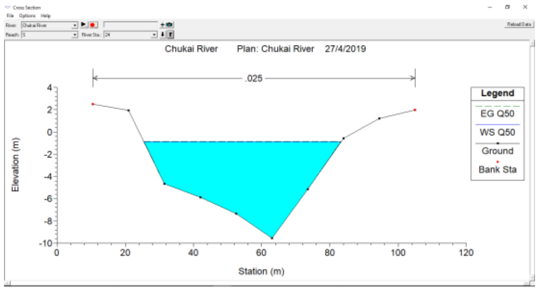


(b) New bridge piers

Figure 4.122 Water level at CH 14,750 of Chukai River –50-year ARI

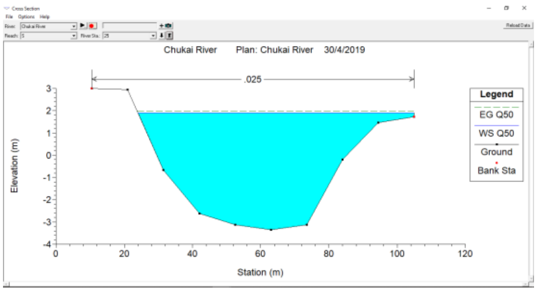


(a) Old bridge piers

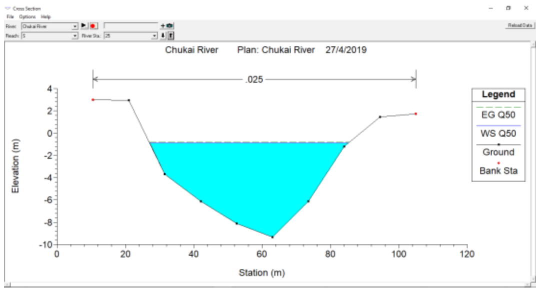


(b) New bridge piers

Figure 4.123 Water level at CH 14,950 of Chukai River –50-year ARI

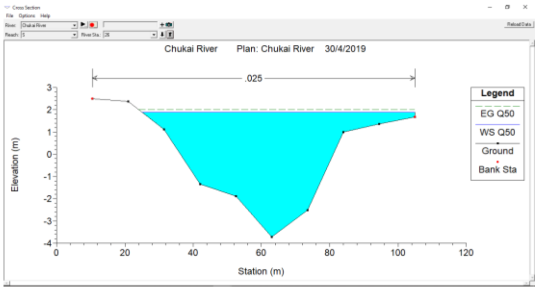


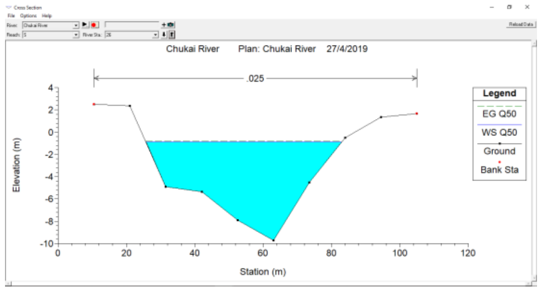
(a) Old bridge piers



(b) New bridge piers

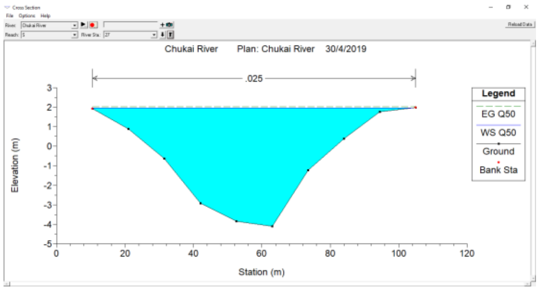
Figure 4.124 Water level at CH 15,150 of Chukai River –50-year ARI

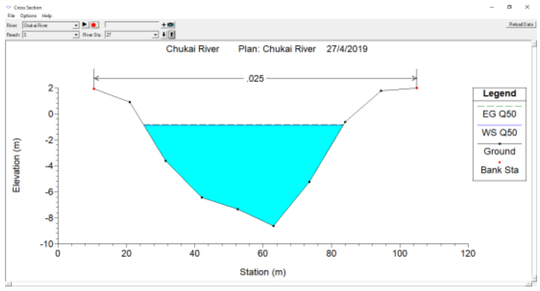




(a) Old bridge piers (b) New bridge piers

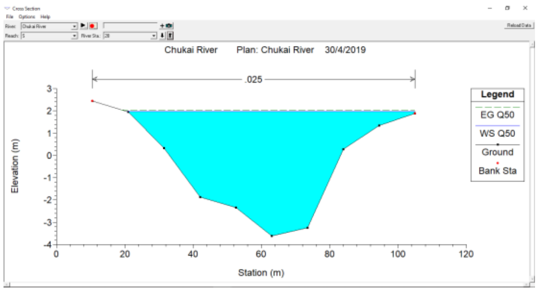
Figure 4.125 Water level at CH 15,350 of Chukai River –50-year ARI

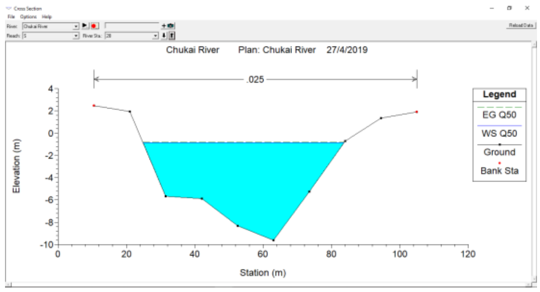




(a) Old bridge piers (b) New bridge piers

Figure 4.126 Water level at CH 15,550 of Chukai River –50-year ARI





(a) Old bridge piers (b) New bridge piers

Figure 4.127 Water level at CH 15,750 of Chukai River –50-year ARI

4.9 WATER LEVEL FOR Q_{100} FOR OLD AND NEW BRIDGE PIERS

The difference in water levels of the Chukai River along the chainage with 100-year ARI is tabulated in Table 4.7. For old bridge, all chainages were overflown except CH 10,350 and CH 10,750 as stated in Table 4.7. For the new bridge, there is no overflown on both banks.

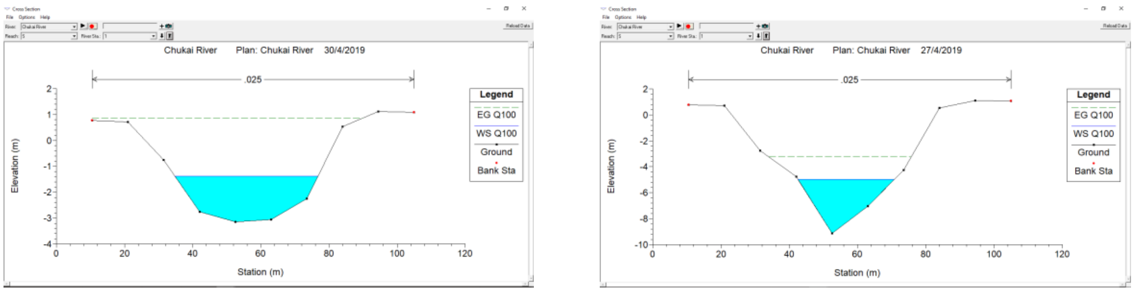
Table 4.7 Difference of water levels with Q_{100}

Q_{100} = 359.50 m³/s							
Chainage	Old Bridge Water levels (m)		New Bridge Water levels (m)		Difference of water levels (m)		Overflow
	Left bank	Right bank	Left bank	Right bank	Left bank	Right bank	
CH 10,350 (1)	-1.38	-1.38	-4.96	-4.96	-3.58	-3.58	
CH 10,550 (2)	0.79	0.79	-3.95	-3.95	-4.74	-4.74	(Right bank-Old)
CH 10,750 (3)	1.47	1.47	-3.20	-3.20	-4.67	-4.67	(Left bank-Old)
CH 10,950 (4)	1.51	1.51	-2.83	-2.83	-4.34	-4.34	(Left bank-Old)
CH 11,150 (5)	1.50	1.50	-2.85	-2.85	-4.35	-4.35	(Left bank-Old)
CH 11,350 (6)	1.55	1.55	-3.05	-3.05	-4.60	-4.60	(Right bank-Old)
CH 11,550 (7)	1.62	1.62	-2.42	-2.42	-4.04	-4.04	(Both banks-Old)
CH 11,750 (8)	1.62	1.62	-2.43	-2.43	-4.05	-4.05	(Right bank-Old)
CH 11,950 (9)	1.67	1.67	-2.46	-2.46	-4.13	-4.13	(Both banks-Old)
CH 121,50 (10)	1.72	1.72	-2.42	-2.42	-4.14	-4.14	(Both banks-Old)
CH 12,350 (11)	1.69	1.69	-2.41	-2.41	-4.10	-4.10	(Both banks-Old)
CH 12,550 (12)	1.68	1.68	-2.37	-2.37	-4.05	-4.05	(Right bank-Old)
CH 12,750 (13)	1.67	1.67	-1.71	-1.71	-3.38	-3.38	
CH 12,950 (14)	2.00	2.00	-0.73	-0.73	-2.73	-2.73	(Right bank-Old)
CH 13,050 (D) (14.5)	2.00	2.00	-0.73	-0.73	-2.73	-2.73	(Right bank-Old)
CH 13,050 (U) (14.5)	2.00	2.00	-0.73	-0.73	-2.73	-2.73	(Right bank-Old)
CH 13,150 (15)	2.02	2.02	-0.73	-0.73	-2.75	-2.75	(Right bank-Old)
CH 13,350 (16)	2.00	2.00	-0.79	-0.79	-2.79	-2.79	(Both banks-Old)
CH 13,550 (17)	2.06	2.06	-0.73	-0.73	-2.79	-2.79	(Both banks-Old)
CH 13,750 (18)	2.05	2.05	-0.72	-0.72	-2.77	-2.77	(Both banks-Old)
CH 13,950 (19)	2.05	2.05	-0.75	-0.75	-2.80	-2.80	(Both banks-Old)
CH 14,150 (20)	2.06	2.06	-0.68	-0.68	-2.74	-2.74	(Both banks-Old)
CH 14,350 (21)	2.08	2.08	-0.65	-0.65	-2.73	-2.73	(Both banks-Old)
CH 14,550 (22)	2.11	2.11	-0.64	-0.64	-2.75	-2.75	(Both banks-Old)
CH 14,750 (23)	2.11	2.11	-0.64	-0.64	-2.75	-2.75	(Both banks-Old)
CH 14,950 (24)	2.11	2.11	-0.64	-0.64	-2.75	-2.75	(Right bank-Old)
CH 15,150 (25)	2.13	2.13	-0.63	-0.63	-2.76	-2.76	(Right bank-Old)
CH 15,350 (26)	2.10	2.10	-0.63	-0.63	-2.73	-2.73	(Right bank-Old)
CH 15,550 (27)	2.18	2.18	-0.63	-0.63	-2.81	-2.81	(Both banks-Old)
CH 15,750 (28)	2.18	2.18	-0.61	-0.61	-2.79	-2.79	(Right bank-Old)

4.9.1 Water level for Q_{100} at upstream of the bridge

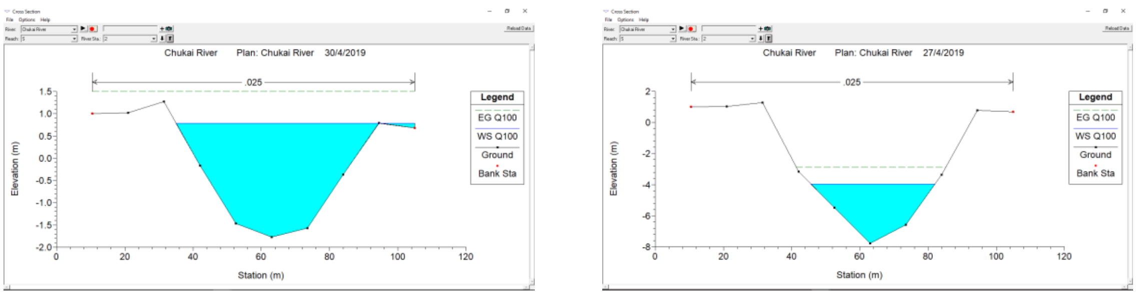
Based on Table 4.7 and Figure 4.128 to Figure 4.140 the water levels were in between -1.38 m to 1.67 m above sea level at the upstream of the bridge with the old bridge piers while with the new bridge piers, the water levels were between -4.96 m to -1.71 m above sea level at the upstream of the bridge. The water levels surpassed the river bank along CH 10,350 to CH 15,750 during rainfall event of 100-year ARI. In other words, the water was overflown except at CH 10,350 and CH 12,750 due to its incline bank surface.

Besides, the differences in water levels along the upstream chainage which start from CH 10,350 to CH 12,750 (Figure 4.128 to Figure 4.140) were between -3.58 m to -3.38 m. These indicated that with the presence of new bridge piers, flooding will not occur.



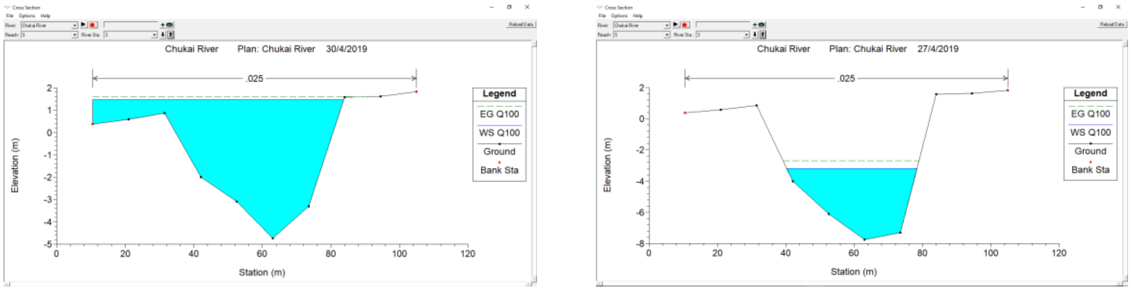
(a) Old bridge piers (b) New bridge piers

Figure 4.128 Water level at CH 10,350 of Chukai River – 100-year ARI



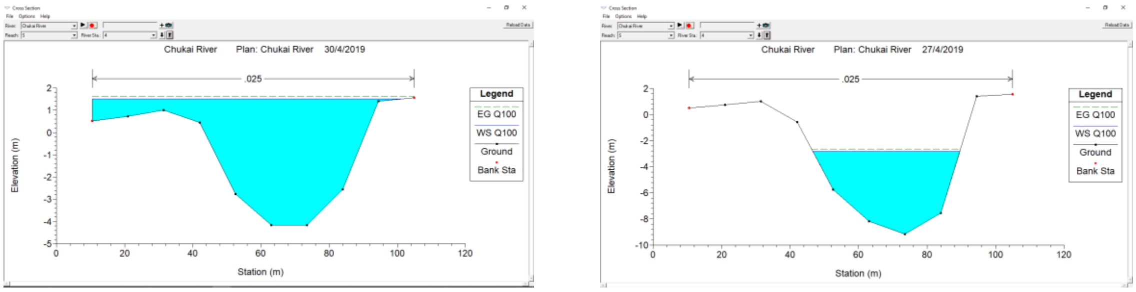
(a) Old bridge piers (b) New bridge piers

Figure 4.129 Water level at CH 10,550 of Chukai River –100-year ARI



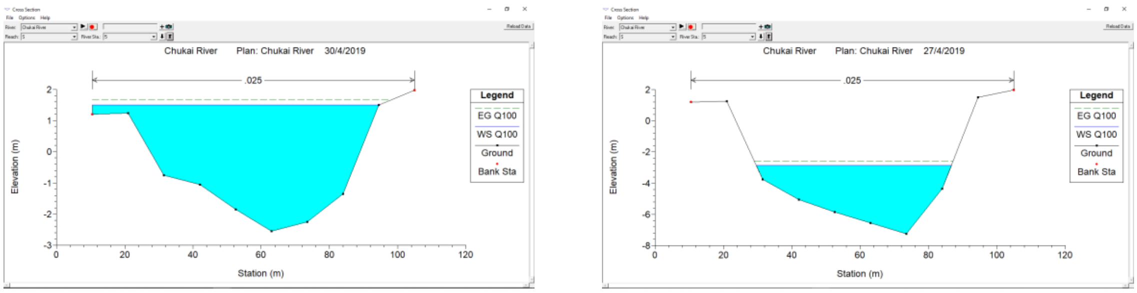
(a) Old bridge piers (b) New bridge piers

Figure 4.130 Water level at CH 10,750 of Chukai River –100-year ARI



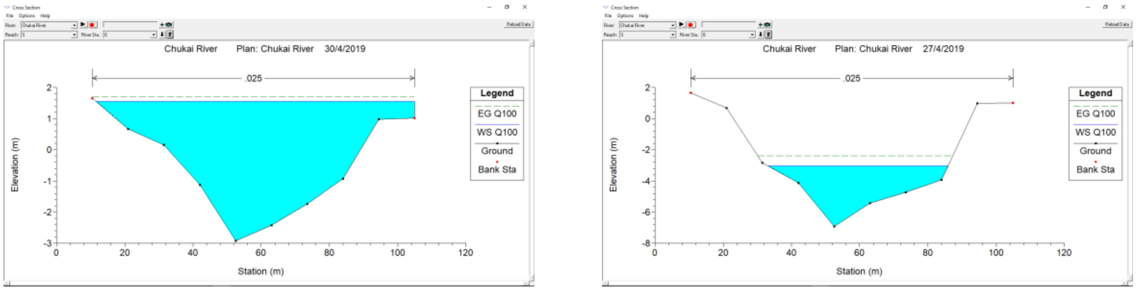
(a) Old bridge piers (b) New bridge piers

Figure 4.131 Water level at CH 10,950 of Chukai River –100-year ARI



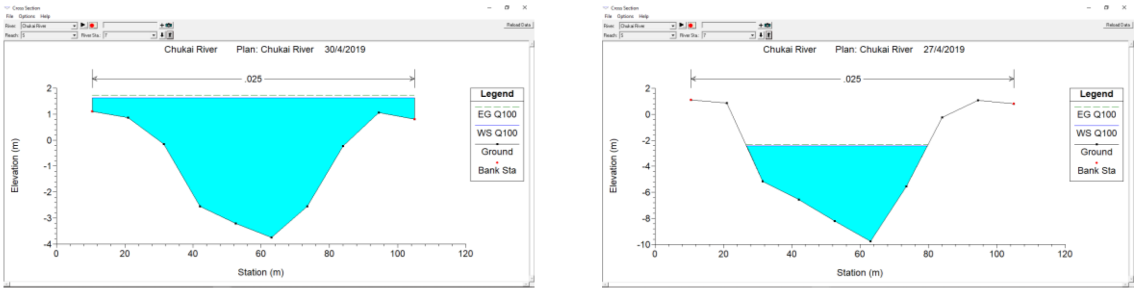
(a) Old bridge piers (b) New bridge piers

Figure 4.132 Water level at CH 11,150 of Chukai River –100-year ARI



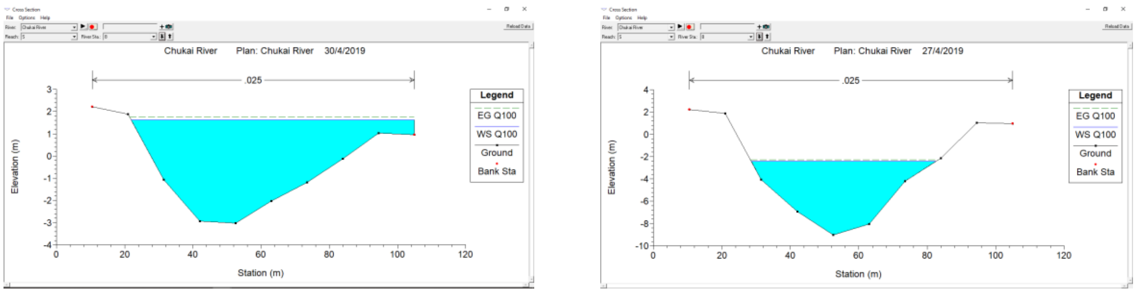
(a) Old bridge piers (b) New bridge piers

Figure 4.133 Water level at CH 11,350 of Chukai River –100-year ARI



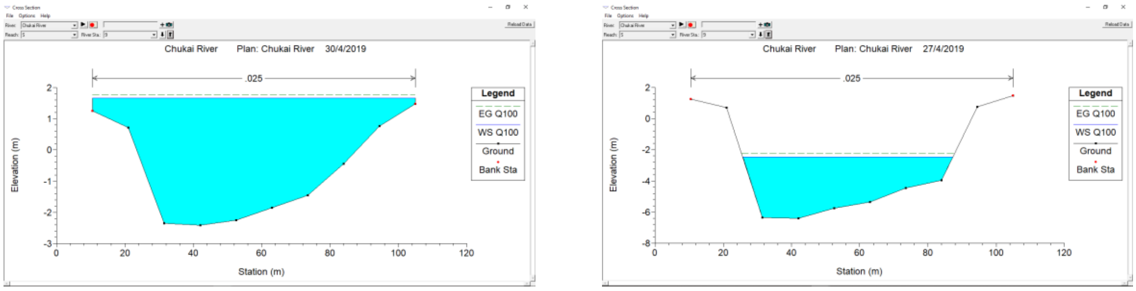
(a) Old bridge piers (b) New bridge piers

Figure 4.134 Water level at CH 11,550 of Chukai River –100-year ARI



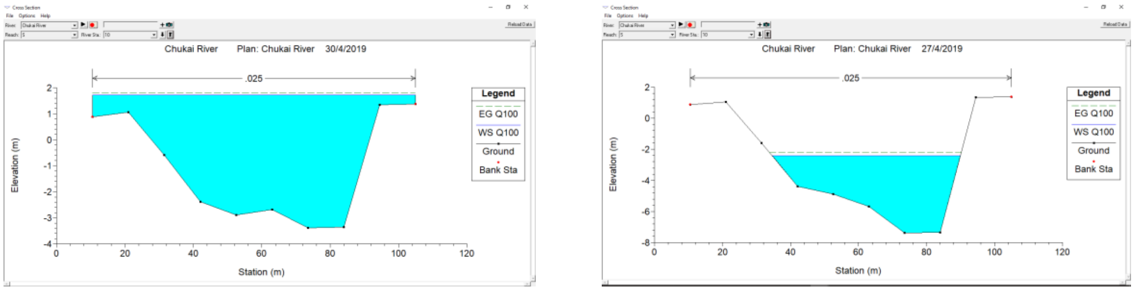
(a) Old bridge piers (b) New bridge piers

Figure 4.135 Water level at CH 11,750 of Chukai River –100-year ARI



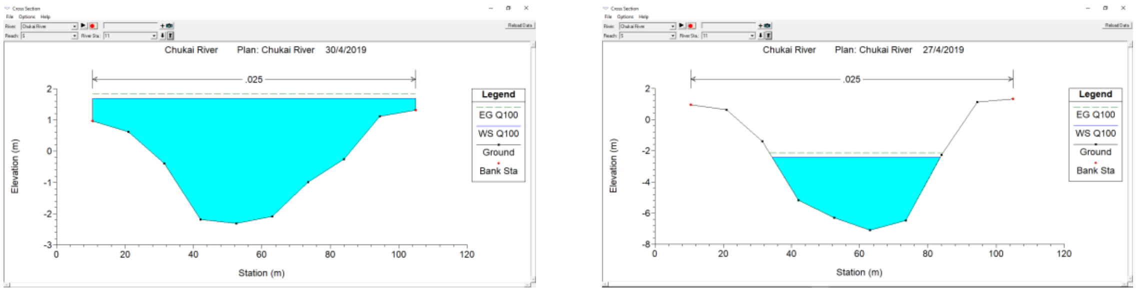
(a) Old bridge piers (b) New bridge piers

Figure 4.136 Water level at CH 11,950 of Chukai River –100-year ARI



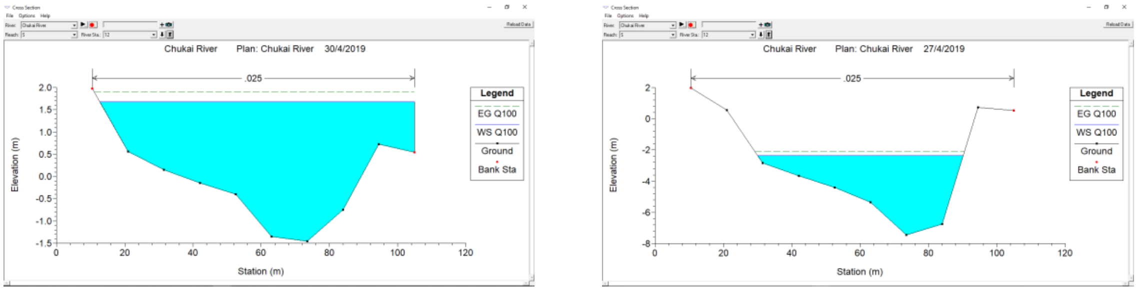
(a) Old bridge piers (b) New bridge piers

Figure 4.137 Water level at CH 12,150 of Chukai River –100-year ARI



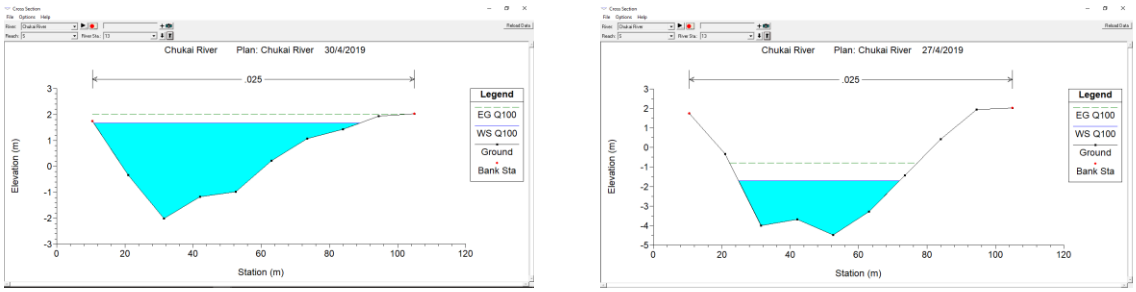
(a) Old bridge piers (b) New bridge piers

Figure 4.138 Water level at CH 12,350 of Chukai River –100-year ARI



(a) Old bridge piers (b) New bridge piers

Figure 4.139 Water level at CH 12,550 of Chukai River –100-year ARI

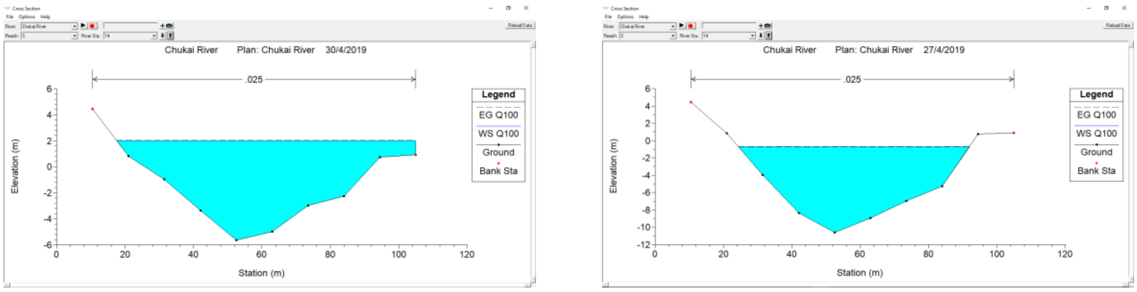


(a) Old bridge piers (b) New bridge piers

Figure 4.140 Water level at CH 12,750 of Chukai River –100-year ARI

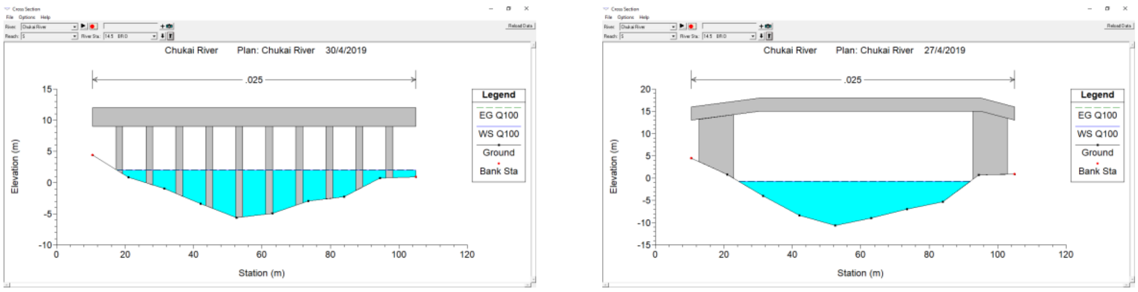
4.9.2 Water level for Q_{100} under the bridge

Table 4.7 and Figure 4.141 to Figure 4.144 presented the water profiles at the bridge areas. These simulations showed that the water level at this chainage with 100-year ARI is 2.00 m above sea level with the old bridge piers and -0.73 m with the new bridge piers. Apart from that, during rainfall event of 100-year ARI, the water level under the bridge cross section which starts from CH 12,950 to CH 13,150 had the difference of -2.73 m.



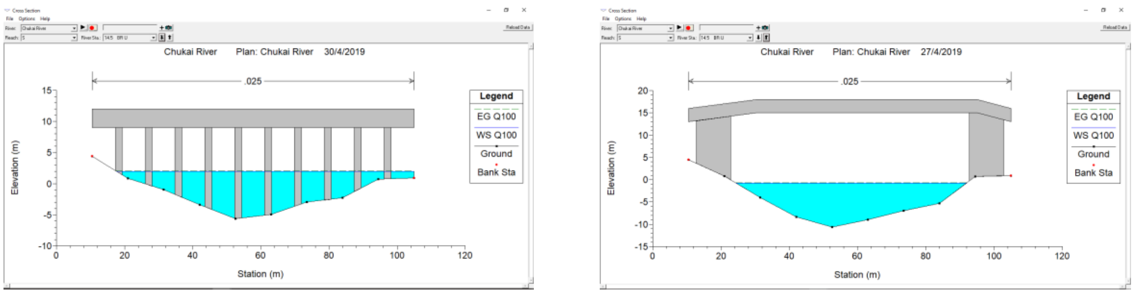
(a) Old bridge piers (b) New bridge piers

Figure 4.141 Water level at CH 12,950 of Chukai River –100-year ARI



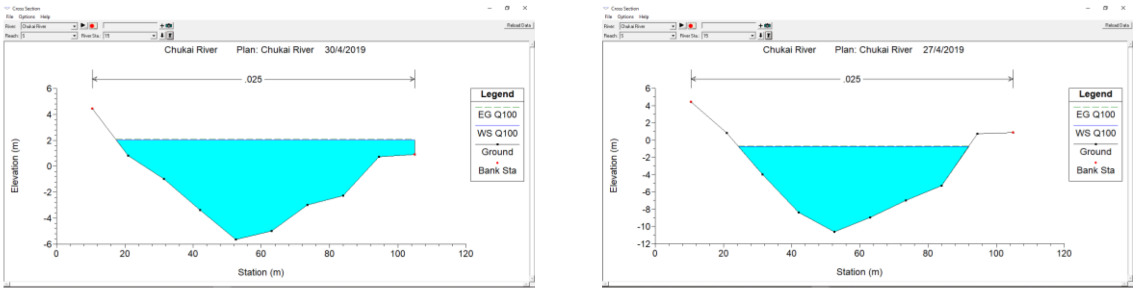
(a) Downstream old bridge piers (b) Downstream new bridge piers

Figure 4.142 Water level at CH 13,050 of Chukai River –100-year ARI



(a) Upstream old bridge piers (b) Upstream new bridge piers

Figure 4.143 Water level at CH 13,050 of Chukai River –100-year ARI



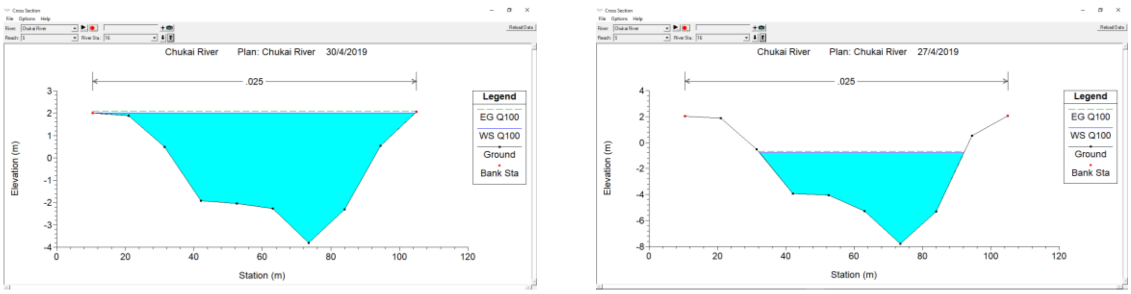
(a) Old bridge piers (b) New bridge piers

Figure 4.144 Water level at CH 13,150 of Chukai River –100-year ARI

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

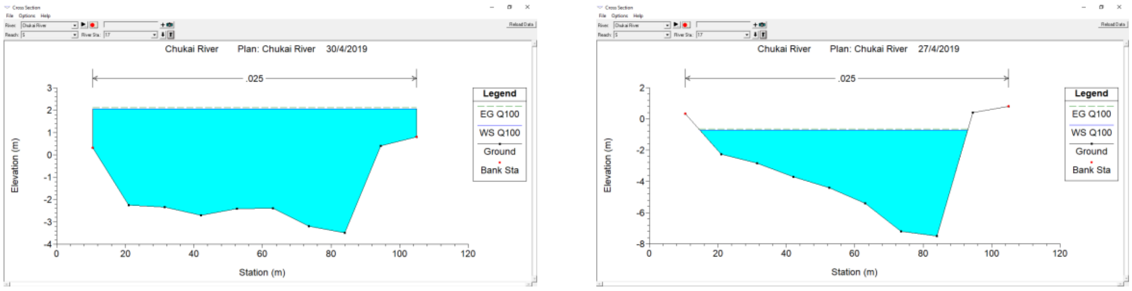
Based on Table 4.7 and Figure 4.145 to Figure 4.157, the water levels were in between 2.00 m to 2.18 m above sea level at the upstream of the bridge with the old bridge piers while with the new bridge piers, the water levels were between -0.79 m to -0.61 m above sea level at the downstream of the bridge.

This study showed that the increases in water levels from CH 13,350 to CH 15,750 had slightly fluctuated. The water levels were from -2.79 m to -2.81 m different along the chainage with 100-year ARI condition.



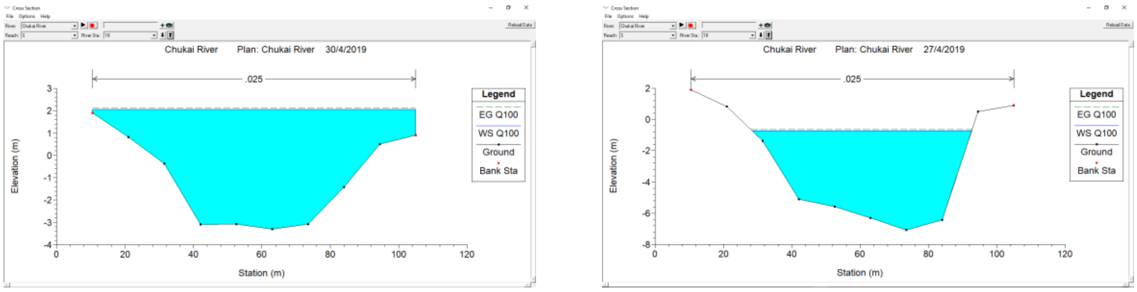
(a) Old bridge piers (b) New bridge piers

Figure 4.145 Water level at CH 13,350 of Chukai River –100-year ARI



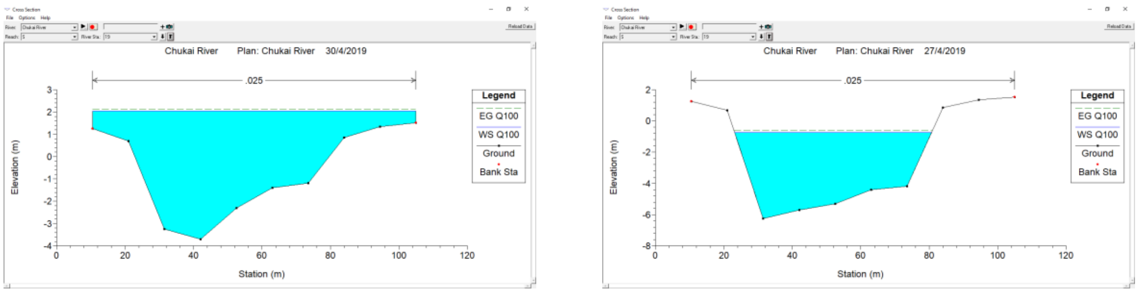
(a) Old bridge piers (b) New bridge piers

Figure 4.146 Water level at CH 13,550 of Chukai River –100-year ARI



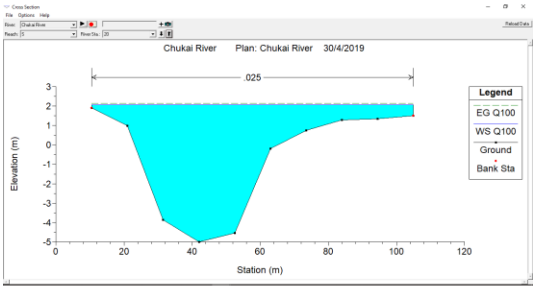
(a) Old bridge piers (b) New bridge piers

Figure 4.147 Water level at CH 13,750 of Chukai River –100-year ARI

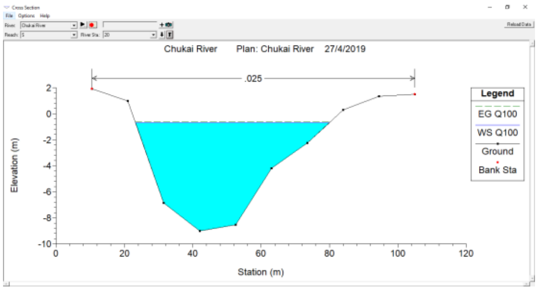


(a) Old bridge piers (b) New bridge piers

Figure 4.148 Water level at CH 13,950 of Chukai River –100-year ARI

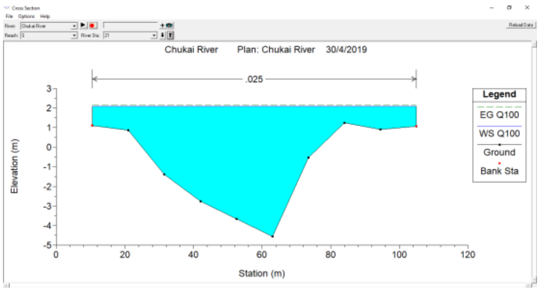


(a) Old bridge piers

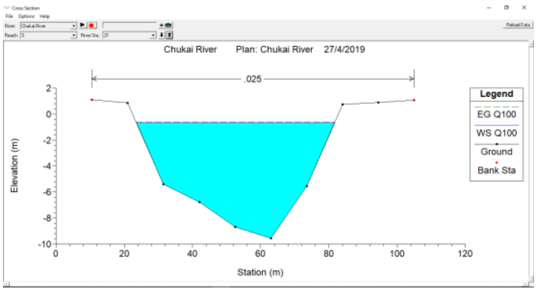


(b) New bridge piers

Figure 4.149 Water level at CH 14,150 of Chukai River –100-year ARI

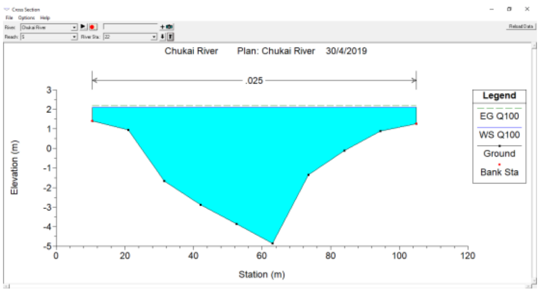


(a) Old bridge piers

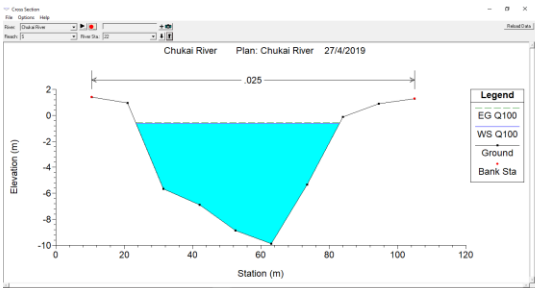


(b) New bridge piers

Figure 4.150 Water level at CH 14,350 of Chukai River –100-year ARI

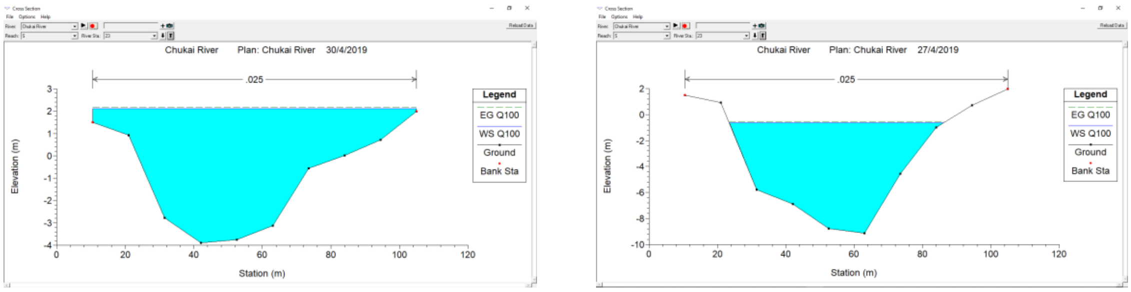


(a) Old bridge piers



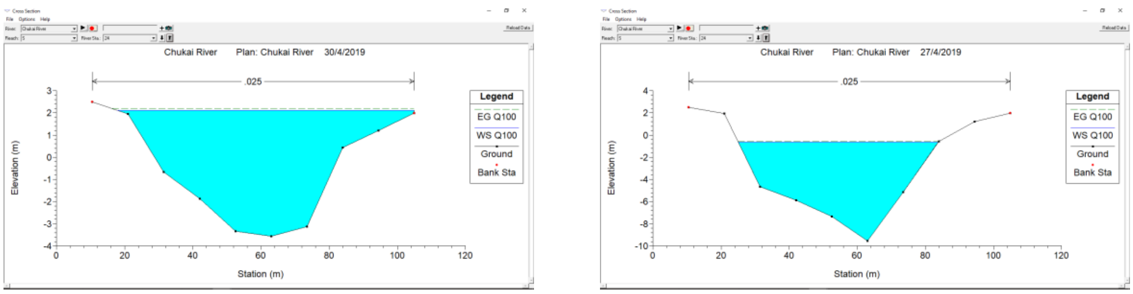
(b) New bridge piers

Figure 4.151 Water level at CH 14,550 of Chukai River –100-year ARI



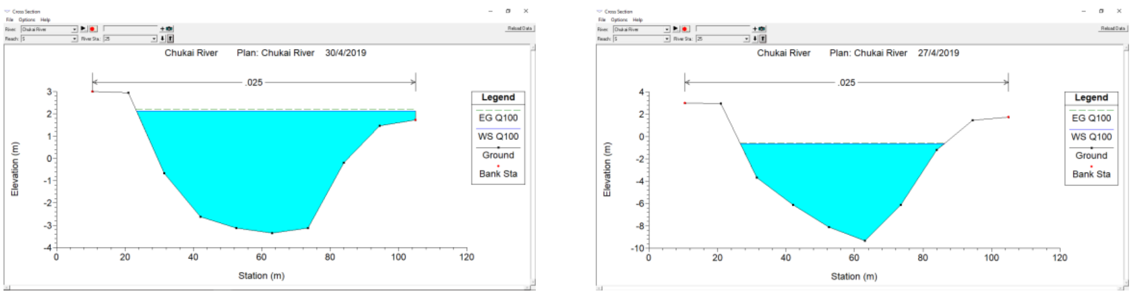
(a) Old bridge piers (b) New bridge piers

Figure 4.152 Water level at CH 14,750 of Chukai River –100-year ARI



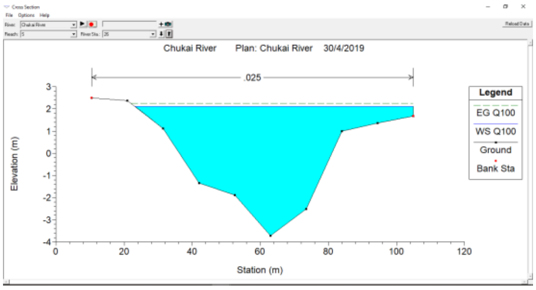
(a) Old bridge piers (b) New bridge piers

Figure 4.153 Water level at CH 14,950 of Chukai River –100-year ARI

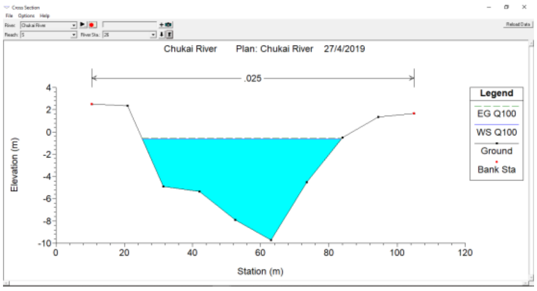


(a) Old bridge piers (b) New bridge piers

Figure 4.154 Water level at CH 15,150 of Chukai River –100-year ARI

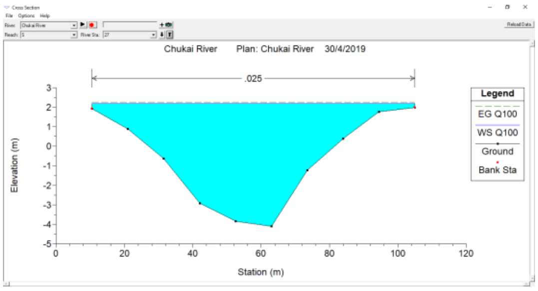


(a) Old bridge piers

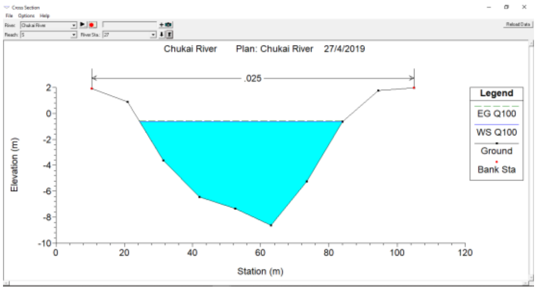


(b) New bridge piers

Figure 4.155 Water level at CH 15,350 of Chukai River –100-year ARI

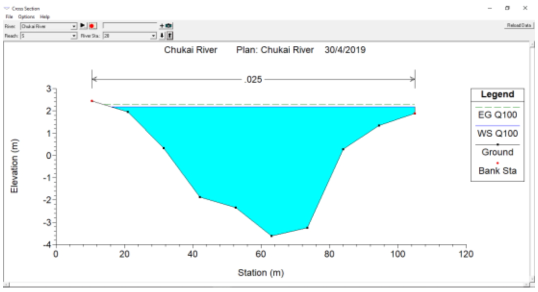


(a) Old bridge piers

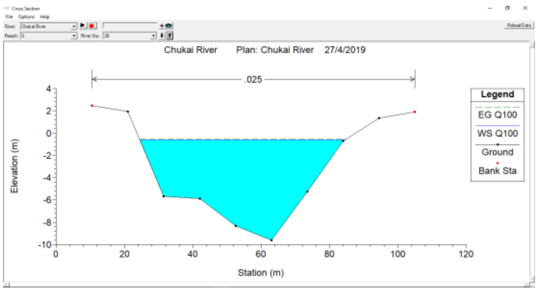


(b) New bridge piers

Figure 4.156 Water level at CH 15,550 of Chukai River –100-year ARI



(a) Old bridge piers



(b) New bridge piers

Figure 4.157 Water level at CH 15,750 of Chukai River –100-year ARI

4.10 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 CH 10,350 to CH 15,750. Figure 4.158 shows the water levels with 5-year, 10-year, 20-year, 50-year, 100-year ARI along the longitudinal cross section of the river with the old bridge piers. 3D plot of the water profiles including all the peak flows with the bridge piers is illustrated in the Figure 4.159.

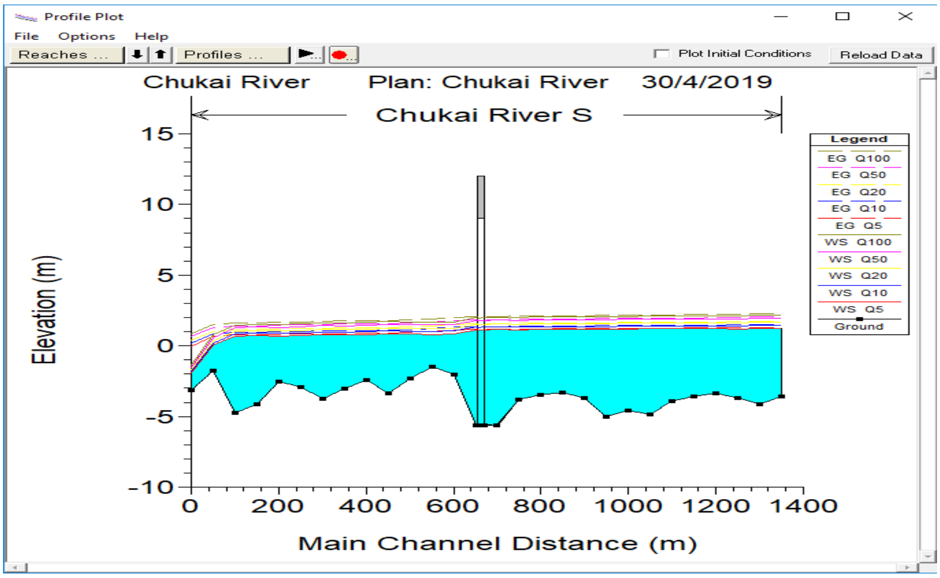


Figure 4.158 Water levels along the longitudinal cross section (old bridge piers)

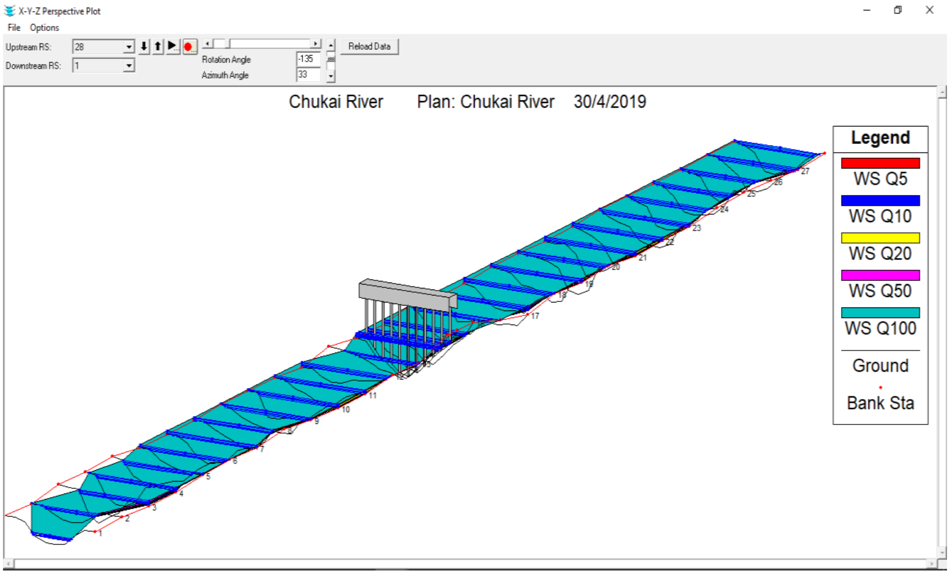


Figure 4.159 3D plot (old bridge piers)

Figure 4.160 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 new bridge piers. 3D plot of the water profiles including all the peak flows with the new bridge piers is illustrated in the Figure 4.161.

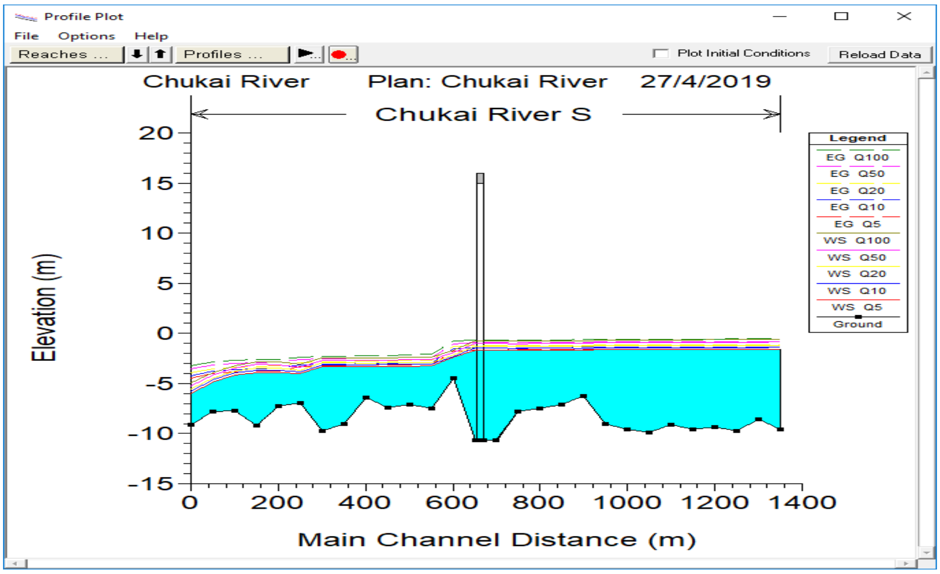


Figure 4.160 Water levels along the longitudinal cross section (new bridge piers)

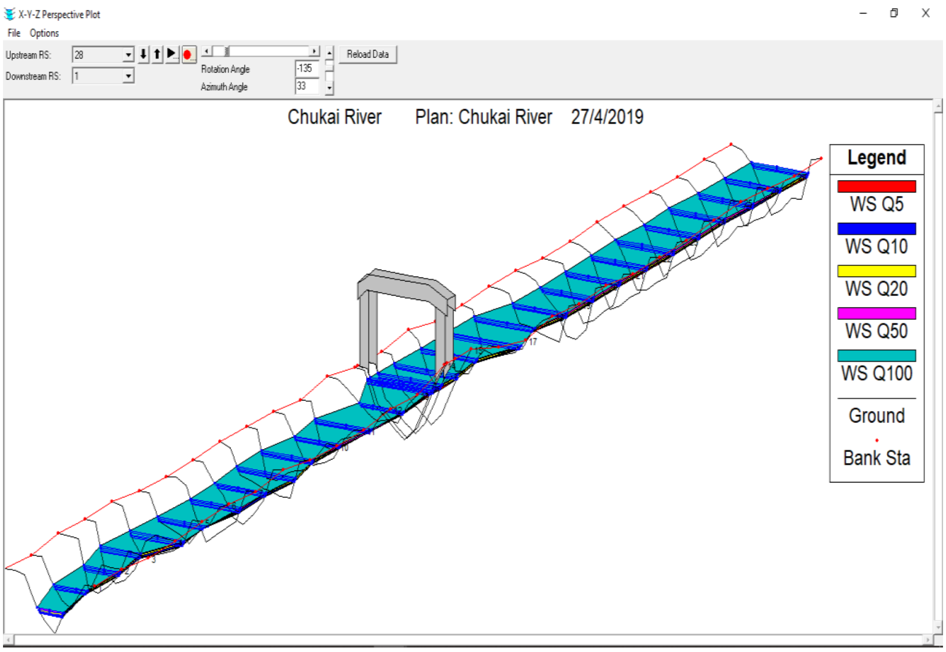


Figure 4.161 3D plot (new bridge piers)

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 new bridge piers does influence the water level. The level with Q_{100} for old and new bridge piers were 2.00 m and -0.73 m respectively.

4.11 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 difference of the water levels at the old and new bridge piers. Simulation with the new bridge piers shows that the water level under the bridge is -0.73 m and it is satisfied the clearance of 15 m for 100-year ARI as designed by the Malaysian Public Works Department. Since the depth of the river is deepen, the water level at 100-year shall not overflow along the river.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The study area is located at Chukai, Kemaman, Terengganu. The catchment area is 20.40 km. This catchment area usually faced the flood problem due to the monsoon season at the end of the year. The main objective of the study was to determine the difference of the water levels at the old and new bridge piers by generating 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. The simulation was run under basic condition on rainfall catchment area by using the Clark's method.

At the early stage of the study, a 2-dimensional river network with a cross section and longitudinal section diagram was produced for river reach that contained twenty-eight chainages at which the distance between chainages is 50m. The surface ground levels of the river were known from the Google Earth. After performing several simulations and study on the river network using the HEC-RAS software developed by Hydrologic Engineering Centre and by data analysis, a several conclusions were made from the study results. It is caused by rainfall event.

The analysis showed that the water levels in the channel increased as the flow increased. Along the downstream of the bridge the water levels rise higher 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. 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.

Furthermore, the simulation showed that the objective of the study was achieved successfully. However, further improvement could be done for a better result. Further improvement 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 Chukai River.

5.2 RECOMMENDATIONS

It is suggested that further experiments shall be carried out to give progressively compelling and gainful outcomes:

i. Conduct a real flood event study and compare the result of the simulation with the real flood event report from DID.
ii. Use AcrGIS software to get more accurate area of the catchment.
iii. Set up a rainfall station near the study area to give an exact rainfall profile of catchment area.

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APPENDIX A

MSMA 2nd Edition, 2012

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Perak	1	4010001	JPS Teluk Intan	54.017	0.198	0.084	0.790
	2	4207048	JPS Setiawan	56.121	0.174	0.211	0.854
	3	4311001	Pejabat Daerah Kampar	69.926	0.148	0.149	0.813
	4	4409091	Rumah Pam Kubang Haji	52.343	0.164	0.177	0.840
	5	4511111	Politeknik Ungku Umar	70.238	0.164	0.288	0.872
	6	4807016	Bukit Larut Taiping	87.236	0.165	0.258	0.842
	7	4811075	Rancangan Belia Perlop	58.234	0.198	0.247	0.856
	8	5005003	Jln. Mtg. Buloh Bgn Serai	52.752	0.163	0.179	0.795
	9	5207001	Kolam Air JKR Selama	59.567	0.176	0.062	0.807
	10	5210069	Stesen Pem. Hutan Lawin	52.803	0.169	0.219	0.838
	11	5411066	Kuala Kenderong	85.943	0.223	0.248	0.909
	12	5710061	Dispensari Keroh	53.116	0.168	0.112	0.820
Perlis	1	6401002	Padang Katong, Kangar	57.645	0.179	0.254	0.826
Selangor	1	2815001	JPS Sungai Manggis	56.052	0.152	0.194	0.857
	2	2913001	Pusat Kwln. JPS T Gong	63.493	0.170	0.254	0.872
	3	2917001	Setor JPS Kajang	59.153	0.161	0.118	0.812
	4	3117070	JPS Ampang	65.809	0.148	0.156	0.837
	5	3118102	SK Sungai Lui	63.155	0.177	0.122	0.842
	6	3314001	Rumah Pam JPS P Setia	62.273	0.175	0.205	0.841
	7	3411017	Setor JPS Tj. Karang	68.290	0.175	0.243	0.894
	8	3416002	Kg Kalong Tengah	61.811	0.161	0.188	0.816
	9	3516022	Loji Air Kuala Kubu Baru	67.793	0.176	0.278	0.854
	10	3710006	Rmh Pam Bagan Terap	60.793	0.173	0.185	0.884
Terengganu	1	3933001	Hulu Jabor, Kemaman	103.519	0.228	0.756	0.707
	2	4131001	Kg. Ban Ho, Kemaman	65.158	0.164	0.092	0.660
	3	4234109	JPS Kemaman	55.899	0.201	0.000	0.580
	4	4332001	Jambatan Tebak, Kem.	61.703	0.185	0.088	0.637
	5	4529001	Rmh Pam Paya Kempian	53.693	0.194	0.000	0.607
	6	4529071	SK Pasir Raja	48.467	0.207	0.000	0.600
	7	4631001	Almuktafibillah Shah	66.029	0.199	0.165	0.629
	8	4734079	SM Sultan Omar, Dungun	51.935	0.213	0.020	0.587
	9	4832077	SK Jerangau	54.947	0.212	0.026	0.555
	10	4930038	Kg Menerong, Hulu Trg	60.436	0.204	0.063	0.588
	11	5029034	Kg Dura. Hulu Trg	60.510	0.220	0.087	0.617
	12	5128001	Sungai Gawi, Hulu Trg	48.101	0.215	0.027	0.566
	13	5226001	Sg Petualang, Hulu Trg	48.527	0.228	0.000	0.547
	14	5328044	Sungai Tong, Setiu	52.377	0.188	0.003	0.558
	15	5331048	Setor JPS K Terengganu	58.307	0.210	0.123	0.555
	16	5426001	Kg Seladang, Hulu Setiu	57.695	0.197	0.000	0.544
	17	5428001	Kg Bt. Hampar, Setiu	55.452	0.186	0.000	0.545
	18	5524002	SK Panchor, Setiu Klinik	53.430	0.206	0.000	0.524
	19	5725006	Kg Raja, Besut	52.521	0.225	0.041	0.560

Figure A1 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	0.316	0.133	0.060	0.060	0.059	0.070	0.019	0.027	0.021
2	0.368	0.193	0.062	0.061	0.067	0.073	0.022	0.028	0.029
3	0.316	0.211	0.084	0.071	0.071	0.083	0.027	0.029	0.030
4		0.202	0.087	0.080	0.082	0.084	0.036	0.033	0.033
5		0.161	0.097	0.110	0.119	0.097	0.042	0.037	0.037
6		0.100	0.120	0.132	0.130	0.106	0.044	0.040	0.038
7			0.115	0.120	0.123	0.099	0.048	0.046	0.042
8			0.091	0.100	0.086	0.086	0.049	0.048	0.048
9			0.087	0.078	0.073	0.084	0.050	0.049	0.053
10			0.082	0.069	0.069	0.083	0.056	0.054	0.055
11			0.061	0.060	0.063	0.070	0.058	0.058	0.058
12			0.054	0.059	0.057	0.064	0.068	0.065	0.067
13							0.058	0.060	0.059
14							0.057	0.055	0.056
15							0.050	0.053	0.053
16							0.050	0.048	0.052
17							0.048	0.046	0.047
18							0.046	0.044	0.041
19							0.043	0.038	0.038
20							0.039	0.034	0.036
21							0.028	0.030	0.033
22							0.025	0.029	0.030
23							0.022	0.028	0.022
24							0.016	0.019	0.020

Figure A2 Normalised design rainfall temporal pattern