Appendix F

West Surat Rail Link Hydraulic Model Report

West Surat Rail Link – Hydraulic study

19 September 2012

Northern Energy Corporation Ltd.

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1. Introduction

This report documents the hydraulic study of waterways crossing the proposed West Surat 36 km spur link as part of the environmental assessment process.

Parsons Brinckerhoff was commissioned to complete hydrological and hydraulic modelling of four major crossings to determine key hydraulic design parameters and to assess hydraulic impacts resulting from the proposed bridges and rail embankment.

Simplified hydrological and hydraulic methods were used to complete an initial sizing of approximately 35 – 40 minor crossings.

1.1 Background

The Elimatta Mine is located in south western Queensland in the Surat Basin coal province, located North West of Xstrata's proposed Wandoan Coal project. New Hope Coal is proposing to develop the thermal coal mine for export via coal export facilities at the port of Gladstone. Parsons Brinckerhoff has previously undertaken a transport options study and developed concept alignments for the rail link as the basis for the initial planning and approvals.

As part of previous investigations, high level flood modelling of the existing scenario was completed for the major waterway crossings. An existing XP-RAFTS hydrological model, calibrated to the Juandah Creek stream-flow gauge located at Windamere was used to estimate design flows at the four major waterway crossings. An existing MIKE-11 hydraulic model of Juandah Creek and HEC-RAS hydraulic models of Horse Creek, Mud Creek and Spring Creek were used to estimate design flood levels under existing conditions.

1.2 Scope of assessment

The scope of this assessment was to:

- develop hydraulic models for identified crossings (major and minor) within the 36km spur link from Elimatta Mine to its crossing at Surat Basin Railway
- run hydraulic models for major crossings
- run four different scenarios for each major crossing which include a scenario for the current condition in order to investigate the impacts of flooding on upstream and downstream of the proposed crossings
- undertake analysis for a range of ARIs (10, 100 and 1000 year)
- size bridge openings and determine hydraulic impacts
- provide flood impact assessment report.

2. Description of proposed development

The Surat West Link project is described as a proposed open access railway commencing at a connection point on the proposed Surat Basin Railway, located north of the Wandoan Township and travelling west for approximately 36 km to the proposed Elimatta Coal Mine with potential for the line to also carry additional tonnages from adjacent projects.

The work undertaken to date shows that the proposed railway will consist of a single track with a passing loop positioned adjacent to the takeoff with the Surat Basin Rail (SBR) and provision within the design to accommodate an additional passing loop on the western side of Juandah Creek to allow for potential future increases in tonnage throughput. The railway has been designed to accommodate trains of up to approximately 2.4 km in length. The railway has been designed for a maximum speed of 80 km/h with maximum ruling grades of 1.25% loaded and 2% unloaded.

To construct the proposed railway the following activities will need to be undertaken:

- earthworks
- major and minor structures
- cross and longitudinal drainage including diversion drains
- road works
- private access crossings
- utilities and services crossings
- signalling and telecommunications
- fencing and revegetation.

All of the required works have a varying impact on the projects permanent land requirements and also maintenance access requirements. These requirements need to be considered when determining the permanent rail corridor.

In addition to the proposed railway, potentially a water pipeline and an overhead electricity transmission line may be required to run adjacent to the railway. The final location of these services needs to be determined with further engineering investigation including consultation with relevant service authorities.

It has been assumed that both the water and electricity services will generally follow the rail alignment where possible. It is recognised that the power line will more than likely divert away from the rail corridor at or before the Leichhardt Highway and head towards the Wandoan substation. At this stage the corridor has been defined with the power line following the full length of the rail alignment from the Elimatta Mine to Nathan Road. Any alternative transmission route will most likely impact a number of common property owners and include a separate transmission line corridor/easement. At this stage the transmission line has been incorporated into the rail and services corridor and further project development may result in a reduction of the corridor requirements and colocation with property owners. It has been assumed that these issues will be discussed by NEC as part of the community consultation.

3. Hydrology

The design flows in watercourses crossing the proposed rail link have been estimated using three hydrological assessment methods as follows:

- flood frequency analysis
- XP-RAFTS hydrological modelling
- rural probabilistic rational method.

Flood frequency analysis was used to determine peak design flows in Juandah Creek at the stream-flow gauge at Windamere, located approximately 1.6 km upstream of the proposed rail crossing. This analysis updates a previous flood frequency analysis by including the 2010 flood event which is the largest event recorded at Windamere.

A hydrological model was previously developed using XP-RAFTS to represent rainfall runoff processes in the Juandah Creek, Mud Creek, Spring Creek and Horse Creek catchments. Historical flood flow data from the stream-flow gauge on Juandah Creek at Windamere and rainfall data was used to calibrate the hydrological model.

The rural probabilistic rational method was used to estimate peak design flows from the minor crossing catchments.

3.1 Catchment descriptions

3.1.1 Juandah Creek

The Juandah Creek catchment extends approximately 45 km to the south of the proposal rail link crossing. The catchment mainly comprises rural farmland.. In its lower reaches, Juandah Creek is characterised by a well-defined and incised main channel flowing through a wide floodplain.

3.1.2 Mud Creek

The Mud Creek catchment extends approximately 25 km to the south-west of the proposed rail link crossing. The catchment mainly comprises rural farmland. In its lower reaches, Mud Creek is characterised by a small main channel with several smaller channels on its floodplain. The proposed rail link also crosses a tributary of Mud Creek (approximately 800m south east of the main Mud Creek crossing).

3.1.3 Spring Creek

The Spring Creek catchment extents approximately 11 km to the south of the proposed rail link crossing. The catchment mainly comprises rural farmland. In its lower reaches, Spring Creek is characterised by a well-defined main channel flowing through a wide floodplain with several smaller flood channels.

3.1.4 Horse Creek (southern alignment)

The Horse Creek catchment extends approximately 41 km south of the proposed southern rail link crossing. The catchment mainly comprises rural farmland. In its lower reaches, Horse Creek is characterised by a well-defined main channel flowing through a wide floodplain with several smaller flood channels.

3.1.5 Horse Creek (northern alignment)

The Horse Creek catchment extends approximately 43.5 kilometres south of the proposed northern rail link crossing. The catchment mainly comprises rural farmland. In its lower reaches, Horse Creek is characterised by a well-defined main channel flowing through a wide floodplain with several smaller flood channels.

3.1.6 Minor catchments

There are numerous minor catchments crossing the proposal rail link. These minor catchments typically comprise rural farming land. Peak flows from the minor catchments have been estimated using the probabilistic rational method and are discussed in Section 4.5.

3.2 Available data

3.2.1 Rainfall data

Rainfall data from several pluviographs (6 minute time interval) and daily rainfall stations are available within or close to the catchments. These rainfall data have been used in the calibration of the rainfall runoff model as discussed in Section 4.4.2.

3.2.2 Stream flow data

Stream flow data are available within the catchment for Juandah Creek at Windamere (130344A), located approximately 1.5 km upstream of the proposal rail link crossing, from 1974. These stream flow data have been used for flood frequency analysis (Section 4.3) and calibration of the rainfall runoff model (Section 4.4.2)

3.3 Flood frequency analysis

Peak design flows for Juandah Creek at the Windamere stream flow gauge were estimated using flood frequency analysis. As recommended in Australian Rainfall and Runoff (Pilgrim, 1987) (ARR87), a Log-Pearson Type III distribution was fitted to the peak annual flows at the site. The maximum recorded flow is 1,057 m^3/s recorded in December 2010. Two low outliers (1986 and 1992) were removed from the annual series to produce a better fit to the data. The results of the flood frequency analysis are shown in Figure 3.1 and Table 3.3.

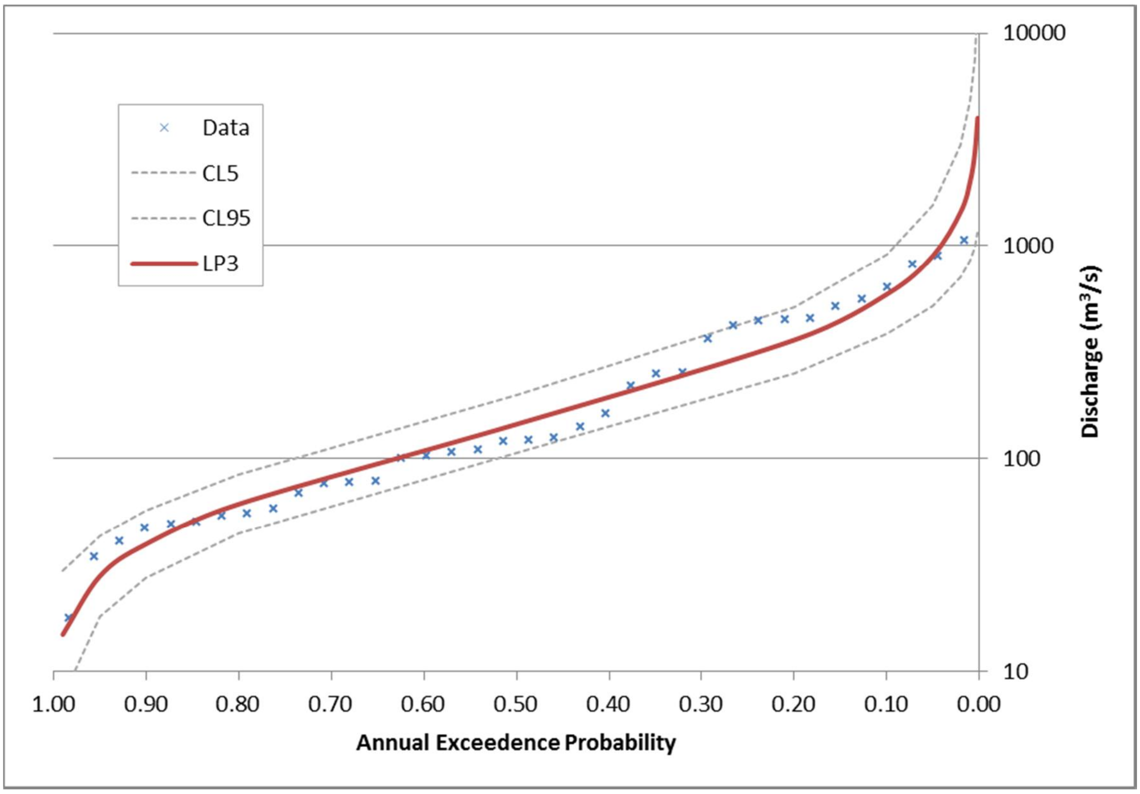

Figure 3.1 Flood frequency curve for Juandah Creek at Windamere (130344A)

3.4 Rainfall runoff modelling

Hydrological models simulate rainfall-runoff processes within catchments. They are used to estimate catchment flows generated from either design storm events or historical rainfall records.

Hydrological models represent catchments and their tributaries as a series of sub-catchment areas which are linked together to replicate the stream network. The model input data include parameters to represent factors such as rainfall patterns (temporal and spatial), catchment size, catchment slope, drainage features, channel and floodplain storage, and variations in catchment land use.

Hydrological modelling software suitable for design flood estimation is described in ARR87. In current Australian engineering practice, examples of commonly used models include XP-RAFTS, RORB, WBNM and URBS.

The XP-RAFTS hydrologic model was chosen to estimate design hydrographs for this assessment as it is capable of representing a range of physical characteristics that influence runoff behaviour such as rainfall patterns, catchment shape, catchment slope, drainage features, channel and floodplain storage, and variations in catchment land use. The XP-RAFTS model converts rainfall to runoff by applying rainfall losses to both the impervious and pervious catchments within the model to produce excess rainfall hyetographs.

The XP-RAFTS model was calibrated by comparing modelled flows for historic events against historic data from the Juandah Creek stream flow gauge.

The soil loss model used in XP-RAFTS was the Australian Representative Basins Model (ARBM). This method was preferred over an initial loss/continuing loss model because it allows the recovery of rainfall losses between rainfall bursts, which is a significant feature of some of the observed rainfall events.

The XP-RAFTS model has been developed to assess catchment flows for the 10, 100 and 1,000 year ARI design events.

The layout and full details of adopted model parameters for the XP-RAFTS model is included in Appendix B.

3.4.1 Model parameters

3.4.1.1 Design rainfall

Intensity Frequency Duration (IFD) parameters from ARR87 Volume 2 adopted for this project are shown below in Table 3.1.

Table 3.1 IFD Parameters

Parameter	Symbol	Value (mm/hr)
2 year ARI, 1 hour rainfall intensity	${}^{2}I_{1h}$	40.6
2 year ARI, 12 hour rainfall intensity 2I12h	${}^{2}I1_{2h}$	5.98
2 year ARI, 72 hour rainfall intensity 2I72h	${}^{2}I_{72h}$	1.5
50 year ARI, 1 hour rainfall intensity 50I1h	${}^{50}I_{1h}$	71.6
50 year ARI, 12 hour rainfall intensity	${}^{50}I_{12h}$	10.9
50 year ARI, 72 hour rainfall intensity	${}^{50}I_{72h}$	3.03
Skewness	G	0.24
Geographical factor (2 year ARI)	F2	4.27
Geographical factor (50 year ARI)	F50	16.7

The CRC FORGE method (Hargraves, 2005) was used to derive rainfall depths for the 1,000 year ARI event.

In accordance with ARR87, temporal patterns for Zone 3 were applied.

Conservative, areal reduction factors were not applied to the design rainfall depths, however a good match was obtained during model calibration (refer Section 4.4.2)

3.4.1.2 Rainfall losses

In the south-west and north-east of the Juandah/Woleebee Creek catchment, there are several areas where the Orallo (or Gubberamunda), Kumbarilla and Hutton sandstone geological units outcrop. These sandstone units form part of the Great Artesian Basin (GAB), and the outcrop areas make up part of the intake/outflow areas for these aquifers, which might be expected to be areas of high infiltration.

The XP-RAFTS model includes two sets of soil loss models:

1. ARBM_GAB_Intake_Bed

2. ARBM_Non_GAB_Intake_Bed.

Parameters for each soil loss model are included in Appendix B. There are no available data to verify the parameters assumed in the soil loss models other than the accepted ranges of values provided in the XP-RAFTS user manual (which refers to research by Goyen, as referenced in the user manual). Though it is expected that the GAB intake beds will capture more rainfall than the non-GAB intake beds, it is acknowledged that the parameters used are somewhat arbitrary and that they have been adjusted to provide a best fit with the stream flow gauge on Juandah Creek at Windamere.

3.4.1.3 Sub-catchment areas, impervious and roughness

Catchment areas have been identified using a digital terrain model (DTM) using survey data from several sources. The XP-RAFTS model, covering a total of approximately 3,600 km^2, extends downstream to the confluence of Horse Creek and Juandah Creek.

A uniform fraction impervious of 0% and Manning's 'n' roughness value of 0.07 were adopted.

3.4.1.4 Catchment lag and channel routing

The storage delay coefficient for each of the sub-catchments in XP-RAFTS is calculated using the Equal Area Method which determines the average vectored slope of the catchment together with catchment area, percentage impervious, Manning's 'n' value, loss rates and rainfall data.

Routing of hydrographs through each channel link was completed using estimated channel cross section dimensions, reach lengths and slopes and the Manning equation open channel flow.

3.4.2 Calibration

Six historic events were identified for the calibration of the XP-RAFTS model using recorded rainfall and stream flow data.

Given the size of the catchment, significant spatial distribution of rainfall occurs during storm events. This has been accounted for by the construction of isohyet maps for each storm event then the application of location specific local storms to each model sub-catchment area.

The XP-RAFTS model was calibrated by adjusting the ARBM loss model to obtain the best fit possible for all storm events. A comparison of the peak flows in given in Table 3.2. Recorded rainfall hyetographs, recorded hydrographs and modelled hydrographs are given in Appendix B.

Table 3.2 XP-RAFTS model calibration events peak flow comparison

Date	Windamere gauge (m^3/s)	XP-RAFTS (m^3/s)
October 1989	366	206
April 1990	441	51
February 1991	828	829
March 1997	264	0

Date	Windamere gauge (m^3/s)	XP-RAFTS (m^3/s)
August 1998	815	914
December 2004	104	40

The results in Table 3.2 and Appendix B show that the XP-RAFTS model provides a reasonable estimate for large flood events however there is appreciable uncertainty for smaller events.

3.4.3 Validation

The results of the flood frequency analysis (Section 3.3) were used to validate the results of the XP-RAFTS model. Table 3.3 shows that there is good agreement between the two methods.

Table 3.3 Comparison of flood frequency analysis and XP-RAFTS modelled peak flows (m^3/s)

ARI (years)	Flood frequency analysis	XP-RAFTS
10	592	637
100	2024	2114

3.5 Probabilistic rational method

The rural probabilistic rational method (PRM) has been utilised for estimation of design flows for minor crossings of the proposed rail link. The rural probabilistic rational method adopts design rainfall data applicable to the area and parameters which describe the catchment (including catchment area, time of concentration for the catchment, and a runoff coefficient describing catchment loss characteristics) to determine peak flows resulting from design rainfall events. The rural PRM procedure as outlined in the Road Drainage Manual (Department of Main Roads 2010) as deemed suitable by ARR87 has been adopted.

The rural PRM for Queensland is applicable for catchments up to 25 km² in area. As outlined in the Road Drainage Manual, the rational method formula is:

$$Q_Y = \frac{1}{360} C_Y . I_{t_c,Y} A$$

Where:

- Q_Y = peak flow rate (m^3/s) of average recurrence interval (ARI) of Y years
- C_Y = runoff coefficient (dimensionless) for ARI of Y years
- A = catchment area (ha)
- $I_{tc,Y}$ = average rainfall intensity (mm/h) for design duration of t_c hours ($t_c = \frac{F \times L}{A^{0.1} \times S_e^{0.2}}$ Brasnsby-Williams formula) and ARI of Y years.

The Bransby-Williams formula for time of concentration, t_c (as detailed above) was adopted for ease of use. The mean catchment slope was adopted in place of the equal area slope.

3.6 Adopted design flows

The peak design flows shown in Table 3.4 have been adopted as inputs to the hydraulic models.

Table 3.4 Peak design flows (m^3/s)

Crossing	XP-RAFTS model node	10 year ARI	100 year ARI	1000 year ARI
Juandah Creek	Gauge	608	2114	3345
Mud Creek	C1	127	318	503
Mud Creek Tributary	C1b	8.5	21	34
Spring Creek	A15C	38	90	144
Horse Creek	A06	199	875	1433

4. Hydraulic modelling – major crossings

Hydraulic modelling is required to determine the flood extents and levels to inform bridge designs at waterway crossings. As part of this modelling, the impacts of the bridge on flood levels can be demonstrated and any afflux shown with consideration to the relative effects of the afflux on surrounding land. Three options have been prepared to simulate the effects of the bridge in three flood events, with a single option recommended. Recommended options have being selected based on afflux impacts and also downstream impacts.

The intent of the hydraulic modelling has been to collate a number of potential bridge design options and determine the afflux of the bridging structure on the flood levels.

It should be noted that the "Bridge Opening" presented in this report are measured at the base of the bridge and therefore additional spans may be required to enable spill-through abutments to be constructed up to the rail level.

Whilst detailed bridge designs have not been completed, for the purpose of this assessment the following bridge design parameters have been included:

- bridge spans of 25 m
- pier diameters of 1.2 m with only singular piers adopted
- bridge width of 4 m
- abutment slopes of 1.5:1
- bridge soffit to be a minimum of 600 mm above the 1,000 year ARI flood level.

4.1 Two-dimensional modelling

Of the four major waterways that impact on the alignment of the proposed rail line, three have river floodplain characteristics that necessitate two-dimensional hydraulic modelling. The models for these waterways are further outlined within this section.

The flood behaviour at Juandah Creek, Mud Creek and Horse Creek is considered two-dimensional. Therefore, separate TUFLOW two-dimensional hydrodynamic models have been developed at these crossings to determine flood conditions under existing and proposed scenarios. Specific details of the TUFLOW models are discussed below and proposed model layouts attached. The models will be used to simulate the 10, 100 and 1,000 year ARI flood events using inflow hydrographs from the XP-RAFTS model.

4.1.1 Juandah Creek

The proposed Juandah Creek 1D/2D TUFLOW model covers approximately 9 km of Juandah Creek and its floodplain from approximately 2 km upstream of the proposed rail crossing to approximately 7 km downstream of the proposed rail crossing. The 1D component of the model comprised cross sections at 100 m intervals and one proposed bridge crossing. The 2D component of the model covers approximately 23 km² using a grid resolution of 10 m.

4.1.2 Mud Creek

The Mud Creek 1D/2D TUFLOW model covers approximately 3.4 km of Mud Creek and approximately 2.6 km of its tributary to its confluence with Mud Creek and their floodplains. The upstream model boundaries are approximately 1.8 km upstream of the proposed rail crossing of Mud Creek and approximately 1.6 km upstream of the proposed rail crossing of its tributary. The downstream model boundary is approximately 1.6 km downstream of the proposed rail crossing of Mud Creek. The 1D component of the model comprised cross sections at 100 m intervals and two bridge crossings. The 2D component of the model covers approximately 9.9 km² using a grid resolution of 10 m.

4.1.3 Horse Creek

Two alternative crossings for Horse Creek are modelled because the track alignment over Horse Creek has not been confirmed at the time of modelling. The northern alignment requires a single bridge similar to Juandah and Mud Creeks, but has a greater length of railway track. The southern alignment has less track length but requires three separate bridges over a wider floodplain.

4.1.4 Topographic and bathymetric data

A digital elevation model (DEM) covering the extent of the models was constructed using the supplied Aerial Laser Survey (ALS) data. Channel cross section data will be ‘cut’ from the DEM. The ALS defines the inverts of Mud Creek and Horse Creek to sufficient detail; however as the low flow channel of Juandah Creek was wet at the time of aerial survey, it will be necessary to manually define the invert of Juandah Creek within the 12D model. The northern Horse Creek alignment was poorly defined in terms of survey available for use in the modelling and further more detailed survey should be obtained prior to detailed design.

4.1.5 Boundary conditions

4.1.5.1 Inflows

Hydrographs from the XP-RAFTS model were applied as inflow boundaries to the TUFLOW models. Point and distributed upstream inflows were investigated. Flow-on-grid inflows were required for both Juandah and Mud Creek.

4.1.5.2 Downstream boundaries

The downstream boundaries of the Juandah Creek, Mud Creek and Horse Creek TUFLOW models were based on a rating curve which describes the flow from the model (assumed to be normal depth) at a range of water levels. The rating curves have been determined using the channel and floodplain topography, slope and assumed roughness values. Initially, the flow in the 1D domain was transferred to the 2D domain before using a full 2D downstream boundary. However, instabilities arose in the TUFLOW model for Juandah Creek, so the 1D network was extended downstream (outside) of the 2D model extent.

The Juandah Creek TUFLOW model downstream boundary is aligned at the same location as the existing MIKE11 model. The rating curve from the MIKE11 model was examined to ensure consistency so that the stage-discharge relationships were similar.

The Mud Creek TUFLOW model downstream boundary is within the existing Mud Creek HEC-RAS model domain. The flow behaviour at the location in the HEC-RAS model was examined to ensure consistency at the downstream boundary so that the stage-discharge relationships were similar.

The Horse Creek TUFLOW model downstream boundary is within the existing Horse Creek HEC-RAS model domain. The flow behaviour at the location in the HEC-RAS model was examined to ensure consistency at the downstream boundary so that the stage-discharge relationships were similar.

4.1.6 Structures

There are no hydraulic structures modelled in the existing scenarios.

The proposed bridges have been modelled using a combined 1D/2D approach (i.e. bridge loss tables where the proposed bridge crosses the 1D channel domain and 2D flow constrictions where it crosses the 2D domain). Hydraulic losses through bridge crossings were verified using either HEC-RAS or hand calculations.

4.1.7 Roughness

A land map was developed using aerial photography of the TUFLOW model extents. Each 2D grid cell and 1D cross section will be assigned roughness values according to its land use. Table 4.1 below shows the proposed roughness values which are based from "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains", United States Geological Survey Water-supply Paper 2339.

Table 4.1 Roughness values

Number	Land use	Manning's 'n' value
1	Roads	0.02
2	Water courses	0.03
3	Rural	0.04
4	Riparian zone / light vegetation	0.06
5	Dense vegetation	0.08
6	Rural buildings	0.10

4.1.8 Validation

We are unaware of any surveyed flood level information available in the vicinity of the crossings; therefore no calibration of the TUFLOW models has been undertaken.

The existing MIKE11 model and HEC-RAS models were used as a check on the TUFLOW model results.

4.1.9 Model scenarios

The TUFLOW models were used to simulate the 10, 100 and 1,000 year ARI events for the existing and three geometry scenarios; existing and proposed. At each crossing model three proposed scenarios have been modelled to assess the hydraulic impacts made by various bridge configurations.

4.1.10 Results

Results for each of the waterway crossings modelling in TUFLOW are presented in the following section. Bridge spans are based on the span at the base of the bridge, with 1.5:1 abutments simulated.

4.1.10.1 Juandah Creek

The geometry of Juandah Creek and its floodplain lends the bridge design towards having a single bridge crossing. The three bridge openings are summarised below in Table 4.2.

Table 4.2 Proposed geometry scenarios

Reference	Bridge Opening (m)	Number of Piers
Option 1	300	11
Option 2	200	7
Option 3	150	5

Results of the three proposed scenarios together with the existing scenario are presented in the three tables below (Tables 4.3, 4.4 and 4.5). The results are also presented in the attached figures in Appendix C.

Table 4.3 10 Year ARI water levels

Selected location	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Immediately upstream of bridge	224.60	224.62	20	224.62	20	224.69	90
Approximately 1km upstream of bridge	225.98	225.98	0	225.98	0	225.98	0

Table 4.4 100 year ARI water levels

Selected location	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Immediately upstream of bridge	227.05	227.13	80	227.19	140	227.31	260

Selected location	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Approximately 1km upstream of bridge	227.63	227.65	20	227.67	40	227.73	100

Table 4.5 1,000 Year ARI water levels

Selected location	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Immediately upstream of bridge	228.07	228.23	160	228.33	260	228.47	400
Approximately 1km upstream of bridge	228.54	228.63	90	228.70	160	228.79	250

Based on the impacts caused by the bridge design (as shown by the above results and the figures attached in Appendix C for Option 2), Option 2 is recommended based on afflux and velocities downstream of the bridge.

4.1.10.2 Mud Creek

The geometry of Mud Creek and its floodplain lends the bridge design towards having a two bridge crossings. The three bridge options modelled are summarised below in Table 4.6.

Table 4.6 Proposed geometry scenarios

Reference	Bridge 1		Bridge 2	
	Bridge opening (m)	Number of piers	Bridge opening (m)	Number of piers
Option 1	300	11	200	7
Option 2	200	7	50	1
Option 3*	150	5	25	0

** Following modelling of Option 3, a review of Bridge 2 was undertaken using the minor crossing hydraulic analysis method. Under this methodology, it was found that a 1,950 mm diameter culvert (or equivalent) would be adequate and would be recommended instead of a bridge.*

Results of the three proposed scenarios together with the existing scenario are presented in the three tables below (Tables 4.7, 4.8 and 4.9). The results are also presented in the attached figures in Appendix C.

Table 4.7 10 Year ARI water levels

Selected location	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)

Selected location	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Immediately upstream of bridge	223.64	223.81	170	223.82	180	223.84	200
Approximately 1 km upstream of bridge	228.70	228.70	0	228.70	0	228.70	0

Table 4.8 100 Year ARI water levels

Selected location	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Immediately upstream of bridge	224.37	224.39	20	224.40	30	224.42	50
Approximately 1 km upstream of bridge	229.11	229.11	0	229.11	0	229.11	0

Table 4.9 1,000 Year ARI water levels

Selected location cross-section	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Immediately upstream of bridge	224.63	224.65	20	224.68	50	224.70	70
Approximately 1 km upstream of bridge	229.30	229.32	20	229.35	50	229.36	60

Based on the impacts caused by the bridge design (as shown by the above results and the figures (Option 3 only) attached in Appendix C), Option 3 is recommended.

4.1.10.3 Horse Creek (South)

The larger floodplain of the southern alignment crossing has been shown in modelling to have a large proportion of conveyance outside of the main channel itself. Therefore, multiple openings are required. These openings are summarised in Table 4.10 below.

Table 4.10 Proposed geometry scenarios

Selected location	Bridge 1		Bridge 2		Bridge 3	
	Bridge opening (m)	Number of piers	Bridge opening (m)	Number of piers	Bridge opening (m)	Number of piers
Option 1	600	23	n/a	n/a	n/a	n/a
Option 2	150	5	150	5	150	5

Selected location	Bridge 1		Bridge 2		Bridge 3	
	Bridge opening (m)	Number of piers	Bridge opening (m)	Number of piers	Bridge opening (m)	Number of piers
Option 3	150	5	150	5	100	3

Results of the three proposed scenarios together with the existing scenario are presented in the three tables below (Tables 4.11, 4.12 and 4.13). The results are also presented in the attached figures (Options 2 and 3 only) in Appendix C.

Table 4.11 10 Year ARI water levels

Selected cross-section	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Immediately upstream of bridge	227.34	227.47	130	227.49	150	227.50	160
Approximately 1 km upstream of bridge	228.24	228.32	80	228.33	90	228.34	100

Table 4.12 100 Year ARI Water Levels

Selected cross-section	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Immediately upstream of bridge	228.13	228.26	130	228.27	140	228.40	270
Approximately 1 km upstream of bridge	229.74	229.76	20	229.76	20	229.76	20

Table 4.13 1,000 Year ARI Water Levels

Selected location	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Immediately upstream of bridge	228.21	228.36	150	228.39	180	228. 843	630
Approximately 1 km upstream of bridge	229.80	229.81	10	229.82	20	229.84	40

Based on the impacts caused by the bridge design (as shown by the above results and the figures attached in Appendix C for options 2 and 3), Option 3 is recommended.

4.1.10.4 Horse Creek (North)

The geometry of the northern Horse Creek and its floodplain lends the bridge design towards having a single bridge crossing. The three bridge openings are summarised below in Tables 4.14, 4.15 and 4.16.

Table 4.14 Proposed geometry scenarios

Reference	Bridge Opening (m)	Number of Piers
Option 1	300	11
Option 2	200	7
Option 3	150m and 75m at the road	5 and 2

Results of the three proposed scenarios together with the existing scenario are presented in the three tables below (Tables 4.15, 4.16 and 4.17). The results are also presented in the attached figures in Appendix C.

Table 4.15 10 Year ARI water levels

Selected location	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Immediately upstream of bridge	224.42	224.44	20	224.44	20	224.44	20
Approximately 1 km upstream of bridge	225.54	225.54	0	225.54	0	225.54	0

Table 4.16 100 Year ARI water levels

Selected location	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Immediately upstream of bridge	225.00	225.22	220	225.34	340	225.32	320
Approximately 1 km upstream of bridge	226.28	226.31	30	226.33	50	226.35	69

Table 4.17 1,000 Year ARI water levels

Selected location	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Immediately upstream of bridge	225.39	225.78	490	225.89	500	225.90	510

Selected location	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
Approximately 1 km upstream of bridge	226.77	226.85	80	226.92	150	226.97	200

Based on the impacts caused by the bridge design (as shown by the above results and the figures attached in Appendix C), Option 3 is recommended.

4.2 One-dimensional modelling

The flood behaviour at Spring Creek is more suited to one-dimensional modelling, therefore HEC-RAS modelling at this crossing is considered appropriate. The existing scenario is based upon the latest hydrology and topographic information available. The model was then used to simulate the proposed scenarios and undertake an impact assessment.

4.2.1 Geometry

A digital elevation model (DEM) covering the extent of the model was constructed using the supplied Aerial Laser Survey (ALS) data. Channel cross section data was ‘cut’ from the DEM at regular intervals.

The conveyance of the Spring Creek channel compared to the conveyance outside of the floodplain is relatively small. The sinuosity of the channel is high with the channel meandering across the floodplain. Therefore, the model has been aligned to represent flow travelling in the direction of the floodplain rather than following the channel which provides cross-sections perpendicular to the floodplain.

Cross-sections have been spaced at approximately 100 m intervals with interpolated sections made where necessary.

The model extends approximately 1,000 m upstream of the proposed rail crossing and approximately 500 m downstream of the proposed rail crossing (relative to the floodplain).

4.2.2 Boundary conditions

4.2.2.1 Inflows

The peak flow rates from the hydrographs produced in the XP-RAFTS model are applied as an inflow to the HECRAS model at the upstream extent of the model.

4.2.2.2 Downstream boundary

The downstream boundary of the Spring Creek is specified as a normal depth representing a 0.1% gradient. This is consistent with the gradient and floodplain of the channel immediately upstream of the model terminus.

4.2.3 Roughness

In HECRAS, roughness coefficients are applied within each cross-section. Spring Creek can be defined as a gently meandering waterway with no major obstructions or vegetation. Within the floodplain, the land-use is predominantly pastoral. Table 4.18 below shows roughness values used which are based off typical published values.

Table 4.18 Roughness values

Location	Land use	Manning's 'n' value
Within Channel	Waterway	0.03
Floodplain	Grazing	0.04

4.2.4 Model scenarios

The HECRAS model was used to simulate the 10, 100 and 1,000 year ARI events for the existing and three proposed geometry scenarios. The three geometry scenarios are summarised below in Table 4.19.

Table 4.19 Proposed geometry scenarios

Reference	Bridge Opening (m)	Number of Piers
Option 1	200	8
Option 2	150	6
Option 3	50	2

4.2.5 Results

The HECRAS model was used to simulate the 10, 100 and 1,000 year ARI events for the existing and three proposed geometry scenarios.

Table 4.20 10 Year ARI water levels

Selected cross-section	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
667 (50 m upstream of bridge)	223.678	223.696	18	223.696	18	223.713	35
755 (140 m upstream of bridge)	224.242	224.242	0	224.242	0	224.242	0
845 (230 m upstream of bridge)	224.662	224.662	0	224.662	0	224.662	0

Table 4.21 100 Year ARI water levels

Selected cross-section	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
667 (50 m upstream of bridge)	223.970	223.919	51	223.950	-2	224.112	+142
755 (140 m upstream of bridge)	224.327	224.346	17	224.345	16	224.329	2
845 (230 m upstream of bridge)	224.743	224.751	8	224.751	8	224.743	0

Table 4.22 1,000 Year ARI water levels

Selected cross-section	Existing	Option 1		Option 2		Option 3	
	Water level (m AHD)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)	Water level (m AHD)	Afflux (mm)
667 (50 m upstream of bridge)	224.057	224.089	32	224.141	184	224.409	352
755 (140 m upstream of bridge)	224.471	224.480	9	224.480	9	224.521	50
845 (230 m upstream of bridge)	224.898	224.902	4	224.902	4	224.897	-1

Note: The Bridge is located at approximate chainage 600.

Based on the impacts caused by the bridge design (as shown by the above results and the figures attached in Appendix C), Option 3 is recommended.

4.3 Summary of major crossings

4.3.1 Juandah Creek

The recommended option for Juandah Creek is Option 2 which is a 200 m wide opening (225m long bridge). For the 100 year event, this option has an afflux of 40 mm approximately 1 km upstream of the rail alignment.

4.3.2 Mud Creek

The recommended option for Mud Creek is Option 3 which is a 150 m wide opening (175m long bridge) with a culvert (1950 mm diameter or equivalent) on the tributary. For the 100 year event, this option has no afflux approximately 1 km upstream of the rail alignment indicating that further reductions to the bridge span could be considered.

4.3.3 Horse Creek (south)

The recommended option for Horse Creek (south) is Option 3 which consists of two 150 m wide openings (175m long bridges) and one 100 m opening over the road (125m long bridge). For the 100 year event, this option has an afflux of 20 mm approximately 1 km upstream of the rail alignment.

The model geometry used for modelling of Horse Creek (south) is relatively coarse and consideration should be made in the detailed design phase to improve upon this geometry to improve result accuracy.

4.3.4 Horse Creek (north)

The recommended option for Horse Creek (north) is Option 3 which consists of one 150 m wide opening (175m long bridge) and one 75 m opening over the road (100m long bridge). For the 100 year event, this option has an afflux of 69 mm approximately 1 km upstream of the rail alignment.

The model geometry used for modelling of Horse Creek (north) is relatively coarse and consideration should be made in the detailed design phase to improve upon this geometry to improve result accuracy. Immediately to the north of the crossing it was necessary to use survey sourced from the Shuttle Radar Topography Mission to fill in missing areas of coverage.

4.3.5 Spring Creek

The recommended option for Spring Creek is Option 3 which consists of a one 50 m wide opening (75m bridge total). For the 100 year event, this option has an afflux of 50 mm approximately 150 m upstream of the rail alignment.

5. Minor crossings

Numerous cross drainage culverts are required on the rail alignment to allow water from minor waterways to pass beneath the rail alignment. The methodology in determining the size and number of culvert units, together with a summary of the culverts is detailed within this section.

5.1 Methodology

Minor cross drainage culverts not subject to detailed hydraulic modelling have been designed to accommodate the peak design flows using a culvert design spreadsheet that has been developed based on the methodology set out in Waterway Design (AUSTROADS, 1994).

The probabilistic rational method was used to estimate peak design flows generated from minor catchments and a culvert design spreadsheet be used to undertake an initial hydraulic sizing of the minor crossings. For any bridge crossings, other than the major crossing summarised previously, an approximate bridge span was determined by modelling the crossing as a large box culvert.

The design approach selects a trial culvert size and assesses whether the culvert is operating under inlet or outlet control. The headwater depth and outlet velocity are then calculated to assess their acceptability and the culvert size refined where necessary.

Culvert location, length, levels and slope have been determined based on survey and rail geometry design details using the computer package 12D.

Where possible, pipe culverts have been designed to have a minimum of 1.0m cover for improved constructability and a water level freeboard of 600mm to top of rail.

Culverts have been designed such that they, and any drainage channels downstream or upstream of the culverts, have a minimum slope of 0.5% to ensure adequate drainage and self-cleansing velocity is achieved.

The minimum culvert size has been adopted as 900mm diameter.

5.2 Summary of culverts required

A summary of the minor cross-drainage locations and nomenclature used can be referred to in Appendix D, Figures 55-58. The following table summarizes the required culvert sizes at each of the locations in Appendix D, Figures 55-58. Further details can be found in Appendix E.

Table 5.1 Summary of culvert sizing's at each location

Culvert ID	Culvert Size (mm)	Number of Culverts	Culvert length
CD0007	900	5	15
CD0041	1650	4	29

Culvert ID	Culvert Size (mm)	Number of Culverts	Culvert length
CD0103	1800	5	30
CD0129	1950	1	31
CD0167	900	6	18
CD0219	1950	2	52
CD0264	1800	2	55
CD0317	900	6	24
CD0360	1200	4	22
CD0505	900	11	12
CD0556	1950	1	31
CD0656	900	12	16
CD0807	1050	4	34
CD0823	900	7	18
CD0950	1800	1	41
CD1012	900	5	14
CD1050	900	2	27
CD1230	1200	5	26
CD1348	1050	6	24
CD1378	1950	1	38
CD1446	1800	2	55
CD2024	1500	2	32
CD2051	900	3	15
CD2080	1500	1	40
CD2137	2100	1	56
CD2225	1650	1	51
CD2410	1650	2	50
CD2450	1500	1	48
CD2573	2100	1	57
CD2795	1950	1	47
CD2826	2100	2	59
CD2968	1950	2	58
CD2983N	2100	3	101
CD3050N	1350	5	31
CD3223N	1350	2	30

Culvert ID	Culvert Size (mm)	Number of Culverts	Culvert length
CD3299N	1200	1	43
CD3317N	900	1	45
CD3709N	1350	2	30
CD3952N	1200	1	61
CD4048N	1200	1	22
CD2983S	1350	3	101
CD3050S	1350	5	31
CD3364S	1350	3	73
CD3609S	900	1	48
CD6868S	900	1	53

6. References

[1] Department of Main Roads 2010, Road Drainage Manual, Queensland Department of Main Roads, Brisbane.

[2] Pilgrim, DH (ed), 1987, Australian Rainfall and Runoff, Engineers Australia, Canberra.

[3] Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains, United States Geological Survey Water-supply Paper 2339.