# Appendix 17

Hydrology and Flooding Assessment Report

# BYERWEN COAL PROJECT

# Hydrology & Flooding Assessment Report

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### Revision History

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# Acronyms and Abbreviations

| | |
|---|---|
| 1D | One dimension |
| 2D | Two dimension |
| AEP | Annual Exceedance Probability |
| AHD | Australian Height Datum |
| AR&R | Australian Rainfall and Runoff |
| ARI | Average Recurrence Interval |
| BOM | Bureau of Meteorology |
| CHPP | Coal Handling Preparation Plant |
| CL | Continuing Loss |
| DEM | Digital Elevation Model |
| DERM | Department of Environment and Resource Management |
| DNRM | Department of Natural Resources and Mines |
| DTM | Digital Terrain Model |
| EA | Environmental Authority |
| EIS | Environmental Impact Statement |
| GS | Gauging Station |
| Ha | Hectare (s) |
| IFD | Intensity Frequency Duration |
| IL | Initial Loss |
| Km | Kilometres (s) |
| $km^2$ | Square kilometres |
| L | Litre (s) |
| L/s | Litres per second |
| M | Metre (s) |
| m/s | Metres per second |
| m3/s | Cubic metres per second |
| ML | Mega litres |
| ML/a | Mega litres per annum |
| Mm | Millimetre (s) |
| Mm/hr | Millimetres per hour |
| Mt | Million tonnes |
| Mtpa | Million tonnes per annum |
| $N/m^2$ | Newtons per metre squared (Shear Stress) |
| PMF | Probable Maximum Flood |
| PMP | Probable Maximum Precipitation |
| Q | Peak flow rate resulting from storm ARI of Y years |
| RL | Raised Level |
| ROM | Run of Mine |
| S | Slope |
| TSS | Total Suspended Solids |
| u/s | Upstream |
| WL | Water Level |
| $W/m^2$ | Watt per square metre |

# 1 Introduction

This report presents results of a flood assessment that was conducted to determine the impact of the QCoal Byerwen Coal Project (the Project) on flooding in the Project area necessary to inform a Water Management Plan that will satisfy EIS requirements.

The construction of waste rock dumps (WRD), mine industrial areas (MIA), coal handling and preparation plants (CHPP) and diversion works could potentially influence the existing creek geomorphology and riparian habitats of the Project area. Detailed hydraulic investigations were undertaken to assess the potential impacts of these changes and to identify required mitigation measures. Potential impacts such as changes to flood levels and changes to flow velocities were also assessed.

This report is structured as follows:

- Section 2 describes the Project area
- Section 3 describes the scope of work and methodology
- Section 4 presents the site hydrological assessment
- Section 5 presents the setup of the hydraulic flood model
- Section 6 presents the flood hydraulic assessment to determine design flood levels, flood extents and flow velocities for the existing conditions and proposed mine
- Section 7 presents the assessment of flow diversions around mine pits
- Section 8 summarises the conclusions of the study
- Section 9 presents the references.

Sections 2 to 6 examine the potential for flooding at the proposed mine from regional flood events and the impacts of the site on existing infrastructure, whilst Section 7 examines local drainage and stream diversions required to access coal reserves and prevent mine flooding.

# 2 Study area

The Project is located approximately 140 km west of Mackay in the Northern Bowen Basin, approximately 30 km north-west of the town of Glenden (see Figure 2.1). The Project involves the development of a coal mine at a ‘greenfield’ site close to the catchment divide between the Suttor River and Bowen River catchments.

The principal seams to be mined are the Goonyella upper and middle, with all open-cut coal hauled by truck to the main deposit area for processing at the either the southern or northern CHPP.

Open-cut and underground coal mining activity already exists in the area with the adjacent Xstrata Newlands Coal project.

## 2.1 EXISTING DRAINAGE NETWORK

The Project is located in the headwaters of the Suttor River catchment which is a tributary of the Burdekin River. The Suttor River catchment covers an area of approximately 65,000 $km^2$ although the catchment area upstream of the proposed mine site is around 900 $km^2$ on the Suttor River and 750 $km^2$ on Suttor Creek.

The Suttor River is an ephemeral watercourse although seasonal waterholes do exist. The catchment upstream of the proposed development is joined by Suttor Creek to the south of the Project. The Suttor River continues in a westerly direction and is joined by the Belyando River and then Rosetta Creek before flowing into the Burdekin Falls Dam impoundment.

The section of Suttor River within the Project site has waterholes and a great deal of woody debris from fallen trees. There is also a natural palustrine wetland on the western side of the Project, between the Suttor River and the Project (in the floodplain of the Suttor River).

Other waterways within the Project area include two unnamed tributaries that drain west to the Suttor River. These tributaries are ephemeral and do not hold permanent water. The southern extent of the Project area also includes a small section of the Suttor Creek floodplain.

Figure 2.2 shows the location of the upper Suttor River subcatchment and relevant tributaries in relation to the Project area, as well as Suttor Creek.

No works are proposed in the Suttor River although disturbance of two tributaries is required within the Project area for open-cut mining in the Byerwen South Pit, West Pit and East Pit. The southern tributary is defined as a watercourse under the *Water Act 2000*, while the northern tributary is not. A corridor is proposed between East Pit 1 and East Pit 2 to avoid the need for a diversion in the upper part of the southern tributary. No works are proposed in Suttor Creek.

## 2.2 POTENTIAL IMPACTS OF MINING OPERATIONS

The potential impacts to surface water and flooding behaviour are as follows:

- The proposed locations of the open-cut pits and waste rock emplacements for the Project are outside the banks of the Suttor River main channel. However, the principal area of concern regarding flooding would be the southern end of the proposed development where waste rock emplacements could encroach into the Suttor River floodplain which may impact surrounding properties and infrastructure.
- Disturbance of two tributaries is required within the Project area for open-cut mining in the Byerwen West Pit 1, South Pit 1 and East Pit 2. The southern tributary is defined as a watercourse under the Water Act 2000, while the northern tributary is not. A corridor is proposed between the Byerwen East Pit 1 and 2 to avoid the need for a diversion in the upper part of the southern tributary.
- The northern part of West Pit and the Southern MIA and CHPP follows the catchment divide between the Suttor River and Bowen River catchments. The terrain is above the Suttor River floodplain and the small gullies in these areas are not of sufficient size to warrant flood modelling for the EIS.
- The flood modelling in this report does not include the north pit, north pit diversions and northern MIA/CHPP/co-disposal. A small drainage diversion is planned to allow water to bypass the North Pit and flow to Kangaroo Creek. This drainage diversion is to be in place before mining operations commence at the North Pit. The northern MIA is located across a tributary of Kangaroo Creek and may require culverts or a bridge to provide access and protect the area. However this infrastructure is not required until late in the mine life and will be assessed later. Assessment of potential final void impacts from Kangaroo Creek is included in the Final Void Assessment, BEW106-TD-WE-REP-0006 (KBR, 2012b).
- The proposed development may obstruct flow paths for local stormwater runoff, including the identified palustrine wetland. The potential impacts on local runoff are addressed in the Mine Water Management Plan, (KBR, 2012a).

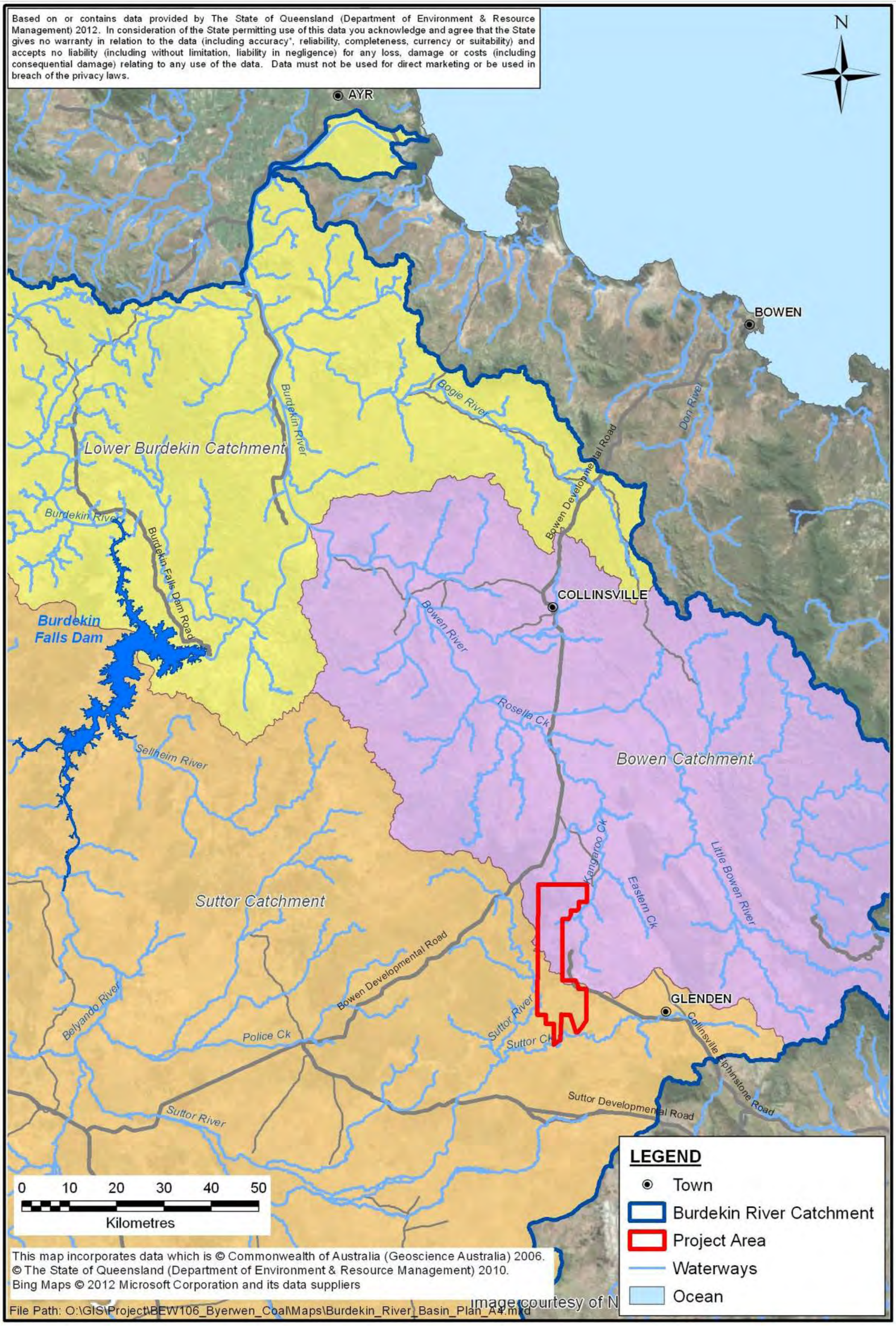


**Figure 2.1**
**PROJECT LOCATION**

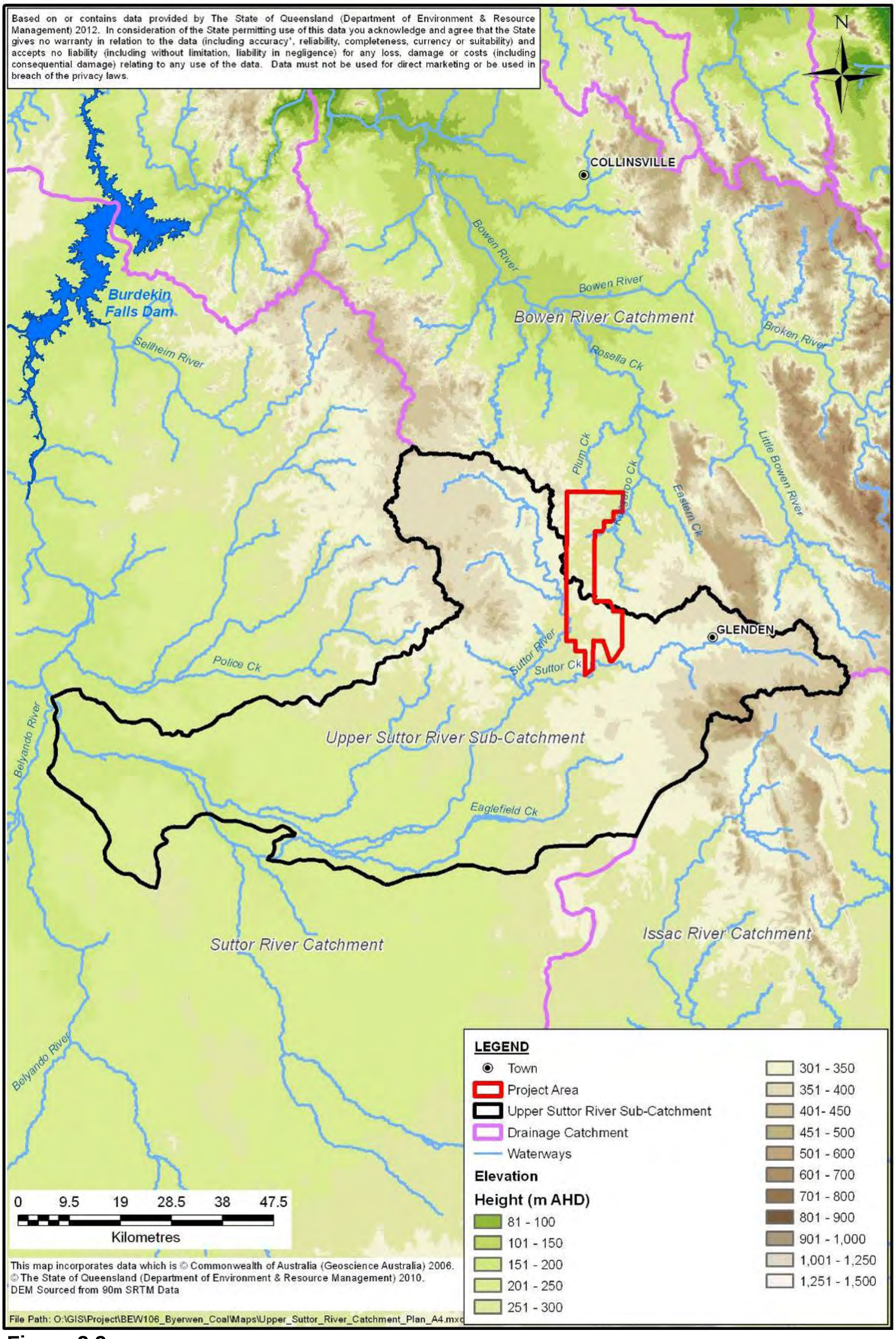


**Figure 2.2**
**UPPER SUTTOR RIVER SUBCATCHMENT**

# 3 Scope of work and methodology

## 3.1 STUDY OBJECTIVES

In response to the identified issues, the following objectives were developed for the study:

- provide input to the design of the Project, based on consideration of flooding impacts and environmental risks
- assess the potential for open pits, MIAs and CHPPs to be flooded
- assess the impacts of the Project on flood levels in the vicinity of the proposed disturbance areas and surrounding properties and infrastructure
- assess the impacts of the Project on waterway and floodplain stability
- determine flood protection levees for representative Project development stages to provide an appropriate level of mine flood immunity
- determine the need for, and impact of, diversions on flooding and channel stability.

## 3.2 SCOPE OF WORK

The scope of work included:

- A review of existing information including topographic data, previous hydrology studies and previous estimates of flood levels.
- Development of a hydrologic model to estimate historical and design average recurrence interval (ARI) flood discharges for the Suttor River and relevant tributaries.
- Development of a two-dimensional (2D) hydraulic model for the Suttor River.
- Development of a one-dimensional (1D) hydraulic model for the unnamed tributaries of Suttor River within the Project area.
- Application of the hydraulic model to assess flood levels and flow patterns for existing and future proposed floodplain conditions.
- Assessment for the need for flood protection levees based on an assessment of flood level and overtopping risk.
- An assessment of and mapping of flood level impacts.
- An assessment of the impacts of altered flow patterns, changes to flow velocity and erosive potential within the waterway channels and floodplains.

## 3.3 METHODOLOGY

The following tasks formed the methodology.

- Review of relevant reports and investigations: this task includes the collation and critical assessment of previous relevant reports and investigations undertaken in the vicinity of the Project area over the last 10 years.
- Undertake data collection and review including hydrometeorological data, survey data, flooding data and cadastral information.
- Hydrologic investigation: Estimate peak discharges in the waterways in the vicinity of the Project. Consider available methods (e.g. flood frequency analysis and rainfall-runoff hydrologic modelling) to obtain an estimate of design floods. A range of design storms were assessed including: 100 and 1,000 year ARI flood events, including Probable Maximum Flood (PMF).
- River hydraulic modelling: Develop a hydraulic model for Suttor River using SOBEK software and hydraulic models of its unnamed tributaries in the Project area using HEC-RAS software. SOBEK is a fully 2D hydrodynamic model developed by DELFT Hydraulics. The model was specifically written for the analysis of complex flow patterns in broad river floodplains and is well suited to the requirements of this study.
- Surface water impact assessment. Design floods developed from the hydrological investigation were simulated in the hydraulic models to compare pre- and post-development stages. The output from this modelling work was used to assess surface water impacts including impacts on flood levels and flow velocities.
- Levee investigations: Review the need for levees based on an assessment of the risk of overtopping and the flood heights obtained from the hydraulic model.
- Diversions: assess the need for, and impact of, diversions on channel geomorphology and stability.

# 4 Hydrology

Hydrological analysis provides the design flood information required for input to the hydraulic model and the flood impact assessment. The objective of the hydrological analysis is to calculate the design floods for a range of required probabilities.

A design flood is a theoretically derived ‘flood’ (or a time series of sequential flow rates) which has a certain likelihood of occurrence, expressed as an annual recurrence interval (ARI) or annual exceedance probability (AEP). A design flood is distinct from a ‘historical flood’ which is a time series of sequential flow rates from an event which has actually occurred in the past. Design floods are used in a hydraulic analysis to estimate flood levels or flow velocities in an event with a defined likelihood of occurrence in the future. The design floods are an integral part of the information used as the basis for all subsequent impact assessments.

Note also that the term ‘design discharge’ is used in conjunction with the term design flood. Discharge is the instantaneous flow rate at a given time within the design flood. The design discharge typically is used to refer to the peak discharge in a design flood.

For additional clarification it is noted that a ‘hydrologic’ analysis is undertaken to estimate flood flows along a river system. This is then the principal input to a ‘hydraulic’ analysis which uses these flows to estimate flood levels and flow velocities. The outcomes from the hydraulic analysis are used to assess the potential impacts of the Project.

Typically there are two possible methods of hydrologic analysis for the estimation of design flood discharges in a river system: flood frequency analysis or rainfall runoff modelling. Accurate flood frequency analysis requires a long period of representative and good quality recorded stream flow data. Such data is available for Suttor River not far downstream of the Project area at the Eaglefield Gauge Station.

The alternative method of rainfall runoff modelling has some distinct advantages and disadvantages when compared with the flood frequency approach. These issues along with the adopted methods for mitigating any shortcomings are discussed below.

The methodology which was adopted for the hydrological analysis includes the following main components:

- produce a hydrologic rainfall-runoff model of the catchments affected by the Project
- verify the model to known historical events, flood frequency analysis and/or previous study results if data are available

- simulate design rainfall for a range of probabilities up to the 1,000 year ARI event, including the Probable Maximum Precipitation (PMP) event
- adopt appropriate design flood hydrographs for input into the hydraulic model.

## 4.1 HYDROLOGIC MODEL

### 4.1.1 Model description

As part of this study, a hydrological model of the Suttor River was developed in XP-RAFTS down to the Eaglefield Gauge Station (DNRM Gauge No. 120304A) on the Suttor River. XP-RAFTS uses the Laurenson non-linear runoff-routing method to generate hydrographs from actual rainfall or design rainfall.

The primary objective of the hydrologic modelling of the Suttor River catchment was to gain an understanding of the flood behaviour of the catchment under varying storm durations and to derive the design flood hydrographs for input into the 1D and 2D hydraulic models.

The hydrologic model was calibrated using recorded rainfall and streamflow data for the March 1988, February 2008 and March 2012 flood events. The calibrated hydrologic model was then used to predict the flooding expected to be generated over the Suttor River catchment for the 100 year, 1,000 year and PMF design events.

### 4.1.2 Catchment delineation

Catchment boundaries for the Suttor River hydrologic model were determined in CatchmentSIM using SRTM data. CatchmentSIM is a stand-alone GIS based terrain analysis program designed to assist in the setup of hydrologic models. The program automatically delineates catchment boundaries and calculates spatial and topographic characteristics of the catchment to estimate relevant hydrologic modelling parameters. These includes subcatchment areas, equivalent slopes and lag times.

The Suttor River catchment upstream of the Eaglefield Gauge was subdivided into 85 subcatchments as shown in Figure 4.1. The delineation was based on the topography and points of interest in the project area.

### 4.1.3 Catchment roughness values

The catchment roughness parameter in the XP-RAFTS model was set globally to 0.07. This is considered an appropriate value for the Manning’s ‘n’ roughness parameter for rural areas and was considered to provide an acceptable representation of roughness of the catchments in the Project area.

Note that this value represents the natural catchment characteristics and areas that have existing mine pits and operational plant may not be represented accurately. However for the purpose of this report, this global assumption is adequate since the areas of interest around the proposed open-cut mining pits are largely undisturbed.

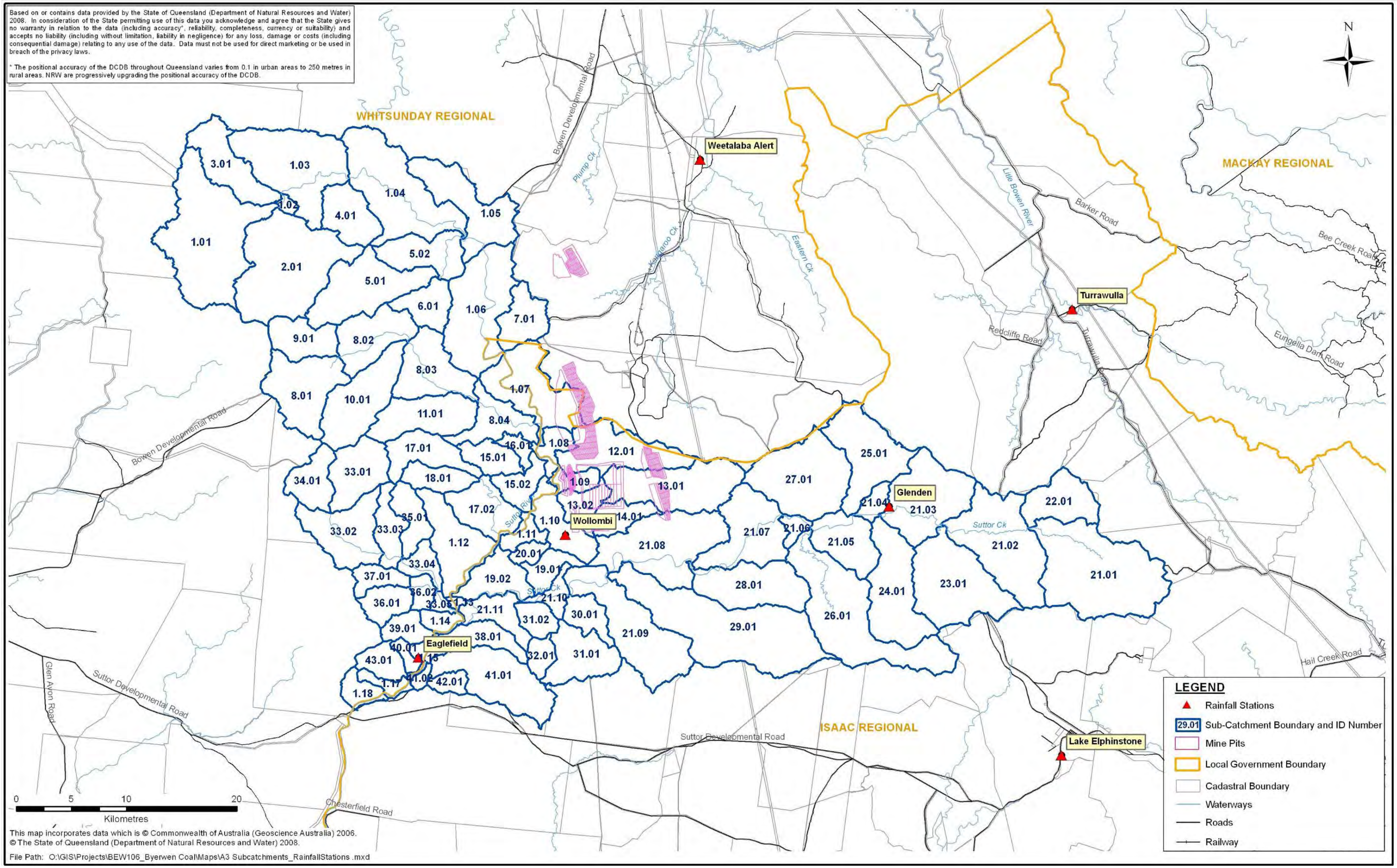


**Figure 4.1**
**SUTTOR RIVER CATCHMENT**

## 4.2 CALIBRATION OF MODEL

In order to use the XP-RAFTS model for catchments in the Project area with some level of confidence, it was necessary to calibrate and/or verify results against historical flood events and/or previous flood peak estimates.

There are sufficient rainfall gauging stations within the catchment and a stream gauge is located a short distance downstream of the Project area which enabled calibration and verification of the model to be undertaken.

The streamflow gauge records at the Eaglefield Gauge Station (DNRM Gauge No. 120304A) on the Suttor River were used as the basis for calibration of the hydrologic model. The gauge site is located approximately 7 km downstream from the junction of Suttor River and Suttor Creek and the total catchment area at the gauging site is approximately 2,000 $km^2$. There were no other streamflow gauging stations located within the hydrological model extent.

### 4.2.1 Historical rainfall data

Daily rainfall data and pluviographic data were collected from the Bureau of Meteorology (BOM) for the March 1988, February 2008 and March 2012 events. The locations and details of the daily and pluviographic stations are shown in Figure 4.1 and Table 4.1.

**Table 4.1 Daily and pluviographic stations**

| Station Name | Station No. | Latitude | Longitude | Source | Type | Calibration Year |
|---|---|---|---|---|---|---|
| Glenden | 34064 | 21.326 S | 148.094 E | BOM | Daily | 1988 |
| Wollombi | 34020 | 21.35 S | 147.83 E | BOM | Daily | 1988 & 2008 |
| Weetalaba Alert | 533081 | 21.044 S | 147.94 E | BOM | Pluvio | 2008 & 2012 |
| Turrawulla | 33169 | 21.166 S | 148.244 E | BOM | Daily | 1988, 2008, 2012 |
| Lake Elphinstone | 34077 | 21.529 S | 148.235 E | BOM | Daily | 1988 |
| Eaglefield | 120304A | 21.45 S | 147.71 E | DNRM | Daily | 2012 |

As shown in Table 4.1, data at five rainfall stations used for the calibration events was available as daily rainfall totals. The only pluviographic rainfall data available was at Weetalaba Alert (Stn No. 533081) which is located outside the Suttor River catchment area.

The distribution of the BOM's daily rainfall stations provided a reasonably good definition of the spatial variation in rainfall depths in the middle and lower parts of the Suttor River catchment. However, the absence of the BOM's daily rainfall gauges in the upper parts of the catchment mean that storm multipliers are required to increase the local rainfall intensity to represent the significant rainfall gradient expected from orographic effects in these areas.

### 4.2.2 RAFTS model runoff routing parameters

Routing for individual subcatchments in XP-RAFTS is carried out using the Muskingum method. The storage within each subcatchment is a non-linear function of the discharge from that subcatchment, with the default value of the storage coefficient model parameter ‘B’ based on a regression equation developed for urban catchments.

The default value of ‘B’ can be adjusted by changing the impervious area fraction and surface roughness of the pervious area (PERN). In addition, XP-RAFTS also allows the universal adjustment of all the calculated ‘B’ parameter values by means of an overall multiplier ‘BX’.

For the current study, the surface roughness of the pervious area of each subcatchment was chosen based on the catchment characteristics and was not changed during the calibration of the hydrological model for the three storm events. The default value of ‘B’ was then adjusted using the overall multiplier ‘BX’ = 0.70 which was used for the design event simulations.

### 4.2.3 Rainfall losses

The initial loss (IL)/continuing (CL) loss model was adopted for all catchments. This method requires an estimate of the loss to simulate the initial wetting of the catchment when no runoff is generated. A constant CL accounts for infiltration once the IL process is complete. Australian Rainfall and Runoff (AR&R) recommends a value between 0 and 35 mm for IL and 2.5 mm/hr for CL when simulating design storms in Queensland.

Rainfall loss parameters derived during the calibration process for the three historical events are shown below in Table 4.2.

**Table 4.2 Rainfall losses for historical storm events**

| Storm Event | Rainfall Losses | | Peak Flow rates ($m^3/s$) | |
|---|---|---|---|---|
| | Initial (mm) | Continuous (mm/hr) | Eaglefield gauge | Calculated (RAFTS) |
| March 1988 | 5.0 | 2.0 | 1596.5 | 1590.5 |
| Feb 2008 | 15.0 | 3.0 | 1040.5 | 1041.7 |
| March 2012 | 15.0 | 3.5 | 935.3 | 944.7 |

The adopted loss parameters applied to the design events of Suttor River hydrologic model are summarised in Table 4.3. A conservative estimate of the initial loss was used for the design events.

**Table 4.3 Rainfall losses for design events**

| ARI (years) | Initial loss (mm) | Continuing loss (mm/h) |
|---|---|---|
| 100 | 0 | 2.5 |
| 1,000 | 0 | 2.5 |
| PMF | 0 | 2.5 |

### 4.2.4 Calibration discussion

The following discussion summarises each of the storm events and how the rainfall data was used as input to the hydrologic model. A summary of the comparison between the Eaglefield gauge on the Suttor River and the XP-RAFTS model results is also provided.

#### March 1988 event

Daily rainfall data at Glenden, Wollombi and Lake Elphinstone were used for modelling this event. Data at Turrawulla station was not used due to the poor quality of data for this storm event. Wollombi station is located in the middle of the catchment while Glenden station is located in the upper Suttor Creek catchment. Lake Elphinstone is outside the catchment on the south-eastern side.

The highest rainfall at these rain gauges for the storm event occurred on 2 March 1988. As there are no rain gauges in the upper Suttor River catchment, a storm multiplier of 1.1 was used to increase the rainfall intensity in the north-western part of the catchment to account for expected orographic effects as discussed in Section 4.4.1.

#### February 2008 event

Pluviograhpic rainfall data at Weetalaba station and daily rainfall data at Turrawulla station was used for this event. Wollombi station was not used due to the poor quality of data for this storm event.

The Weetalaba rain gauge is located outside the Suttor River catchment to the north and the highest rainfall at this gauge occurred on 11 and 12 February. Turrawulla station is also located outside the catchment to the north east and the max rainfall at this station occurred slightly later on 12 and 13 February.

#### March 2012 event

Pluviograhpic rainfall data at Weetalaba station was combined with daily rainfall data at Turrawulla and Eaglefield stations for this storm event. The Eaglefield gauge is located at the downstream end of the Suttor River hydrologic model. The highest rainfall at Eaglefield station occurred on 18 and 19 March whereas at Weetalaba and Turrawulla stations the peak was recorded later on 20 and 21 March.

#### Results comparison

The simulated flood hydrograph from the XP-RAFTS model was compared with the recorded discharge at Eaglefield river gauge station on the Suttor River (DERM Gauge No. 120304A) for the March 1988, February 2008 and March 2012 flood events and is shown in Figures 4.2 to 4.4.

The figures show that the calibrated XP-RAFTS model matches the peak flows at Eaglefield gauge very well. The rising and falling limbs of the main hydrograph predicted by the model are also satisfactory given that most of the rainfall data is recorded daily.

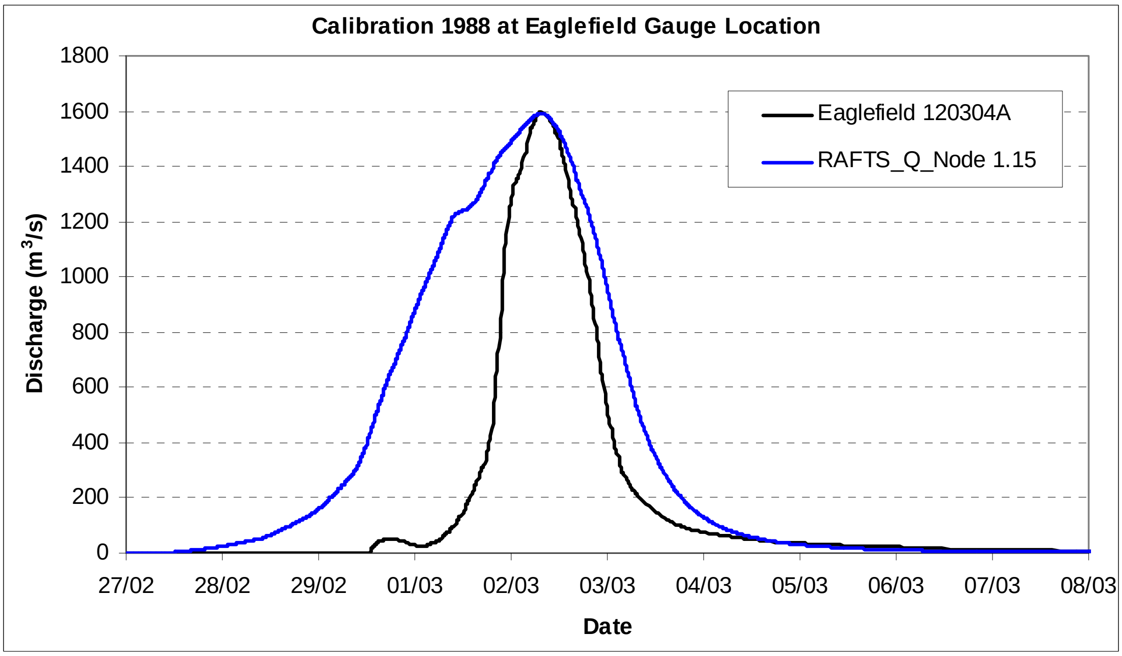


**Figure 4.2**
**CALIBRATION FOR MARCH 1988 EVENT**

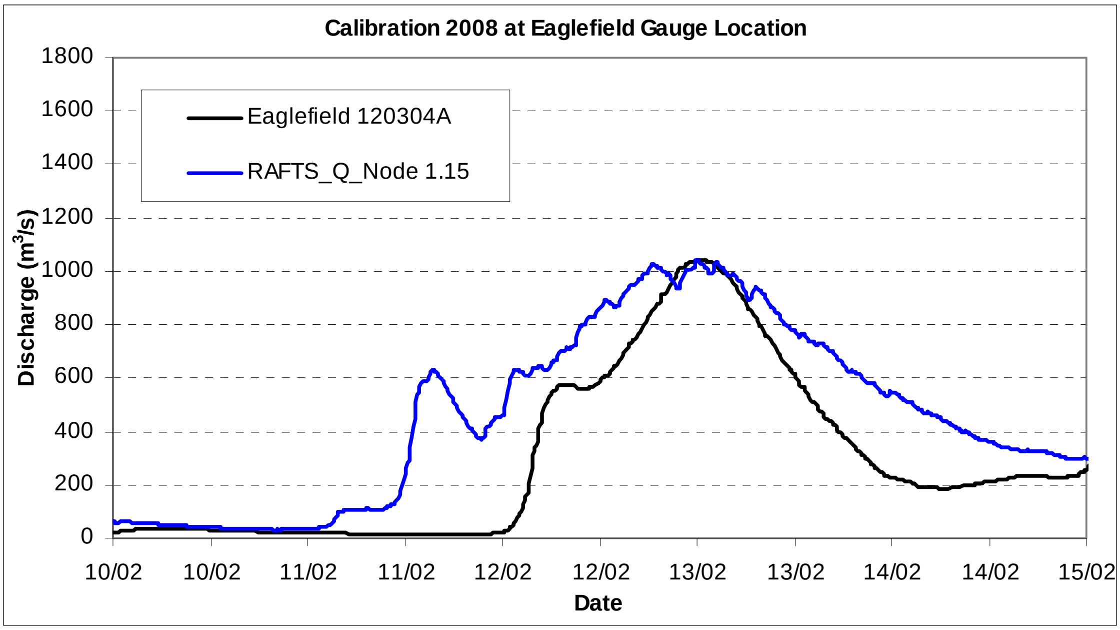


**Figure 4.3**
**CALIBRATION FOR FEBRUARY 2008 EVENT**

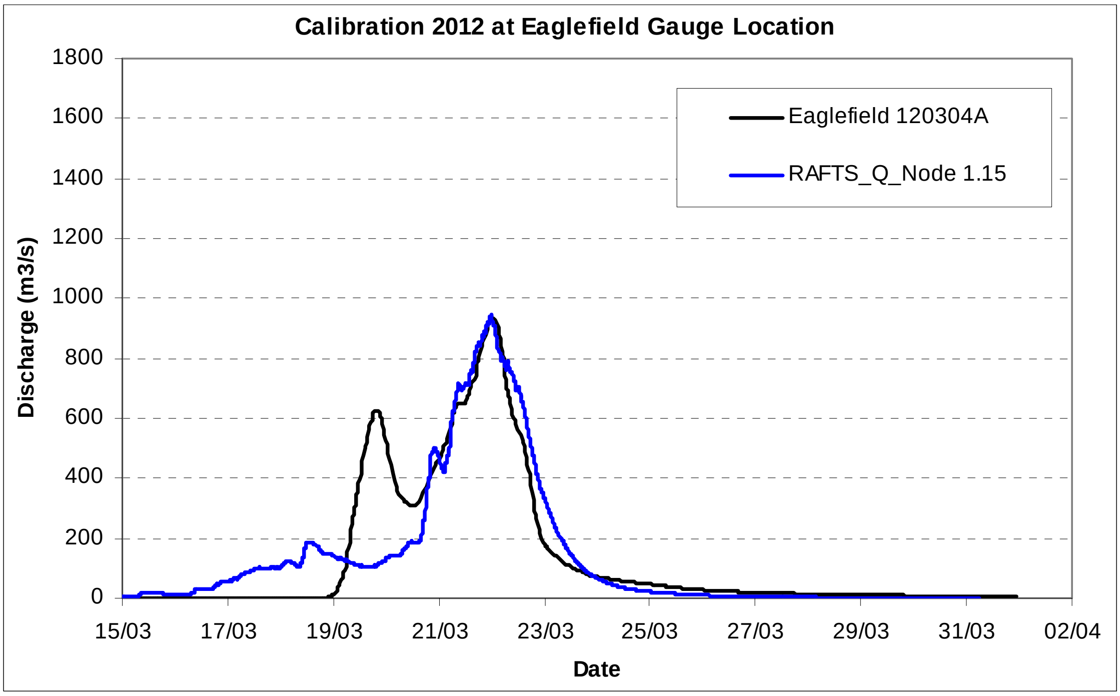


**Figure 4.4**
**CALIBRATION FOR MARCH 2012 EVENT**

## 4.3 DESIGN EVENTS

For this project, the 100 and 1,000 year ARI rainfall depths together with the PMP were derived. Design flood hydrology has been undertaken in accordance with AR&R guidelines and the PMP estimation technique from the BOM.

### 4.3.1 Design rainfall

Rainfall intensities determined by the CRC-FORGE method were compared with those derived using the BOM web based Intensity Frequency Durations (IFDs). The CRC-FORGE rainfall intensities were adopted as they have been developed using more recent rainfall data and are generally considered to give better estimates. The temporal patterns adopted were those for Region 3 (ARR, 2001).

CRC-FORGE also provides an estimate of areal reduction factors and improved estimates for events greater than the 100 year ARI event. Sensitivity analysis was carried out to represent the estimated impacts of future climate change by increasing peak rainfall intensities increased by 20% as recommended in the Final report on the Inland Flooding Study (DERM, 2010). Table 4.4 provides the rainfall intensities for the Suttor River catchment based on the CRC-FORGE method.

**Table 4.4 Design rainfall intensities for Suttor River catchment**

| Duration (hrs) | Intensity (mm/hr) | | |
|---|---|---|---|
| | 100 year | 100 year with climate change | 1,000 year |
| 3.0 | 37.43 | 44.92 | 55.59 |
| 6.0 | 23.14 | 27.77 | 34.37 |
| 12 | 14.34 | 17.21 | 21.30 |
| 18 | 11.20 | 13.44 | 16.64 |
| 24 | 9.38 | 11.25 | 13.93 |
| 48 | 6.23 | 7.48 | 9.01 |
| 72 | 4.69 | 5.63 | 6.84 |

### 4.3.2 Probable Maximum Precipitation

The PMP event estimates are based on procedures developed by the BOM. Probable Maximum Precipitation is defined by the World Meteorological Organization (1986) as 'the greatest depth of precipitation for a given duration meteorologically possible for a given size storm area at a particular location at a particular time of year'.

There are two methods available for estimating PMP depths in tropical zones:

- Generalised Short Duration Method – GSDM (BOM, 2003): Method for estimating PMP depths for storm durations up to 6 hours.
- Generalised Tropical Storm Method – GTSMR (BOM, 2003): Method for estimating PMP depths for storm durations greater than 6 hours and up to 120 hours.

Due to the relatively large size of the catchment, the critical duration of the PMP was greater than 6hrs and rainfall estimates using the GSDM method were not calculated.

The GTSMR method involves the determination of an initial PMP depth based on catchment area and storm duration which is adjusted to account for site specific influences such as topography, elevation and moisture availability.

The GTSMR adjustment parameters determined for the Suttor River catchment are shown in Table 4.5. Results from the hydrologic model indicate a critical duration of 24 hours for the PMP event.

**Table 4.5 GTSMR adjustment parameters**

| Parameter | Value |
|---|---|
| Topographic Adjustment Factor (TAF) | 1.0 |
| Decay Amplitude factor (DAF) | 1.0 |
| Moisture Adjustment Factor (MAF) | 0.78 |

The final PMP estimates for Suttor River catchment area are summarised in Table 4.6. These are point estimates and apply for catchment areas up to 150,000 km$^2$, which are appropriate for the catchment area being assessed.

**Table 4.6 GTSMR estimate of PMP rainfall**

| Storm Duration (hours) | PMP Estimate | |
|---|---|---|
| | Depth (mm) | Intensity (mm/hr) |
| 24 | 970 | 40.42 |
| 36 | 1,140 | 31.67 |
| 48 | 1,310 | 27.29 |
| 72 | 1,610 | 22.36 |

## 4.4 VERIFICATION OF MODEL

### 4.4.1 Flood Frequency Analysis

Peak discharge data at the Eaglefield Gauge Station (DERM Gauge No. 120304A) on the Suttor River has been collected and includes a 46 year recording period from 1967 to 2012. Flood frequency analysis was carried out using the streamflow records to validate the RAFTS model predictions for different design events.

A Weibull probability distribution was used for the frequency analysis as it is an appropriate distribution for analysing annual peak flows. The Weibull distribution was fitted against the ranked annual peak flows measured at the Eaglefield gauge. Confidence limits for the FFA analysis were derived based on the binomial distribution to indicate the confidence of the analysis.

After an initial analysis, the five lowest ranking peak annual flows were excluded from the Weibull analysis so that the probability distribution curve more accurately fitted the larger records. Table 4.7 shows the design discharge at Eaglefield Gauge station for different return periods and the fitted Weibull distribution is presented in Figure 4.5.

It is noted that large extrapolation of the fitted curve derived from a flood frequency analysis is not recommended by ARR. According to Book VI of ARR, the 100 year ARI flood is the largest event that should be estimated by direct frequency analysis for major works. The maximum flood that should be estimated by this means under any circumstances is the 500 year ARI event. Note that this does not invalidate the XP-RAFTS estimates for 1:1,000 ARI and PMF flood events, but that these events should not be compared with against FFA estimates.

**Table 4.7 Frequency analysis at Eaglefield Gauge**

| ARI (Years) | 90% confidence (Upper limit) | Weibull Prediction | 90% confidence (Lower limit) |
|---|---|---|---|
| 2 | 419 | 256 | 157 |
| 5 | 890 | 648 | 471 |
| 10 | 1,322 | 964 | 703 |
| 20 | 1,810 | 1,293 | 924 |
| 50 | 2,526 | 1,742 | 1,202 |
| 100 | 3,113 | 2,091 | 1,404 |
| 200 | 3,734 | 2,446 | 1,602 |
| 500 | 4,602 | 2,924 | 1,858 |
| 1,000 | 5,292 | 3,292 | 2,048 |

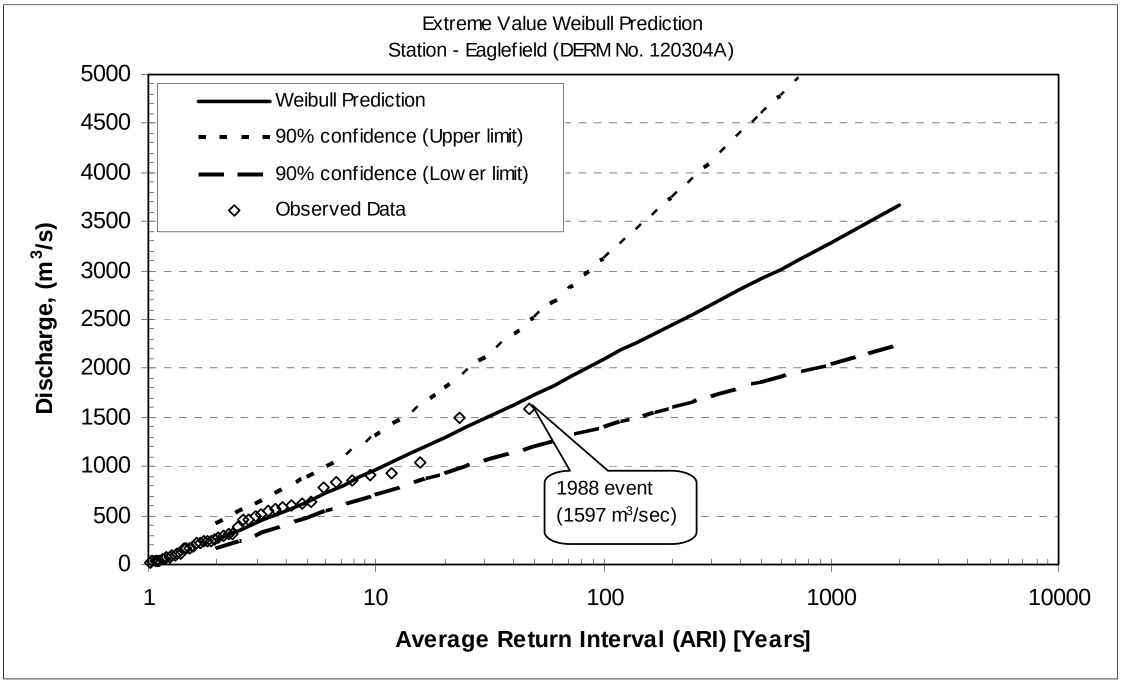


**Figure 4.5**
**WEIBULL PROBABILITY DISTRIBUTION FOR EAGLEFIELD GAUGE ON SUTTOR RIVER**

### 4.4.2 Verification of XP-RAFTS model prediction

The calibrated Suttor River XP-RAFTS model was verified against the flood frequency analysis for data recorded at the Eaglefield gauge for the 100 year and 200 year ARI design events. This verification has been conducted to confirm that the adopted XP-RAFTS parameters are consistent with the regional catchment response to rainfall.

The 24 hour design storm was found to be critical for the Suttor River catchment at Eaglefield gauge location and Table 4.8 provides the comparison of the XP-RAFTS prediction against the flood frequency analysis.

**Table 4.8 Model comparison against the Flood Frequency Analysis**

| | Peak Discharge ($m^3$/s) | |
|---|---|---|
| ARI (years) | XP-RAFTS Model | Flood Frequency Analysis |
| 100 | 2072 | 2091 |
| 200 | 2461 | 2446 |

The table shows that the XP-RAFTS parameters from the calibrated model produce design event discharges very similar to the flood frequency analysis. The agreement between flood peaks of the XP-RAFTS model and the regional flood frequency analysis allows greater confidence to be placed in the calibrated hydrologic model.

## 4.5 DESIGN DISCHARGES

The previous sections have presented the development and calibration of the Suttor River XP-RAFTS hydrologic model. The model parameters have been adjusted through this process and results compare well with the historic flood events and the Flood Frequency Analysis.

The design flood peaks from XP-RAFTS using the CRC-FORGE rainfall data have been estimated and a summary of the results at relevant sites in the Project area are shown in Table 4.9.

**Table 4.9 Summary of design flood peaks for Suttor River**

| Watercourse | Location | Flood Peak ($m^3/s$) | | |
|---|---|---|---|---|
| | | 100 yr ARI | 1,000 yr ARI | PMF |
| Suttor River | Upstream boundary of SOBEK model | 856 | 1,425 | 4,950 |
| Rockingham Creek | Western tributary | 515 | 875 | 2,502 |
| Northern U/T | Upstream of NML rail crossing | 73 | 131 | 408 |
| Southern U/T | Upstream of NML rail crossing | 136 | 238 | 727 |

# 5 Suttor River hydraulics

The Project has the potential to cause changes to flood extents, depths and velocities. Flood models were established to predict the:

- impact of mine pits and waste rock dumps on surrounding property infrastructure
- proximity of floodwaters to waste rock dumps and residual voids.

A 2D hydrodynamic model (SOBEK) was used for the Suttor River to assess the impact of mine pits and waste rock dumps on flooding behaviour. The model is a fully 2D hydrodynamic suite developed by Delft Hydraulics. The suite was specifically written for the analysis of complex flow patterns in broad river floodplains and is very well suited to assessing the effects of subsidence.

The flood model covers all areas of the mine at risk from regional flooding of the Suttor River. Therefore it does not include the North Pit or South Pit 2. Previous investigations for the GAP Newlands Rail Line indicate that the Byerwen Coal Mine is not at risk of flooding from Suttor Creek.

The 100 year ARI, 1,000 year ARI and PMF design flood events were assessed for this study and a summary is provided in Table 5.1. The design pit immunity stipulated for this Project was the 1,000 year ARI flood event.

Figure 5.1 shows the extent of the hydraulic model established for the Suttor River. The model extent has been optimised to include all floodplain area within the PMF extent.

**Table 5.1 Summary of flood and drainage assessment for each investigation area**

| Study Area Location | | Suttor River | Diversions |
|---|---|---|---|
| Model | 1D | | Y |
| | 2D | Y | |
| Assessment | Q2 | | Y |
| | Q10 | | Y |
| | Q50 | | Y |
| | Q100 | Y | Y |
| | Q1,000 | Y | Y |
| | PMF | Y | |

*Note Diversions are discussed in Section 7*

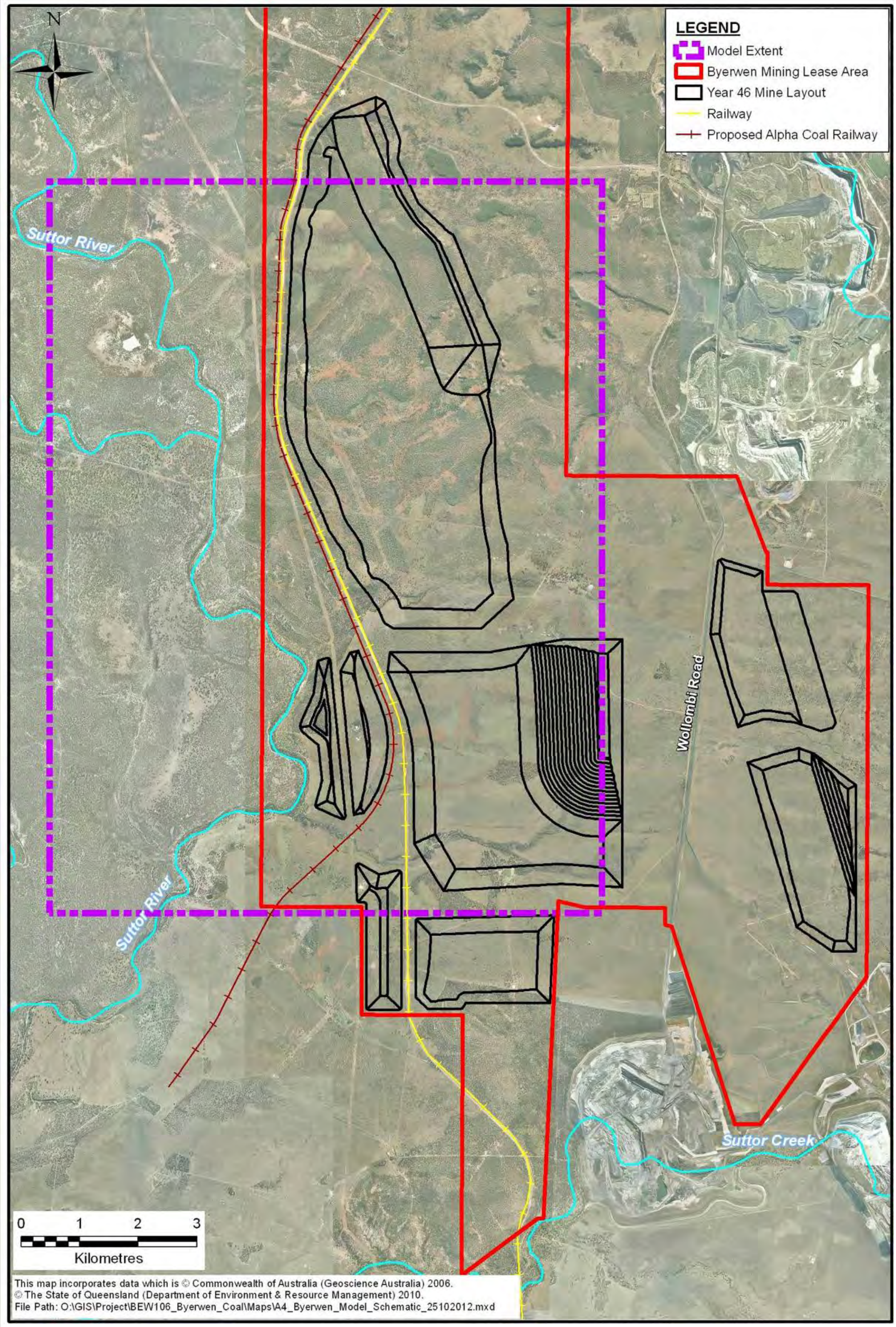


**Figure 5.1**
**FLOOD MODEL EXTENT**

A visit to the Project site was undertaken on 29 September 2012 to inspect the area to be included in the flood model and confirm details regarding hydraulic roughness, debris loadings and to inspect key hydraulic structures.

## 5.1 TOPOGRAPHY

Elevation data used in this study was sourced from the proponent and is in the form of 1 m contours which were converted into a 15 m digital elevation model (DEM) for the study area using the civil design software 12D. The size of the DEM is considered appropriate to capture key hydraulic features including flow in the Suttor River and its tributaries.

The Northern Missing Link (NML) railway alignment was constructed after the elevation data was captured for the study area. Details were obtained from 'as-constructed' information and the vertical alignment of the railway embankment was included in the flood model DEM.

## 5.2 ROUGHNESS

The section of Suttor River within the Project site has waterholes and a large amount of woody debris from fallen trees. Manning's 'n' roughness values for the 2D model extent were estimated based on aerial photography and notes made during the site visit. The adopted Manning's 'n' values for the Suttor River model are provided in Table 5.2.

**Table 5.2 Hydraulic roughness for Suttor River model**

| Description | Manning's value |
|---|---|
| Suttor River channel and banks | 0.05 |
| Cleared land | 0.06 |
| Light to medium vegetation | 0.07 |
| Rail embankments | 0.02 |

## 5.3 BOUNDARY CONDITIONS

Inflow hydrographs developed from the XP-RAFTS model were input at the upstream end of each waterway. These were the inflow hydrographs for the 100 year ARI, 1,000 year ARI and PMF design flood events.

A total of ten separate inflow locations were included in the model to simulate inflow from the upper Suttor River and feeding tributaries, as well as in-stream flow accumulation. These are summarised in Table 5.3 and the location of RAFTS subcatchments can be seen in Figure 4.1.

The downstream boundary condition adopted in the model for the Suttor River was a stage-flow hydrograph developed using HEC-RAS. A detailed cross section of the downstream river was taken from the DEM and input to HEC-RAS. A range of discharges from the RAFTS model were simulated in HEC-RAS to generate the rating curve which was specified as the downstream boundary in the SOBEK model.

**Table 5.3 Summary of design inflow flood peaks**

| Name | RAFTS ID | 100 year ARI | 1,000 year ARI | PMF |
|---|---|---|---|---|
| | | Total | | |
| Suttor River | S1.07 | 856 | 1,425 | 4,950 |
| Rockingham Creek | S8.04 | 516 | 875 | 2,502 |
| North U/T Suttor River | S12.01 | 73 | 131 | 408 |
| South U/T Suttor River | S13.02 | 136 | 239 | 727 |
| West 1 U/T Suttor River | S16.01 | 19 | 30 | 78 |
| West 2 U/T Suttor River | S15.01 | 48 | 78 | 200 |
| | | Local | | |
| Suttor River in stream | S15.02 | 27 | 48 | 147 |
| Suttor River in stream | S1.08 | 16 | 29 | 103 |
| Suttor River in stream | S1.09 | 27 | 44 | 114 |
| Suttor River in stream | S1.10 | 52 | 91 | 264 |

## 5.4 HYDRAULIC STRUCTURES

There are only a small number of structures within the model extent and all of them relate to the Northern Missing Link rail corridor. Table 5.4 presents the list of structures and their details. The dimensions of these structures have been estimated from measurements and photos taken during the site visit. Figure 5.2 presents a picture of one bridge, the other bridge being identical.

The main reason for including these structures in the SOBEK model is to allow flood waters from the Suttor River to pass through the NML alignment and fill the floodplain on the other side. The culverts have been modelled as 1D links in the 2D SOBEK model, whereas the two bridges are modelled as gaps in the rail embankment with higher roughness to represent the bridge piers and abutments. Invert levels for the culverts are based on the DEM cell elevations at the upstream and downstream openings.

**Figure 5.2**
**NML BRIDGE OVER UNNAMED TRIBUTARY OF THE SUTTOR RIVER**

**Table 5.4 Summary of NML hydraulic structures**

| ID | Type | Details | Invert level (upstream) (mAHD) | Length (m) |
|---|---|---|---|---|
| 1 | Bridge | 2x 20m spans | | |
| 2 | Bridge | 2x 20m spans | | |
| 3 | Culvert | 4x 0.9 RCP | 292.0 | 14.7 |
| 4 | Culvert | 1x 0.9 RCP | 293.0 | 17.6 |
| 5 | Culvert | 4x 0.9 RCP | 289.4 | 24.6 |
| 6 | Culvert | 6x 0.9 RCP | 290.3 | 24.6 |
| 7 | Culvert | 1x 2.5x2.5m RCBC | 290.8 | 29.7 |
| 8 | Culvert | 1x 0.45 RCP | 292.8 | 22.2 |

# 6 Impact assessment

## 6.1 PRE-DEVELOPMENT FLOOD EXTENTS

The Suttor River hydraulic model depth and velocity results for the existing conditions are shown in Figures 6.1 to 6.6 for the 100 year ARI, 1,000 year ARI and PMF flood events.

As expected, the flow capacity of the main Suttor River channel is exceeded in the storm events modelled and flow enters the floodplain. Relatively shallow flooding is evident in the location of the proposed Byerwen South Pit 1 and West Pit for the 1,000 year ARI event.

Flow velocities along the Suttor River for the 100 year ARI event are shown in Figure 6.2 and shows the average flow velocity in the channel is between 1.0 to 2.5 m/s within the mining lease area. Within the backwater affected tributaries the velocities are lower at around 0.5 m/s.

Figure 6.7 presents the pre-development inundation extents for the 1, 2 and 5 year ARI flood events. The figure shows that the palustrine wetland located to the west of the operational areas of the site is not inundated during these more frequent flood events. This indicates that flooding from the Suttor River is not the main source of water for the wetland. The Northern Missing Link elevated rail embankment and associated culverts is located between the wetland and the Project. Potential impacts of the Project on this area are addressed in the Mine Water Management Plan (KBR, 2012a) and the ecological component of the EIS.

## 6.2 POST-DEVELOPMENT FLOOD EXTENTS

This section discusses changes to flood extents, water levels (afflux) and areas of inundation relative to the location of mining pits and waste rock dumps that form the Byerwen Coal Mine project development scenario.

This was implemented by adjusting the DEM of the SOBEK model to include the mining pits to the east of the NML, waste rock dumps in the Suttor River floodplain and the South Pit diversion channels upstream of the NML. The catchment inflows of the Suttor River remain unchanged from the existing scenario.

Mine life scenarios at major intervals have not been assessed however it is understood that diversion channels will be constructed at the start of mine operations and therefore impacts on inflows from unnamed tributaries to the Suttor River should be negligible. Additionally, the levees that form part of the water course diversions works will be extended to protect the South Pit 1 and West Pit 1 from 1,000 year ARI regional flooding in the Suttor River at all intervals of the mine life.

The post-development flood depths and velocities are shown in Figures 6.8 to 6.13. There is no significant change in the flood extent for the post-development scenario compared to the existing case, except in the PMF event where the WRDs block some of the floodplain flow.

In the 1,000 year ARI event flood waters reach the western waste rock dump with depths up to 2.0 m and velocities in the order of 1.0 m/s adjacent to the waste rock in some locations. This will require armouring up to the 1,000 year ARI flood level (refer to Figure 6.10) such that it is non-erodible when in contact with flood waters. Alternately the toe of the spoil dump can be relocated outside the flood extent. It should be noted that the peak depth and velocities at the face of the WRD do not occur together and where the velocity is highest adjacent the WRD the depth is 0.8 m.

The diversion channel assessment (Section 7) includes levees to contain the 1,000 year ARI local catchment flows from the unnamed tributaries of the Suttor River. These levees have been included in the SOBEK model and extended to high ground at their downstream extent to completely protect the mine pits.

These levees protect the mining pits from backwater flooding in the Suttor River 1,000 year ARI event. The levee of the southern diversion channel is not overtopped for the 1:1000 ARI; however it is overtopped for the PMF. The impacts to the mine operation and in the longer term are assessed against the 1,000 year ARI event, which do not overtop the levee.

Figures 6.14 and 6.15 present the change in peak flood level (afflux) in the post-development scenario for the 100 year and 1,000 year ARI flood events respectively. The SOBEK results of the Byerwen Coal assessment indicate negligible change in peak flood levels, generally less than 0.02 m, with a localised maximum of no more than 0.2 m.

The GAP Newlands Rail Line is not overtopped in the pre-development Suttor River 100 year ARI flood event and this flood immunity is not affected by the proposed mine development. There are minor impacts at the GAP Newlands Rail Line (maximum 0.15 m for the 100 year ARI flood event) and negligible impacts at proposed Alpha Coal Project Rail Line.

The minor changes that are predicted are attributable to available floodplain storage and flow paths in the pre-development scenario being modified by the mine pits and diversion channels.

Figures 6.16 and 6.17 present the change in average velocity in the post-development scenario for the 100 year and 1,000 year ARI flood events respectively. Generally the velocity change is negligible, with a few localised areas where velocities increase by about 0.1 m/s. This occurs near the WRDs and at the downstream end of diversions near the NML alignment.

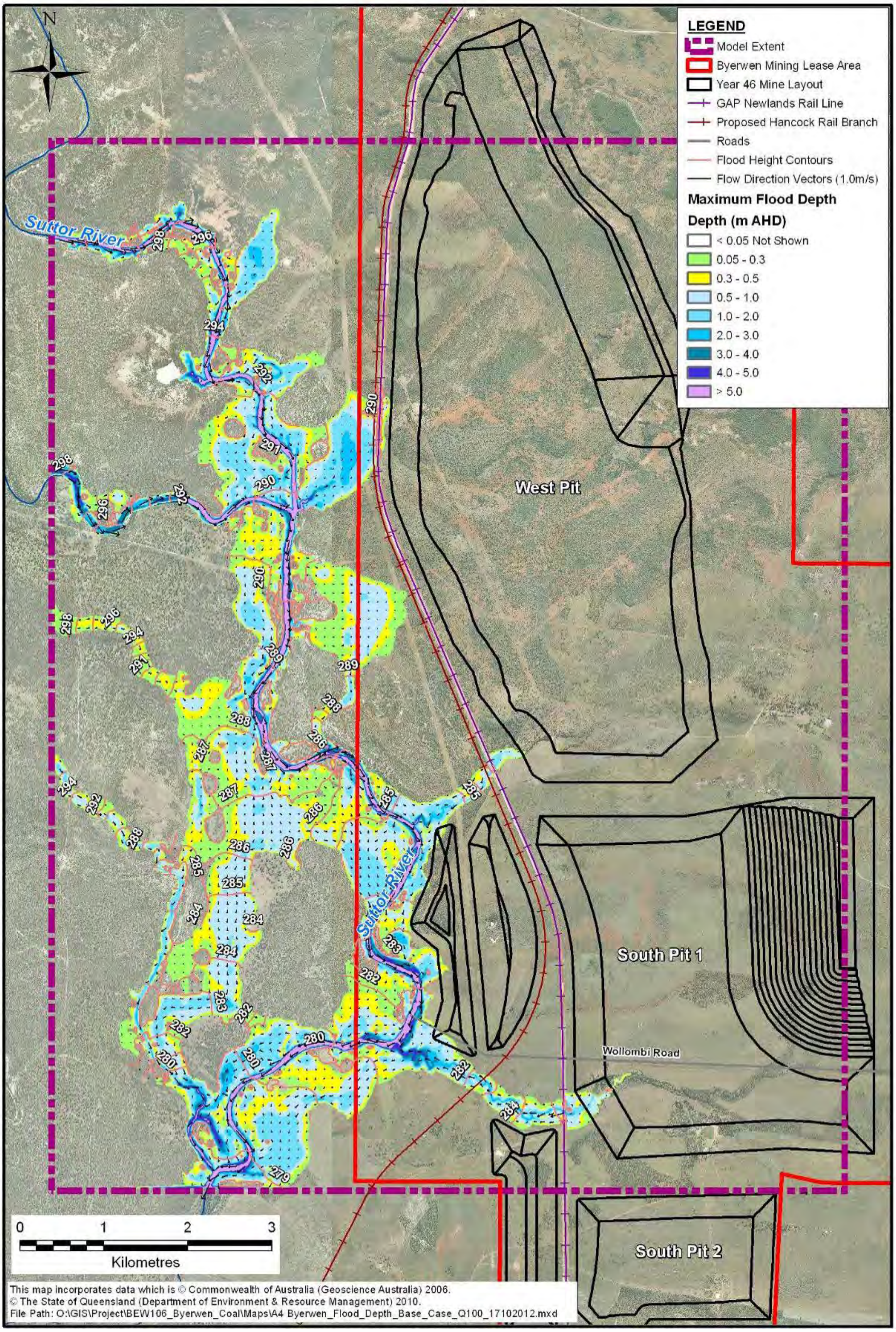


**Figure 6.1**
**SUTTOR RIVER 100 YEAR ARI FLOOD DEPTHS (PRE-DEVELOPMENT)**

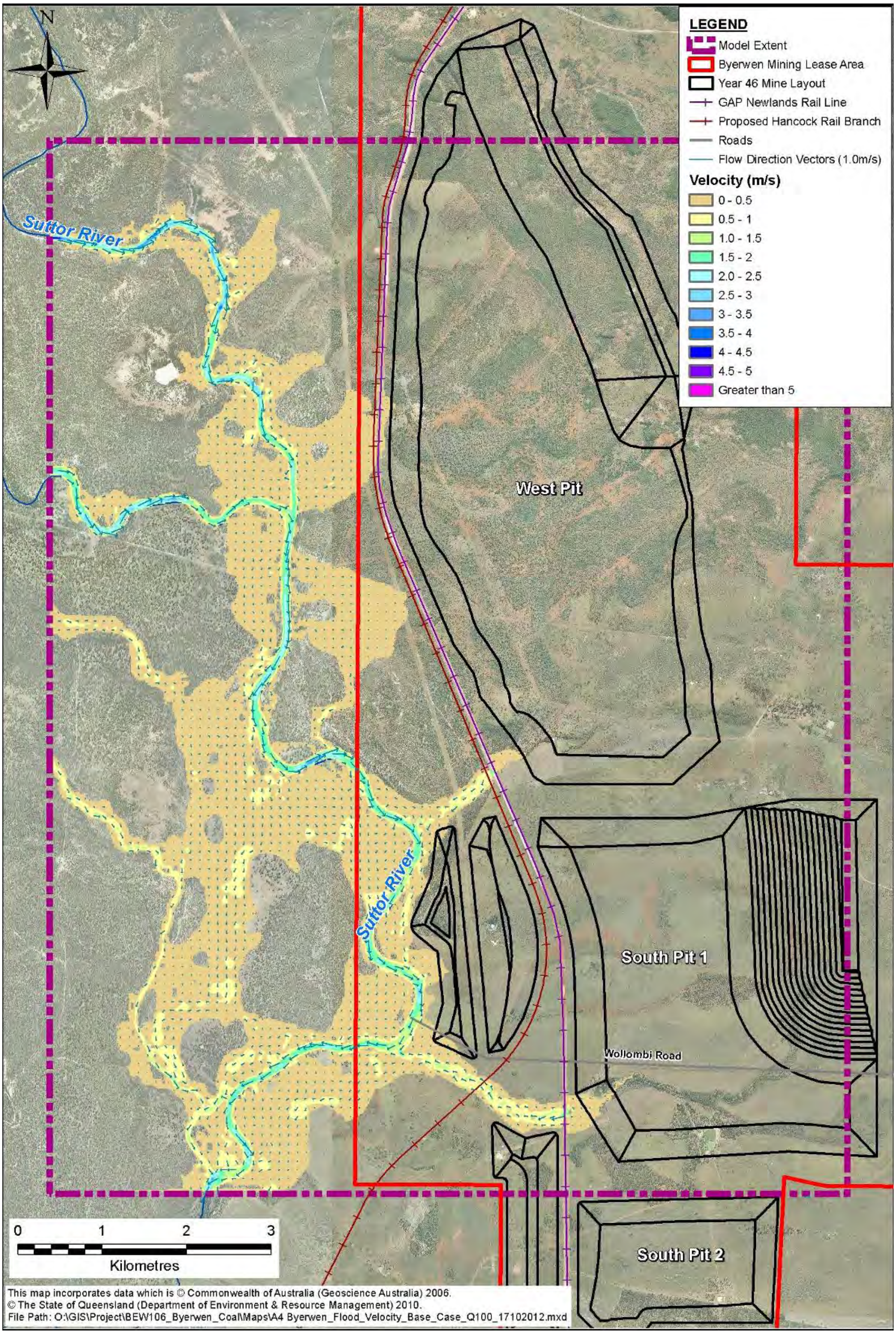


**Figure 6.2**
**SUTTOR RIVER 100 YEAR ARI FLOOD VELOCITY (PRE-DEVELOPMENT)**

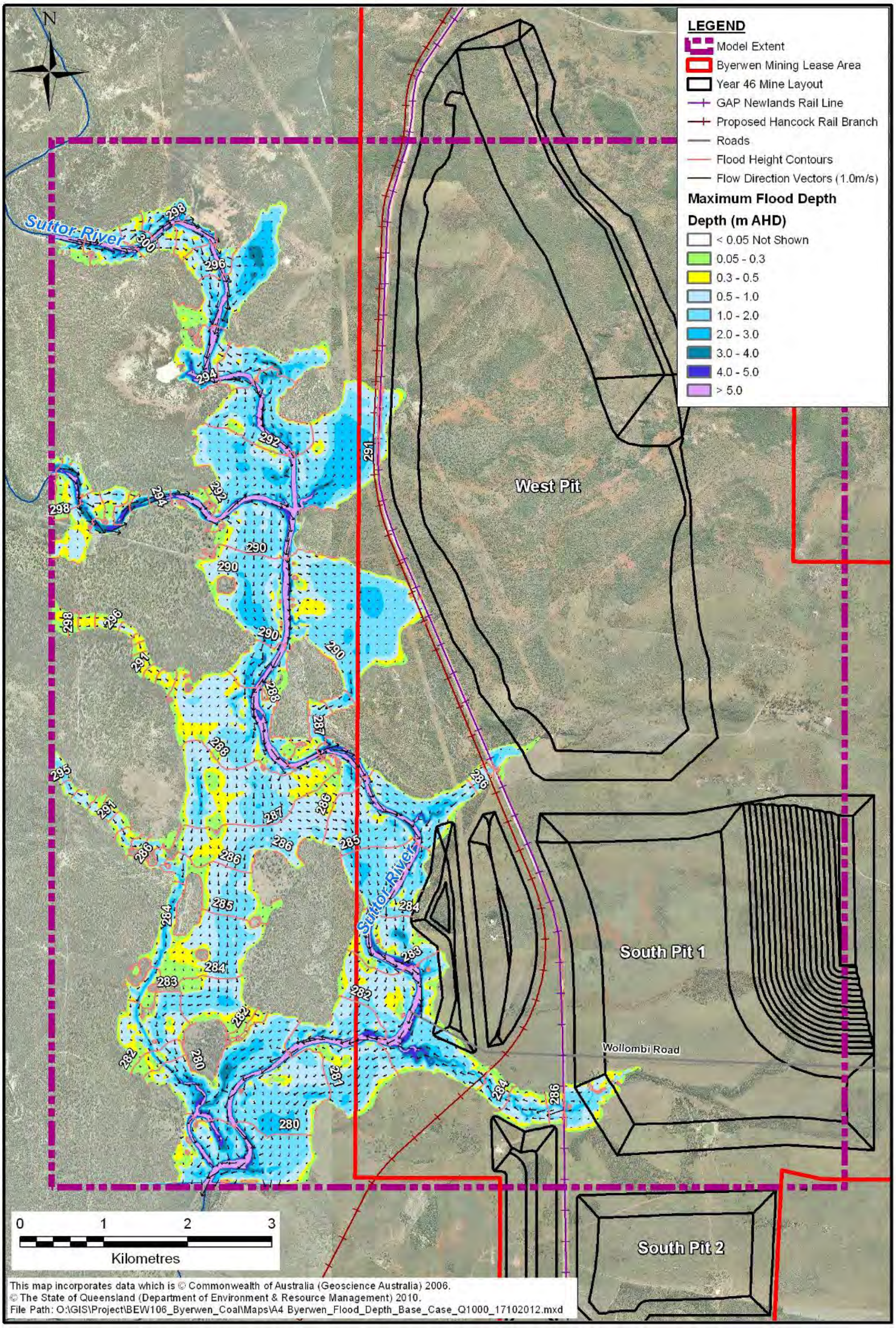


**Figure 6.3**
**SUTTOR RIVER 1,000 YEAR ARI FLOOD DEPTHS (PRE-DEVELOPMENT)**

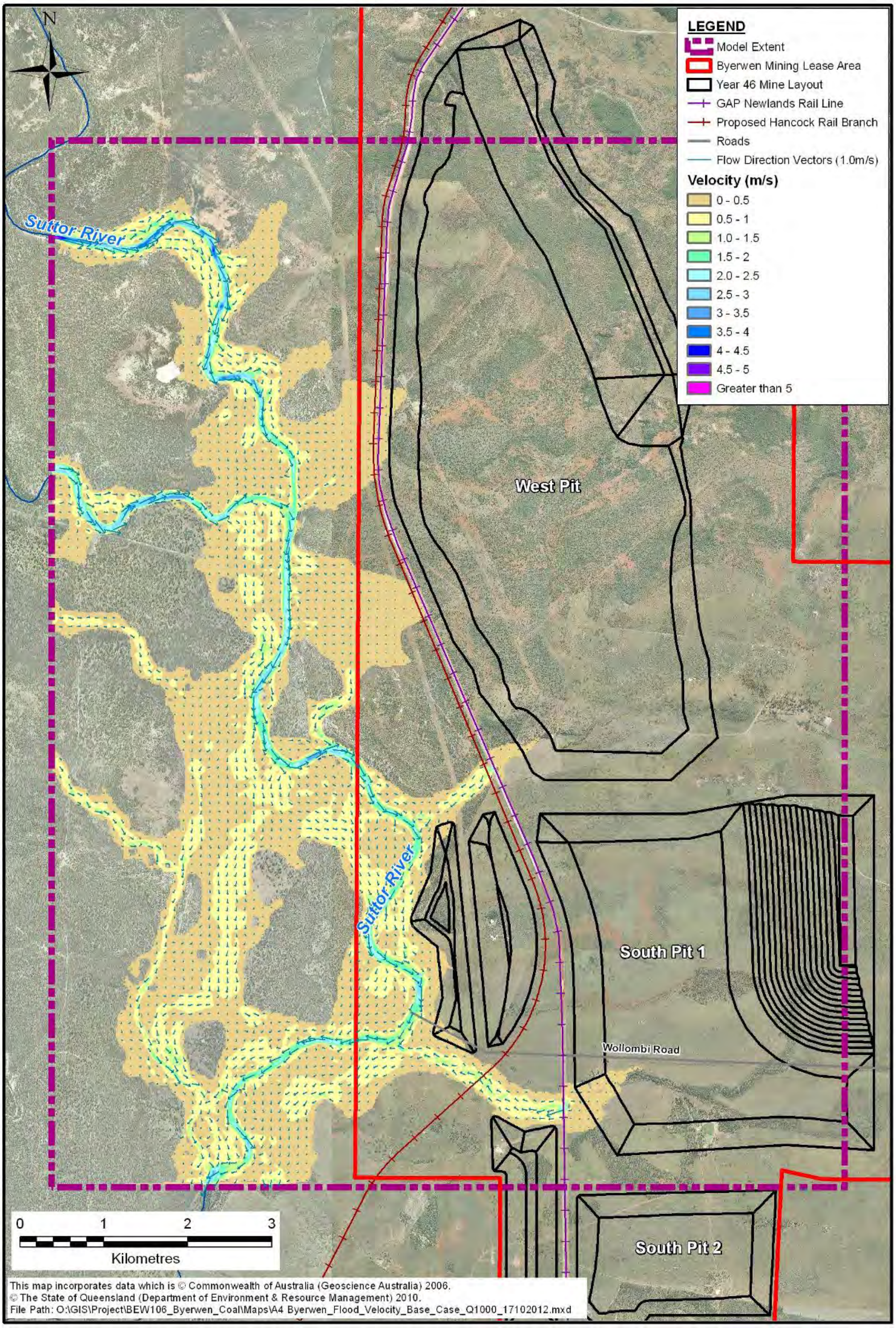


**Figure 6.4**
**SUTTOR RIVER 1,000 YEAR ARI FLOOD VELOCITY (PRE-DEVELOPMENT)**

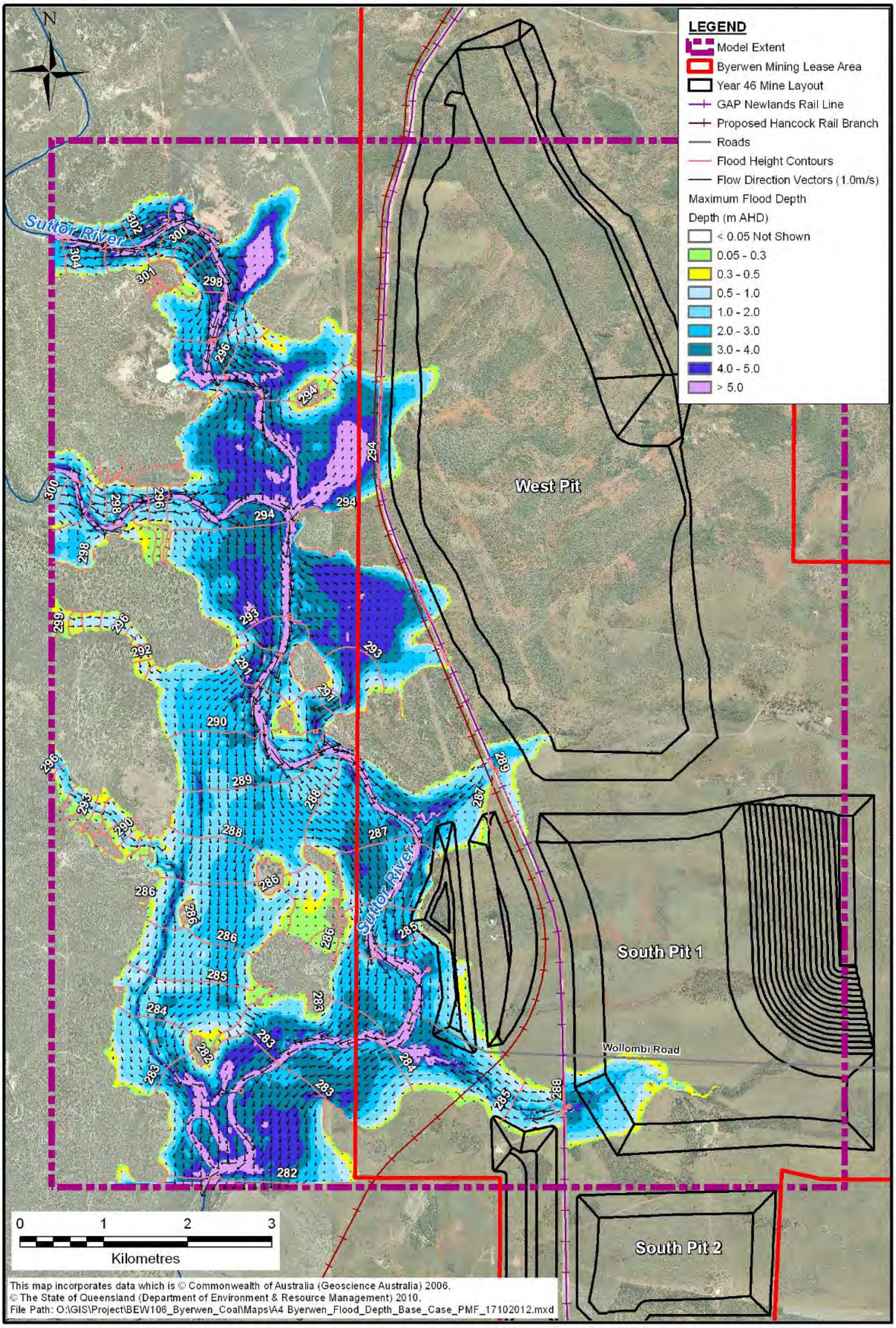


**Figure 6.5**
**SUTTOR RIVER PMF FLOOD DEPTHS (PRE-DEVELOPMENT)**

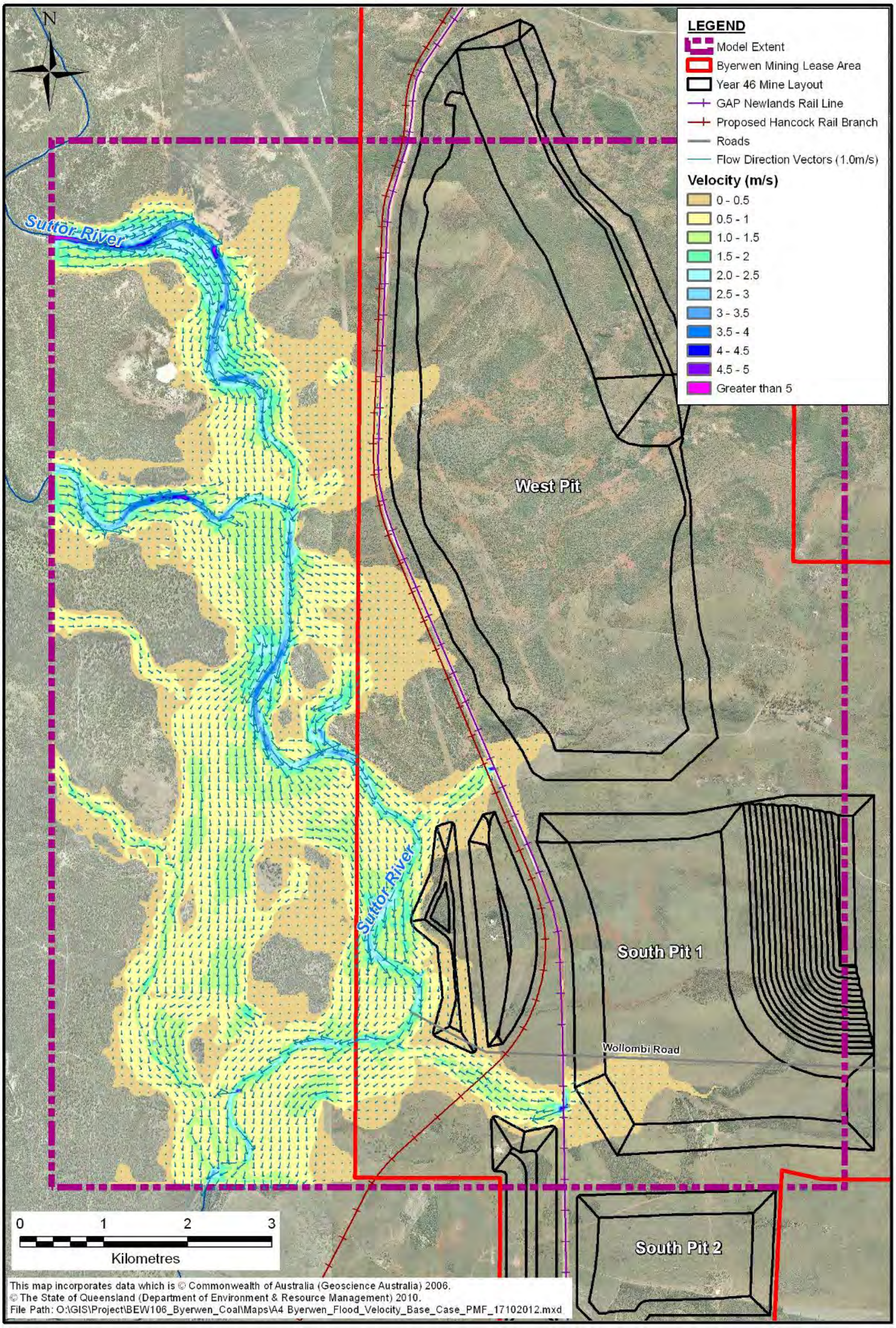


**Figure 6.6**
**SUTTOR RIVER PMF FLOOD VELOCITY (PRE-DEVELOPMENT)**

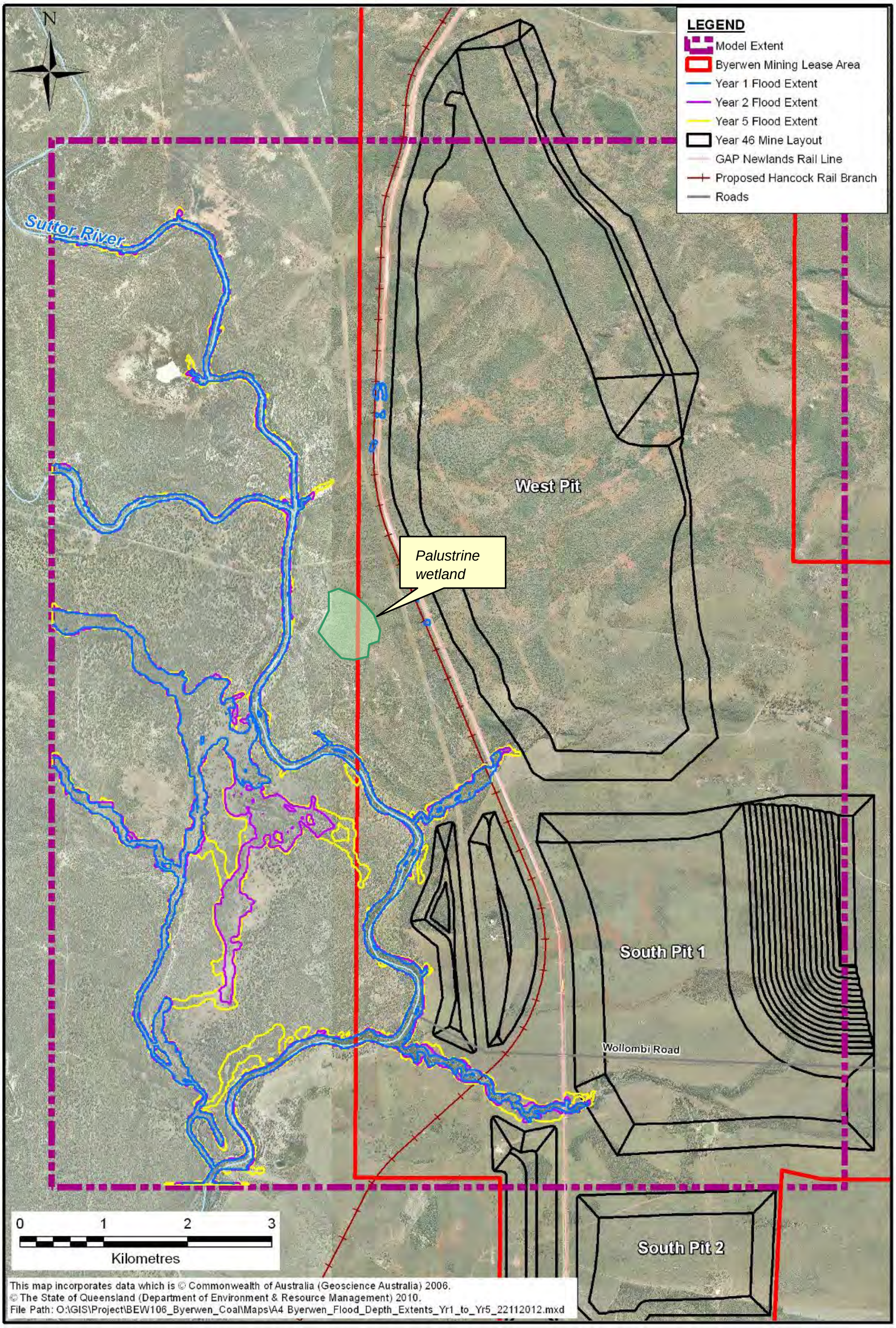


**Figure 6.7**
**SUTTOR RIVER FREQUENT FLOOD INUNDATION (PRE-DEVELOPMENT)**

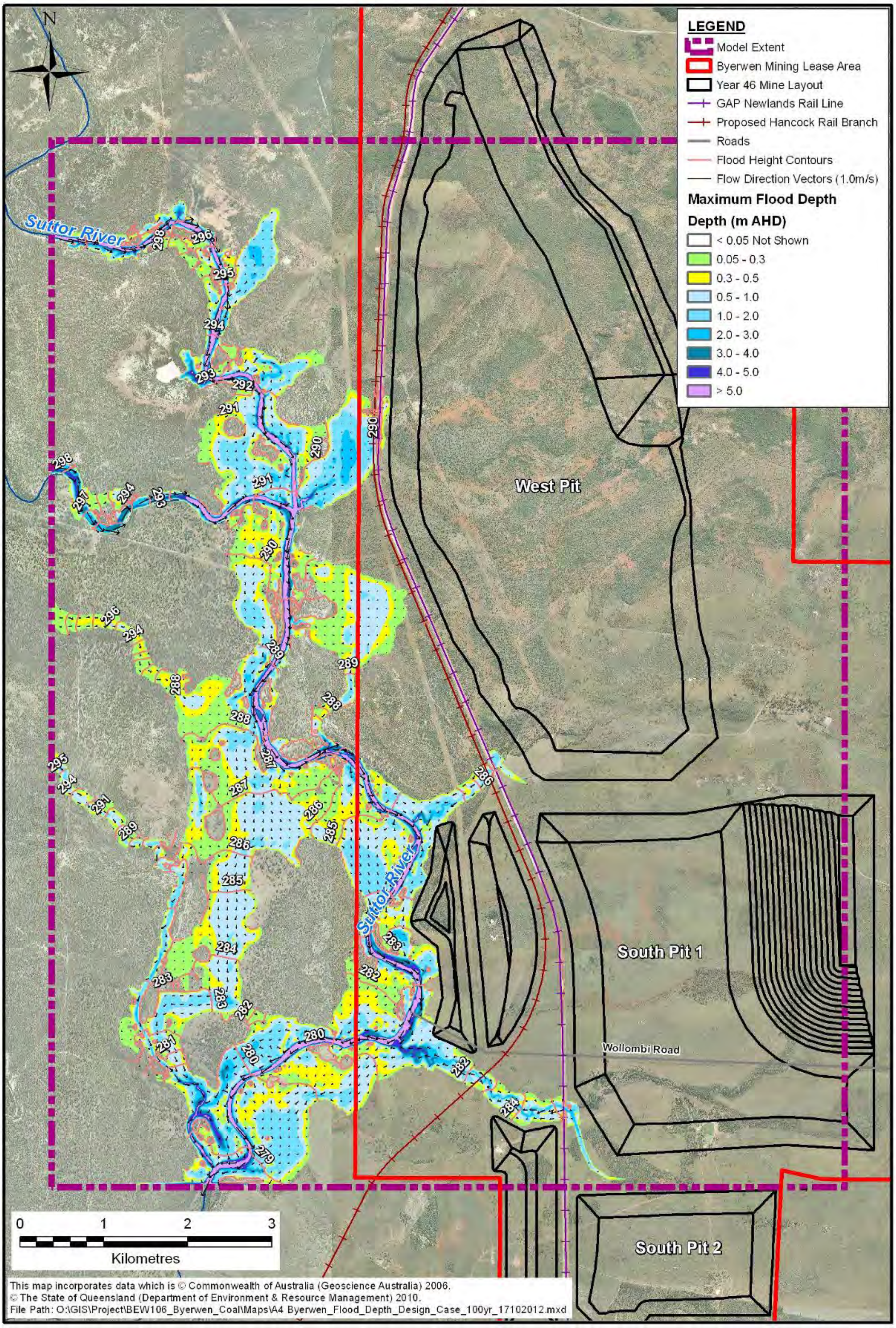


**Figure 6.8**
**SUTTOR RIVER 100 YEAR ARI FLOOD DEPTH (POST-DEVELOPMENT)**

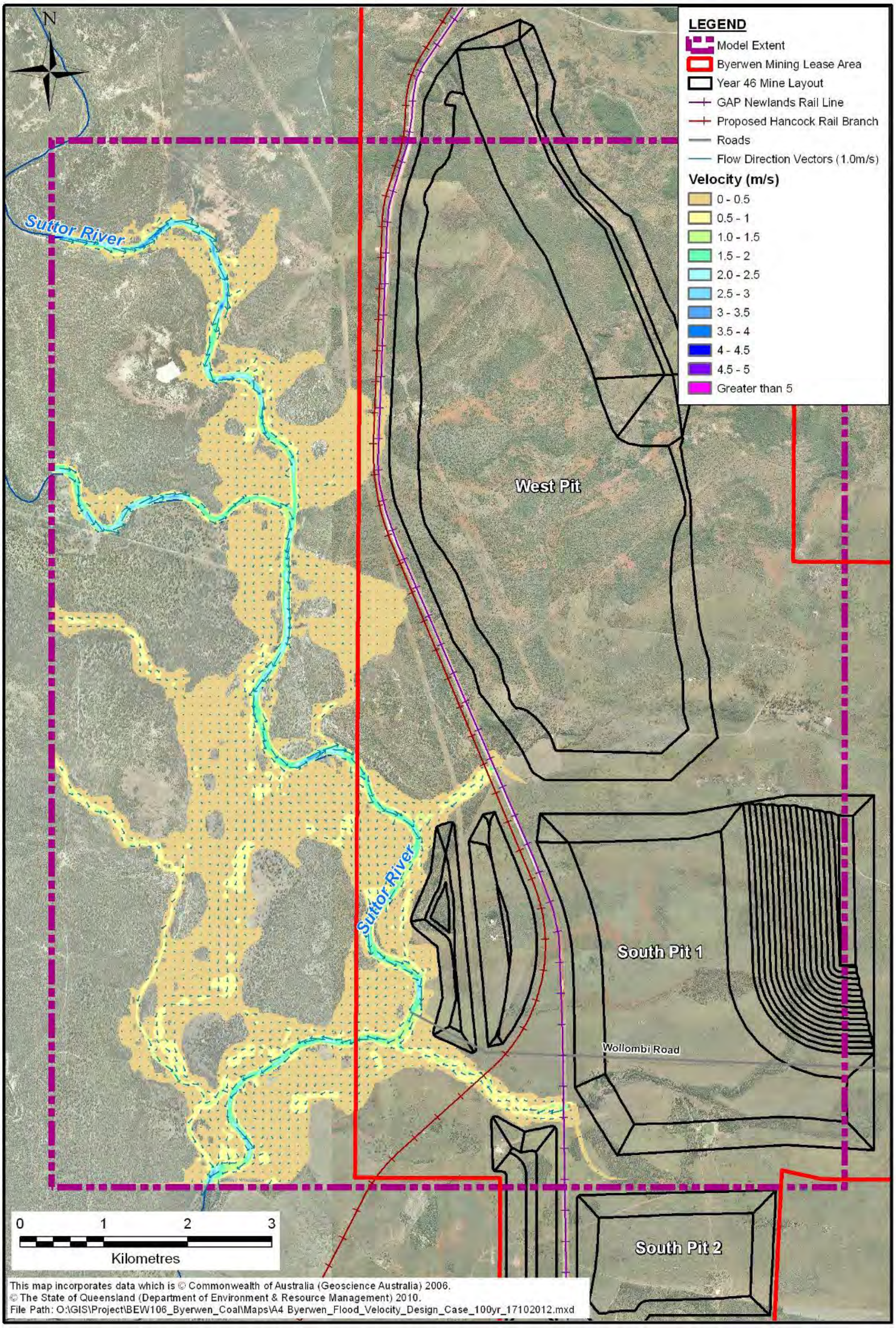


**Figure 6.9**
**SUTTOR RIVER 100 YEAR ARI FLOOD VELOCITY (POST-DEVELOPMENT)**

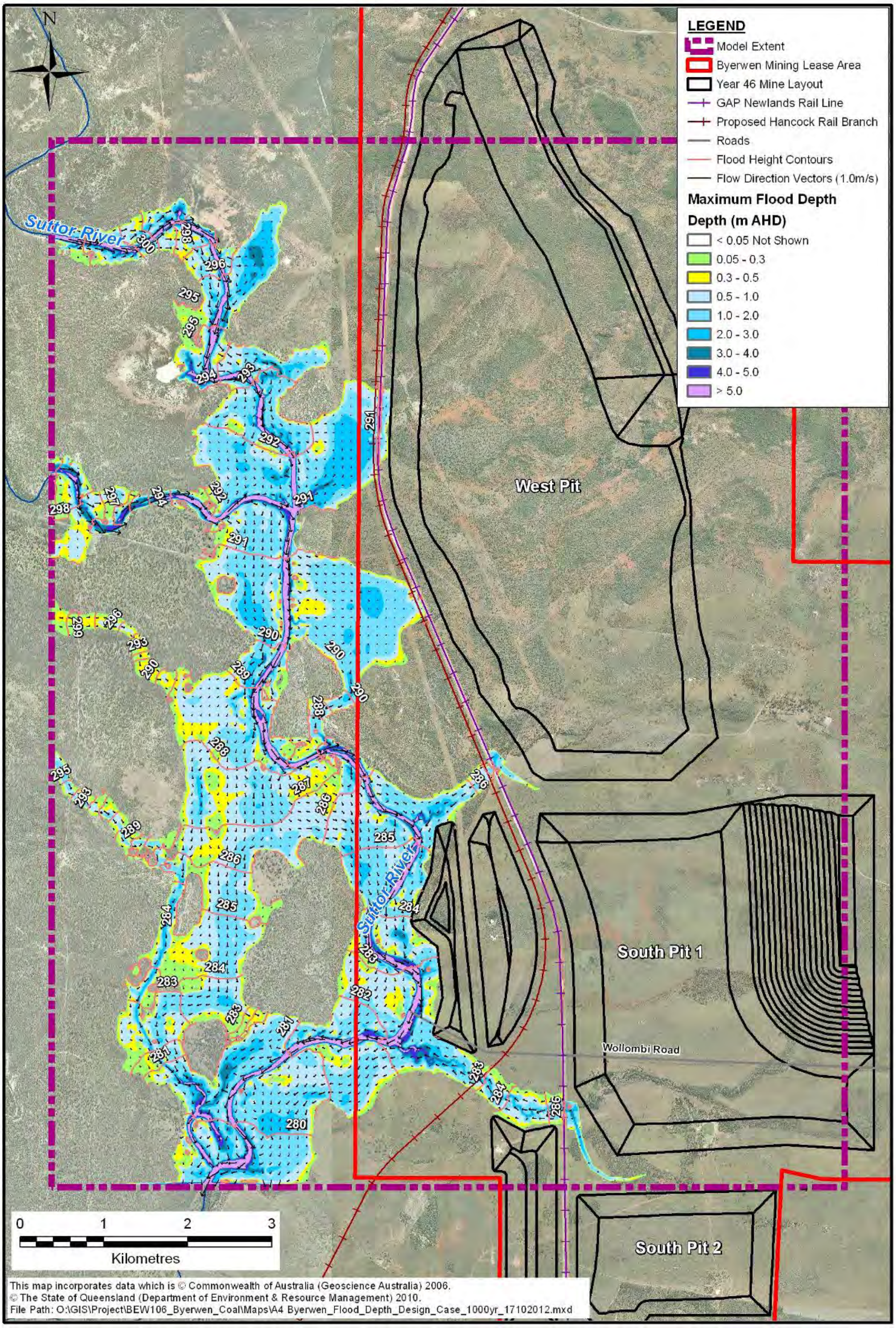


**Figure 6.10**
**SUTTOR RIVER 1,000 YEAR ARI FLOOD DEPTH (POST-DEVELOPMENT)**

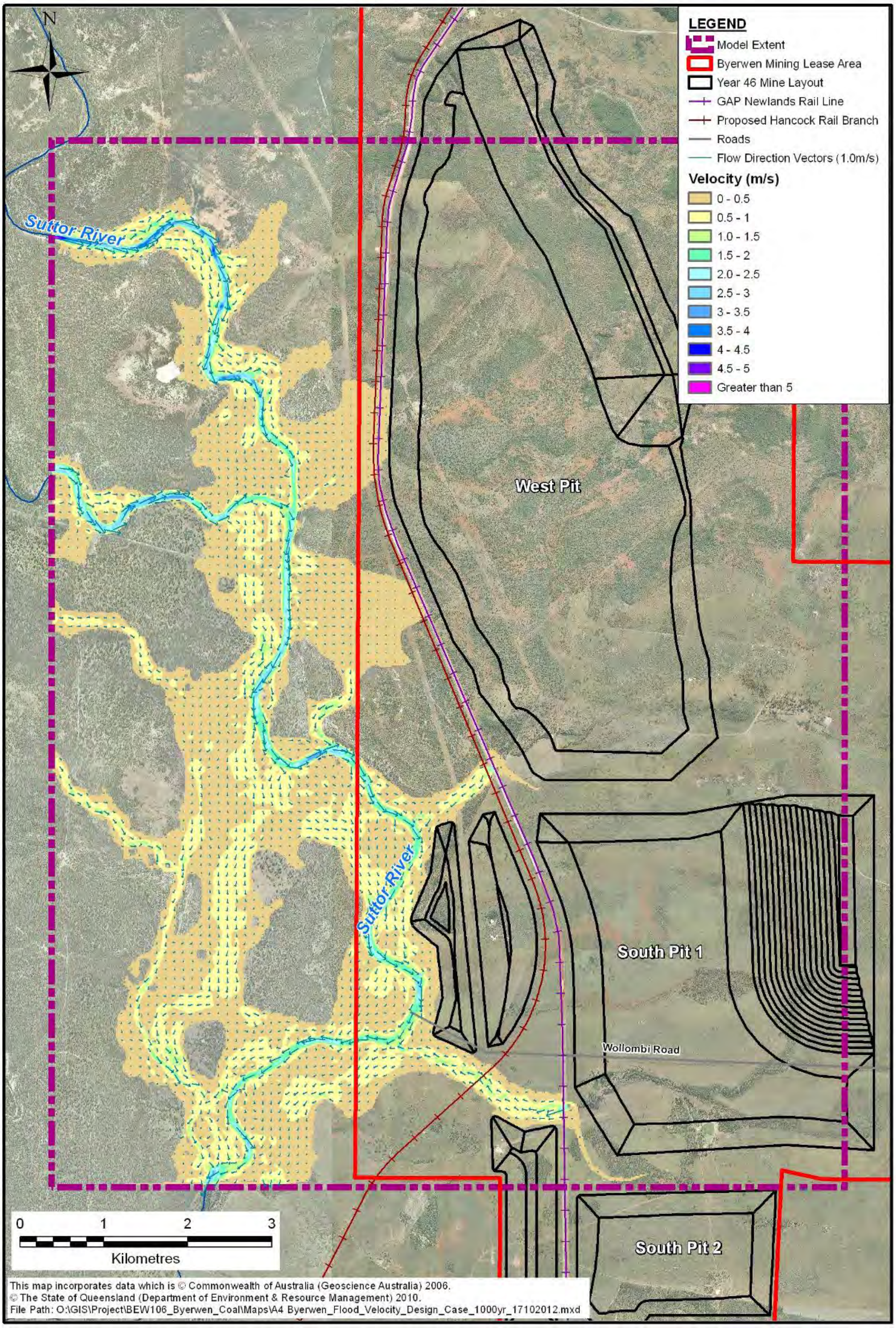


**Figure 6.11**
**SUTTOR RIVER 1,000 YEAR ARI FLOOD VELOCITY (POST-DEVELOPMENT)**

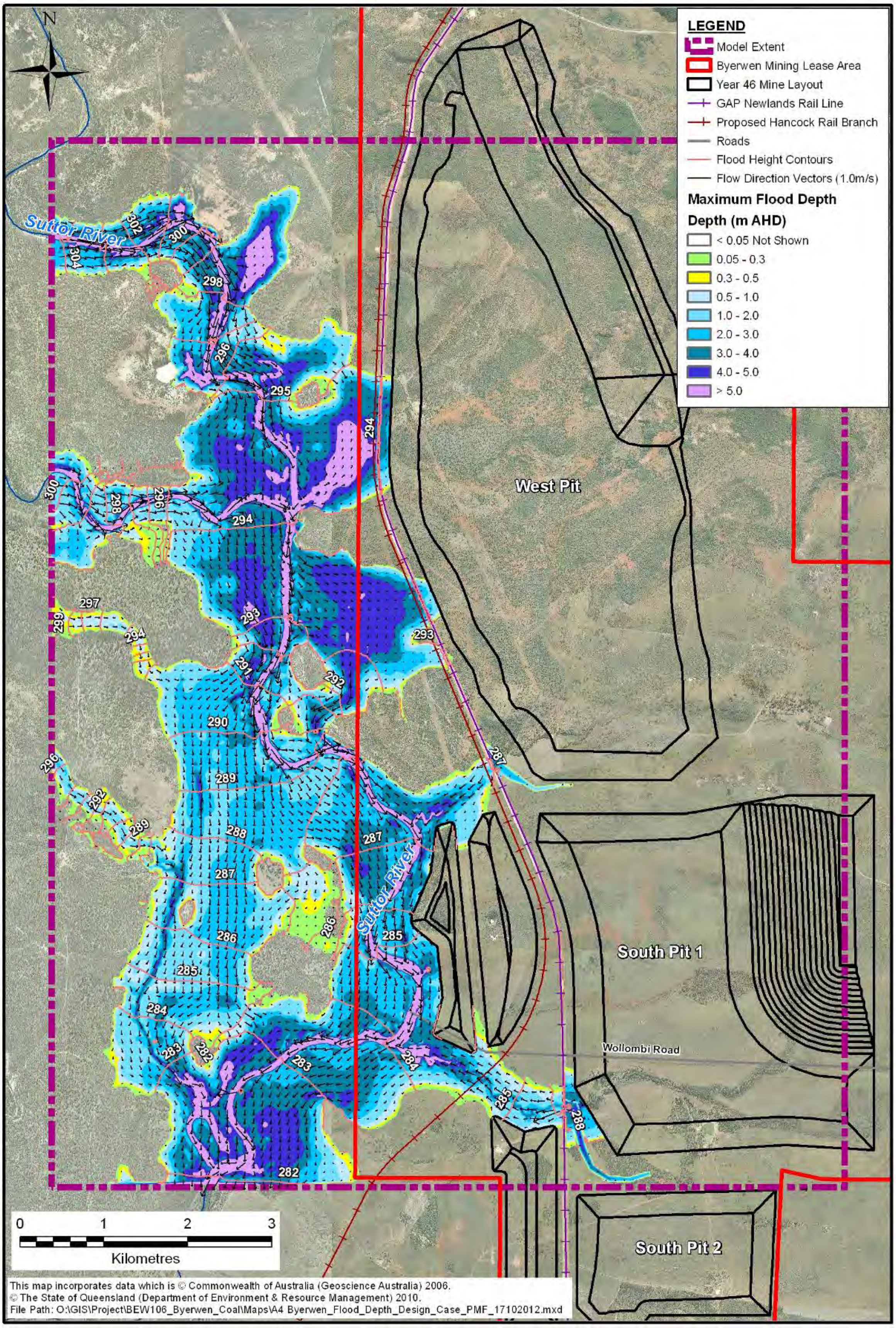


**Figure 6.12**
**SUTTOR RIVER PMF FLOOD DEPTH (POST-DEVELOPMENT)**

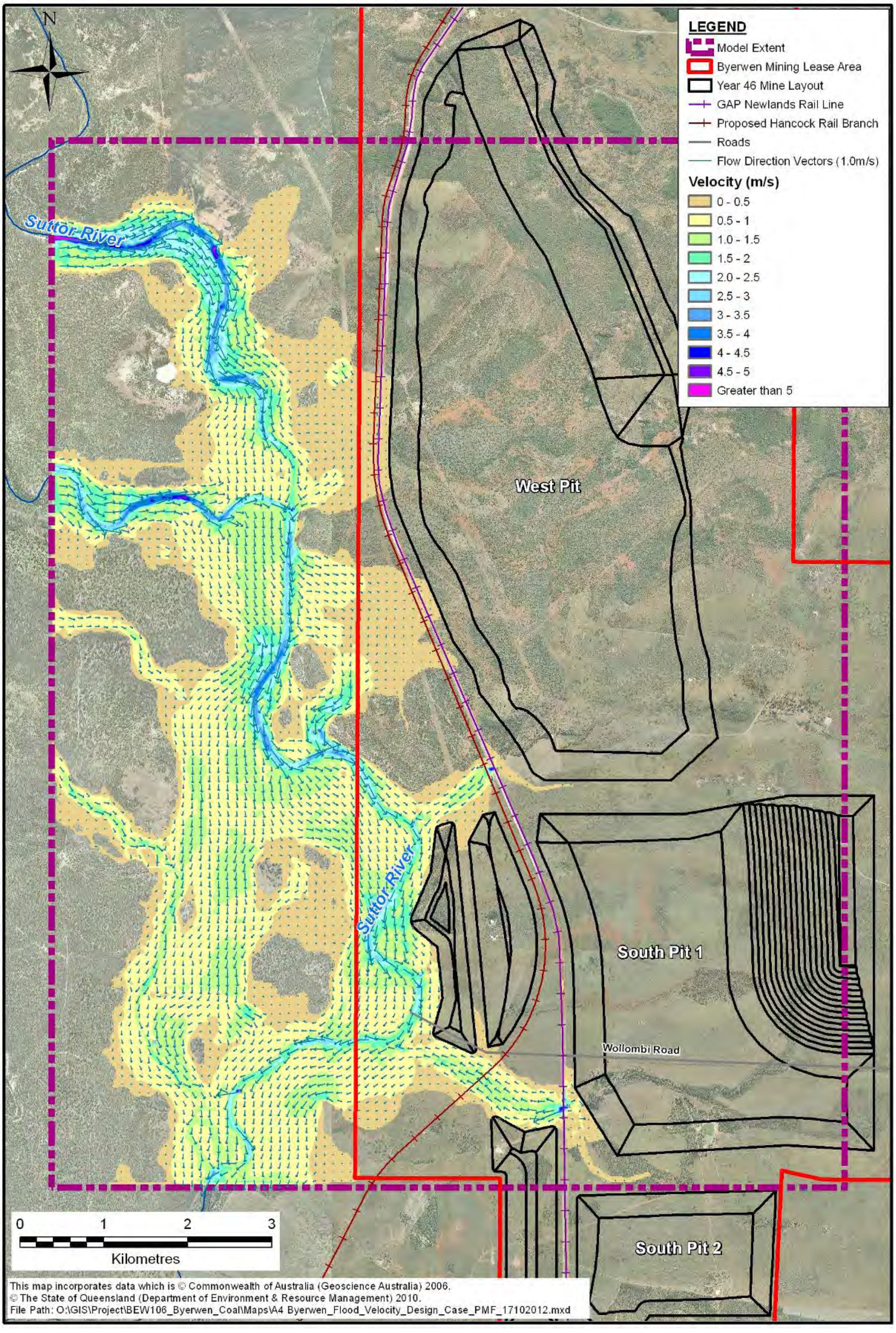


**Figure 6.13**
**SUTTOR RIVER PMF FLOOD VELOCITY (POST-DEVELOPMENT)**

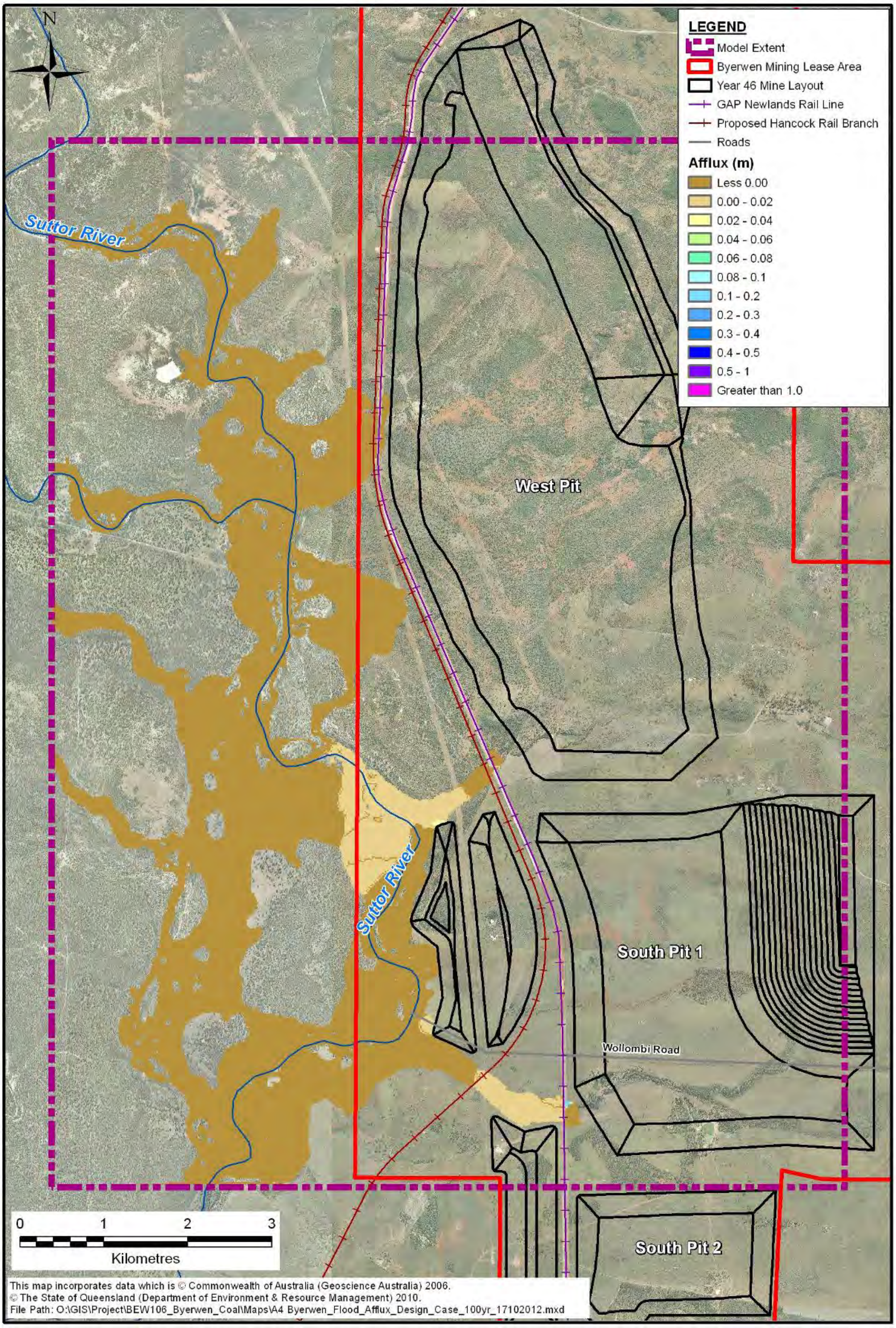


**Figure 6.14**
**SUTTOR RIVER 100 YEAR ARI FLOOD AFFLUX (POST-DEVELOPMENT)**

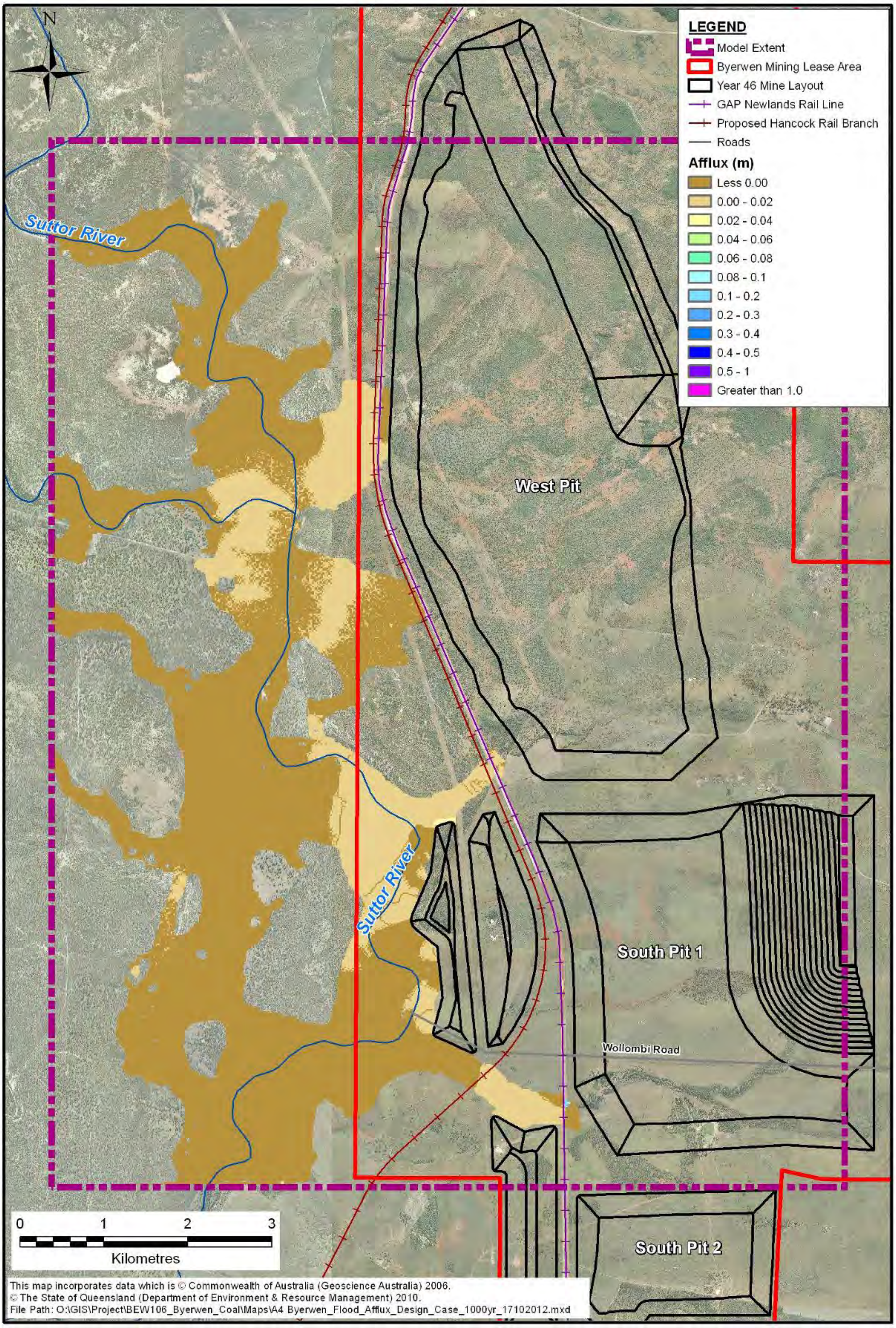


**Figure 6.15**
**SUTTOR RIVER 1,000 YEAR ARI FLOOD AFFLUX (POST-DEVELOPMENT)**

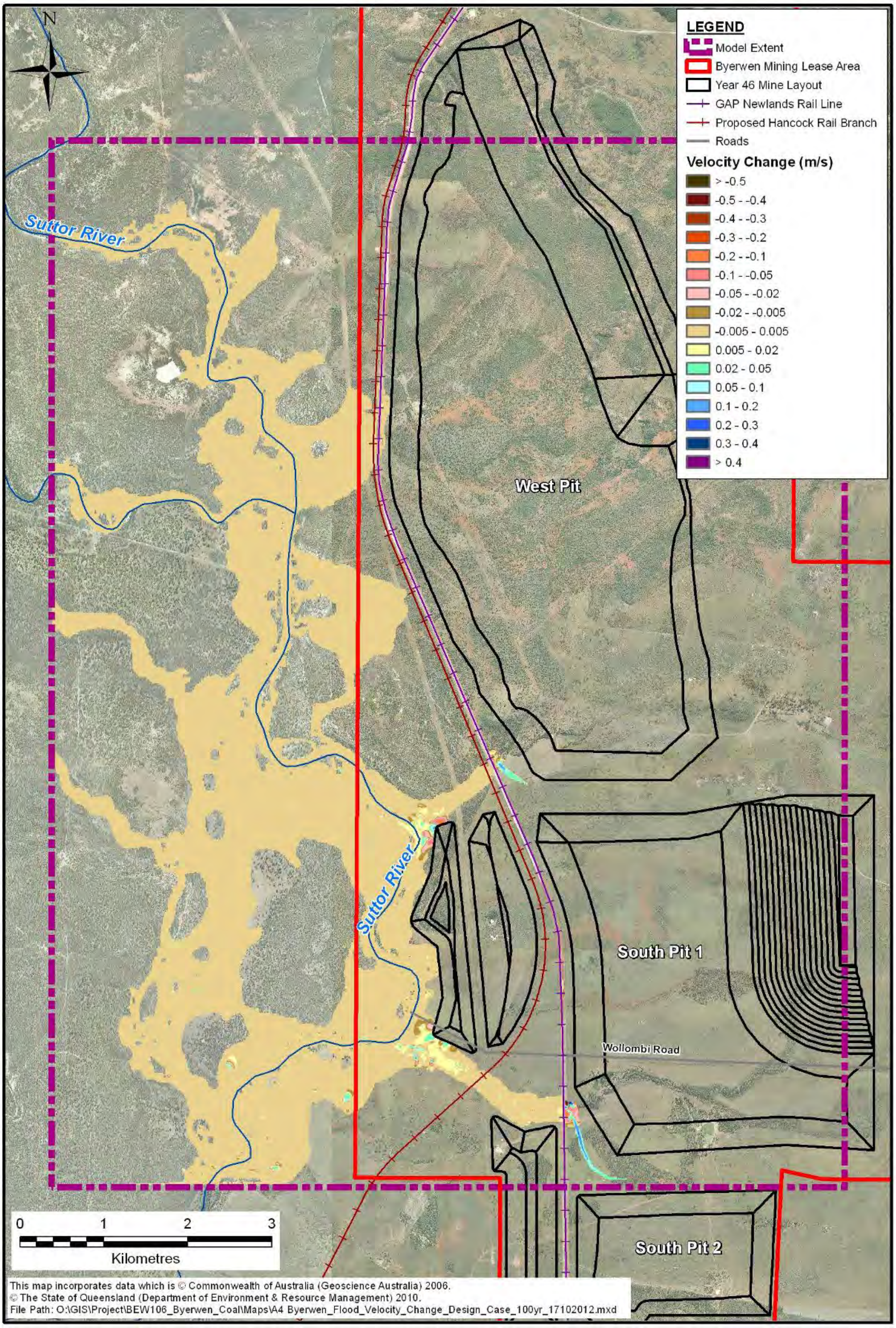


**Figure 6.16**
**SUTTOR RIVER 100 YEAR ARI FLOOD VELOCITY CHANGE (POST-DEVELOPMENT)**

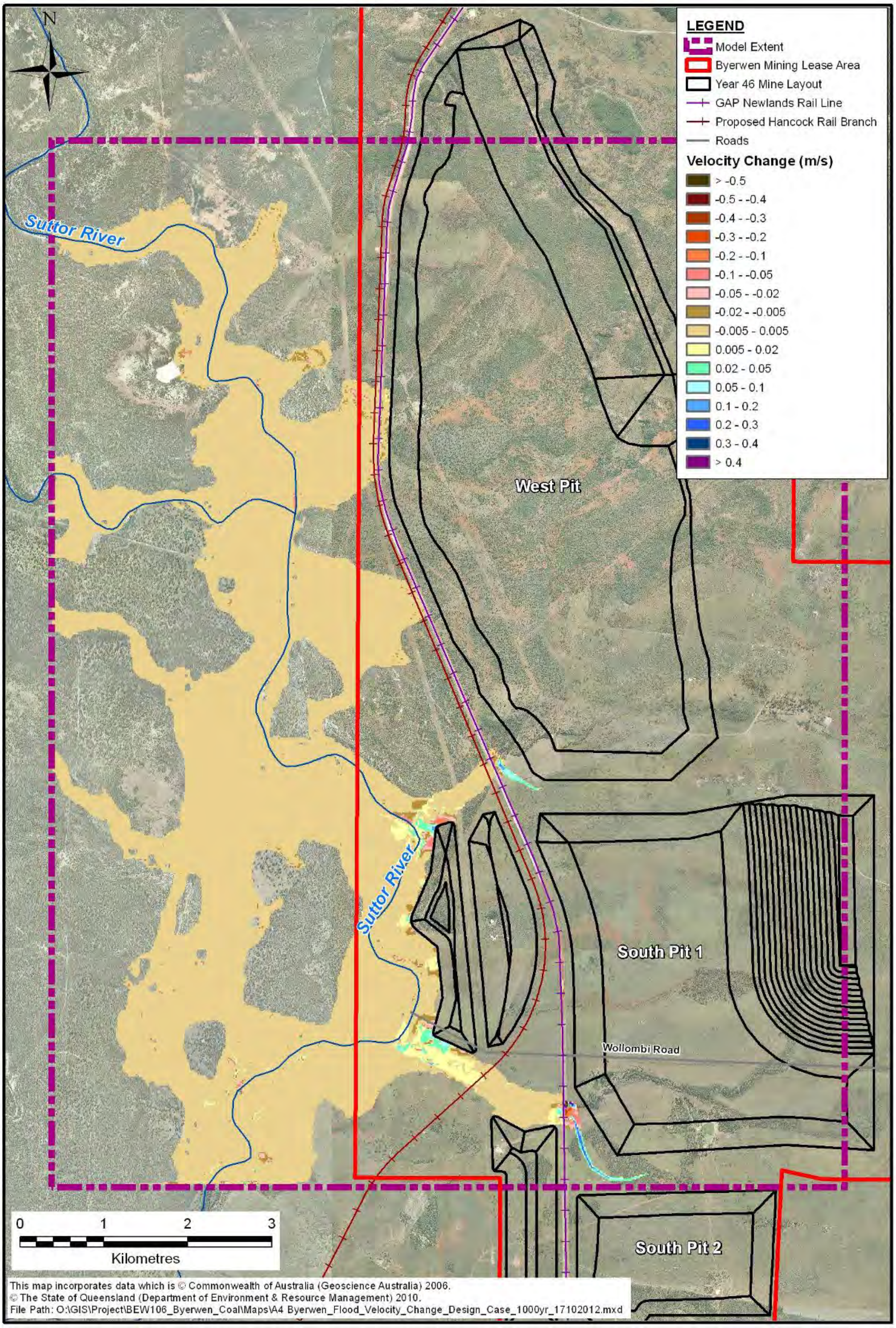


**Figure 6.17**
**SUTTOR RIVER 1,000 YEAR ARI FLOOD VELOCITY CHANGE (POST-DEVELOPMENT)**

# 7 Diversions

## 7.1 INTRODUCTION

A number of existing watercourses are located within the Project mining area and several stream diversions are proposed to gain access to coal reserves.

Disturbance of two tributaries is required within the Project area for open-cut mining in the Byerwen West Pit 1, South Pit 1 and East Pit 2. The southern tributary is defined as a watercourse under the *Water Act 2000*, while the northern tributary is not. A corridor is proposed between the Byerwen East Pit 1 and 2 to avoid the need for a diversion in the upper part of the southern tributary.

Stream diversions can introduce a wide range of issues such as changes to catchment hydrology, localised flooding, geomorphology and ecological integrity. Realignment of these tributaries requires detailed hydrological and hydraulic assessment to enable management of downstream impacts.

Conceptual stream diversions have been determined with stream lengths similar to the natural condition (where possible) to limit erosion potential, with a typical stream cross-section and appropriate roughness parameters (based on vegetation to be established along the watercourses) applied for modelling purposes.

The diversion channels have been designed to cater for local catchment discharges up to and including the 0.1% AEP event discharge plus a 0.5 m freeboard to the top of the bank or levee in accordance with the DERM Manual for Assessing Hazard Categories and Hydraulic Performance of Dams (DERM, 2012).

It should be noted that this is a conceptual study and not for detailed design or construction purposes. The following sections outline the design discharge and the conceptual design of the diversion channels.

### 7.1.1 Proposed diversions

The location of all proposed diversion channels for the Byerwen Coal Project, other than the northern diversion, is presented in Figure 7.1. The preliminary design of the diversion channels will need to be reviewed during detailed design, and will rely on geotechnical advice to confirm the design parameters and stability of the banks to control erosion and scour.

They should be designed as stable systems and maintained over the life of the mine, with refinements made if needed, resulting in diversions and levees that are self sustaining, stable and which require no maintenance post closure.

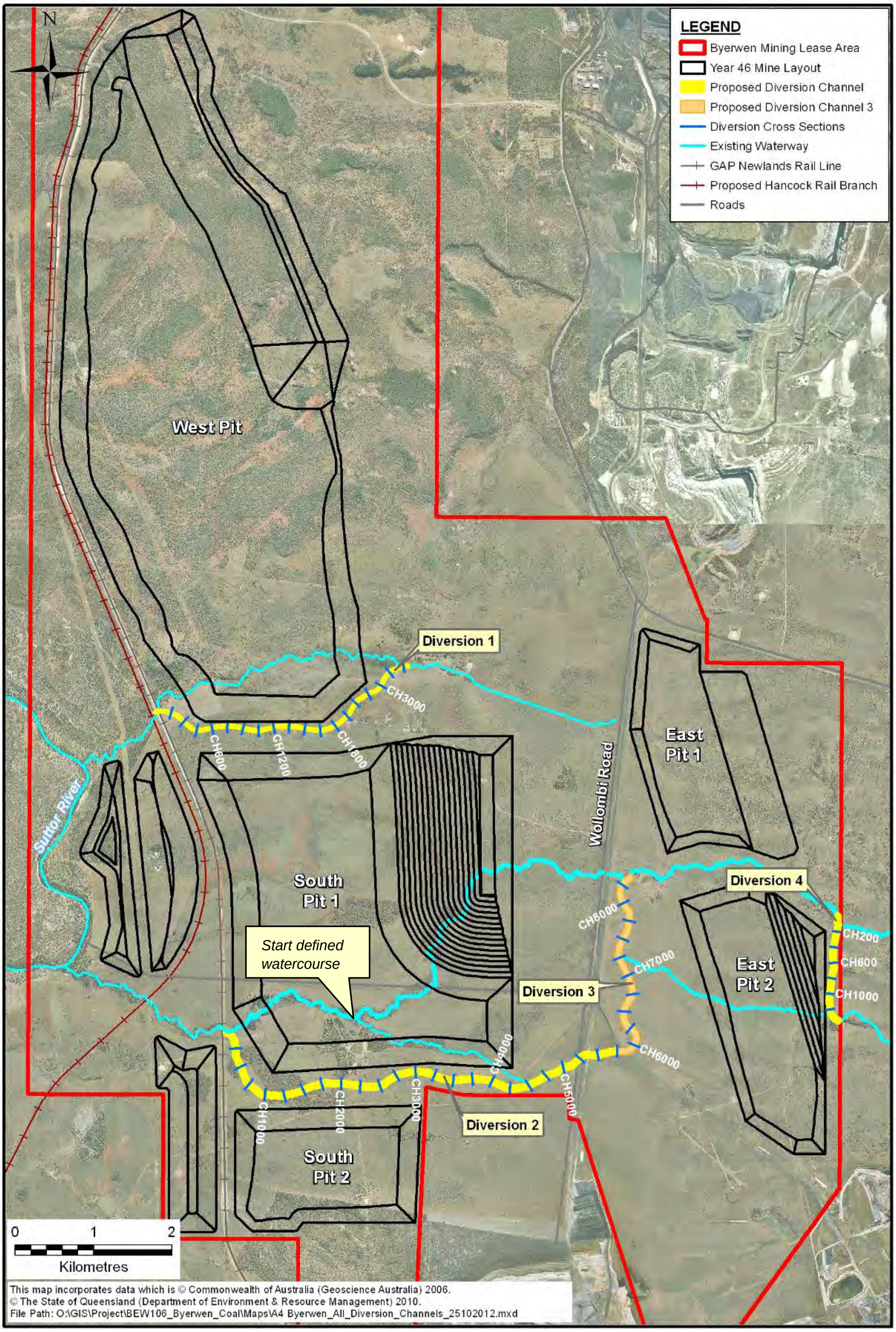


**Figure 7.1**
**PROPOSED DIVERSION CHANNELS**

### Diversion 1 – West Pit 1

This diversion is for the Suttor River north tributary which flows through the proposed West Pit 1. The route of the existing watercourse flows across the southern end of the pit area. The natural topography for the initial 1,200 m of the proposed route of the diversion channel rises approximately 4 m to 300 mAHD before falling 14.5 m to 285.5 mAHD over the remaining 2,500 m. Where necessary, levee banks have been included in the diversion channel feasibility design to contain the 0.1% AEP design discharge.

### Diversion 2 – South Pit 1

The natural drainage line which intersects South Pit 1 would be diverted and would separate South Pit 1 and South Pit 2. The natural topography along the route of the diversion starts at chainage 5,750 m with a gradual fall in elevation of 6.1 m from 301.6 to 295.5 mAHD for the initial 1,330 m (CH5750 to CH4420). The surface topography between CH4420 and CH1100 undulates between 294 and 297 mAHD before falling 11 m over the remaining 1,100 m to an elevation of 283 mAHD at the diversion channel outlet at the Northern Missing Link (NML) railway crossing.

Levee banks have been included in the diversion channel feasibility design to encompass this varying topography where flood flows would spill from the design channel without them.

The proposed diversion channel will require part of the Wollombi Road to be removed. The existing road crossing of the tributary has limited flood immunity. A causeway can be constructed to reconnect the road with flood immunity similar to the existing crossing, or it may be increased as required by the mine by raising the road above the bottom of the diversion channel using culverts for cross drainage.

### Diversion 3 – South Pit 1

Flows from the tributary that runs between East Pit 1 and East Pit 2 are prevented from entering South Pit 1 by Diversion 3 which redirects the flow into Diversion 2. Running in a southerly direction is Diversion 3, approximately 2,630 m in length. It has an upstream elevation of 302 mAHD and remains at a reasonably flat level of 302 to 303 mAHD for the first 1,600 m of channel length (CH6750 to CH8500), climbing to a maximum elevation of 305 mAHD at CH6250. From this location, the terrain begins a gradual decline in elevation from 305 to 301.6 mAHD at CH5750.

The catchment which remains after the drainage realignment would be dammed to prevent surface runoff from entering the mining areas of South Pit 1.

### Diversion 4 – East Pit 2

This diversion is located between the mining pit and the mine lease boundary to convey a small tributary that flows through East Pit 2. The diversion flows in a northerly direction to a natural tributary which flows through a corridor between East Pit 1 and East Pit 2.

The natural topography along the route of the diversion has an upstream elevation of 312 mAHD for the first 200 m before gradually increasing in elevation to 317 mAHD over the next 600 m (CH1200 to CH600, refer to Figure 1). From CH600 the topography gradually falls over the remaining 600 m of the diversion channel to an elevation of 310 mAHD at CH0.

### Diversion 5 – North Pit 1

A small drainage diversion is planned to allow water to bypass the North Pit and flow to Kangaroo Creek. This drainage diversion is to be in place before mining operations commence at the North Pit. The drainage diversion put in place would remain as a permanent structure to divert water around the North Pit and its final void.

The flood modelling in this report does not include the north pit, north pit diversions and northern MIA/CHPP/co-disposal which relates to flooding in the Kangaroo Creek catchment and has no bearing on flooding in the Suttor River.

This infrastructure is not planned to commence until approximately year 15-17 and will therefore be assessed once further confirmatory surveys have been completed in that specific area. However, assessment of potential final void impacts from Kangaroo Creek is included in the Final Void Assessment, BEW106-TD-WE-REP-0006 (KBR, 2012b).

### Guidelines

The DERM guideline on Watercourse Diversions – Central Queensland Mining Industry (DERM, 2011) provides advice on an established range of stream powers, velocities and shear stresses that are considered to be the upper range for natural Bowen Basin watercourses, which were derived based on research conducted by ACARP project C8030, and shown in Table 7.1.

- Stream power represents the energy that is available to do work in and on the channel and is a function of the discharge, slope and width of a channel. Higher stream powers can indicate an elevated erosion potential.
- High flow velocities have the potential to damage the channel through erosion and additional bank protection is required where estimated velocities exceed the criteria. This can be achieved by lining the channel with more dense vegetation or rock armouring.
- Shear stress is another indicator of erosion potential and measures the force exerted on the channel surface. It determines the threshold of motion for bed material.

**Table 7.1 DERM guideline values for hydraulic parameters in natural Bowen Basin watercourses**

| Scenario | Stream Power (Watts/$m^2$) | Velocity (m/s) | Shear Stress (N/$m^2$) |
|---|---|---|---|
| 50% AEP | <35 | <1.0 | <40 |
| 2% AEP | <220 | <2.5 | <80 |

Additionally, the DERM Manual for Assessing Hazard Categories and Hydraulic Performance of Dams (DERM, 2012) outlines the hydrological design criteria for a number of regulated structures, including levees, and specifies the required crest level for levee embankments must contain the 0.1% AEP with an additional 0.5 m freeboard.

## 7.2 ASSESSMENT METHODOLOGY

### 7.2.1 Design discharges

**Diversions 1 and 2**

The design stream flows applied in the assessment of diversion channels were obtained from the Gap 50 Design Report undertaken by CoalConnect for the Northern Missing Link Project (NML) in November 2009 (CoalConnect, 2009). The Gap 50 Design study was conducted to determine the drainage design for the NML railway line between Newlands Junction and North Goonyella Junction.

The NML hydrology study determined design flows for the watercourses that are part of the proposed diversions for the Byerwen Coal Project. The contributing catchment for the NML study was the entire area upstream of the railway corridor.

Design events considered in the NML study included the 100%, 50%, 20%, 10%, 5%, 2%, 1% and 0.05% Annual Exceedance Probability (AEP) events and their peak discharges are presented in Table 7.2.

A design event discharge for the 0.1% AEP event was estimated by logarithmic interpolation between the 1% and 0.05% AEP results from the NML study.

**Table 7.2 Design events peak discharges upstream of the NML railway crossing**

| | Annual Exceedance Probability (AEP)[1] | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Peak Discharge ($m^3/s$) | 100% | 50% | 20% | 10% | 5% | 2% | 1% | 0.1%[2] | 0.05% |
| Diversion 1 | 12.2 | 23.1 | 46 | 59.6 | 80.4 | 108.2 | 133.8 | 228 | 267.7 |
| Diversion 2 | 11.4 | 23.2 | 51.7 | 68.3 | 92 | 125.6 | 157.7 | 271.5 | 319.7 |

*Note1: AEP is equivalent to the reciprocal of the ARI values that have been used previously. AEP is being used to describe design storm frequency in this report.*

*Note 2: the 0.1% AEP peak design discharge has been interpolated.*

**Diversions 3 and 4**

The design stream flows for Diversions 3 and 4 were determined using CatchmentSIM and XP-RAFTS (2009). CatchmentSIM is a GIS based terrain analysis program designed to assist in setting up hydrologic models in XP-RAFTS where rainfall is converted to stream flows which are then routed through the catchment.

Subcatchment areas were determined in CatchmentSIM using Shuttle Radar Topography Mission (SRTM) data with 100 m grid size. Fifteen subcatchments were delineated up to the Northern Missing Link (NML) bridge crossing at the downstream end of the South Pit diversion. Note that the contributing catchment has been based on the existing watercourse with mining pits not included.

Hydraulic roughness for the catchment were specified as a Mannings ‘n’ value of 0.05 based on the vegetative cover of the catchment. It should be noted that impervious areas were not considered to be significant for the subcatchments in this study. Routing of flow between subcatchments was based on the Bransby-Williams lag equation.

The design storm temporal patterns used in XP-RAFTS were adopted from Zone 3, which is consistent with Australian Rainfall and Runoff (AR&R). In addition, initial and continuing loss values were specified for the various design events as outlined in Table 7.3.

**Table 7.3 Loss parameters used in XP-RAFTS modelling**

| ARI (years) | Initial Loss (mm) | Continuing Loss (mm/hr) |
|---|---|---|
| 1 | 15 | 2.5 |
| 2 | 15 | 2.5 |
| 5 | 15 | 2.5 |
| 10 | 15 | 2.5 |
| 20 | 10 | 2.5 |
| 50 | 5 | 2.5 |
| 100 | 0 | 2.5 |
| 1,000 | 0 | 2.5 |
| 2,000 | 0 | 2.5 |

The design intensities listed in Table 7.4 were determined using the BOM Intensity Frequency Duration (IFD) calculator for the 100% to 1.0% AEP events and CRC-FORGE for the 0.1% and 0.05% AEP events.

**Table 7.4 Intensity frequency duration (mm/hr)**

| Duration | 100% | 50% | 20% | 10% | 5% | 2% | 1.0% | 0.1% | 0.05% |
|---|---|---|---|---|---|---|---|---|---|
| 30Mins | 47.8 | 60.9 | 75.4 | 83.7 | 95.2 | 110 | 122 | 198.3 | 221.0 |
| 1Hr | 32.4 | 41.1 | 50.6 | 56.1 | 63.7 | 73.6 | 81.1 | 134.4 | 149.7 |
| 2Hrs | 20.1 | 25.6 | 31.8 | 35.4 | 40.3 | 46.8 | 51.8 | - | - |
| 3Hrs | 14.7 | 18.9 | 23.7 | 26.5 | 30.4 | 35.5 | 39.4 | 63.0 | 70.2 |
| 6Hrs | 8.5 | 11 | 14.2 | 16 | 18.6 | 21.9 | 24.5 | 38.7 | 43.2 |
| 12Hrs | 5.0 | 6.5 | 8.6 | 9.8 | 11.5 | 13.7 | 15.4 | 23.9 | 26.6 |
| 24Hrs | 3.0 | 4.0 | 5.3 | 6.2 | 7.2 | 8.7 | 9.9 | 15.4 | 17.1 |
| 48Hrs | 1.8 | 2.4 | 3.3 | 3.8 | 4.5 | 5.5 | 6.2 | 9.7 | 10.7 |
| 72Hrs | 1.3 | 1.7 | 2.4 | 2.8 | 3.3 | 4.0 | 4.6 | 7.2 | 8.0 |

The critical storm duration for the study catchment was determined to be 3 hours for the East Pit 2 Diversion and 6 hours for the remainder of the catchment downstream of this diversion including the South Pit 1-East Pit 2 Diversion. The critical storm duration for the 0.1% and 0.05% AEP event in the South Pit 1-East Pit 2 diversion was 3 hours. The peak design discharges are listed in Table 3.

Design events considered in this study included the 100%, 50%, 20%, 10%, 5%, 2%, 1% and 0.05% AEP events and their peak discharges are presented in Table 7.5. This table includes the 0.1% AEP event which was adopted in this study for the design of the diversion channels.

**Table 7.5 East pit diversion design event peak discharges ($m^3/s$)**

| Peak Discharges ($m^3/s$) | Annual Exceedance Probability (AEP) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 100% | 50% | 20% | 10% | 5% | 2% | 1% | 0.1% | 0.05% |
| Diversion 3 | 18.7 | 33 | 55.4 | 69.5 | 94.9 | 127.3 | 149 | 279.8 | 321.7 |
| Diversion 4 | 4.3 | 7.6 | 11.4 | 13.9 | 19.1 | 25.4 | 30.2 | 56.1 | 64.2 |

*Note1: AEP is equivalent to the reciprocal of the ARI values that have been used previously. AEP is being used to describe design storm frequency in this report.*

### 7.2.2 Hydraulic modelling

Elevation data used in this study was sourced from QCoal and is in the form of 1 m contours. The diversion channel design process involves using the existing 1 m contour data in 12D to draw the existing watercourse and the proposed diversion channel.

Cross sections are then exported from 12D to HEC-RAS, where the hydraulic modelling of the diversion channel is performed. In HEC-RAS, a number of parameters are required to model the proposed channel design including the channel depth, channel width (both base and top widths), channel slopes and the channel bank slopes. In addition, levees are specified if required, which includes consideration of freeboard allowances. The Manning's 'n' channel roughness values were set at 0.045 for the main channel section and 0.07 for the channel banks.

Consideration was given to the NML railway bridge at the downstream end of the South Pit 1 and West Pit 1 diversion channels. Characteristics specified in the HEC-RAS model to accurately represent the hydraulic effect of these bridge structures included the bridge height, span, pier dimensions and abutments. The NML railway bridge details were obtained from the HEC-RAS model developed for Gap 50 Design (CoalConnect, 2009).

### 7.2.3 Channel design

The key principles that influence the design of the diversion channels are discussed below.

- The diversion channels were designed to have a channel slope similar to the natural watercourse conditions.
- The maximum required width to construct the channels includes an allowance for 10 m wide levees and an additional allowance of 30 m either side of the construction width to allow for design modifications and future possible erosion.
- The proposed channels were designed to meander to match the diverted channel length to the natural creek. The meanders increase the total required channel easement.
- The diversion channels were designed with a 1:5 bank slope, however this slope is dependent on soil characteristics and a geotechnical study of the area is required to confirm the design parameters and stability of the banks.

- The diversion channels were designed to meet the DERM guidelines on an established range of stream powers, velocities and shear stresses that are considered to be the upper range for natural Bowen Basin watercourses.
- The trapezoidal cross sections do not include channel features such as terraces, benches, riffles, etc. The design does not make an assessment of water surface superelevation around bends and the freeboard on the outside of bends may need to be increased locally where this occurs. These features will need to be considered in the detailed design and may increase the top width of the channel.
- Refer to Appendix A which includes indicative cross sections, long sections and velocity profiles for all diversions assessed.

**Table 7.6 Summary details of diversions**

| Detail | Diversion 1 | Diversion 2 | Diversion 3 | Diversion 4 |
|---|---|---|---|---|
| Minimum depth (m) | 3.6 | 4.2 | 4.4 | 2.4 |
| Minimum top width (m) | 56 | 62 | 64 | 40 |
| Bottom width (m) | 20 | 20 | 20 | 20 |
| Corridor length (m) | 3,700 | 5,750 | 2,580 | 1,700 |
| Corridor width (m)[1] | 215 | 325 | 355 | 130 |
| Deepest cut (m) | 7 | 9 | 7.5 | 6.5 |
| Highest levee bank (m) | 1.5 | 2 | 2 | 2 |
| Grade (%) | 0.30 | 0.22 | 0.23 | 0.20 |
| Bank slope | 1:5 | 1:5 | 1:5 | 1:5 |

*Note 1: Includes allowance for any meanders and levees*

### Diversion 1 – West Pit 1

The proposed diversion channel was designed to be a minimum depth of 3.6 m and minimum top width of 56 m with a 1:5 bank slope, to give a bottom width of 20 m. This caters for flows up to and including the 0.1% AEP event with a minimum freeboard of 0.5 m.

The diversion channel was designed to have a channel slope of approximately 0.3% grade (0.27 m fall every 100 m along the channel), similar to the natural watercourse conditions. This grade resulted in an increased cutting depth for the first 1,200 m (CH3600 to CH2400) of the diversion channel where the natural topography was rising. In this case a cutting depth of up to 7 m was required, with the greatest depth between CH2400 and CH2550 to allow for the 0.1% AEP design event discharges.

In the downstream section of the diversion channel (CH 0 to CH1350) the terrain is naturally declining and consequently levee banks will be needed to adequately provide for large event discharges. Levee banks will also be required in the first 400 m of the diversion channel (CH3200 to Ch3600).

The 0.5 m freeboard used for the levee design is the recommended value for levees outlined in the DERM Manual (DERM, 2012). The hydrological design criteria for a number of regulated structures including levees, specifies that the required crest level for levee embankments must be for a 0.1% AEP with an additional 0.5 m freeboard.

### Diversion 2 – South Pit 1

The proposed diversion channel was designed to be a minimum depth of 4.2 m and minimum top width of 62 m with a 1:5 bank slope, to give a bottom width of 20 m. This caters for flows up to and including the 0.1% AEP event with a minimum freeboard of 0.5 m.

The proposed channel was designed to meander to match the diverted channel length to the natural creek. The maximum required width of the channel construction is approximately 113 m where the required cut is the deepest at CH1250. The meanders will increase the total required channel easement to approximately 325 m width.

In this upper section of the channel (CH5400 to CH4200) the terrain is naturally declining and consequently levee banks will be needed to adequately provide for the 0.1% AEP design event discharge. Levee banks will also be required from CH400 downstream to the channel outlet at the NML. From CH4200 to CH400, the natural terrain is such that the diversion channel does not require levees.

The diversion channel was designed to have a channel slope of approximately 0.22% grade (0.224 m fall every 100 m along the channel), similar to the natural watercourse conditions.

### Diversion 3 – South Pit 1

The combined watercourse length of Diversion 2 and Diversion 3 is 8,380 m. This replaces the natural watercourse that crosses South Pit 1 and has a stream length of 8,300 m.

The proposed Diversion 3 channel was designed to be a minimum depth of 4.4 m, with a minimum top width of 64 m and a 1:5 bank slope, to give a bottom width of 20 m.

The proposed channel was designed to meander to reduce stream velocities and better represent natural flow conditions, similar to the existing waterway. The maximum easement width required is approximately 355 m at CH6250 where the deepest cut is required.

The upstream 1,600 m of the diversion channel requires levee banks to cater for flows up to and including the 1,000 year ARI event. In the areas of higher terrain between CH6800 and CH5750, a large cut will be required.

The diversion channel was designed to have a channel slope of approximately 0.23% grade (0.23 m fall every 100 m along the channel), similar to the natural watercourse conditions.

### Diversion 4 – East Pit 2

The diversion channel replaces a length of 1,724 m of natural watercourse across East Pit 2 and its spoil dump. By diverting the channel north to an alternative watercourse with a longer stream length, the diversion channel does not need to provide compensatory channel length with wide meanders, however the channel was designed to meander within the available space to mimic as much as possible the natural creek.

The diversion channel was designed to be a minimum depth of 2.4 m, with a top width ranging between 40 m to 69 m and a 1:5 bank slope, to give a bottom width of 20 m. This caters for flows up to and including the 0.1% AEP event with a minimum freeboard of 0.5 m.

The maximum required depth of the channel construction is approximately 6.6 m at CH600 where the required cut is the deepest. The total required channel easement is approximately 130 m width at this location.

Levee banks are required to convey the design event discharges between CH1385 and CH1,000. Levee banks will also be required from CH200 downstream to the diversion channel outlet at the junction with the existing tributary flowing between East Pit 1 and East Pit 2. From CH1,000 to CH200, the natural terrain is such that the diversion channel does not require levees and the channel will be in cut.

The fall in elevation over the route of the diversion channel was 2.8 m, which gives a channel slope of approximately 0.2% grade (0.2 m fall every 100 m along the channel), similar to the natural watercourse conditions.

### 7.2.4 Hydraulic characteristics

The DERM guidelines on Watercourse Diversions provides advice on an established range of stream powers, velocities and shear stresses that are considered to be the upper range for natural Bowen Basin watercourses, which are shown in Table 7.1.

As a check on the hydraulic characteristics of the diversion channel, the maximum stream power, velocity and shear stress in the diversion channels were assessed against the DERM guidelines. The results are shown in the Table 7.4. As can be seen the values of the hydraulic parameters in the diversion channel are all within the guideline values for natural watercourses in the Bowen Basin.

**Table 7.7 Maximum values of hydraulic parameters in the diversion channel**

| Scenario | Max. Stream Power (Watts/$m^2$) | Max. Velocity (m/s) | Max. Shear Stress (N/$m^2$) |
|---|---|---|---|
| DERM guideline on Watercourse Diversions | | | |
| 50% AEP | <35 | <1.0 | <40 |
| 2% AEP | <220 | <2.5 | <80 |
| Diversion 1 | | | |
| 50% AEP | 21 | 1.0 | 21 |
| 2% AEP | 71 | 1.6 | 44 |
| 0.1% AEP | 148 | 2.2 | 68 |
| Diversion 2 | | | |
| 50% AEP | 17 | 0.9 | 18 |
| 2% AEP | 68 | 1.6 | 42 |
| 0.1% AEP | 144 | 2.2 | 66 |
| Diversion 3 | | | |
| 50% AEP | 22 | 1.0 | 21.5 |
| 2% AEP | 66 | 1.6 | 41 |
| 0.1% AEP | 59 | 2.0 | 118 |

| Scenario | Max. Stream Power (Watts/m$^2$) | Max. Velocity (m/s) | Max. Shear Stress (N/m$^2$) |
|---|---|---|---|
| | | Diversion 4 | |
| 50% AEP | 7.0 | 0.7 | 11 |
| 2% AEP | 19 | 1.0 | 19 |
| 0.1% AEP | 37 | 1.3 | 29 |

As can be seen from the results in Table 7.4, the values of the hydraulic parameters in all diversion channels are all within the guideline values for natural watercourses in the Bowen Basin.

### 7.2.5 Assessment of receiving waterway stability

#### Diversions 1, 2 and 3

The drainage diversion channels 1, 2 and 3 will move the flow of water around the mining area, but still enter their receiving waterways at the same location. Therefore negligible change in flow or velocity is expected to occur in the receiving waterways.

#### Diversion 4

Diversion channel 4 redirects a small part of the catchment upstream of East Pit 2 into the natural waterway between East Pit 1 and East Pit 2. This increases the contributing catchment of the natural waterway by less than 10%, however, this is not expected to significantly alter the flow or velocity of the waterway.

### 7.2.6 Impact of natural watercourse on East Pit 1 and East Pit 2

During modelling of the pit diversions modelling results of the existing watercourse between East Pit 1 and East Pit 2 have indicated that the 0.1% AEP flood levels will encroach into the area allocated for the spoil heap of East Pit 1 and may clip the south-western corner of East Pit 1. Table 7.6 below summarised the 0.1% and 1% flood levels at the pit and spoil heaps.

**Table 7.8 Flood water levels at key locations in the natural watercourse between East Pit 1 and East Pit 2**

| Location | 0.1% AEP flood level (mAHD) | 1% AEP flood level (mAHD) |
|---|---|---|
| East Pit 1 | | |
| South-west side of spoil heap | 305.41 | 304.34 |
| South-east side of spoil heap | 305.49 | 305.12 |
| South-west side of pit | 305.90 | 305.66 |
| Centre of southern edge of pit | 307.72 | 307.40 |
| South-east side of pit | 309.12 | 308.81 |
| East Pit 2 | | |
| North-west side of spoil heap | 305.41 | 304.34 |
| North-east side of spoil heap | 305.49 | 305.12 |
| North-west side of pit | 306.39 | 306.11 |
| Centre of northern edge of pit | 308.66 | 308.30 |
| North-east side of pit | 311.18 | 310.76 |

The toe of the spoil dump will either need to be relocated outside the flood extent, or constructed in a manner such that it is non-erodible when in contact with flood waters.

# 8 Conclusions

The hydrology and hydraulics of the Suttor River in the vicinity of the proposed Project have been investigated in this study.

The flood hydrology of the river has been modelled using a verified rainfall-runoff model to estimate design flood discharges. A high-quality calibration of the hydrologic model against historic flood events was achieved. The design flood estimates obtained for this investigation are based on CRC-FORGE rainfall intensities with temporal patterns adopted for Region 3 (ARR, 2001).

A fully 2D hydrodynamic model (SOBEK) of the Suttor River and floodplain was developed to assess the impact of mine pits and waste rock dumps on surrounding property and infrastructure. The SOBEK model was used to assess the flood levels for a range of flood events for the existing waterway conditions. The model was then altered to simulate changes to the floodplain resulting from the Project.

Flooding impacts were expressed in terms of increases to flood levels (afflux), changes to the extent of inundation and changes in channel and overland flow velocity.

The modelling results indicated that the affluxes are restricted to several localised areas, and are of low order. Velocity impacts tend to be localised in constricted points and adjacent to the waste rock dumps. The results showed that while there is a measurable change (with respect to existing conditions) the relative increase is small and it does not suggest a significant additional widespread scour risk.

The impact due to the Project can be summarised as follows:

- There is no significant change in the flood extents for the post-development scenario.
- The pre-development inundation extents for the 1, 2 and 5 year ARI flood events show that the palustrine wetland located to the west of the operational areas of the site is not inundated during these more frequent flood events. This indicates that flooding from the Suttor River is not the main source of water for the wetland. The Northern Missing Link elevated rail embankment and associated culverts is located between the wetland and the Project. Potential impacts of the Project on this area are addressed in the Mine Water Management Plan (KBR, 2012a) and the ecological component of the EIS.
- In the 1,000 year ARI event flood waters reach the western waste rock dump with depths up to 2.0 m and velocities in the order of 1.0 m/s adjacent to the waste rock in some locations. This will require armouring up to the 1,000 year ARI flood level such that it is non-erodible when in contact with flood waters. Alternately the toe

of the spoil dump can be relocated outside the flood extent. It should be noted that the peak depth and velocity at the face of the WRD do not occur together.

- The GAP Newlands Rail Line is not overtopped in the pre-development Suttor River 100 year ARI flood event and this flood immunity is not affected by the proposed mine development. There are minor impacts at the GAP Newlands Rail Line (maximum 0.15 m for the 100 year ARI flood event) and negligible impacts at proposed Alpha Coal Project Rail Branch.
- The northern part of West Pit and the Southern MIA and CHPP follows the catchment divide between the Suttor River and Bowen River catchments. The terrain is above the Suttor River floodplain and the small gullies in these areas are not of sufficient size to warrant flood modelling for the EIS.
- A review of the flood assessments for the Northern Missing Link project indicates that South Pit 2 is not at risk from Suttor Creek and therefore South Pit 2 is not included in the SOBEK flood model.
- Assessment of all final void impacts is included in the Final Void Assessment (KBR, 2012b).

1D steady state (HEC-RAS) models were developed where diversions to existing watercourses are necessary to gain access to coal reserves. The diversion channels included levees to contain the 1,000 year ARI local catchment flows from unnamed tributaries of the Suttor River. These levees have been included in the SOBEK model and extended to high ground at their downstream extent to protect the mine pits from backwater flooding in the Suttor River 1,000 year ARI event.

The flood modelling in this report does not include the north pit, north pit diversions and northern MIA / CHPP/ co-disposal which relates to flooding in the Kangaroo Creek catchment and has no bearing on flooding in the Suttor River.

A small drainage diversion is planned to allow water to bypass the North Pit and flow to Kangaroo Creek. This drainage diversion is to be in place before mining operations commence at the North Pit. The drainage diversion put in place would remain as a permanent structure to divert water around the North Pit and its final void. The northern MIA is located across a tributary of Kangaroo Creek and may require culverts or a bridge to provide access and protect the area.

This infrastructure is not planned to commence until approximately year 15–17 and will therefore be assessed once further confirmatory surveys have been completed in that specific area. However, assessment of potential final void impacts from Kangaroo Creek is included in the Final Void Assessment, BEW106-TD-WE-REP-0006 (KBR, 2012b).

# 9 References


CoalConnect (2009), Gap50 Design Report, Brisbane, Queensland

Department of Environment and Resource Management (2011), Watercourse Diversions – Central Queensland Mining Industry, Central West Region, Queensland

Department of Environment and Resource Management (2012), Manual for Assessing Hazard Categories and Hydraulic Performance of Dams, Brisbane, Queensland

Department of Infrastructure and Planning (2010), Increasing Queensland's resilience to inland flooding in a changing climate: Final report on the Inland Flooding Study

Institution of Engineers, 1987 & 2001. Australian Rainfall and Runoff, A Guide to Flood Estimation

KBR, 2012a, Technical Report: Mine Water Management Strategy, prepared for QCoal Pty Ltd, Brisbane, Queensland.

KBR, 2012b, Technical Report: Final Void Assessment, prepared for QCoal Pty Ltd, Brisbane, Queensland.

*Appendix A*

# CROSS SECTIONS AND LONG SECTIONS OF PROPOSED DIVERSION CHANNELS

*Appendix A*

# Cross sections and Long sections of proposed diversion channels

## DIVERSION 1 – WEST PIT 1

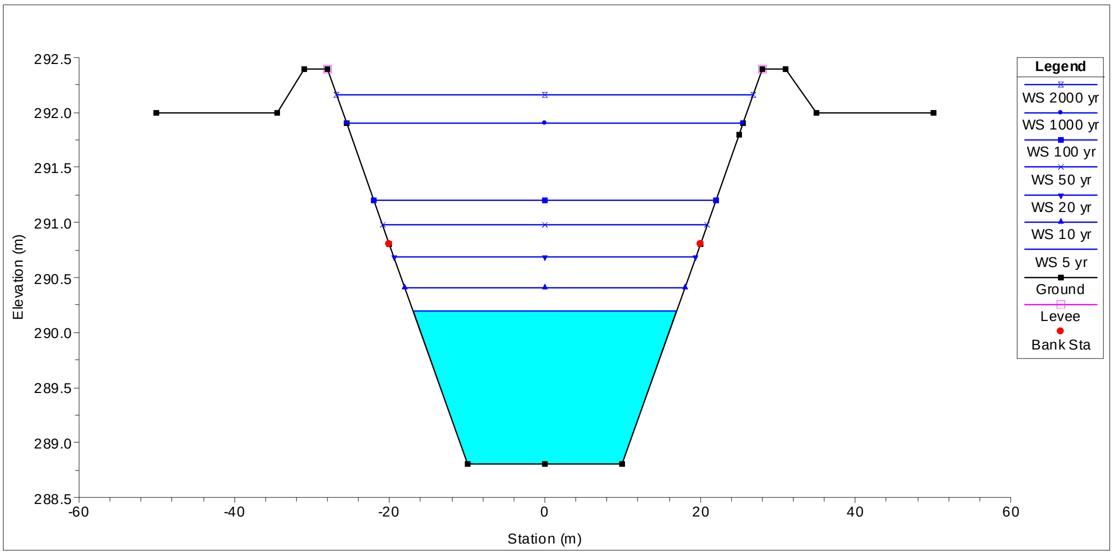


**Figure 1**
**DIVERSION CHANNEL 1 – CROSS SECTION at CH900**

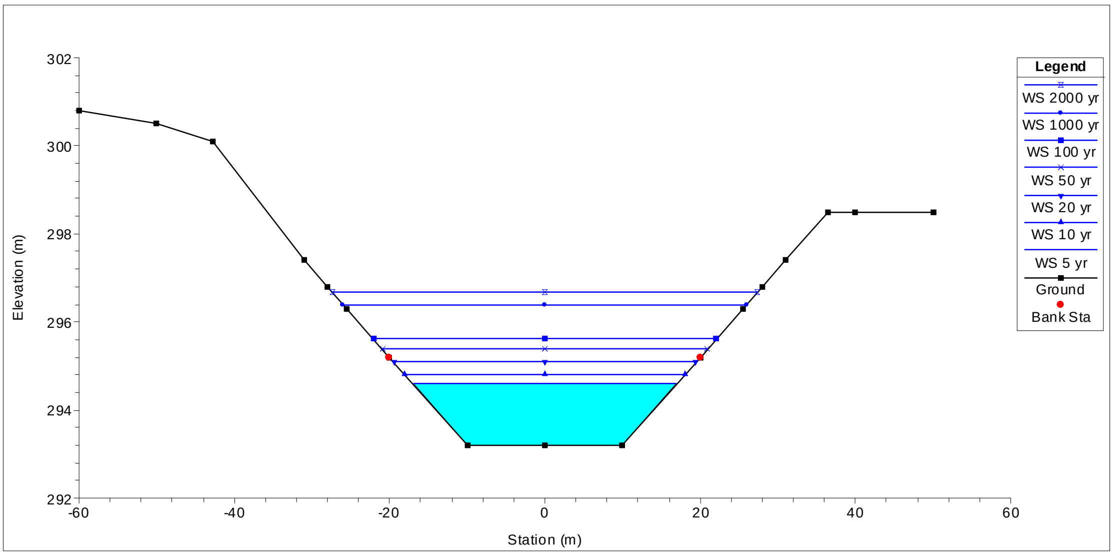


**Figure 2**
**DIVERSION CHANNEL 1 – CROSS SECTION at CH2550**

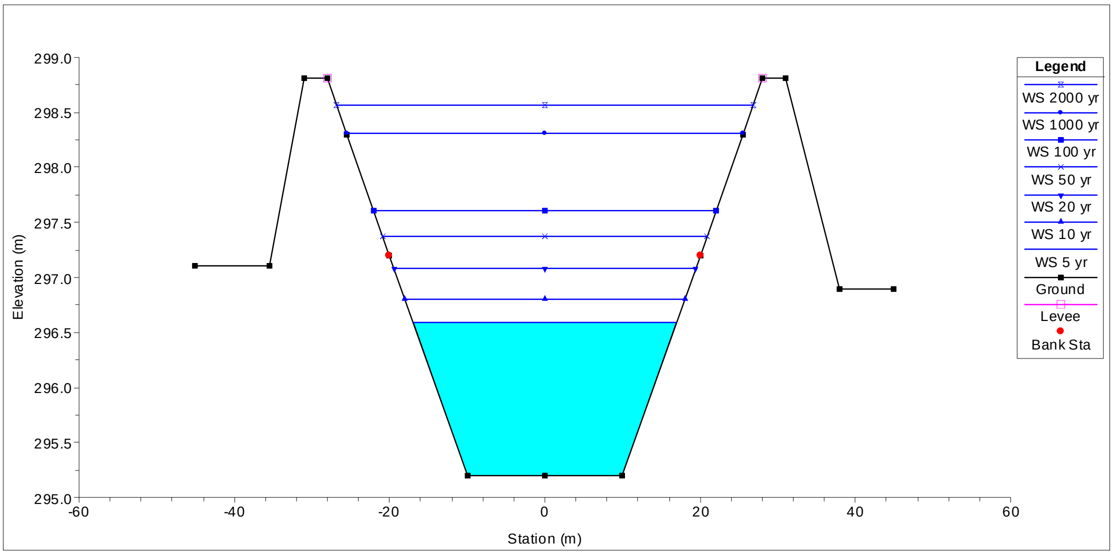


**Figure 3**
**DIVERSION CHANNEL 1 – CROSS SECTION at CH3300**

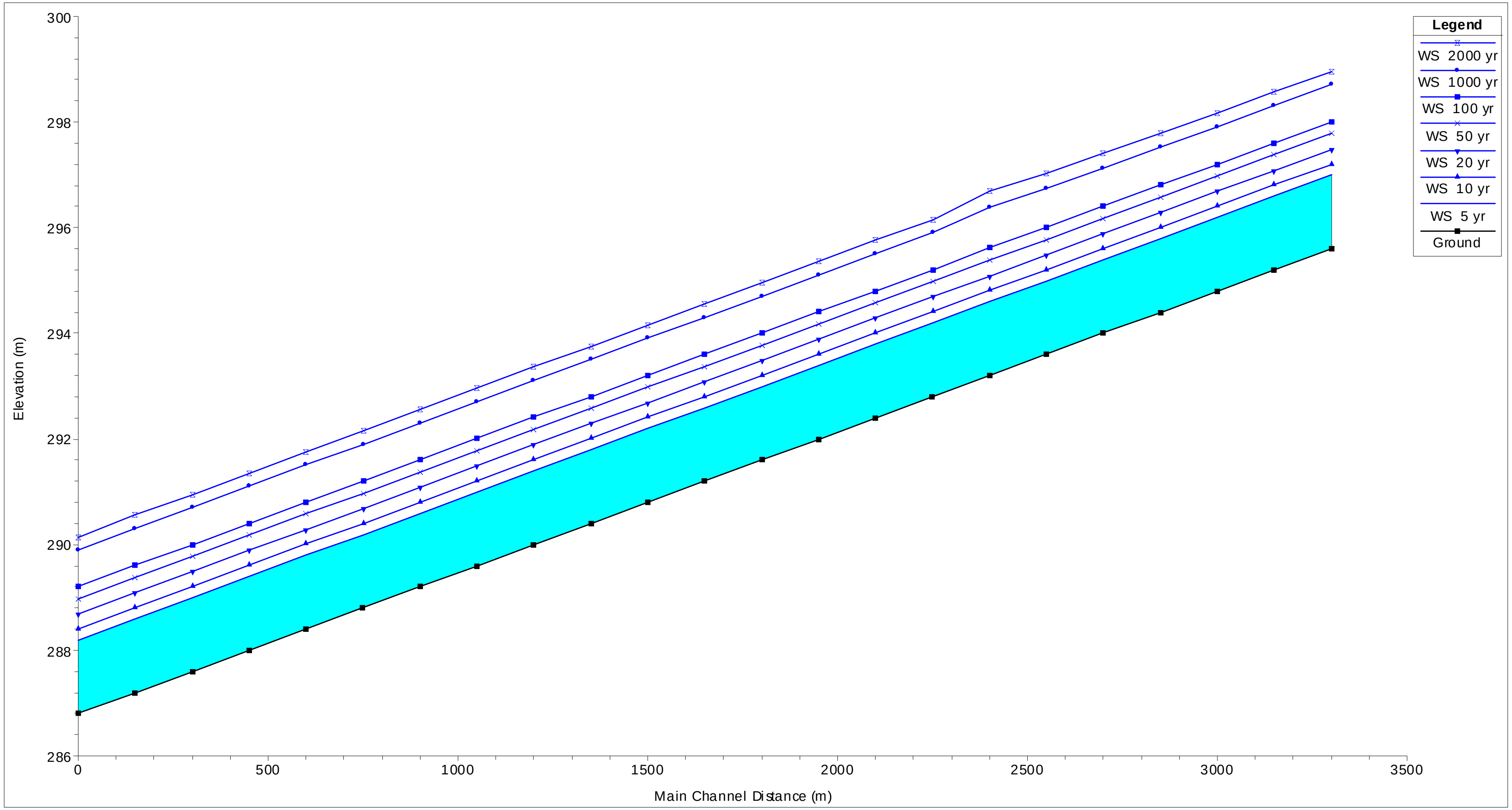


**Figure 4**

**DIVERSION CHANNEL 1 – LONG SECTION**

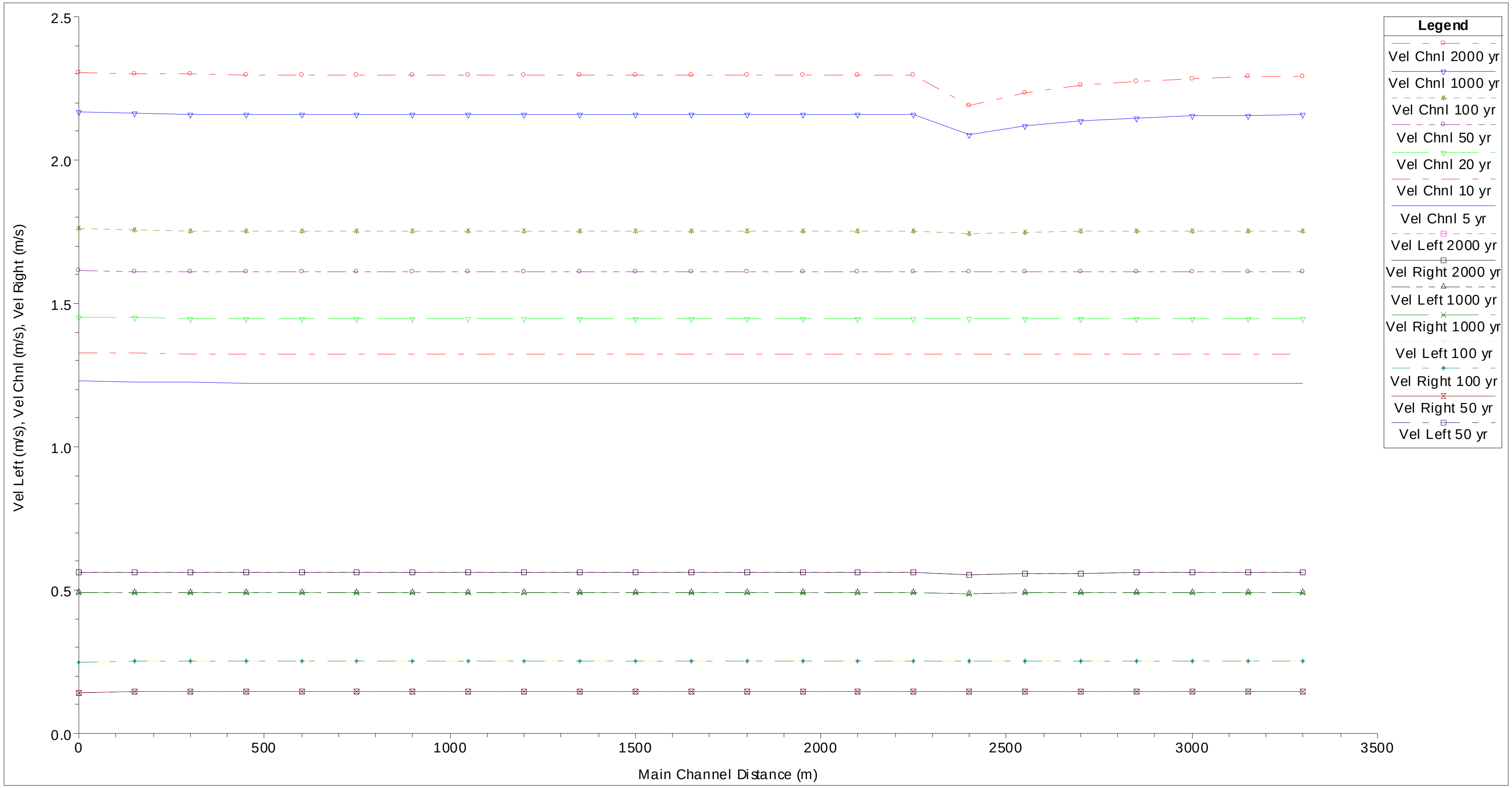


**Figure 5**
**DIVERSION CHANNEL 1 – VELOCITY PROFILE**

## DIVERSION 2 – SOUTH PIT 1

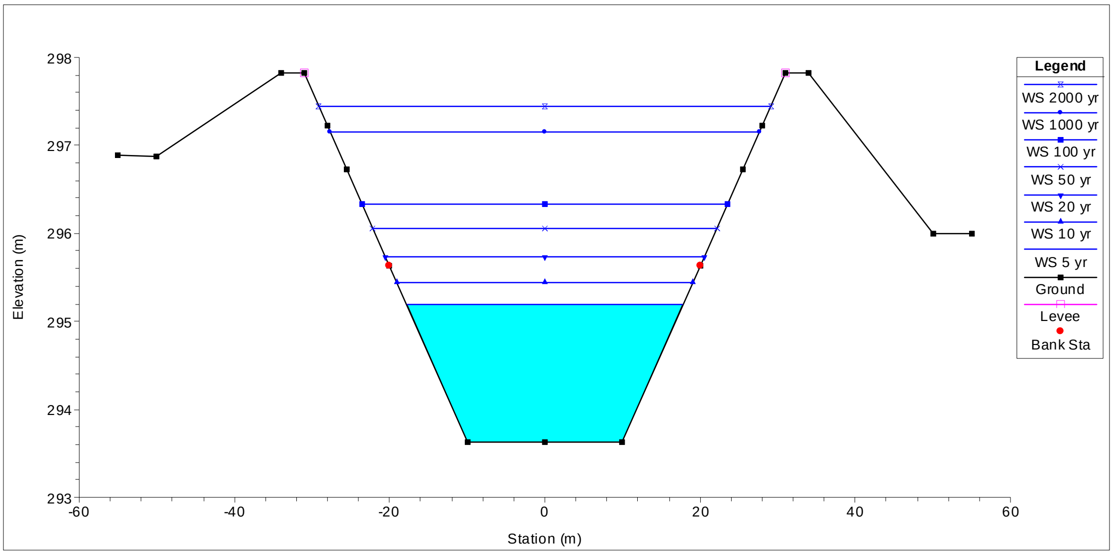


**Figure 6**
**DIVERSION CHANNEL 2 – CROSS SECTION at CH4750**

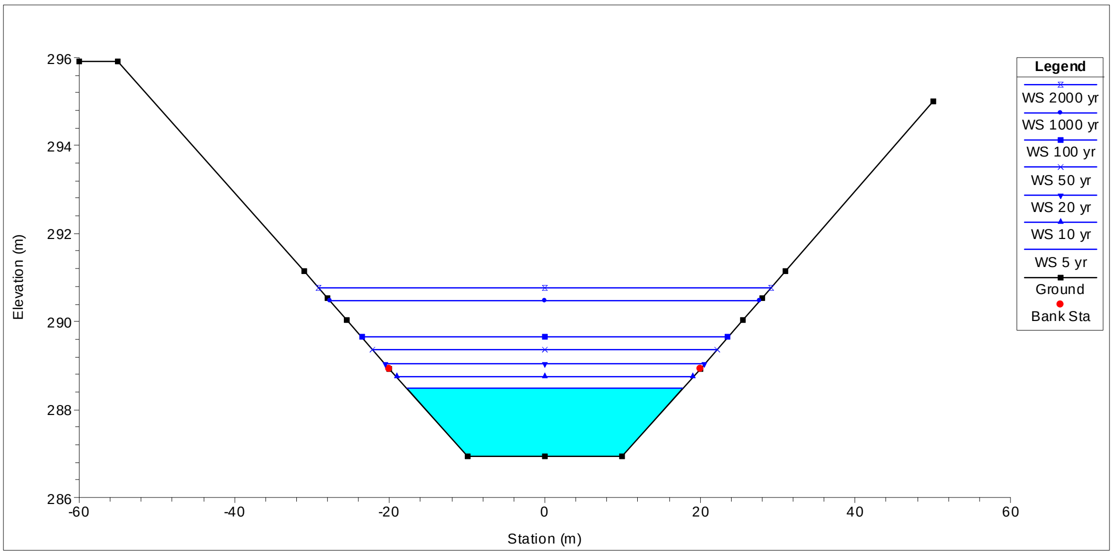


**Figure 7**
**DIVERSION CHANNEL 2 – CROSS SECTION at CH1750**

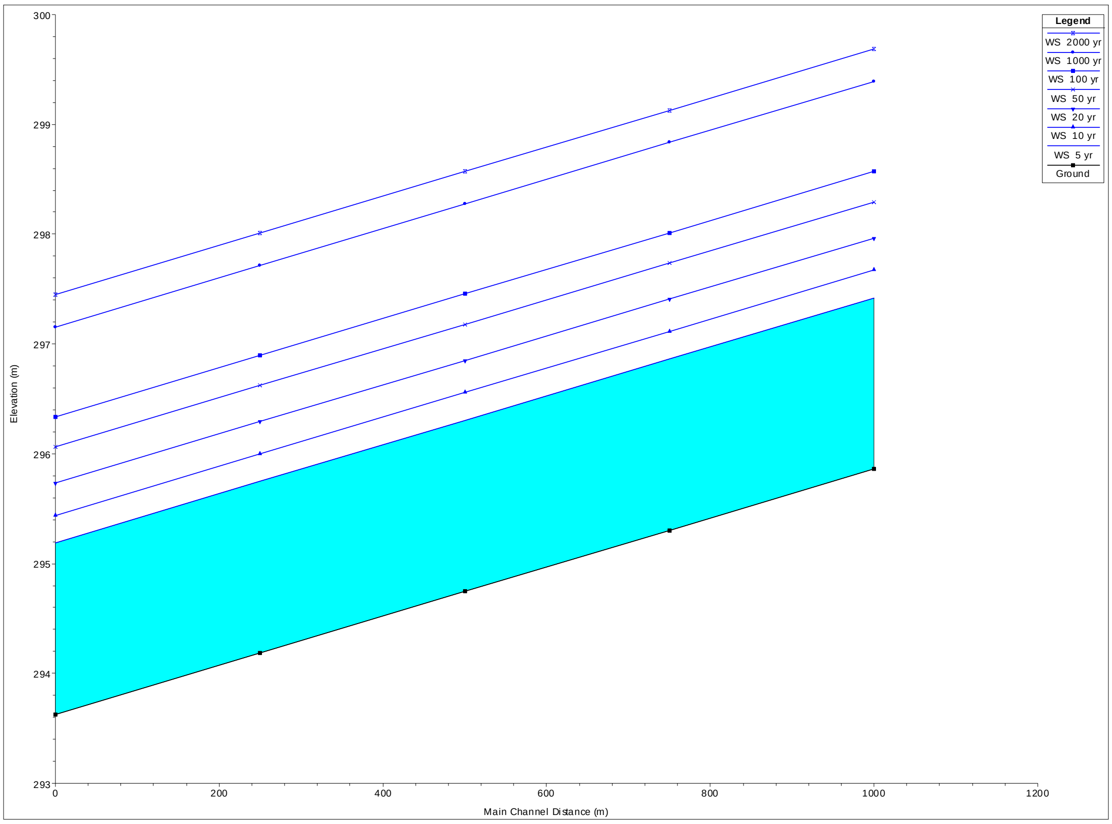


**Figure 8**
**DIVERSION CHANNEL 2 – LONG SECTION CH5750 to CH4750**

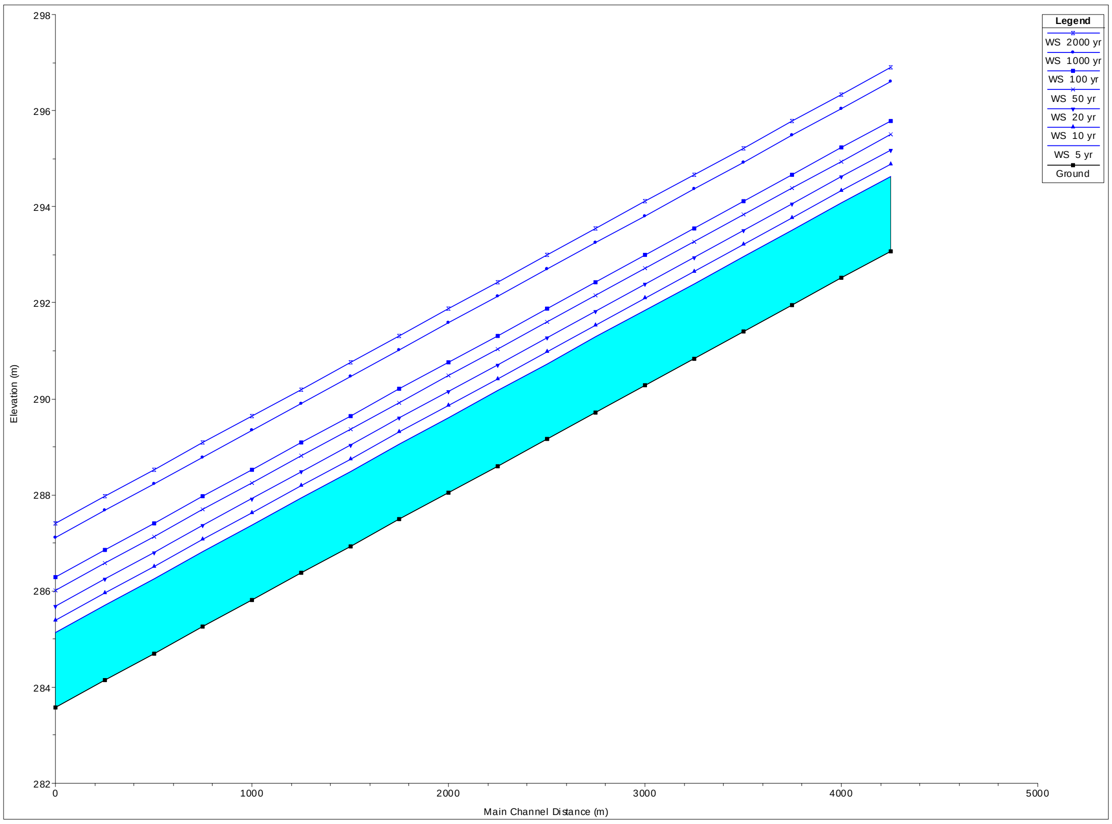


**Figure 9**
**DIVERSION CHANNEL 2 – LONG SECTION CH4500 to CH0**

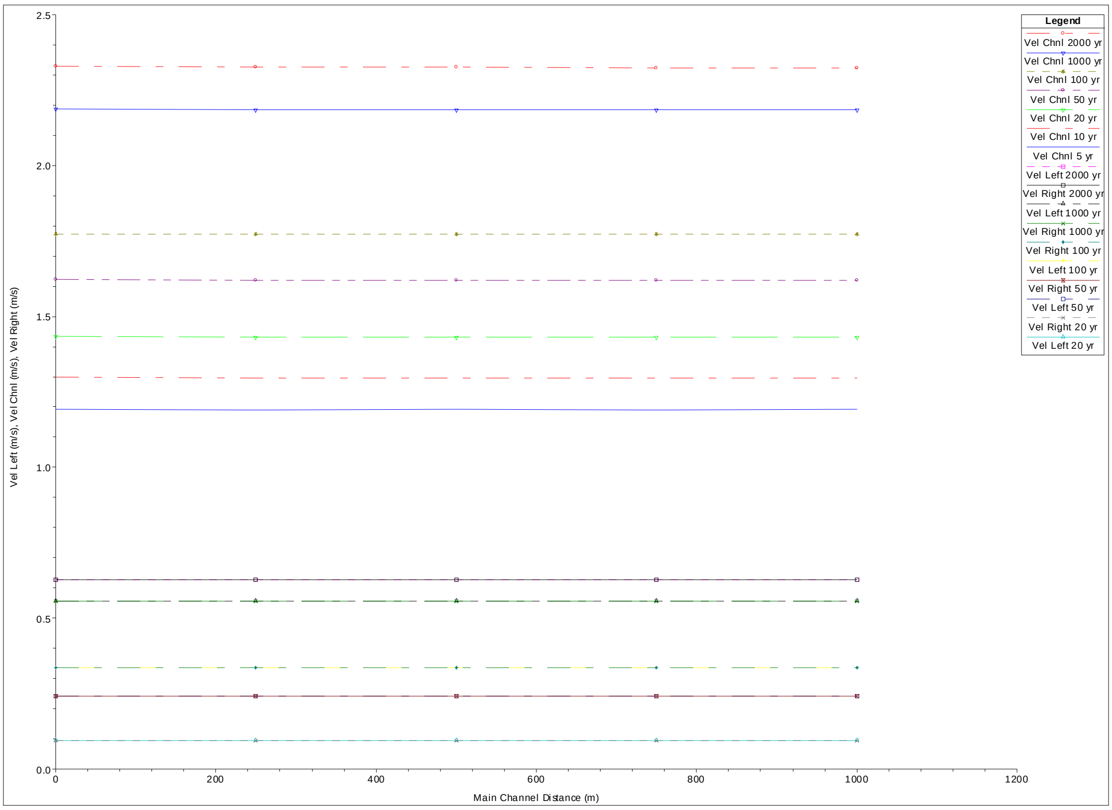


**Figure 10**
**DIVERSION CHANNEL 2 – VELOCITY PROFILES CH5750 to CH4750**

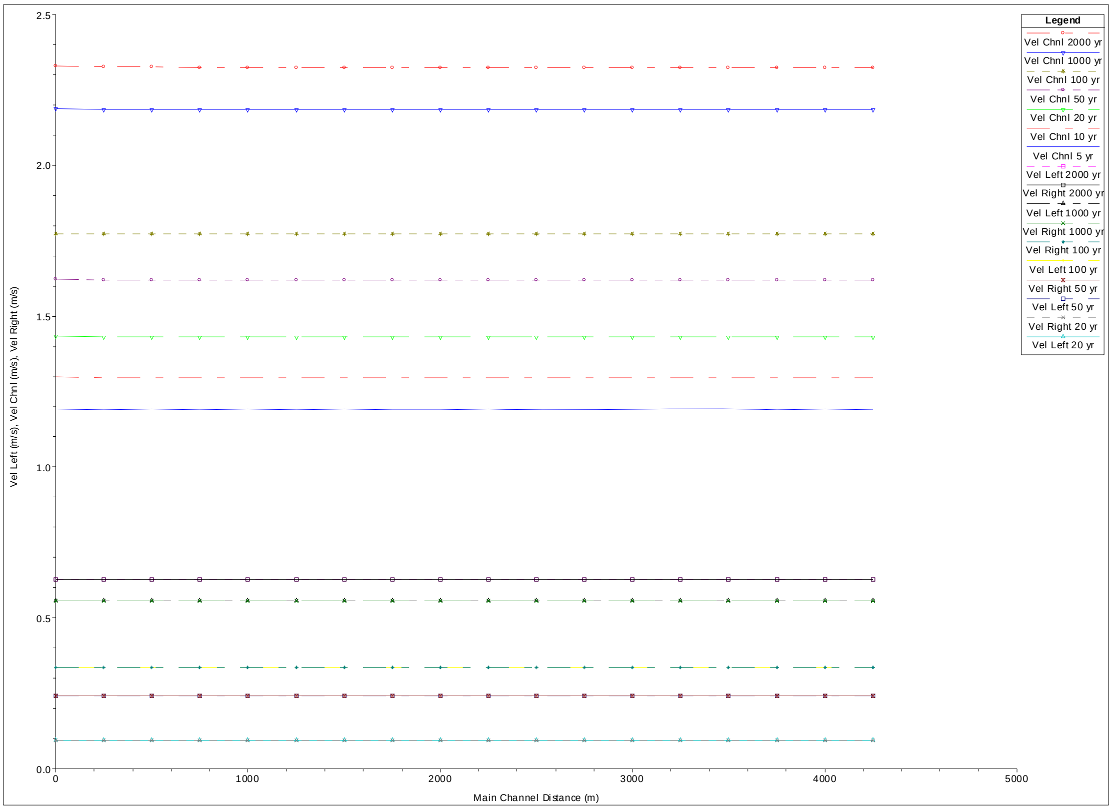


**Figure 11**
**DIVERSION CHANNEL 2 – VELOCITY PROFILES CH4500 to CH0**

## DIVERSION 3 – SOUTH PIT 1

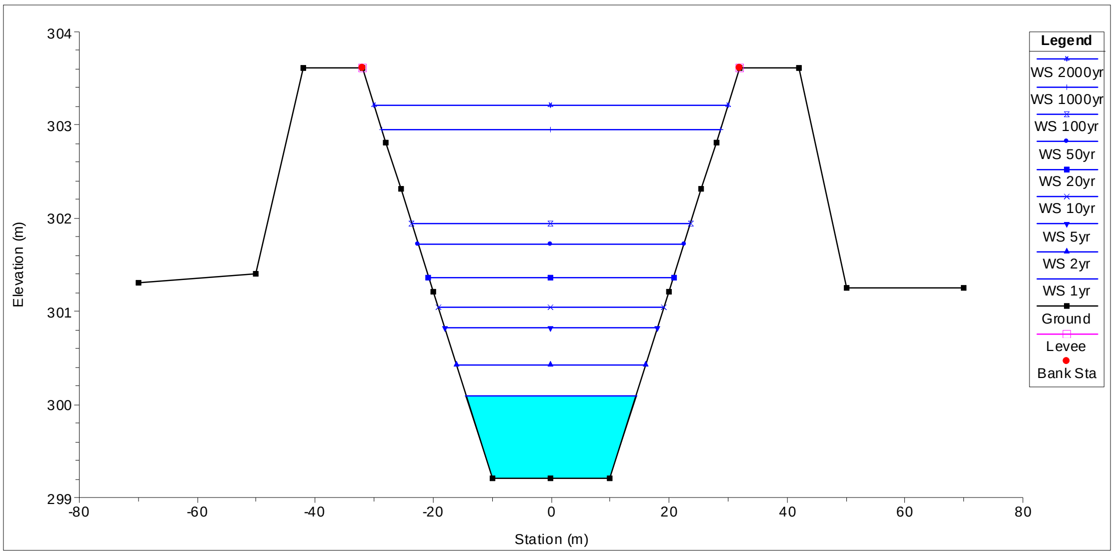


**Figure 12**
**DIVERSION CHANNEL 3 – CROSS SECTION at CH7250**

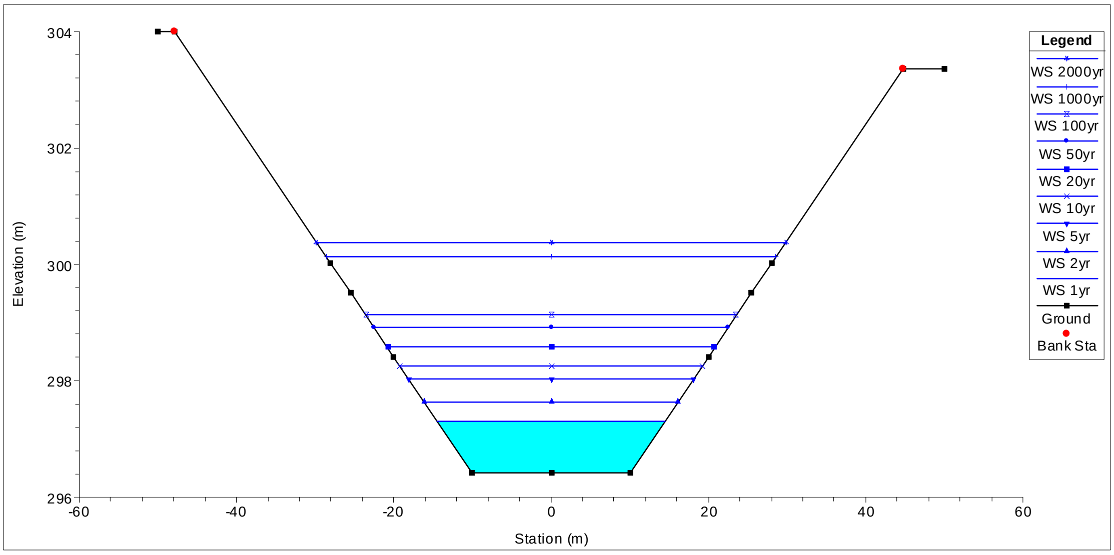


**Figure 13**
**DIVERSION CHANNEL 3 – CROSS SECTION at CH6000**

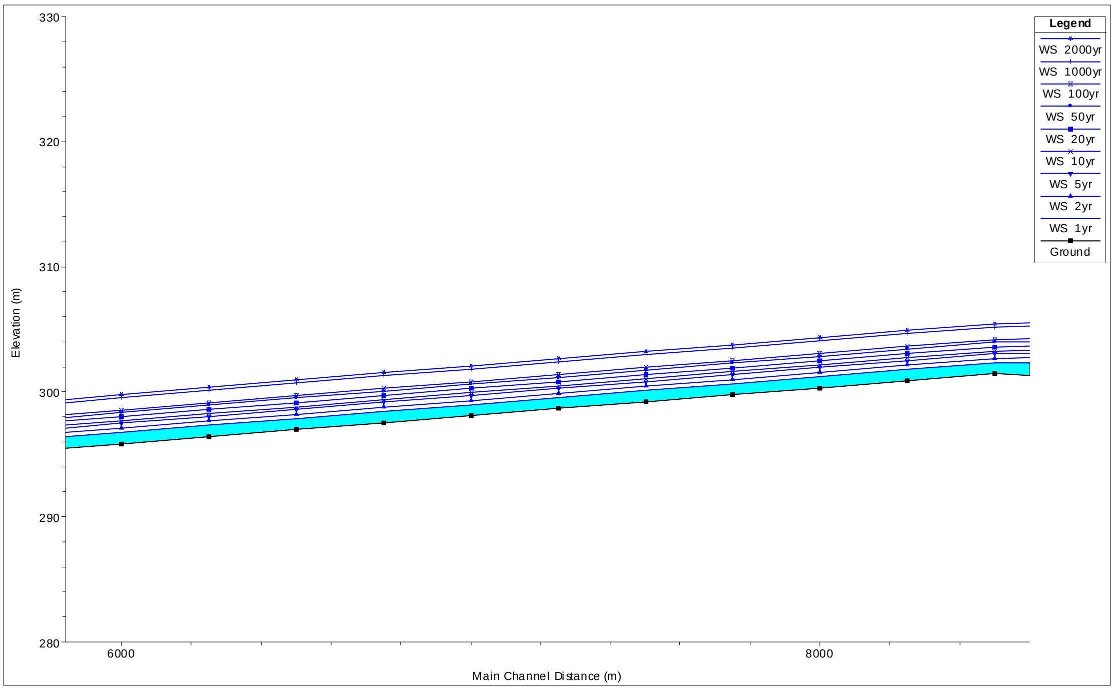


**Figure 14**
**DIVERSION CHANNEL 3 – LONG SECTION CH8500 to CH5750**

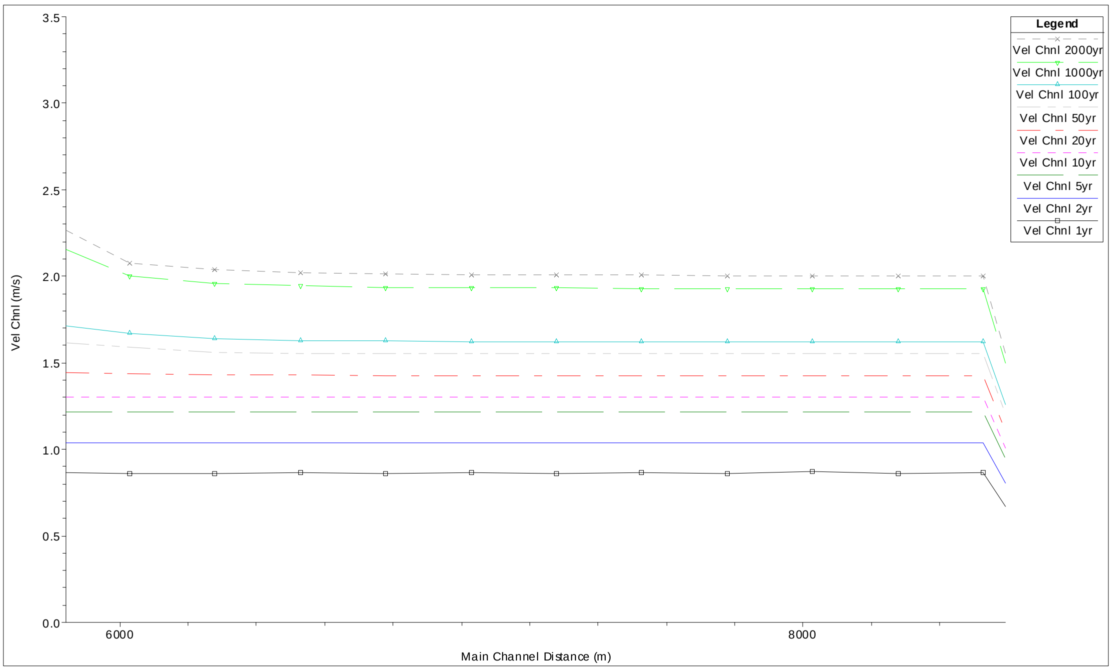


**Figure 15**
**DIVERSION CHANNEL 3 – VELOCITY PROFILES CH8500 to CH5750**

## DIVERSION 4 – EAST PIT 2

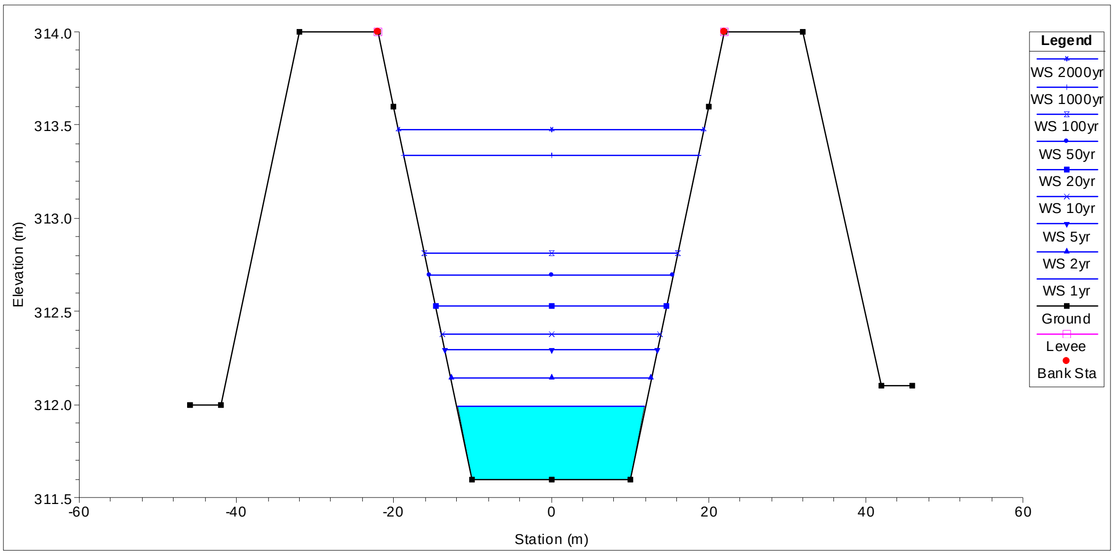


**Figure 16**
**DIVERSION CHANNEL 4 – CROSS SECTION at CH1200**

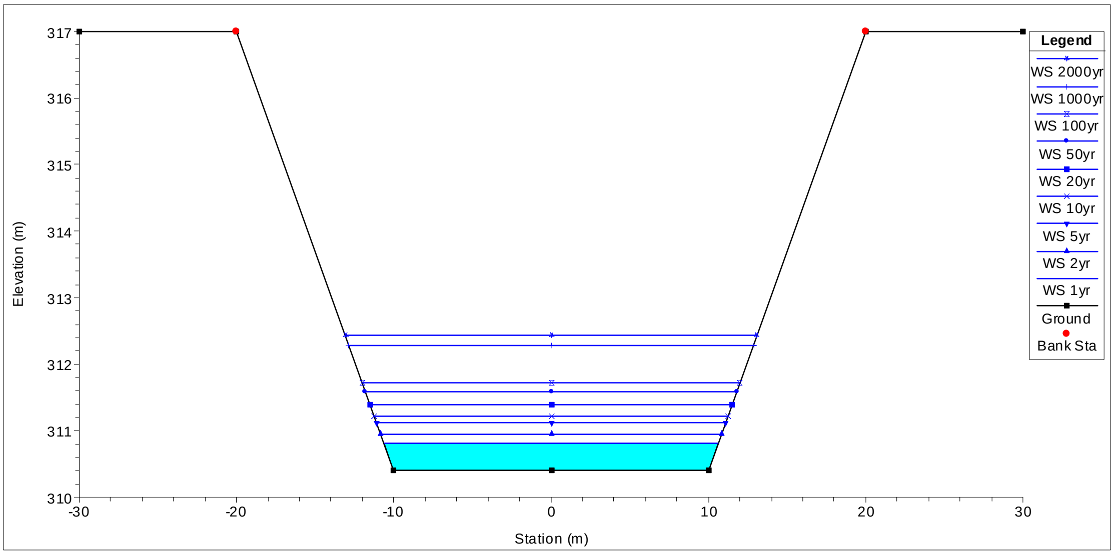


**Figure 17**
**DIVERSION CHANNEL 4 – CROSS SECTION at CH600**

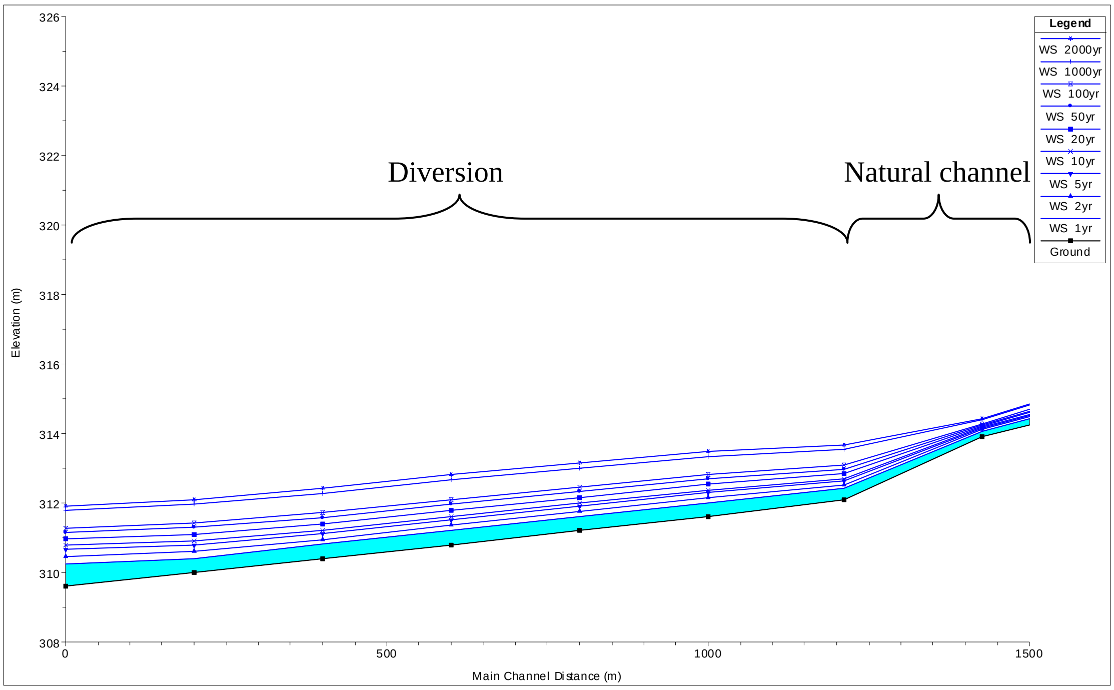


**Figure 18**
**DIVERSION CHANNEL 4 – LONG SECTION CH1385 to CH0**

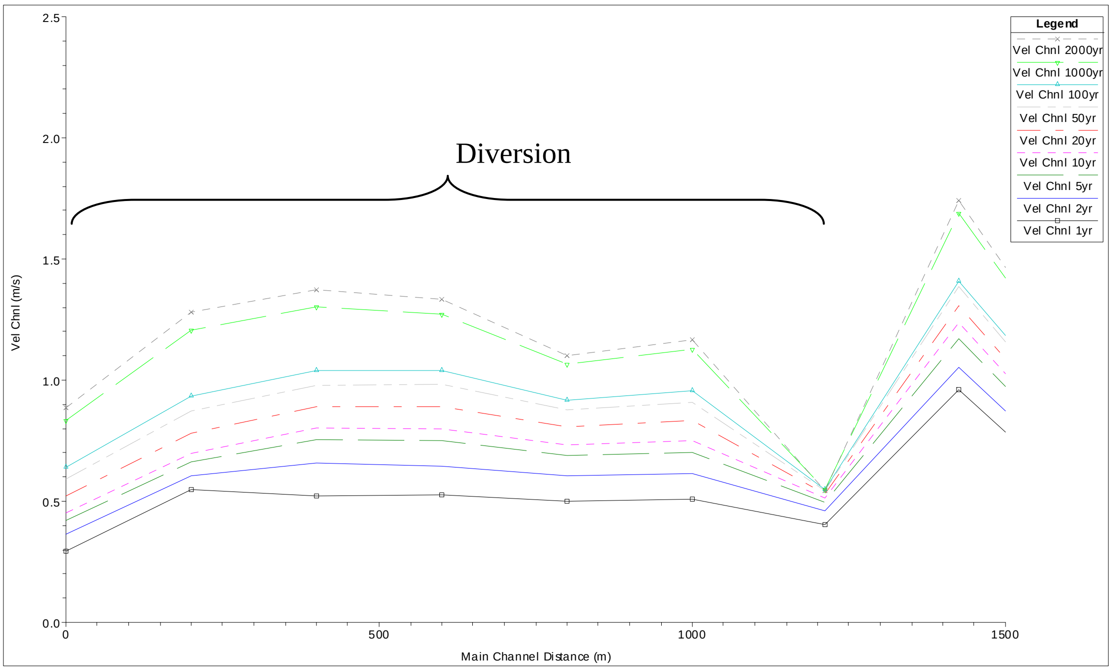


**Figure 19**
**DIVERSION CHANNEL 4 – VELOCITY PROFILES CH1385 to CH0**