PROJECT CHINA STONE

Open Cut Mine Drainage Report

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Project China Stone - EIS Open Cut Mine Drainage Assessment

Hansen Bailey

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Glossary

AEP	Annual Exceedance Probability (%)
AHD	Australian Height Datum
ARI	Average Recurrence Interval
BoM	Bureau of Meteorology
D/S	Downstream
EIS	Environmental Impact Statement
ha	Hectares
km	Kilometre
m	Metre
m^2	Square metre
m^3	Cubic metres
m^3/s	Cubic metres per second
mAHD	Metres Australian Height Datum
Mtpa	Million tonnes per annum
N/m^2	Bed shear stress (pascals)
PMF	Probable maximum flood
PMP	Probable maximum precipitation
RCBC	Reinforced concrete box culvert
ROM	Run Of Mine
tpa	Tonnes per annum
TSF	Tailings Storage Facility
TUFLOW	Two dimensional hydraulic modelling software
U/S	Upstream
V:H	Vertical : Horizontal
WRM	WRM Water and Environment Pty Ltd
W/m^2	Stream Power (power)
XP-RAFTS	Rainfall runoff routing model

1 Introduction

1.1 PROJECT OVERVIEW

WRM Water & Environment Pty Ltd (WRM) was commissioned by Hansen Bailey on behalf of MacMines Austasia Pty Ltd (the proponent) to complete an open cut mine drainage assessment as part of the Environmental Impact Statement (EIS) for Project China Stone (the project).

The project involves the construction and operation of a large-scale coal mine on a greenfield site in Central Queensland. The project site (the area that will ultimately form the mining leases for the project) is remote, being located approximately 270 km south of Townsville and 300 km west of Mackay at the northern end of the Galilee Basin (Figure 1.1). The closest townships are Charters Towers, approximately 285 km by road to the north, and Clermont, approximately 260 km by road to the south-east. The project site comprises approximately 20,000 ha of well vegetated land, with low-lying scrub in the south and east and a densely vegetated ridgeline, known as 'Darkies Range', running north to south through the western portion of the site.

The mine will produce up to approximately 55 million tonnes per annum (Mtpa) of Run of Mine (ROM) thermal coal. Coal will be mined using both open cut and underground mining methods (Figure 1.2). Open cut mining operations will involve multiple draglines and truck and shovel pre-stripping. Underground mining will involve up to three operating longwalls. Coal will be washed and processed on site and product coal will be transported from site by rail. It is anticipated that mine construction will commence in 2016 and the mine life will be in the order of 50 years.

The majority of the mine infrastructure will be located in the eastern portion of the project site (Figure 2). Infrastructure will include coal handling and preparation plants (CHPPs), stockpiles, conveyors, rail loop and train loading facilities, workshops, dams, tailings storage facility (TSF) and a power station. A workforce accommodation village and private airstrip will also be located in the eastern part of the project site.

Figure 1.1 Project China Stone Locality Map (Source: Hansen Bailey)

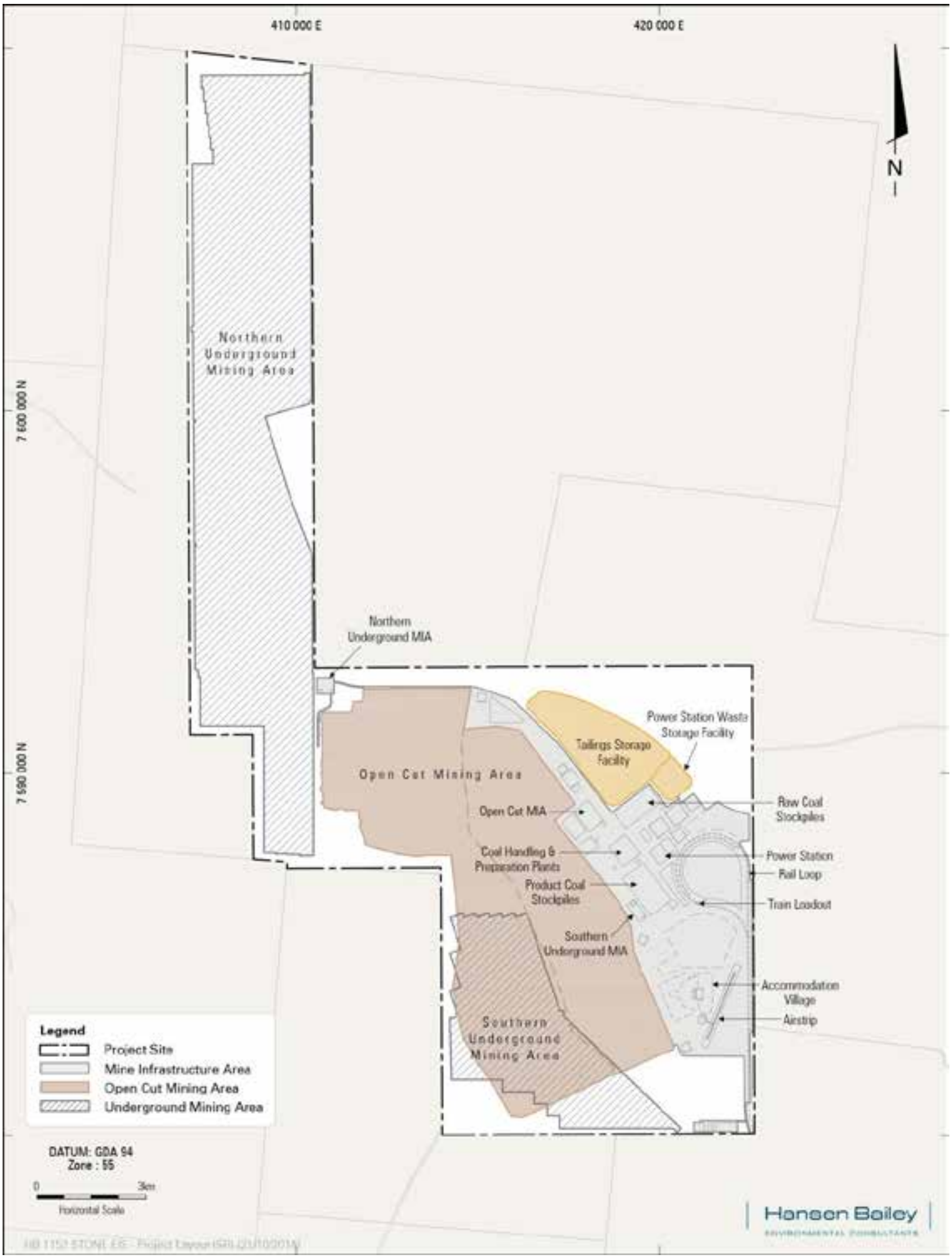

Figure 1.2 Project China Stone Layout (Source: Hansen Bailey)

1.2 STUDY SCOPE

The open cut mining area and mine infrastructure are located in the southern section of the project site (refer Figure 1.2). This area is referred to as the study area for the purposes of this report. The study area is within the headwaters of the North Creek and Tomahawk Creek catchments (refer Figure 1.3). Darkies Range, located at the western boundary of the project site, forms the western limit of both the North Creek catchment and the Tomahawk Creek catchment. The study area drains to the east via numerous ephemeral drainage lines (upper tributaries of North Creek and Tomahawk Creek). The drainage lines form steep gullies in the west of the study area within the steeper topography associated with Darkies Range. The gullies transition to wide flat overland flowpaths in the eastern portion of the study area where the topography is relatively flat. There are no watercourses, as defined under the Water Act 2000, within the project site (DNRM, 2014a). Tomahawk Creek and North Creek become watercourses approximately 20 km and 8km downstream of the project site, respectively.

A drainage strategy for the open cut mining area and mine infrastructure area has been developed as an integral component of project planning. The site drainage strategy was designed to ensure suitable drainage arrangements and associated flood protection are provided for both the operations phase and post mine closure. The site drainage strategy involves diverting runoff from truncated catchment areas upstream of the open cut pit around the open cut mine and mine infrastructure area. This will be achieved by the construction of drains along the final highwall of the open cut pit and the establishment of drainage corridors at the northern and southern ends of the open cut mine and infrastructure areas (refer Figure 1.3).

The highwall drains will minimise the contributing catchment areas of the open cut pits during operations. This will provide flood protection for the operating pits and limit the generation of mine affected pit water. The highwall drains and the northern and southern drainage corridors will remain in place after mine closure. They have therefore been designed to ensure they will remain stable in the long term. They have been designed with capacity to convey the peak flows from the Probable Maximum Flood (PMF).

This report presents the conceptual designs for the proposed highwall drains and an assessment of the performance of the northern and southern drainage corridors during representative stages of the operations phase and post mine closure. It also provides an assessment of the potential impacts of the drainage strategy on downstream flood levels, flow velocities and waterway and floodplain stability.

The report is structured as follows:

- Section 1 - Introduction: provides an overview of the project and the study scope;
- Section 2 – Study Area Drainage Setting;
- Section 3 – Proposed Drainage Strategy;
- Section 4 – Methodology;
- Section 5 – Design Discharge Estimation;
- Section 6 - Existing Flood Conditions;
- Section 7 – Highwall Drain Concept Design;
- Section 8 – Operations and Closure Phase Flooding;
- Section 9 – Downstream Impacts; and
- Section 10 – Summary;

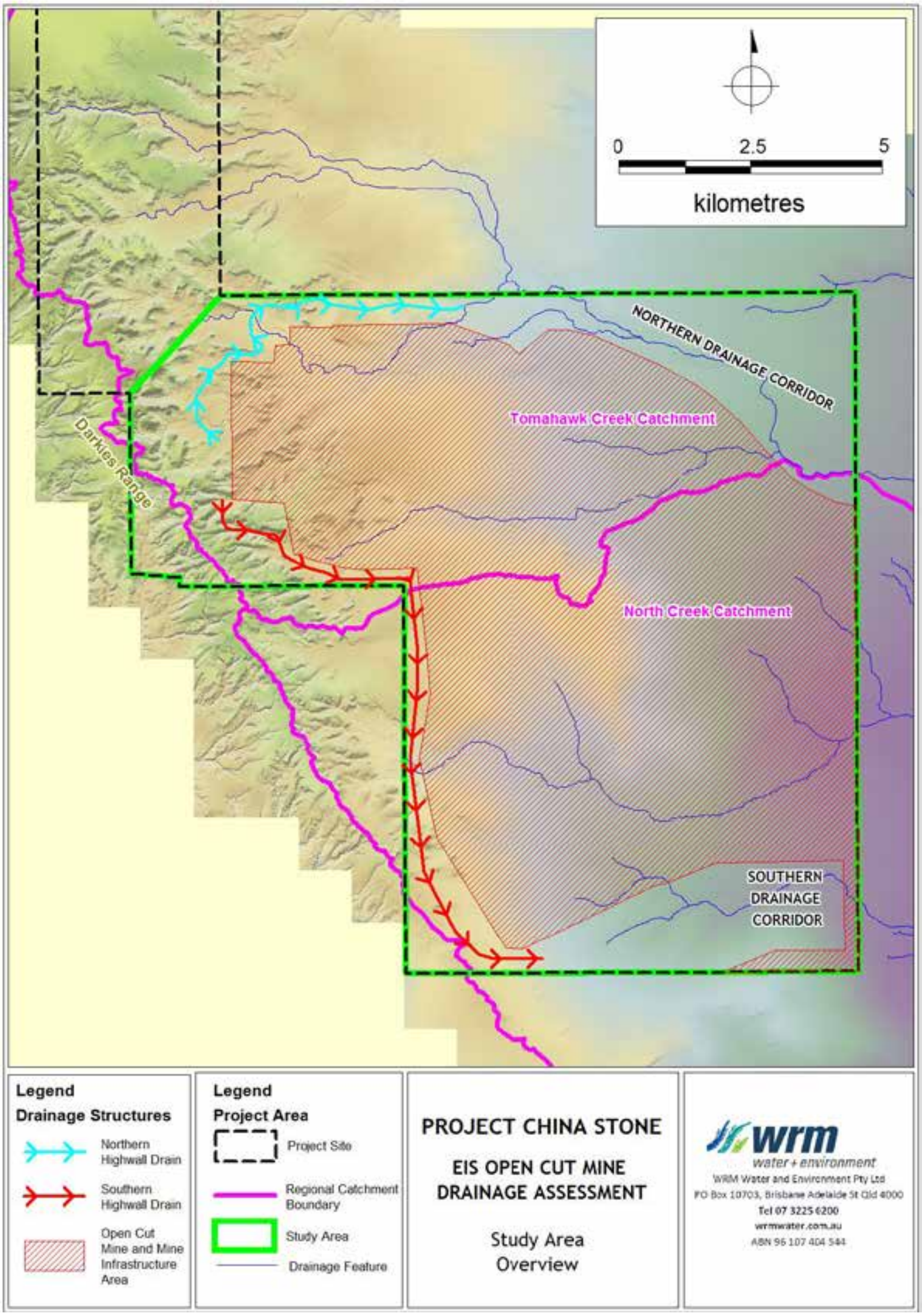

Figure 1.3 Study area overview

2 Study area drainage setting

2.1 OVERVIEW

The open cut mine and mine infrastructure area is located in the headwaters of the Tomahawk Creek and North Creek catchments and covers an area of approximately 135km^2. This is referred to as the study area for the purposes of this report. The study area is drained by upper tributaries of Tomahawk and North Creek. Both Tomahawk Creek and North Creek are tributaries of the Belyando River, which is a tributary of the Suttor River, which in turn is a tributary of the Burdekin River.

The study area is bounded to the west by the Darkies Range which is characterised by steep rocky escarpments as shown in Figure 2.1. The topography of the majority of the study area is characterised by flatter terrain and loose sandy soils as shown in Figure 2.2. The entire project area is covered with scattered trees and grass. Elevations at the project site range from about 450mAHD along the crest of Darkies Range to about 270m along the eastern boundary.

Figure 2.1 Steep, rocky terrain along Darkies Range

Figure 2.2 Flat terrain with sandy soils, showing scattered trees and grass cover

2.2 CATCHMENTS AND DRAINAGE FEATURES

Figure 2.3 shows the drainage features of the study area. There are no watercourses, as defined by the Water Act 2000, on the project site (DNRM, 2014a). The characteristics of drainage features within the study area differ substantially from the steep upper catchment adjacent to Darkies Range, to the wide flat overland flowpaths evident across the majority of the study area. The drainage features within the study area are described in detail below.

2.2.1 Regional catchment setting

The study area is split between the Tomahawk Creek and North Creek catchments. The northern half of the study area is located in the catchment of Tomahawk Creek, and the southern half is located in the catchment of North Creek. The study area is traversed by a number of unnamed tributaries of Tomahawk Creek and North Creek that originate along Darkies Range. The unnamed tributaries of Tomahawk Creek are referred to as drainage feature 1, 2, 3 and 4, and the unnamed tributaries of North Creek are referred to as drainage feature 5 and 6.

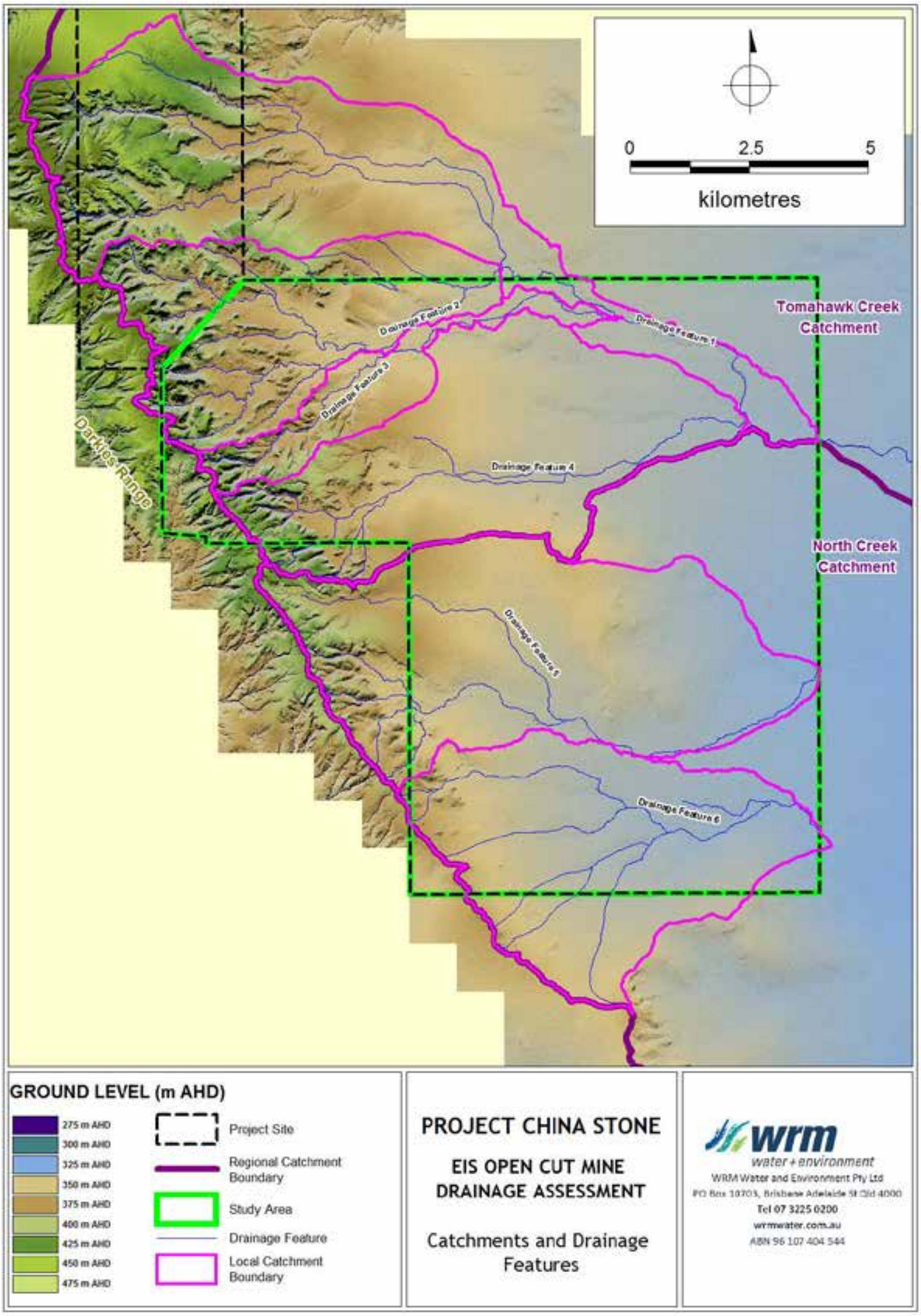

Figure 2.3 Catchments and drainage features

2.2.2 Drainage features

Drainage feature 1 is the largest of the unnamed Tomahawk Creek tributaries, with a catchment area of approximately 3,470ha upstream of the northern boundary of the study area. Despite the large catchment size, drainage feature 1 does not have a defined channel along much of its length upstream of the study area. The drainage feature 1 catchment area increases to approximately 5,842ha at the confluence with drainage feature 2. Figure 2.4 is a photograph of the drainage feature 1 channel downstream of the confluence with drainage feature 2. Figure 2.4 clearly shows that despite the large catchment area, the drainage feature 1 channel has minimal flow conveyance capacity, with the majority of flood flows likely conveyed via the floodplain.

Figure 2.4 Channel of drainage feature 1 downstream of confluence with drainage feature 2

The catchment area of drainage feature 1 at the eastern boundary of the study area is approximately 10,260ha (including the catchments of drainage features 2, 3 and 4). Figure 2.5 is photograph of the drainage feature 1 channel upstream of the eastern boundary of the project site. Figure 2.5 confirms that the drainage feature 1 channel has limited capacity, despite having a large upstream catchment.

Drainage features 2 to 6 originate along Darkies Range at the western boundary of the study area, via a number of confined, steep and rocky channels. The drainage features transition to wide, shallow overland flowpaths in the flatter regions of the project site, typically with no defined channels.

Figure 2.6 is a photograph of the steep upper reaches of drainage feature 2 (approximately 700m downstream of the catchment divide), showing the steep and confined channel representative of the upper catchment drainage features. It is likely that runoff is confined to the channel during flow events, with high velocities and significant flow depths. Channel bed slopes in the upper reaches of the drainage features at the project site range from 1% to 2.5%.

Figure 2.7 is a photograph of drainage feature 6 in the flat, sandy part of the study area, about 3.4km upstream of the eastern boundary of the project site. Despite a significant upstream catchment, there is no channel evident here, indicating that runoff is conveyed via wide, shallow, slow moving sheet flow. Bed slopes along the drainage features in the flatter portion of the project site range from 0.2% to 0.5%.

Figure 2.5 Channel of drainage feature 1 upstream of eastern boundary of project site

Figure 2.6 Upper reach of drainage feature 2

Figure 2.7 Middle/lower reach of drainage feature 6

3 Proposed drainage strategy

3.1 YEAR 5 MINE PLAN

The proposed layout of the open cut mine and mine infrastructure for Year 5 is shown in Figure 3.1. Key drainage control structures include:

- A northern highwall drain. This drain receives runoff from an area of approximately 1,350ha, consisting of the upper reaches of drainage feature 2. The highwall drain diverts runoff in the upper reaches of drainage feature 2 around the northern end of the open cut mining area before releasing it into the existing alignment of drainage feature 2 upstream of the confluence with drainage feature 1;
- The development of the Tailing Storage Facility (TSF), the outer embankment of which will protect some areas of the mine infrastructure area from flooding in the northern drainage corridor. The TSF embankment will be designed and constructed to ensure it is suitable to serve this purpose.
- A southern highwall drain which receives runoff from an area of approximately 960ha (including the upper reaches of drainage feature 3 and drainage feature 4) and diverts it to the south of the proposed open cut mining area to the southern drainage corridor.
- The highwall drains will be sized to capture and divert runoff from the upstream catchments for all events, up to and including the Probable Maximum Flood (PMF) event.

The open cut pits, overburden emplacements and mine infrastructure areas are generally located in the middle and lower catchments of drainage features 2, 3 and 4.

The proposed road and rail loop along the eastern boundary of the project site will cross drainage feature 6, and are located downstream of the outlet of the southern highwall drain. The rail loop and haul road will include culvert openings to pass discharges from the southern drainage corridor whilst maintaining immunity from flooding for up to and including the 1 in 50 Annual Exceedance Probability (AEP) event. The proposed locations of the culverts are shown on Figure 3.1. Table 3.1 gives preliminary concept designs for the culverts.

Table 3.1 Railway embankment culverts concept design

Location	Culvert Dimensions	Number of Barrels	Nominal Invert Level (mAHD)
A	0.9m H x 2.7m W RCBC	10	283.75
B	0.9m H x 2.7m W RCBC	10	281.3
C	1.2m H x 3.6m W RCBC	33	281.4
D	1.2m H x 3.6m W RCBC 0	27	282.1
E	0.9m H x 2.7m W RCBC	10	281.7
F	2.1m H x 3.6m W RCBC	3	275.0
	1.2m H x 3.6m W RCBC	42	275.9

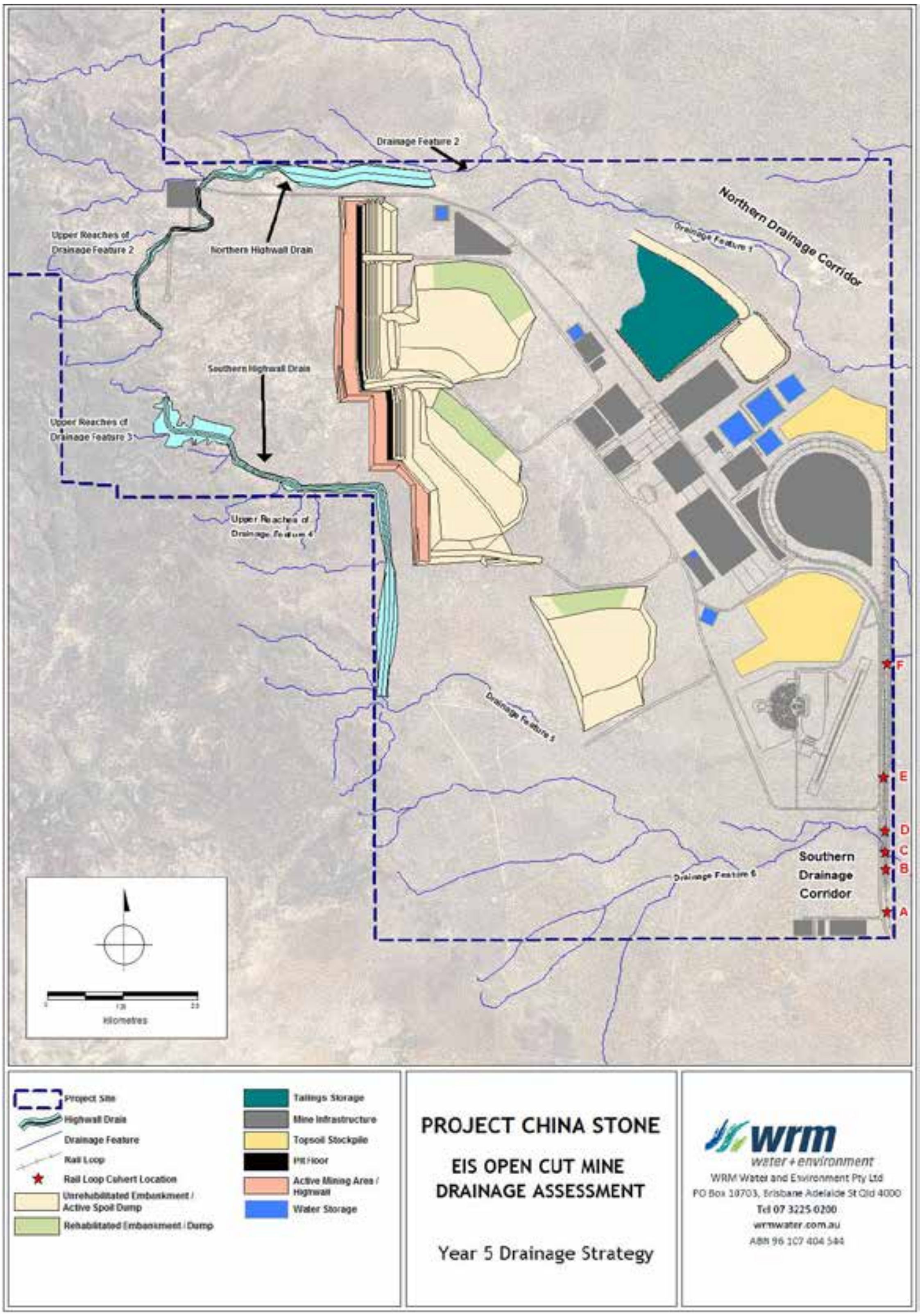

Figure 3.1 Year 5 open cut mine layout plan and railway culvert locations

3.2 YEAR 30 MINE PLAN

The proposed layout of the open cut mine and mine infrastructure for Year 30 is shown in Figure 3.2. Key drainage control structures include:

- The northern highwall drain and is unchanged from Year 5.
- The TSF has expanded to occupy its final footprint.
- The southern highwall drain has been extended and now receives runoff from an area of approximately 1,700ha, including the upper reaches of drainage feature 5 and 6 and diverts it to the southern boundary of the project site before releasing it into drainage feature 6; and
- The open cut pit and associated overburden emplacement have expanded to the south, forming the northern edge of the southern drainage corridor.

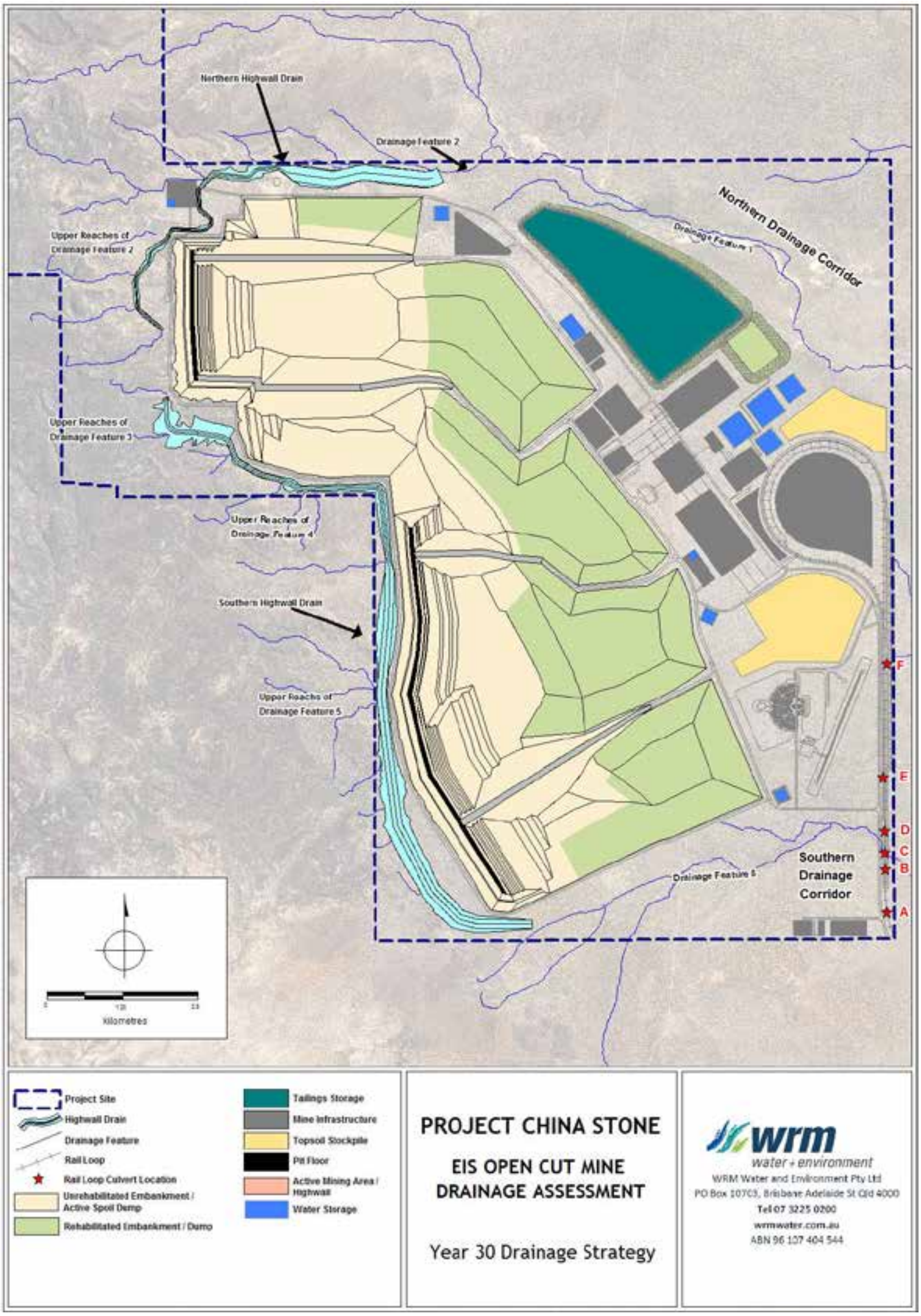

Figure 3.2 Year 30 open cut mine layout

3.3 FINAL LANDFORM

Figure 3.3 shows the proposed final landform for the open cut mine and mine infrastructure area. All mine infrastructure will be removed, and only the rehabilitated overburden emplacements, TSF and power station waste storage facility will remain, along with the final void and highwall drains.

The highwall drains will ensure that no runoff from the upstream catchments drains to the final void for any event up to and including the PMF.

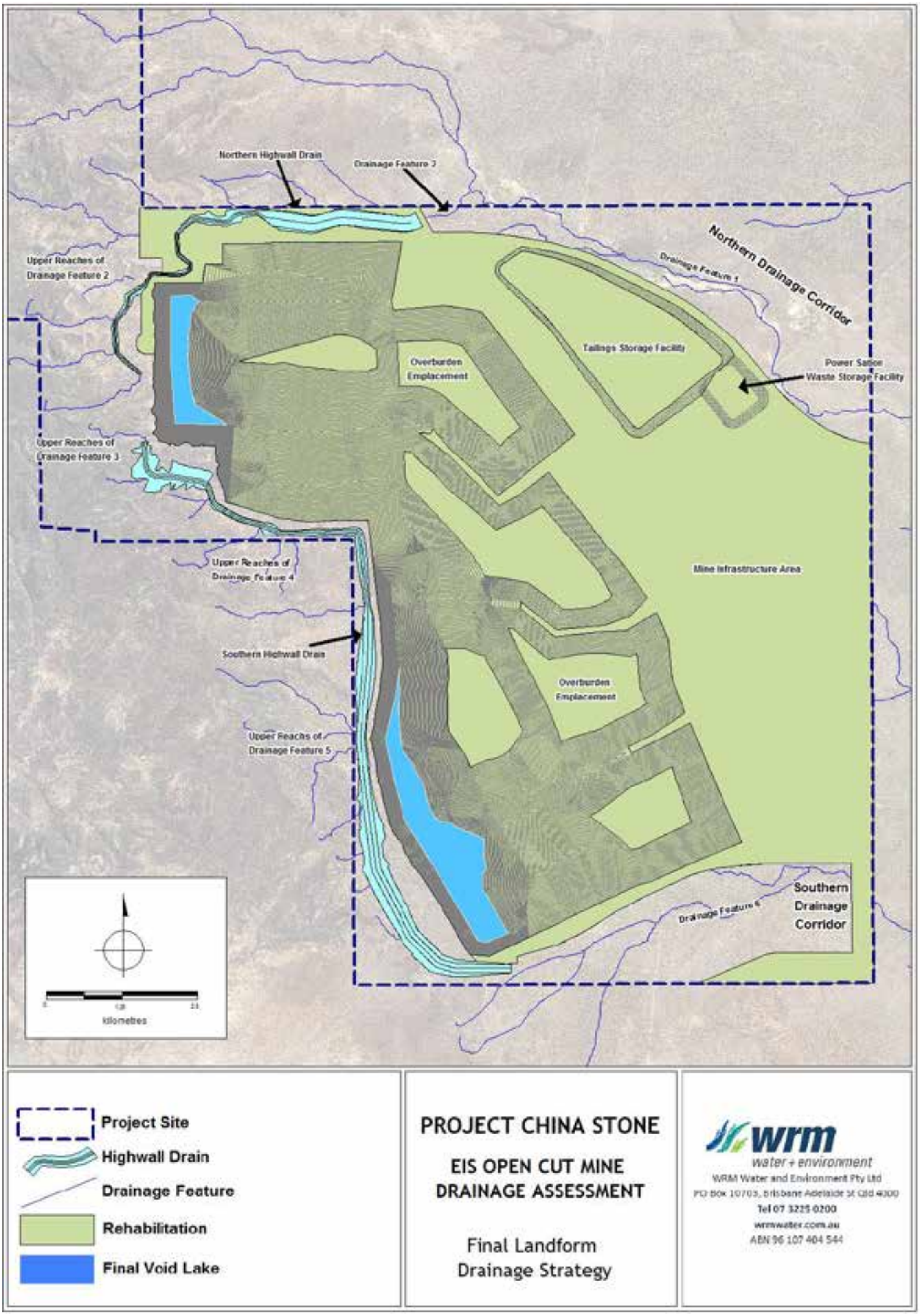

Figure 3.3 Open cut mine final landform

4 Methodology

4.1 FLOOD MODELLING

Hydrological and hydraulic models have been developed of the drainage features at the project site. Models have been developed for existing conditions, Year 5 and Year 30 of the operations phase, and post mine closure. The objectives of these models is as follows:

- The year 5 scenario assesses the flood immunity of the proposed mine infrastructure during this interim development phase.
- The year 30 scenario assesses the flood immunity of the final mine infrastructure. It has also been used to assess the impact of the project on downstream flood levels and velocities.
- The post mine closure scenario assesses the flood immunity of the final void.

XP-RAFTS rainfall runoff routing models (XP Software, 2009) of the study area catchments were developed to estimate design flood discharges. The XP-RAFTS model results were validated against design discharges estimated using the Rational Method in accordance with the procedures given in Australian Rainfall and Runoff (AR&R) (Pilgrim, 1998). No stream flow data is available in the area of interest to calibrate the model.

The TUFLOW model (BMT WBM, 2010) was used to estimate the flooding behaviour along the drainage features within the study area (including the proposed highwall drains). TUFLOW estimates flood levels on a fixed grid pattern by solving the full two-dimensional depth averaged momentum and continuity equations for free surface flow. The model automatically calculates breakout points and flow directions within the study area.

4.2 HIGHWALL DRAIN HYDRAULIC ASSESSMENT

The concept designs of the highwall drains have been assessed against the hydraulic criteria given in the Queensland Government Watercourse diversion guidelines (DNRM, 2014b). These criteria are based on the research undertaken by the Australian Coal Association Research Program (ACARP) (Fisher Stewart, 2002). ACARP recommends that the hydraulic characteristics of the waterway to be diverted, or an adjacent waterway, be used as a 'template' to assist with the design of a proposed diversion. It is likely that the proposed highwall drains will perform in a similar manner during runoff events and be stable in the long term if they have similar hydraulic characteristics to the existing channels.

The ACARP assessment criteria is based around the hydraulic characteristics of the waterways for the 1 in 2 Annual Exceedance Probability (AEP) event (1 in 2 year) and 1 in 50 AEP (1 in 50) design events.

- An assessment of the 1 in 2 AEP design flood was used to represent the behaviour of the main channel of the existing drainage features and highwall drains at bank full flow conditions. In geomorphologic studies, the bank full flow is often considered to be the stream forming flow because it often exerts the greatest influence on channel geometry.
- An assessment of the 1 in 50 AEP design flood was used to represent the behaviour of the existing drainage features and the proposed highwall drains during large floods. Flood extents for the two scenarios were estimated to determine whether a channel break out (avulsion) could alter the course of the existing drainage feature or highwall drain during large flood events.

Guideline values of velocity, stream power and shear stress for the 1 in 2 AEP and 1 in 50 AEP design events, based on Bowen Basin streams, where the adjacent waterways cannot be used as a template, are also provided. Given the locations of the proposed highwall drains, it is not possible to imitate all of the characteristics of the existing drainage features. However, the results of the TUFLOW model, described above, were used to assess the adequacy of the highwall drain designs. The main hydraulic characteristics of interest for the assessment include flood depth, velocity, bed shear and stream power. A brief description of each of these hydraulic characteristics is provided below:

- Flood depths have been used to show the extent of inundation for both existing, operational phase and post mine closure conditions.
- Stream velocity provides a measure of the speed of water draining across the floodplain. There is not a direct relationship between velocity and the force exerted on soil particles at the boundary and thus stream power and shear stress are used as more reliable indicators of erosion potential. However it provides a recognisable characteristic that can be used to identify a potential change in stream behaviour.
- Shear stress provides a measure of the tractive force acting on sediment particles at the boundary of a stream, and is used to determine the threshold of motion for bed material. It is determined from the hydraulic depth and gradient and provides an indication of the potential for erosion of cohesive sediments or movement of non-cohesive sediments at the channel boundary.
- Stream power is a function of discharge, hydraulic gradient and flow depth. It represents the energy that is available to do work in and on the channel. High stream powers are indicative of elevated erosion potential.

Note that the long term stability of the highwall drains is dependent upon the nature of the subsoils, the successful revegetation of the bed and banks of the drains and the geotechnical stability of the bank batters. Further work on the design of the drains including a subsoil characterisation, revegetation plan and geotechnical assessment will be undertaken during detailed design.

5 Design discharge estimation

5.1 OVERVIEW

This section presents details of the hydrologic modelling of the study area undertaken to estimate design discharges. Two models were developed, representing the northern and southern areas of the study area. Delineation of drainage paths and catchment boundaries was undertaken using the CatchmentSIM software package (CatchmentSIM, 2005).

The XP-RAFTS model was used to estimate design discharges for the 1 in 2, 1 in 50 and 1 in 1000 AEP events. Design discharges were also estimated for the PMF event.

5.2 HYDROLOGIC MODEL DEVELOPMENT

5.2.1 Existing conditions

Figure 5.1 shows the configuration of the XP-RAFTS model. Key points with regard to the adopted XP-RAFTS model parameters are:

- Routing link lengths and catchment slopes were determined from supplied LiDAR data using CatchmentSIM;
- Subcatchment boundaries were adjusted to reflect the proposed highwall drains and mine infrastructure area in order to simplify modelling.
- Subcatchment PERN ‘n’ and fraction impervious in the XP-RAFTS model were adjusted to match peak discharges estimated by the model with Rational Method estimates:
  - A global value of 0.035 was adopted for PERN ‘n’;
  - A global value of 7% was adopted for fraction impervious;
- A channel routing ‘X’ factor of 0.2 was adopted for all routing links;
- Channel routing ‘K’ factors were determined based on assumed stream velocities determined using average catchment slope;
- A global ‘Bx’ factor of 1.0 was adopted in the XP-RAFTS model.
- Rational method checks were undertaken for a range of catchment scales, based on the following methodology:
  - Runoff coefficient (C) determined in accordance with *Queensland MRD Bridge-Branch Method* (Pilgrim, 1998);
  - Overland flow time estimated using Friend’s equation (QUDM, 2013);
  - Channel flow time estimated based on assumed stream velocities;
- Rational Method peak discharge estimates were typically within +/- 20% of XP-RAFTS peak discharge estimates. The XP-RAFTS mooel parameters have therefore been adopted for this study.

5.2.2 Year 5

The Year 5 XP-RAFTS model was based on the existing conditions model, with modifications made to subcatchment boundaries and routing links to represent the proposed highwall drains and open cut mine drainage strategy. No changes were made to XP-RAFTS rainfall and runoff routing model parameters within the mine disturbance area, as it was assumed that site drainage management measures would result in no change to runoff generation from these catchments. Subcatchment areas were adjusted to reflect the removal of catchment due to capture of runoff in the open cut mine and mine infrastructure areas.

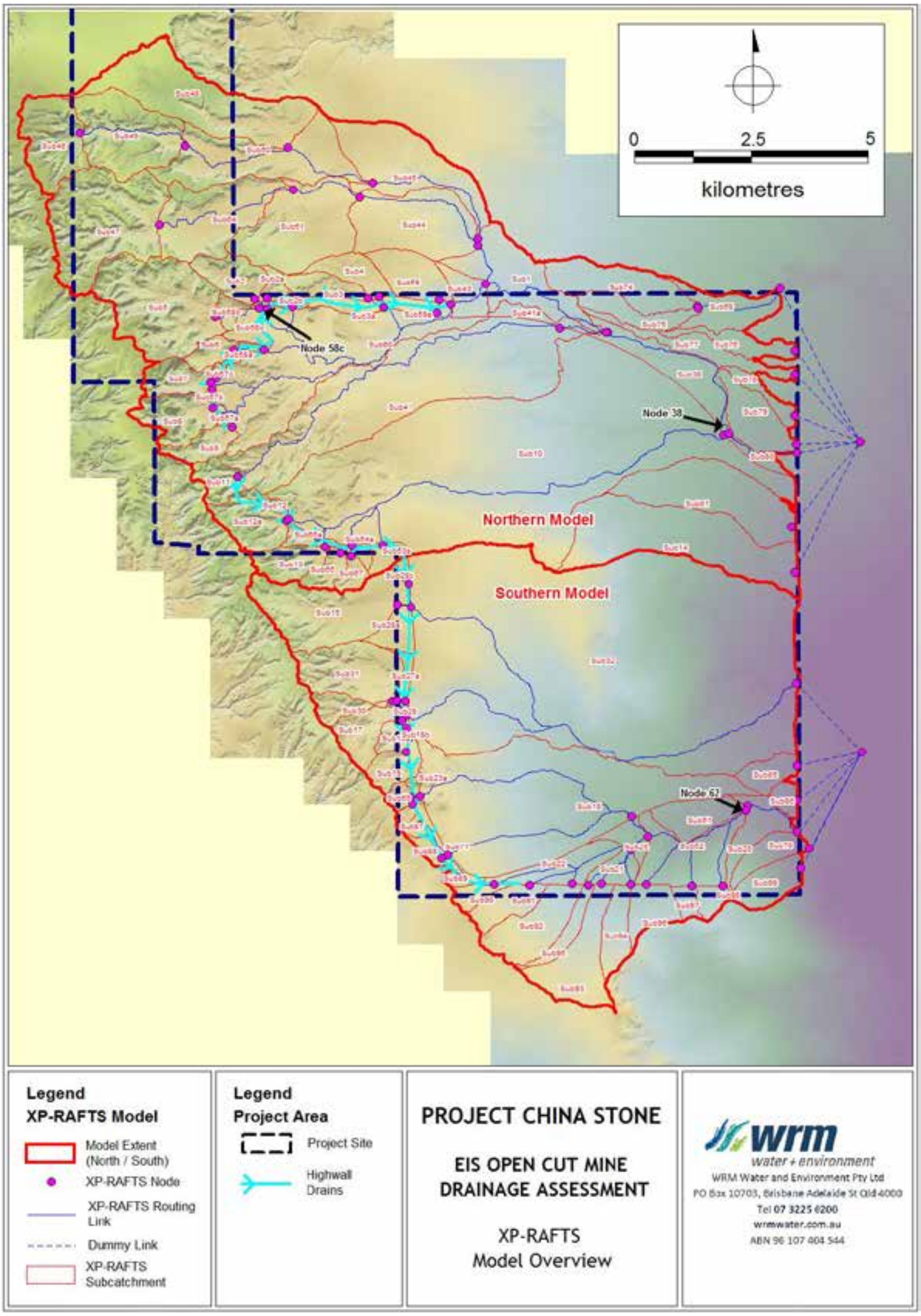

Figure 5.1 Existing conditions XP-RAFTS model layout

5.2.3 Year 30

The Year 30 XP-RAFTS model was based on the Year 5 model, with modifications made to subcatchment boundaries and routing links to reflect the extension of the southern highwall drain, and southern open cut pits and overburden emplacements. The Year 30 XP-RAFTS model was also adopted for final landform modelling, as there are minimal changes to catchments from Year 30 of mine life.

5.3 DESIGN RAINFALL DATA

5.3.1 1 in 2 and 1 in 50 AEP events

Design rainfall data for the 1 in 2 and 1 in 50 AEP events was obtained from the BOM AR&R87 IFDs tool (BOM, 2014). Table 5.1 shows the design rainfall intensities adopted. Temporal patterns for the 1 in 2 and 1 in 50 AEP events were adopted from the *Australian Rainfall and Runoff: A Guide to Flood Estimation* (Pilgrim, 1998) for Zone 3.

Table 5.1 Adopted design rainfall intensities for 1 in 2 and 1 in 50 AEP events

Duration (min)	Intensity (mm/h)	
	1 in 2 AEP	1 in 50 AEP
30	58.2	118
60	39.5	79.4
120	24.5	49.2
180	18.0	36.2
360	10.4	21.1
720	6.25	12.7

5.3.2 1 in 1000 AEP event

Design rainfall data for the 1 in 1000AEP event was obtained using CRC-Forge (DNRM, 2005). Table 5.2 shows the design rainfall intensities adopted for the 1 in 1000AEP event. The temporal pattern for the 1 in 1000AEP design events for durations up to and including 6 hours was adopted from *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method* (BOM, 2003).

Table 5.2 Adopted design rainfall intensities for 1 in 1000AEP event

Duration (min)	Intensity (mm/h)
15	275
30	200
60	139
180	61.1
360	36.0

5.3.3 Probable Maximum Precipitation

Probable Maximum Precipitation (PMP) design rainfall depth estimates for durations up to 6 hours were determined using the Generalised Short Duration Method (GSDM) (BOM, 2003). PMP rainfalls are used to derive the PMF. The notional AEP of the PMP design event is 1 x 10^{-5}% (BOM, 2003).

Estimates of PMP design rainfall depths were obtained for two locations, representative of key design rainfall events at the following locations:

- The northern drainage corridor at the northern boundary of the study area. Rainfall depths were calculated for the combined catchment area discharging through node 44 (refer Figure 5.1) and entering the study area; and
- Small catchments discharging into the highwall drains. Rainfall depths were calculated for the small catchments (approximately 1km^2) discharging directly into the proposed highwall drains. Subcatchment 67 (refer Figure 5.1) was selected as a representative subcatchment.

The design rainfall depths and intensities adopted for the PMP event are listed in Table 5.3

Table 5.3 Adopted design rainfall depths and intensities for PMP events

Duration (min)	Northern Drainage Corridor		Highwall drain catchments	
	Depth (mm)	Intensity (mm/h)	Depth (mm)	Intensity (mm/h)
15	170	680	210	840
30	250	500	310	620
60	370	370	450	450
120	560	280	670	335
180	670	223	820	273
360	900	150	1090	182

5.4 RAINFALL LOSSES

The design rainfall losses adopted are shown in Table 5.4.

Table 5.4 Adopted design rainfall losses

Design Events	Initial Loss (mm)	Continuing Loss Rate (mm/hr)
1 in 2 AEP	15	2.5
1 in 50 AEP	15	2.5
1 in 1000 AEP	0	1.5
PMP	0	1.5

5.5 DESIGN DISCHARGES & CRITICAL DURATIONS

5.5.1 Existing conditions

The existing conditions RAFTS models were used to identify the design storms that produce the highest peak discharges in the northern drainage corridor at the project site, and also in the upper reaches of the drainage features that discharge into the highwall drains. These storm durations are referred to as the critical durations and will typically vary with catchment area. Critical durations and peak discharges were assessed at the following locations:

- Subcatchments 6, 9, 11, & 19 (refer Figure 5.1). These drainage features will drain directly into the highwall drains.
- Node 38 (refer Figure 5.1). This node is located in the northern drainage corridor adjacent to the proposed TSF (and downstream of the northern highwall drain outlet).

- Node 62 (refer Figure 5.1). This node is located in drainage feature 6 upstream of the eastern project site boundary (and downstream of the southern highwall drain outlet).

Table 5.5 shows critical durations and peak discharges for each of these locations for the 1 in 2 and 1 in 50 AEP events. The 60 minute duration storm was adopted as the critical duration for drainage features upstream of the highwall drains, and the 180-minute and 120-minute duration storms for the northern and southern drainage corridors, respectively.

Table 5.5 Existing conditions peak discharge and critical durations at key locations

Node	1 in 2 AEP		1 in 50 AEP	
	Peak Discharge (m^3/s)	Critical Duration (min)	Peak Discharge (m^3/s)	Critical Duration (min)
6	4.00	60	13.2	60
9	4.91	60	14.1	60
11	6.20	60	19.9	60
19	4.18	60	14.1	60
38	161	180	456	180
62	71.3	120	221	120

5.5.2 Year 5

Year 5 design discharges for the 1 in 50 AEP event were identified at the following locations:

- Nodes 58c (located halfway along the northern highwall drain), 3a (outlet of northern highwall drain) and 28a (Year 5 outlet of southern highwall drain).
- Node 38 (refer Figure 5.1). This node is located in the northern drainage corridor adjacent to the proposed TSF (and downstream of the northern highwall drain outlet).
- Node 62 (refer Figure 5.1). This node is located in drainage feature 5 upstream of the eastern boundary of the project site (and downstream of the southern highwall drain outlet).

Table 5.6 shows critical durations and peak discharges for each of these locations for the 1 in 50 AEP event. The 60 minute duration storm was found to produce the highest peak discharges for highwall drain flooding in Year 5, and the 180-minute storm for flooding in the northern and southern drainage corridors. The critical duration in the southern drainage corridor increases due to the increase in catchment area draining to this point following the construction of the southern highwall drain.

Table 5.6 Year 5 peak discharge and critical durations at key locations

Node	1 in 50 AEP	
	Peak Discharge (m^3/s)	Critical Duration (min)
58c	164	60
3a	194	60
28a	145	60
38	347	180
62	355	180

5.5.3 Year 30

Year 30 design discharges for the 1 in 2, 1 in 50 and 1 in 1000 AEP events were identified at the following locations:

- Nodes 58c (halfway along the northern highwall drain), 3a (outlet of northern highwall drain), 28a (halfway along southern highwall drain) and node 71 (outlet of southern highwall drain).
- Node 38 (refer Figure 5.1). This node is located in the northern drainage corridor adjacent to the proposed TSF (and downstream of the northern highwall drain outlet).
- Node 62 (refer Figure 5.1). This node is located in the southern drainage corridor upstream of the eastern boundary of the project site (and downstream of the southern highwall drain outlet).

Table 5.7 shows critical durations and peak discharges for each of these locations for the 1 in 2, 1 in 50 and 1 in 1000 AEP events. The-60 minute duration storm was found to produce the highest peak discharge for highwall drain flooding in Year 30, the 180-minute storm for flooding in the northern drainage corridor, and the 120-minute storm for flooding in the southern drainage corridor.

Table 5.7 Year 30 peak discharge and critical durations at key locations

Node	1 in 2 AEP		1 in 50 AEP		1 in 1000 AEP	
	Peak Discharge (m^3/s)	Critical Duration (min)	Peak Discharge (m^3/s)	Critical Duration (min)	Peak Discharge (m^3/s)	Critical Duration (min)
58c	48.4	60	164	60	371	60
3a	57.4	60	194	60	441	60
28a	41.8	60	145	60	332	60
71	69.0	60	235	60	536	60
38	124	180	358	180	779	180
62	125	120	372	120	968	120

5.5.4 Final landform

Post-mining design discharges for the PMF event were identified at locations outlined in Section 6.5.3. Table 5.8 shows critical durations and peak discharges for each of these locations for the PMF event. The-60 minute duration storm was found to produce the highest peak discharge for highwall drain PMF flooding, the 240-minute storm for PMF flooding in the northern drainage corridor, and the 120-minute storm for PMF flooding in the southern drainage corridor.

It should be noted that the peak discharge and critical duration at node 38 have been estimated using the northern drainage corridor catchment PMF rainfalls outlined in Table 5.3. Discharges for all other locations are based on the highwall drain catchment PMF rainfall estimates, which are considered to be conservatively high for locations at the downstream end of the highwall drains.

Table 5.8 Post-mining final landform peak discharge and critical durations at key locations

Node	PMF	
	Peak Discharge (m^3/s)	Critical Duration (min)
58c	1,408	60
3a	1,658	60
28a	1,260	60
71	2,022	60
38	3,298	240
62	3,468	120

6 Existing flood conditions

6.1 OVERVIEW

This section presents details of the existing conditions TUFLOW two-dimensional hydrodynamic models (BMT WBM, 2010) developed for the assessment. Two TUFLOW models were developed, a northern model of drainage features 1 to 4, and a southern model of drainage features 5 and 6). The results of the existing conditions modelling is also presented.

6.2 HYDRAULIC MODEL DEVELOPMENT

6.2.1 Topographic Data

Hansen Bailey provided Light Detection and Ranging (LiDAR) survey data covering the project site, and extending some 3km east of the project site boundary.

6.2.2 Cell size, topography & model extent

Both northern and southern models use a cell size of 5m (5m x 5m cell). The adopted cell size is a compromise between computational run times, and providing an adequate representation of flow characteristics in the drainage features at the project site.

Figure 6.1 shows the extent of the northern and southern TUFLOW models.

6.2.3 Structures

There are no existing structures within the extent of the TUFLOW models.

6.2.4 Boundaries

Figure 6.1 shows the location of all inflow and outflow boundaries in the hydraulic model.

Local and total runoff hydrographs generated by the XP-RAFTS model described in Section 6 are used as inflow boundaries within the hydraulic model. The northern model includes 47 local hydrograph inflows and two total inflow hydrographs. The southern model includes 38 local inflow hydrographs.

The downstream (outflow) model boundaries were configured as 'normal depth' boundaries, with water levels determined based on upstream ground and flood slope. The downstream model boundary locations were determined by the extent of available survey.

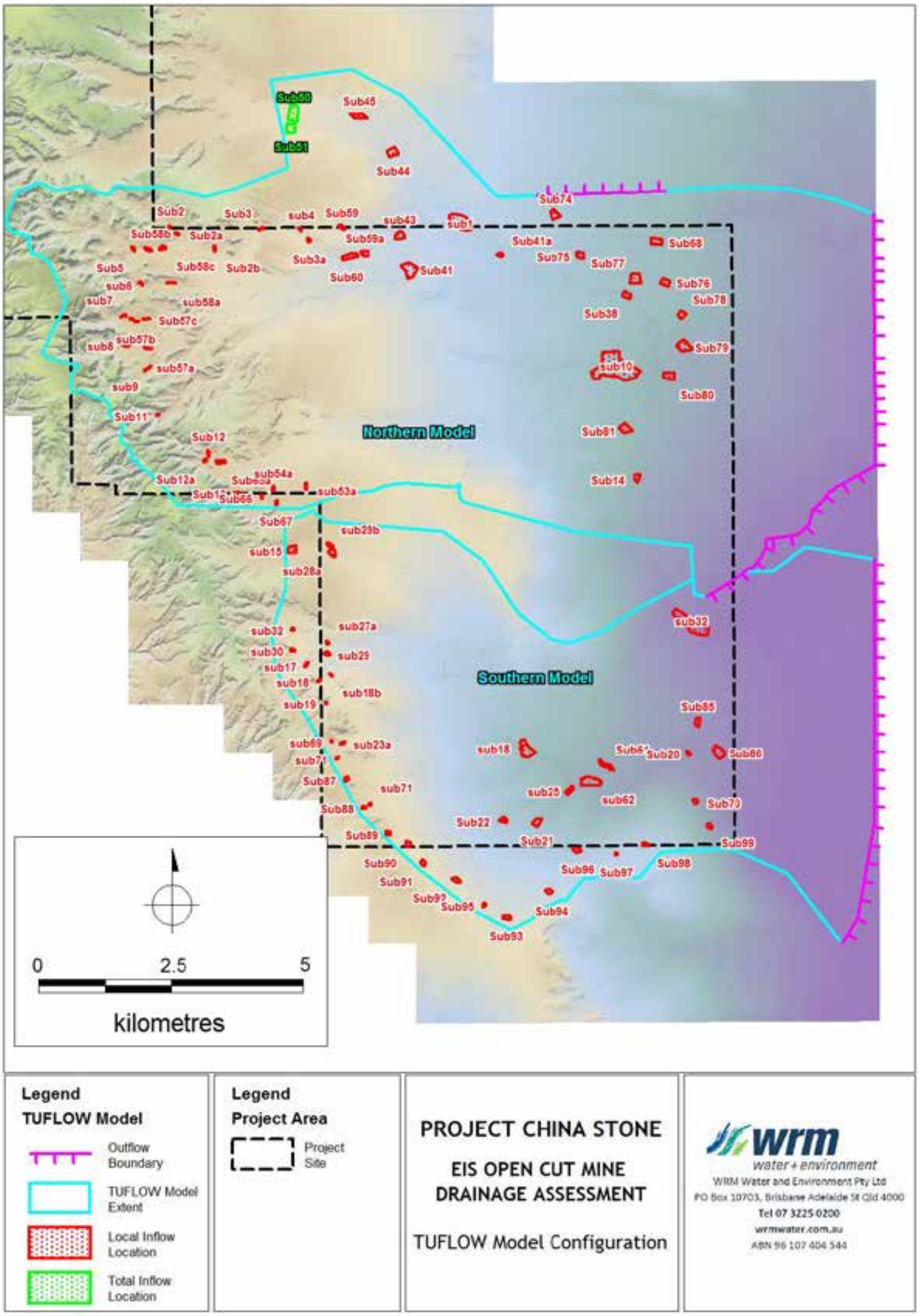

Figure 6.1 Existing conditions TUFLOW model configuration

6.2.5 Hydraulic roughness / Mannings ‘n’

Table 6.1 lists the adopted Manning’s ‘n’ (hydraulic roughness) values within the hydraulic model.

Table 6.1 Adopted TUFLOW model Manning’s ‘n’

Landuse	Manning’s ‘n’
Dirt road	0.045
Drainage feature (sandy / eroded bed)	0.05
Drainage feature (grassed / vegetated)	Water depth 0m-0.3m: 0.1 Water depth > 0.3m: 0.06
Scrub / light vegetation	Water depth 0m-0.5m: 0.09 Water depth > 0.5m: 0.07
Dense scrub / heavy vegetation	0.09
Marsh / swamp	0.08
Waterbody / dam area	0.03
Highwall drain	0.035

6.3 METHODOLOGY

The existing conditions TUFLOW models (north and south) were used to estimate existing conditions flood levels, depths, flow velocities, bed shear stress and stream power for the drainage features at the project site.

Existing conditions simulations were undertaken for the 1 in 2 and 1 in 50 AEP design events, for the following durations (as estimated by the existing conditions XP-RAFTS model):

- 60 minute duration (northern and southern model);
- 120 minute duration (southern model only); and
- 180 minute duration (northern model only).

Maximum result grids were then generated from all durations for water surface elevation, depth, velocity, bed shear stress and stream power.

Longitudinal profiles of flow velocity, bed shear stress and stream power were produced for the upper reaches of four of the drainage features at the project site. The location of the reaches used to produce the longitudinal profiles is shown in Figure 6.2. These reaches were selected in order to provide some guidance on the design of the highwall drains. Profile 1 and profile 2 are located within the catchment of the northern highwall drain, and profile 3 and profile 4 are located within the catchment of the southern highwall drain. The longitudinal profiles presented in Section 8.4 represent average values across the flowpath at a given chainage for each event, and do not represent the variation in expected flow velocities, bed shear stresses and stream power across the flowpath cross section.

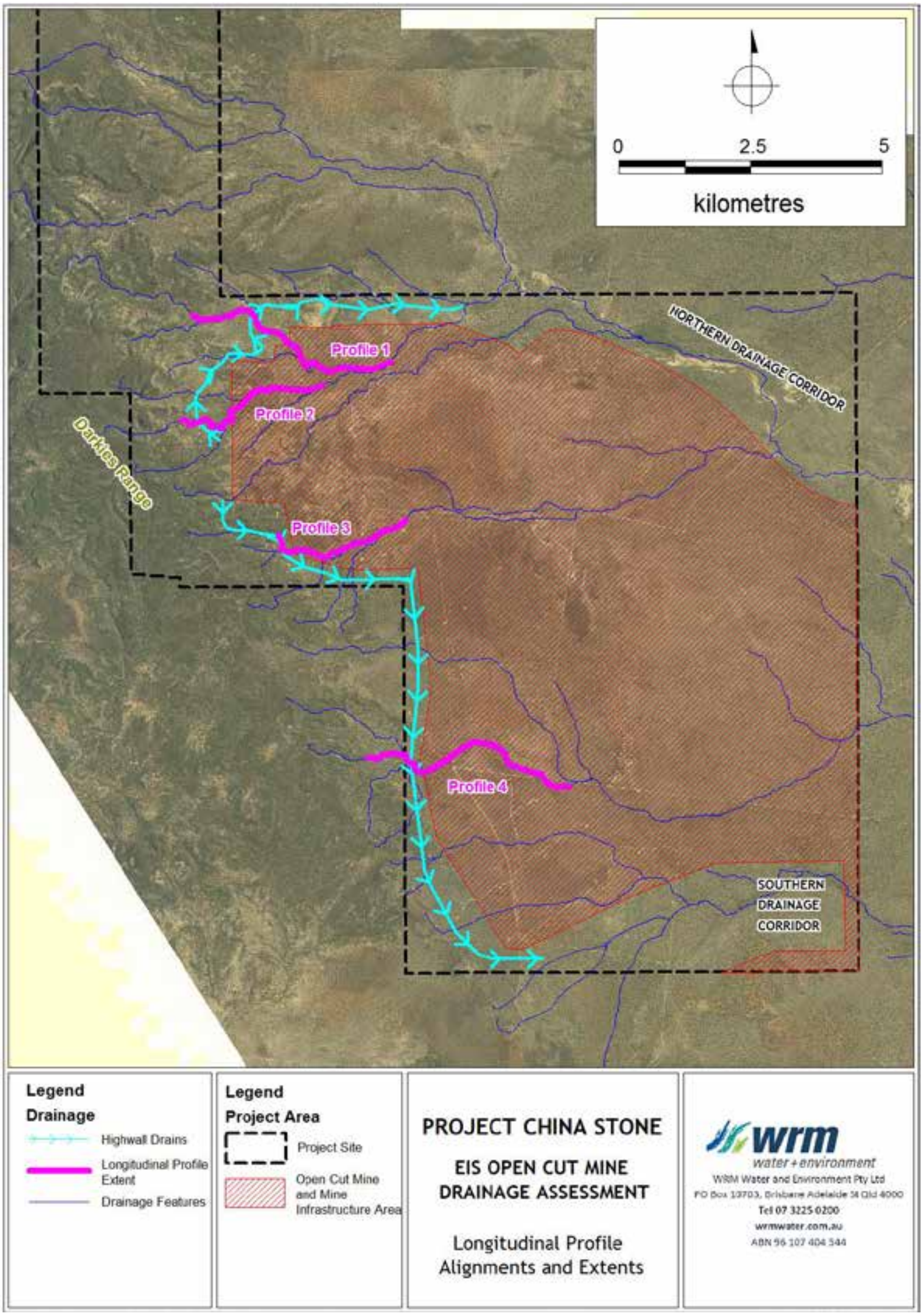

Figure 6.2 Alignment and extent of longitudinal profiles of existing conditions flooding

6.4 FLOOD LEVELS, DEPTHS AND EXTENTS

Figure A1 and A2, Appendix A show the predicted extent and depth of flooding at the project site for the 1 in 2 and 1 in 50 AEP events under existing conditions. Peak water surface contours for each event are also shown. The following is of note:

- Flooding at the project site during 1 in 2 and 1 in 50 AEP events is typically via wide shallow sheet flow in areas where there are no defined channels, with more than 80% of the flood extent inundated to a depth of 0.5m or less.
- Flood depths exceed 0.5m in some parts of the lower and middle reaches of drainage feature 4 and 5, although flooding is still typically broad and shallow in these areas.
- Flooding is more confined in the upper reaches of the drainage features traversing the project site, with flood depths ranging from 1m to 2m.
- Flood depths of between 2m and 3m occur in isolated regions where the channel of drainage feature 1 is well defined.
- Existing conditions flood levels along drainage feature 1 within the project site range from about 292mAHD to 322mAHD for the 1 in 2 AEP event. Due to the wide floodplain, 1 in 50 AEP flood levels along drainage feature 1 are typically less than 1m higher than 1 in 2 AEP levels.

6.5 FLOW VELOCITIES

Figure A3 and A4, Appendix A show the peak flow velocity grids for existing conditions for the 1 in 2 and 1 in 50 AEP events. The following is of note:

- During the 1 in 2 AEP event, peak flow velocities are less than 0.5m/s for more than 90% of the inundated area. Less than 0.5% of the inundated area experience velocities greater than 1m/s.
- Velocities are increased during a 1 in 50 AEP event with approximately 20% of the inundated area experiencing velocities of greater than 0.5m/s.
- Higher velocities tend to be limited to the upper reaches of the drainage features adjacent to Darkies Range and within drainage feature 1. Flow velocities within the wide shallow flowpaths across the majority of the project site are generally less than 0.5m/s.
- The steep upper reaches of drainage features are predicted to experience peak channel velocities of greater than 1.5m/s during a 1 in 2 AEP event, and greater than 2m/s during a 1 in 50 AEP event.
- Areas of high flow velocity typically correspond to deeper areas of inundation.

6.6 LONGITUDINAL PROFILES

6.6.1 Flow velocity

Figure 6.3, Figure 6.4, Figure 6.5 and Figure 6.6 show the predicted flow velocity along the profiles shown in Figure 6.2. Table 6.2 summarises the results of the longitudinal plots. The following is of note:

- Flow velocity along profile 1 and profile 2 (drainage feature 2) typically decreases from upstream to downstream as the drainage feature transitions from a narrow confined channel to a wider shallower flowpath.
- Within profile 1, velocity during a 1 in 2 AEP event ranges from 0.2m/s to 1.2m/s, with the maximum velocity occurring at chainage 800m. During a 1 in 50 AEP event, velocities range from about 0.5m/s to 1.3m/s.

- Within profile 2, velocity during a 1 in 2 AEP event ranges from 0.3m/s to 1.8m/s, with the maximum velocity occurring at chainage 400m. During a 1 in 50 AEP event, velocities range from 0.6m/s to 2.1m/s.
- Flow velocities along profile 3 (drainage feature 4) and profile 4 (drainage feature 5) are more constant over the channel reaches, likely due to the flatter channel grade and wider flowpath along these channels compared to profiles 1 and 2.
- Within profile 3, velocity during a 1 in 2 AEP event ranges from 0.3m/s to 0.7m/s, with the maximum velocity occurring at chainage 20m. During a 1 in 50 AEP event, velocities range from 0.4m/s to 1.3m/s.
- Within profile 4, velocity during a 1 in 2 AEP event ranges from 0.2m/s to 1.2m/s, with the maximum velocity occurring at chainage 4280m, which corresponds to a location where the wide shallow flowpath transitions rapidly to a very confined deeper channel. During a 1 in 50 AEP event, velocities range from 0.3m/s to 1m/s.

Table 6.2 Summary of existing conditions flow velocity

Profile	1 in 2 AEP Channel Velocity (m/s)			1 in 50 AEP Channel Velocity (m/s)		
	Min.	Mean	Max.	Min.	Mean	Max.
1	0.2	0.5	1.2	0.5	0.8	1.3
2	0.3	0.8	1.8	0.6	1.1	2.1
3	0.3	0.4	0.7	0.4	0.8	1.3
4	0.2	0.4	1.2	0.3	0.6	1.0

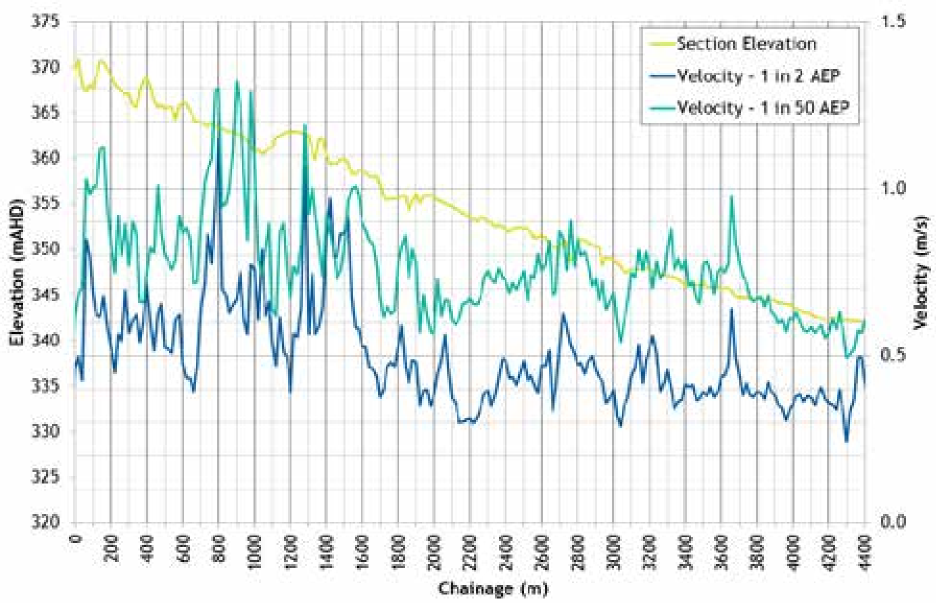

Figure 6.3 Existing conditions flow velocity, longitudinal profile 1

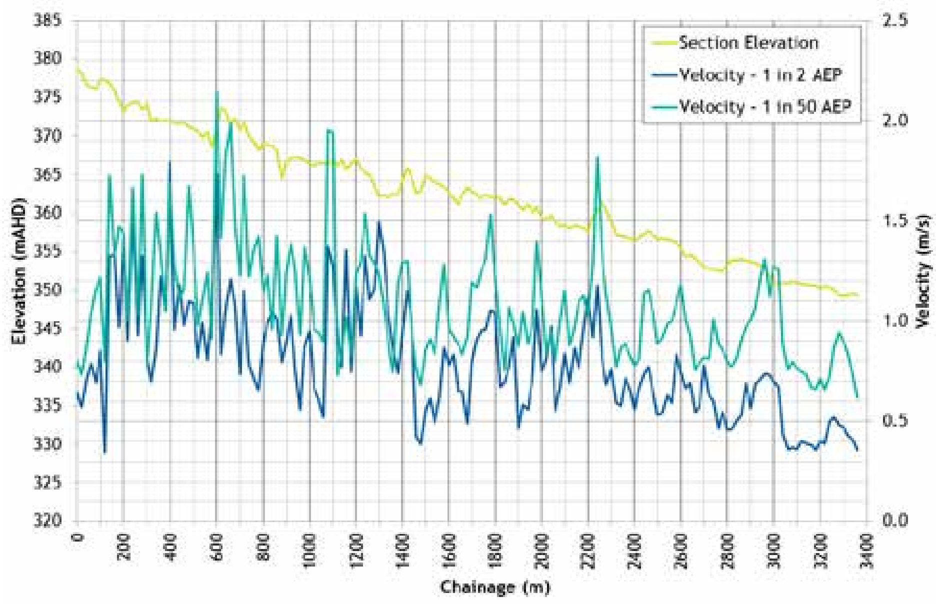

Figure 6.4 Existing conditions flow velocity, longitudinal profile 2

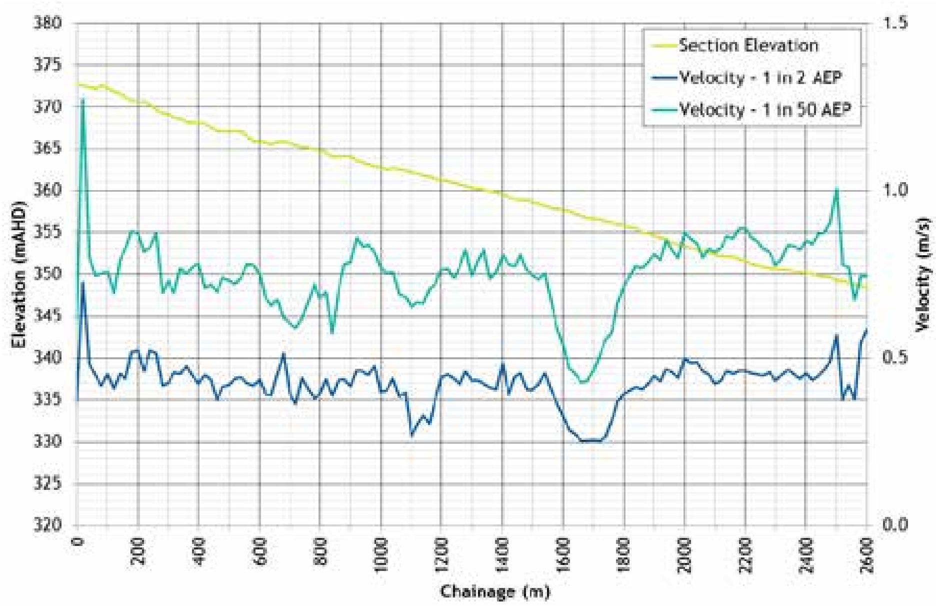

Figure 6.5 Existing conditions flow velocity, longitudinal profile 3

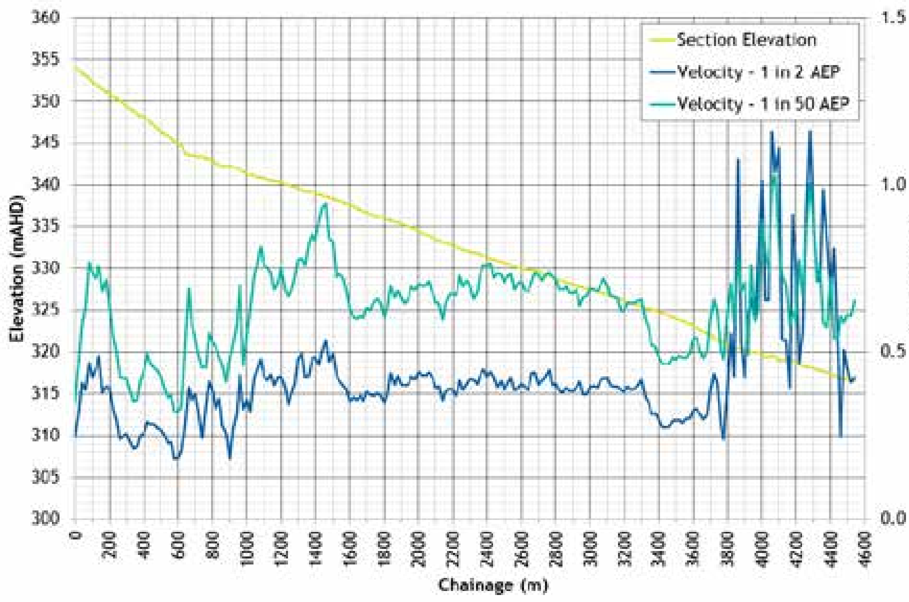

Figure 6.6 Existing conditions flow velocity, longitudinal profile 4

6.6.2 Bed shear stress

Figure 6.7, Figure 6.8, Figure 6.9 and Figure 6.10 show the predicted bed shear stress along the profiles shown in Figure 6.2 during 1 in 2 and 1 in 50 AEP events. Table 6.3 summarises the results of the longitudinal plots. The following is of note:

- Bed shear stress along profile 1 and profile 2 typically decreases from upstream to downstream as the drainage feature transitions from a narrow confined channel to a wider shallower flowpath.
- Within profile 1, bed shear stress during a 1 in 2 AEP event ranges from $4N/m^2$ to $53N/m^2$, with the maximum bed shear stress occurring at chainage 740m. During a 1 in 50 AEP event, bed shear stresses range from $14N/m^2$ to $97N/m^2$.
- Within profile 2, bed shear stress during a 1 in 2 AEP event ranges from $6N/m^2$ to $100N/m^2$, with the maximum bed shear stress occurring at chainage 600m. During a 1 in 50 AEP event, bed shear stresses range from $18N/m^2$ to $157N/m^2$.
- Bed shear stresses along profile 3 and profile 4 are more constant over these channel reaches, likely due to the flatter channel grade and wider flowpath along these channels compared to profiles 1 and 2.
- Within profile 3, bed shear stress during a 1 in 2 AEP event ranges from $8N/m^2$ to $69N/m^2$, with the maximum bed shear stress occurring at chainage 20m. During a 1 in 50 AEP event, bed shear stresses range from $17N/m^2$ to $156N/m^2$.
- Within profile 4, bed shear stress during a 1 in 2 AEP event ranges from $4N/m^2$ to $45N/m^2$, with the maximum bed shear stress occurring at chainage 3800m, which corresponds to a location where the wide shallow flowpath transitions rapidly to a very confined deeper channel. During a 1 in 50 AEP event, bed shear stresses range from $11N/m^2$ to $63N/m^2$.

Table 6.3 Summary of existing conditions bed shear stress

Profile	1 in 2 AEP Bed Shear Stress (N/m^2)			1 in 50 AEP Bed Shear Stress (N/m^2)		
	Min.	Mean	Max.	Min.	Mean	Max.
1	4	20	53	14	44	97
2	6	27	100	18	53	157
3	8	25	69	17	61	156
4	4	17	45	11	37	63

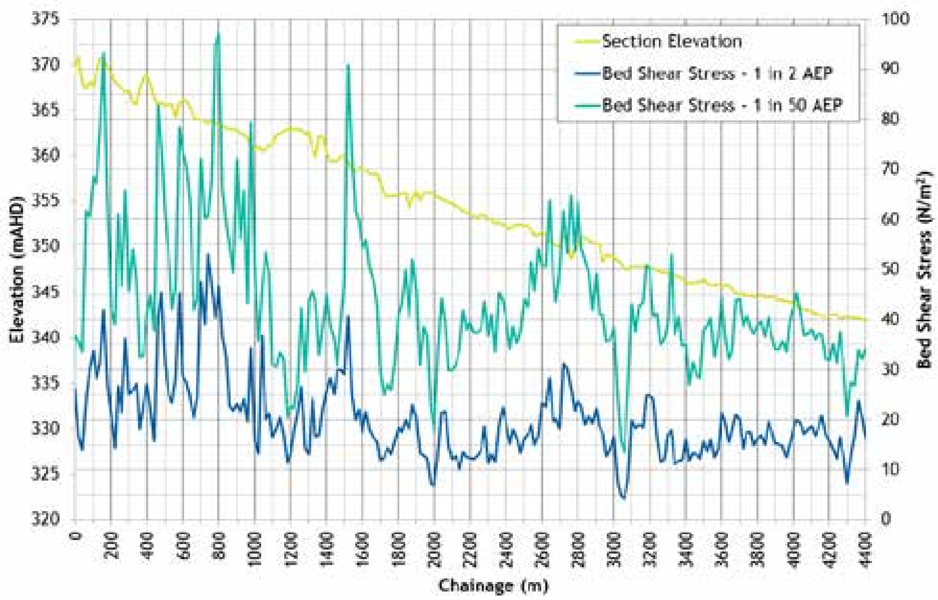

Figure 6.7 Existing conditions channel bed shear stress, longitudinal profile 1

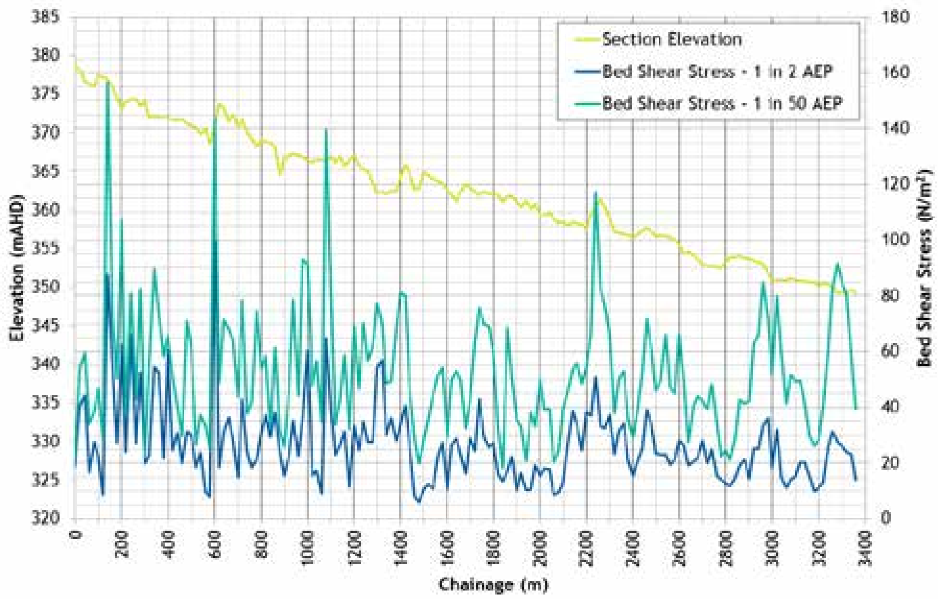

Figure 6.8 Existing conditions channel bed shear stress, longitudinal profile 2

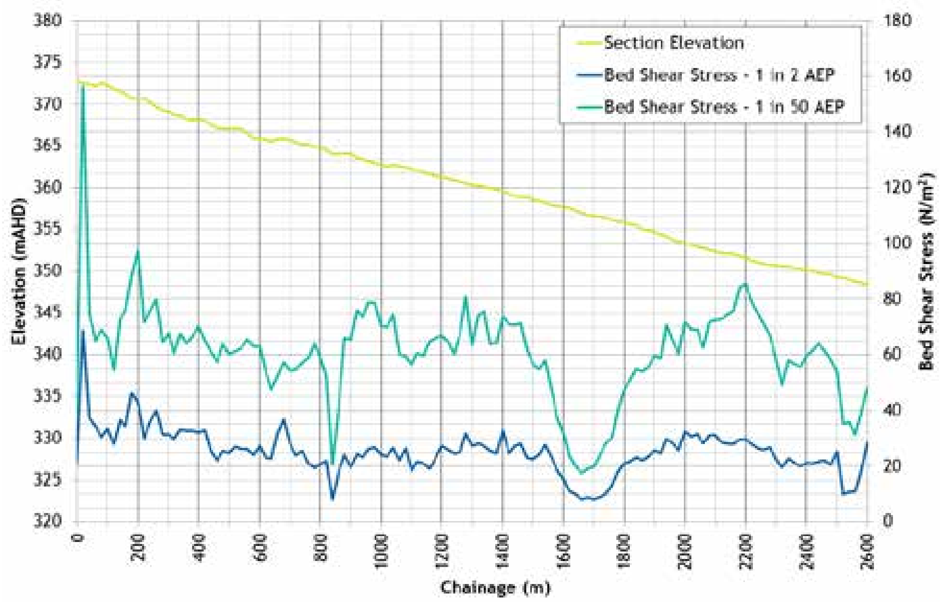

Figure 6.9 Existing conditions channel bed shear stress, longitudinal profile 3

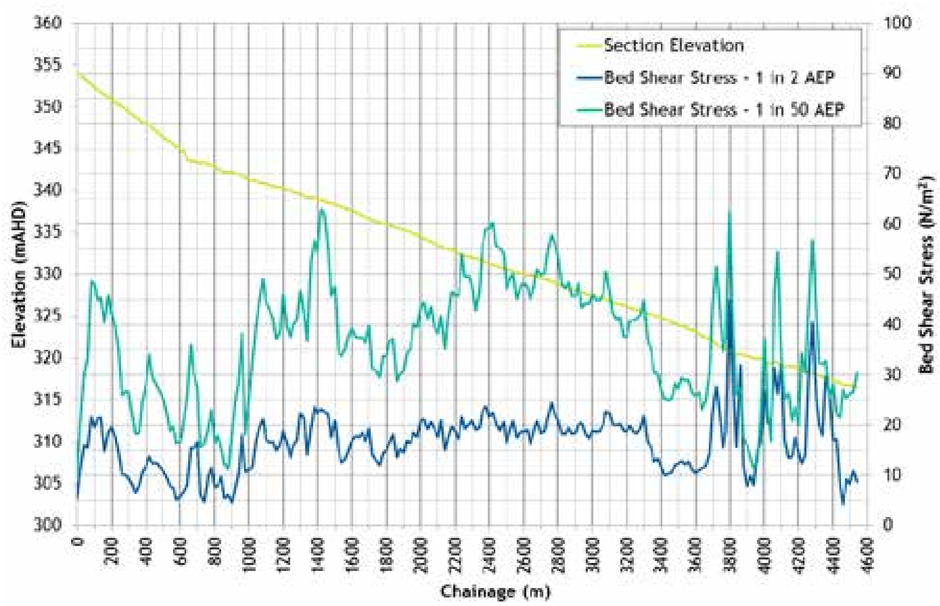

Figure 6.10 Existing conditions channel bed shear stress, longitudinal profile 4

6.6.3 Stream power

Figure 6.7, Figure 6.8, Figure 6.9 and Figure 6.10 show the predicted stream power along the profiles shown in Figure 6.2 during 1 in 2 and 1 in 50 AEP events. Table 6.3 summarises the results of the longitudinal plots. The following is of note:

- Stream power along profile 1 and profile 2 typically decreases from upstream to downstream as the drainage feature transitions from a narrow confined channel to a wider shallower flowpath.

- Within profile 1, stream power during a 1 in 2 AEP event ranges from 1.1W/m^2 to 56.0W/m^2, with the maximum stream power occurring at chainage 800m. During a 1 in 50 AEP event, stream power ranges from 6.3W/m^2 to 167.0W/m^2.
- Within profile 2, stream power during a 1 in 2 AEP event ranges from 3.4W/m^2 to 199.6W/m^2, with the maximum stream power occurring at chainage 600m. During a 1 in 50 AEP event, stream power ranges from 12.4W/m^2 to 386.5W/m^2.
- Stream power along profile 3 and profile 4 is more constant over these channel reaches, likely due to the flatter channel grade and wider flowpath along these channels compared to profiles 1 and 2.
- Within profile 3, stream power during a 1 in 2 AEP event ranges from 2.1W/m^2 to 52.1W/m^2, with the maximum stream power occurring at chainage 20m. During a 1 in 50 AEP event, stream power ranges from 6.6W/m^2 to 182.8W/m^2.
- Within profile 4, stream power during a 1 in 2 AEP event ranges from 1.1W/m^2 to 46.4W/m^2, with the maximum stream power occurring at chainage 2480m, which corresponds to a location where the wide shallow flowpath transitions rapidly to a very confined deeper channel. During a 1 in 50 AEP event, stream power ranges from 5.6W/m^2 to 85.3W/m^2.

Table 6.4 Summary of existing conditions stream power

Profile	1 in 2 AEP Stream power (W/m^2)			1 in 50 AEP Stream power (W/m^2)		
	Min.	Mean	Max.	Min.	Mean	Max.
1	1.1	13.4	56.0	6.3	41.6	167.0
2	3.4	26.4	199.6	12.4	68.8	386.5
3	2.1	14.0	52.1	6.6	52.9	182.8
4	1.1	8.8	46.4	5.6	27.4	85.3

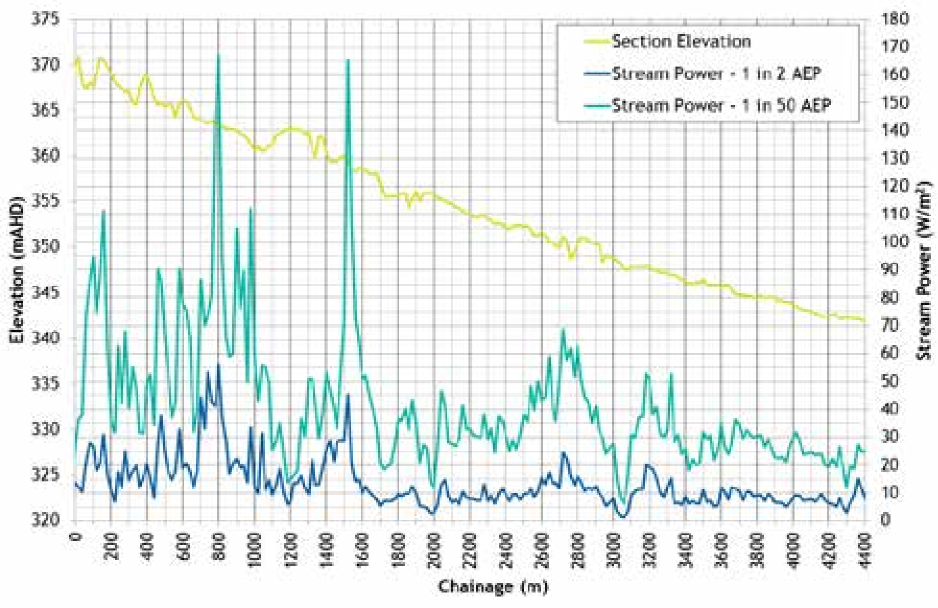

Figure 6.11 Existing conditions channel stream power, longitudinal profile 1

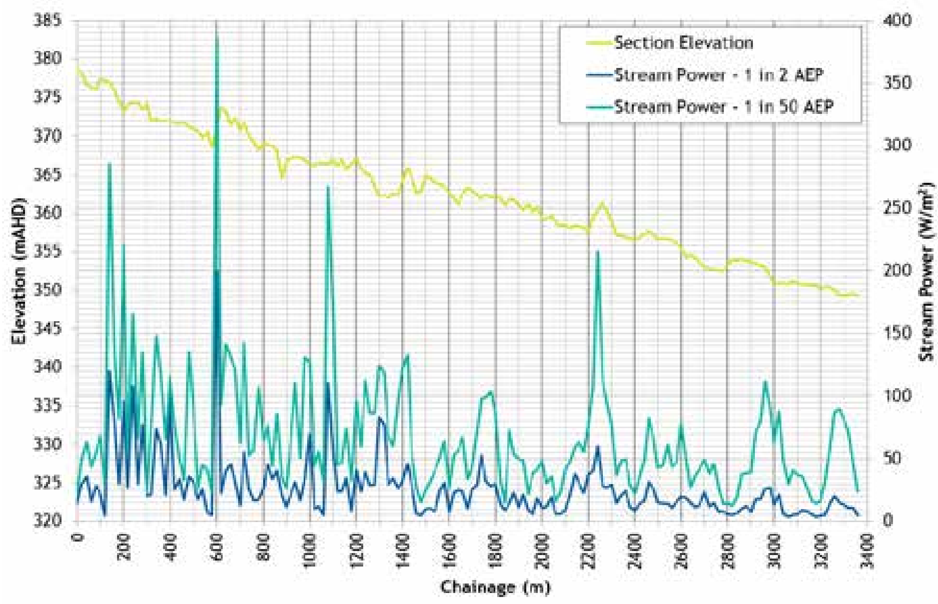

Figure 6.12 Existing conditions channel stream power, longitudinal profile 2

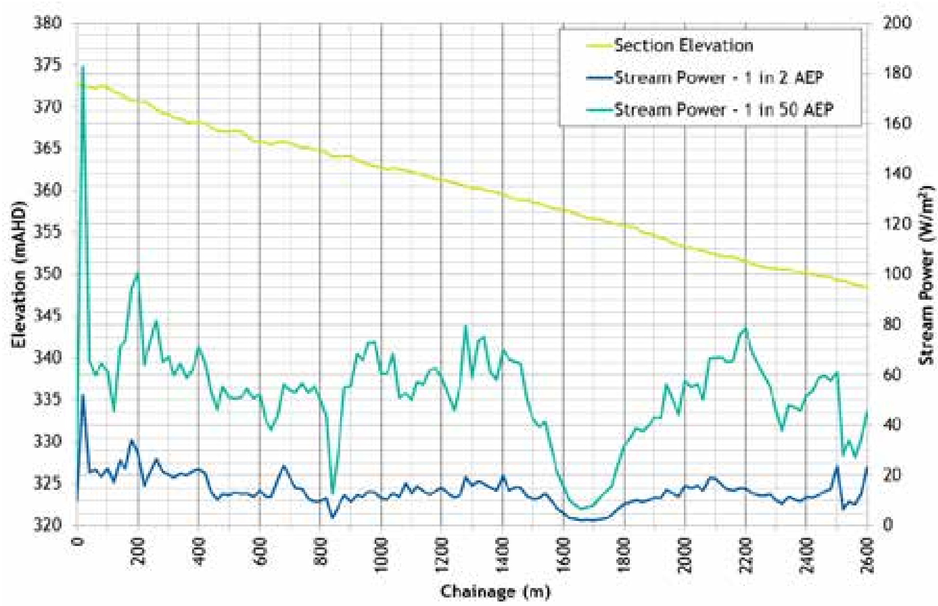

Figure 6.13 Existing conditions channel stream power, longitudinal profile 3

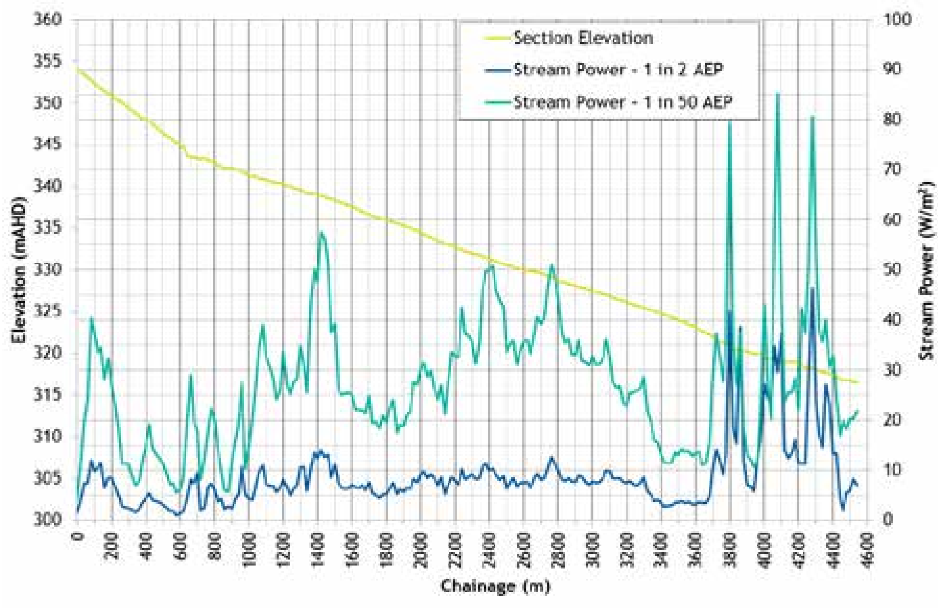

Figure 6.14 Existing conditions channel stream power, longitudinal profile 4

7 Highwall drain concept design

7.1 OVERVIEW

This section presents details of the highwall drain concept designs including the design objectives, design constraints and design criteria.

7.2 DESIGN OBJECTIVES

The drains were designed and located to minimise the catchment area draining into the open cut pits and final void, and to provide the open cut pits and final voids with protection from flooding from the undisturbed upstream catchment for all events up to and including the PMF.

Given the location and confined nature of the drains, it is not possible to replicate all of the geomorphological characteristics of the drainage features which they replace. Instead, the drains have been designed to replicate the hydraulic characteristics of the existing channels such as stream velocity, bed shear and stream power. In addition, the hydraulic characteristics of the drains have been compared to the ACARP hydraulic design criteria described in Section 4.2.

The proposed highwall drains will behave in a similar manner to the existing drainage features and be stable in in the long term if:

- The highwall drains meet the hydraulic criteria outlined in Section 4.2 and have similar hydraulic characteristics to the existing drainage features;
- The highwall drains are sufficiently vegetated; and the highwall drains have batters that are geotechnically stable and protected against rill erosion.

7.3 DESIGN CONSTRAINTS

The alignment of the highwall drains was determined by the proposed extent of the open cut pit and location of the proposed mining lease boundary, with consideration given to the invert levels of the existing drainage features that the highwall drains would intercept. The drains were designed such that a 25m buffer from the proposed mining lease boundary and a 100m buffer from the final highwall of the open cut pit were provided.

It was also imperative for the base of the drains to intercept the base of each interrupted drainage feature to limit the possibility of headward erosion up the drainage feature. At these locations, bunds will be required along the downstream edge of the highwall drain to prevent water overflowing out of the drain and continuing downstream along the drainage feature. These bunds have been designed to not be overtopped during the PMF event. The proposed highwall drain bunds have a trapezoidal section, with 1(V) in 3(H) slopes, and minimum crest width of 3m.

7.4 ADOPTED DESIGN CRITERIA

Table 7.1 and Table 7.2 summarises the existing hydraulic characteristics of the upper reaches of the drainage features at the project site, and compare them to the DNRM (2014b) guideline upper limit values. The following is of note:

- Average flow velocities, bed shear stresses and stream power are generally well below the DNRM (2014b) guideline values.
- Maximum flow velocities in the drainage feature at longitudinal profile 2 exceed the DNRM (2014b) upper limits for both the 1 in 2 AEP and 1 in 50 AEP events.
- Maximum bed shear stress during a 1 in 2 AEP event exceeds the DNRM (2014b) upper limit guideline value in all of the profiled drainage features.

- Maximum bed shear stress during a 1 in 50 AEP event exceeds the DNRM (2014b) upper limit guideline value in profile 1, profile 2 and profile 3.
- Maximum stream power in the drainage feature at longitudinal profile 2 exceeds the DNRM (2014b) upper limits for both the 1 in 2 AEP and 1 in 50 AEP events.
- Maximum flow velocities and stream power in profile 1 and 4 during a 1 in 2 AEP event corresponds to the upper limit values proposed in DNRM (2014b).
- Maximum flow velocities and stream power in profile 1, 2 and 3 during a 1 in 50 AEP event are typically substantially less than the DNRM (2014b) upper limits.

The highwall drains have been designed such that flow velocities, bed shear stresses and stream power in the drains does not exceed the upper limits given in the DNRM (2014b) guidelines, and where possible replicates the hydraulic characteristics of the existing drainage features.

Table 7.1 Comparison of existing conditions drainage feature hydraulic characteristics for 1 in 2 AEP event and DNRM (2014b) guideline upper limits

Profile	1 in 2 AEP Event Averages			1 in 2 AEP Event Maximums		
	Flow Velocity (m/s)	Bed Shear Stress (N/m^2)	Stream power (W/m^2)	Flow Velocity (m/s)	Bed Shear Stress (N/m^2)	Stream power (W/m^2)
1	0.5	20.1	13.4	1.2	53.0	56.0
2	0.8	26.6	26.4	1.8	100.0	199.6
3	0.4	25.2	14.0	0.7	68.8	52.1
4	0.4	16.7	8.8	1.2	44.8	46.4
DNRM (2014b) Vegetated Channel	<1.5	<40	<60	<1.5	<40	<60

Table 7.2 Comparison of existing conditions drainage feature hydraulic characteristics for 1 in 50 AEP event and DNRM (2014b) guideline upper limits

Profile	1 in 50 AEP Event Averages			1 in 50 AEP Event Maximums		
	Flow Velocity (m/s)	Bed Shear Stress (N/m^2)	Stream power (W/m^2)	Flow Velocity (m/s)	Bed Shear Stress (N/m^2)	Stream power (W/m^2)
1	0.8	43.8	41.6	1.3	97.0	167.0
2	1.1	53.2	68.8	2.1	156.5	386.5
3	0.8	60.7	52.9	1.3	156.4	182.8
4	0.6	36.6	27.4	1.0	63.0	85.3
DNRM (2014b) Vegetated Channel	<2.5	<80	<220	<2.5	<80	<220

7.5 PROPOSED HIGHWALL DRAIN DESIGN

7.5.1 Alignment and catchment area

Figure 7.1 and Figure 7.2 show the proposed alignment and extent of the northern and southern highwall drains. Table 7.3 summarises the catchment area of each drain at key chainage locations (ie. where runoff from a drainage feature enters the highwall drain), including the associated XP-RAFTS model node (see Section 4). Note that the southern highwall drain will be developed in two stages, with the upper 5,737m of the drain constructed by Year 5, and the remaining length of drain constructed by Year 10. The entire length of the northern highwall drain will be constructed by Year 5. The highwall drains will only be constructed after any subsidence of the drain alignments due to project longwall mining has been completed. The existing ground levels presented in this section are therefore the final subsided ground levels after the completion of any longwall mining in the vicinity of the drain alignments.

Table 7.3 Highwall drain catchment areas

Northern Highwall Drain			Southern Highwall Drain		
Chainage (m)	Catchment Area (ha)	RAFTS Subcatchment	Chainage (m)	Catchment Area (ha)	RAFTS Subcatchment
0	74	Sub9	0	74	Sub11
516	225	Sub8	1,742	232	Sub12a
1,063	329	Sub7	2,732	362	Sub13
2,004	390	Sub6	3,212	474	Sub66 & 67
3,512	1,094	Sub5 & 2	5,737	964	Sub15 – Year 5 Outlet
6,147	1,232	Sub3	8,117	1,440	Sub31, 17, 18
6,247	1,323	Sub4	8,810	1,527	Sub9
7,435	1,357	Outlet	13,113	1,717	Year 30 Outlet

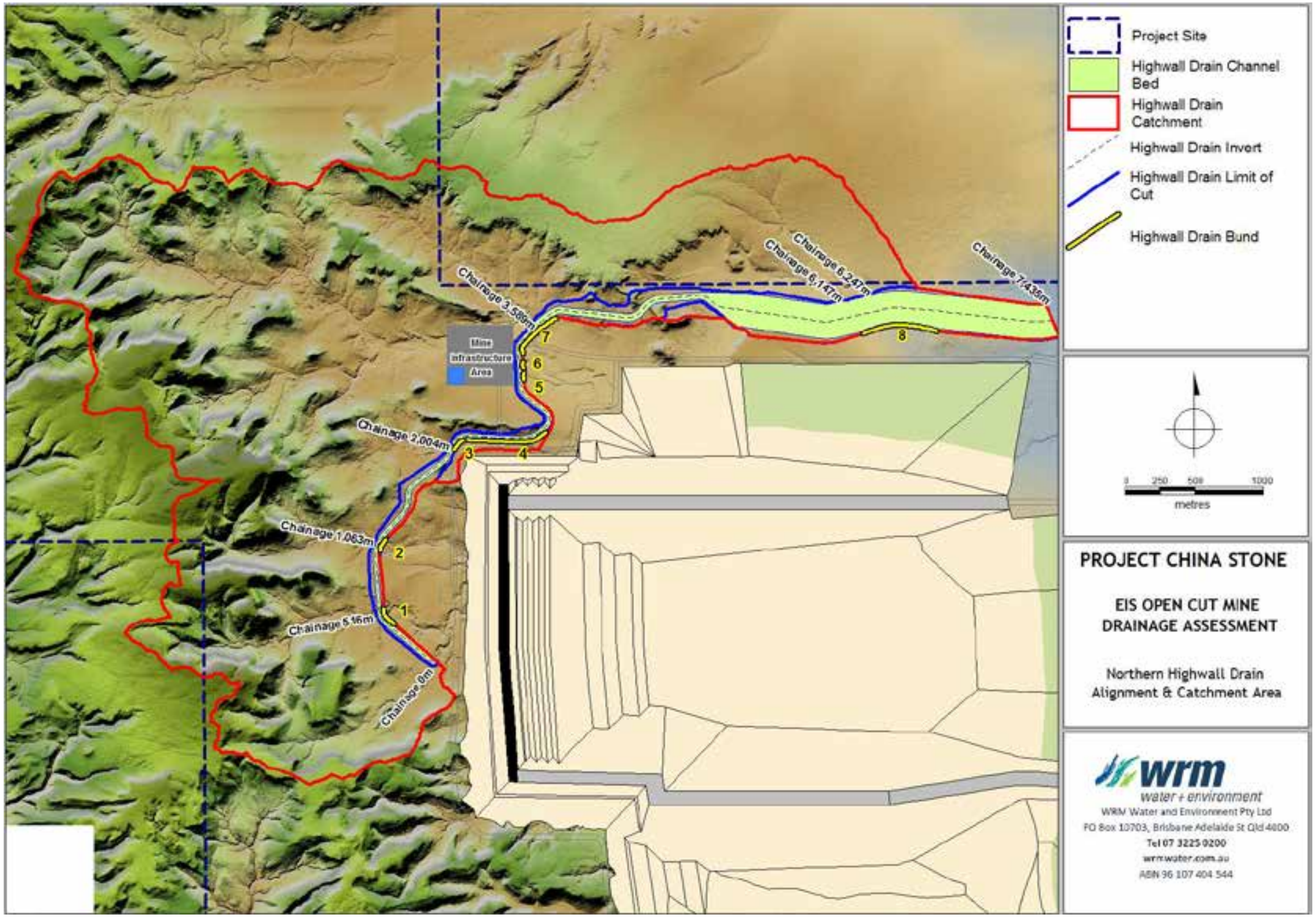

Figure 7.1 Northern highwall drain alignment and catchment area

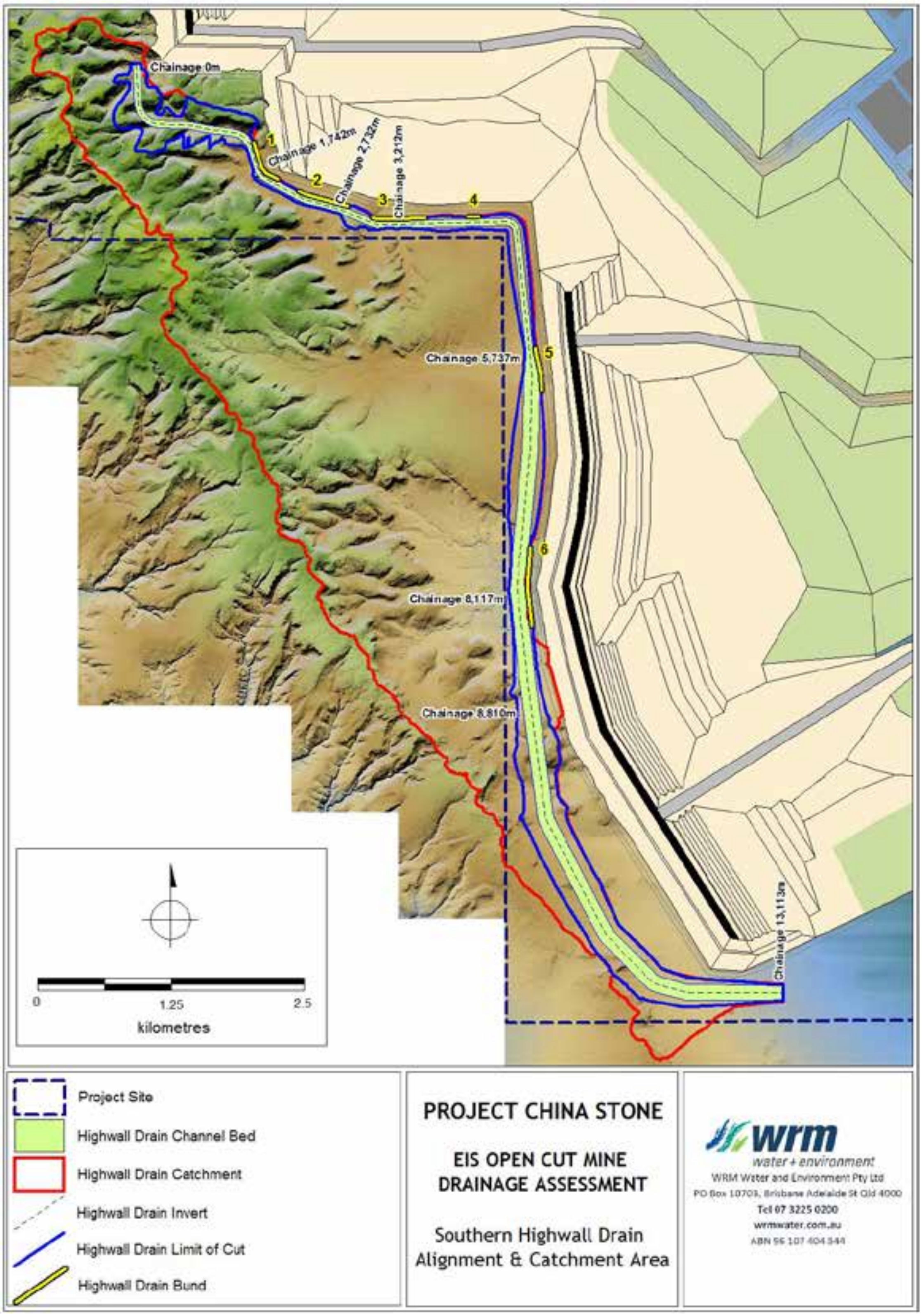

Figure 7.2 Southern highwall drain alignment and catchment area

7.5.2 Longitudinal grade

Figure 7.3 and Figure 7.4 show long sections along the invert of the proposed northern and southern highwall drains, including existing ground levels along the drain alignment. Channel bed slopes in the highwall drains range from 0.11 in 100 to 1.0%. These slopes are within the range of existing bed slopes in the drainage features intercepted by the northern highwall drain. The bed slopes of the various reaches of the highwall drains are variable, due to the constraint of having to match the invert of each intercepted drainage feature that it intercepts.

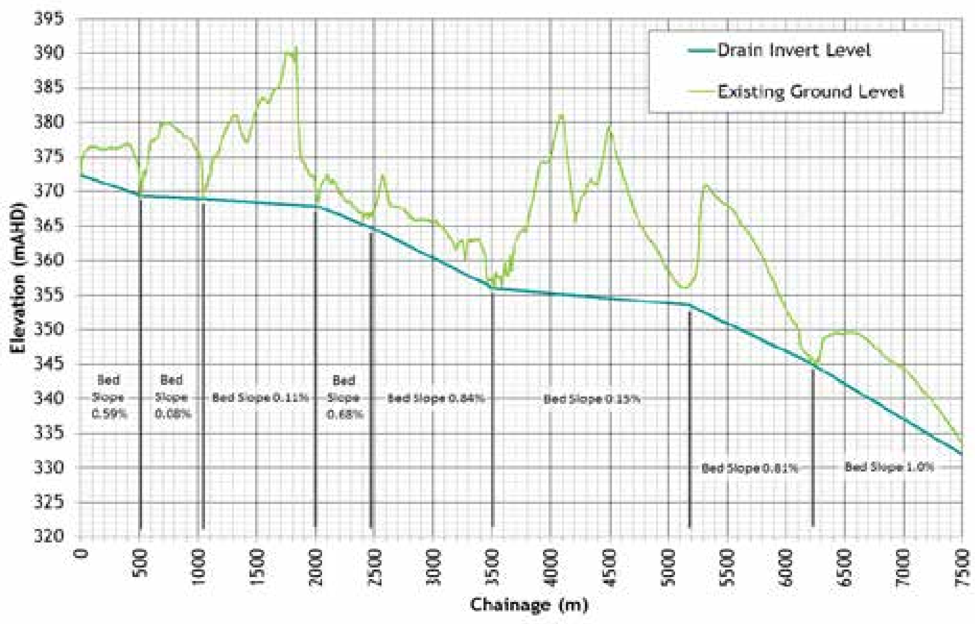

Figure 7.3 Northern highwall drain long section

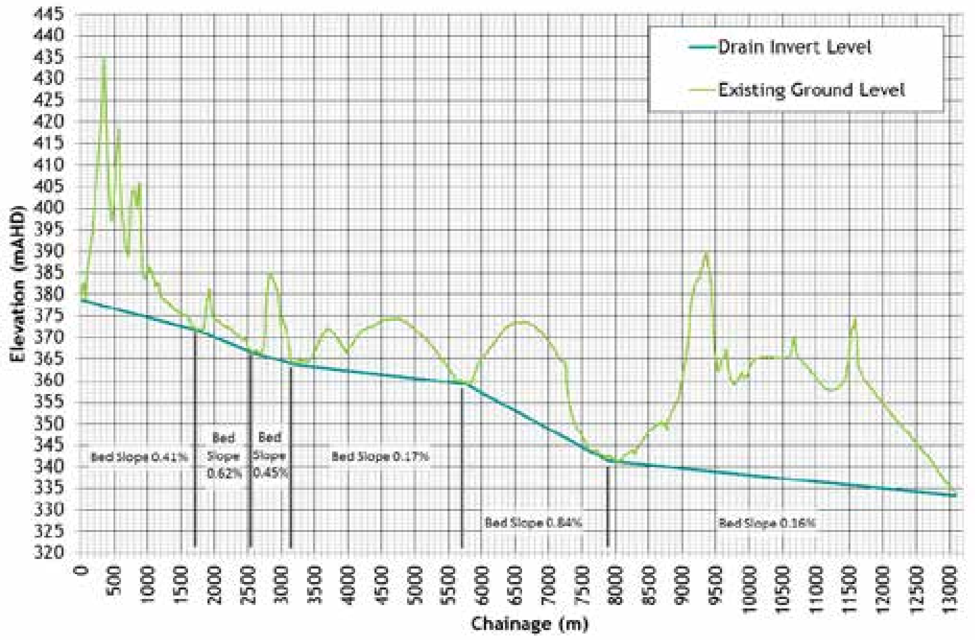

Figure 7.4 Southern highwall drain long section

7.5.3 Typical cross sections

An idealised trapezoidal channel cross section was developed for each reach of the highwall drains. The drain cross section dimensions are dependent on drain bed slope and design discharge in the drain. The base width of the drain was widened in various sections to reduce flow depths sufficiently to meet the adopted design criteria (See Section 8.2).

Figure 7.5, Figure 7.6 and Figure 7.7 show the adopted cross section dimensions for the northern highwall drain. Figure 7.8 and Figure 7.9 show the adopted cross section dimensions for the southern highwall drain.

The northern highwall drain has three cross sections, increasing in drain base width as the drain catchment increases. The smaller drain section (base width 34m) is applied from chainage 0m to 3,512m of the northern highwall drain. The 54m base width section is applied from chainage 3,512m to 5,182m. The largest drain section (base width 250m) is applied from chainages 5,182 to the outlet. The larger drain section through this reach of drain is required due to the steeper bed slopes in this reach (about 1 in 100).

The southern highwall drain has two cross sections, also increasing in drain base width as the drain catchment increases. The smaller drain section (base width 70m) is applied from chainage 0m to 5,573m of the southern highwall drain (corresponding to the Year 5 drain outlet point). From this point onwards the 150m base width section is applied due to the increased drain catchment and reaches of steeper slope (up to 0.85% bed slope).

The following key points are of note with regards to the drain cross sections:

- All drain cross sections have a 1 in 100 crossfall from the edge of the drain to the invert; and
- All drain cross sections have a 1V:3H cut slope for the embankments.

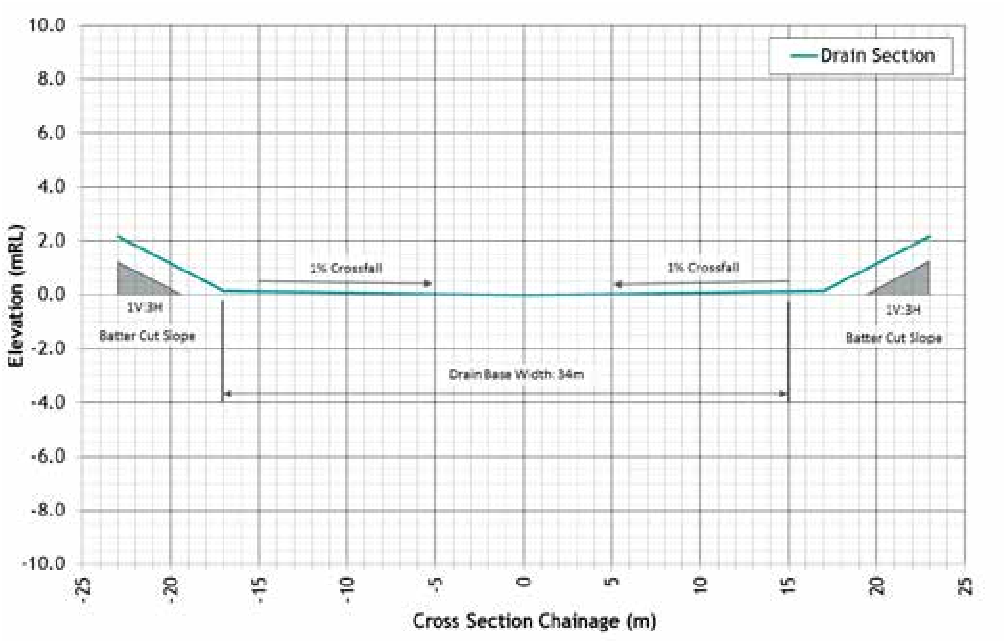

Figure 7.5 Northern highwall drain cross section, chainage 0m to 3,512m

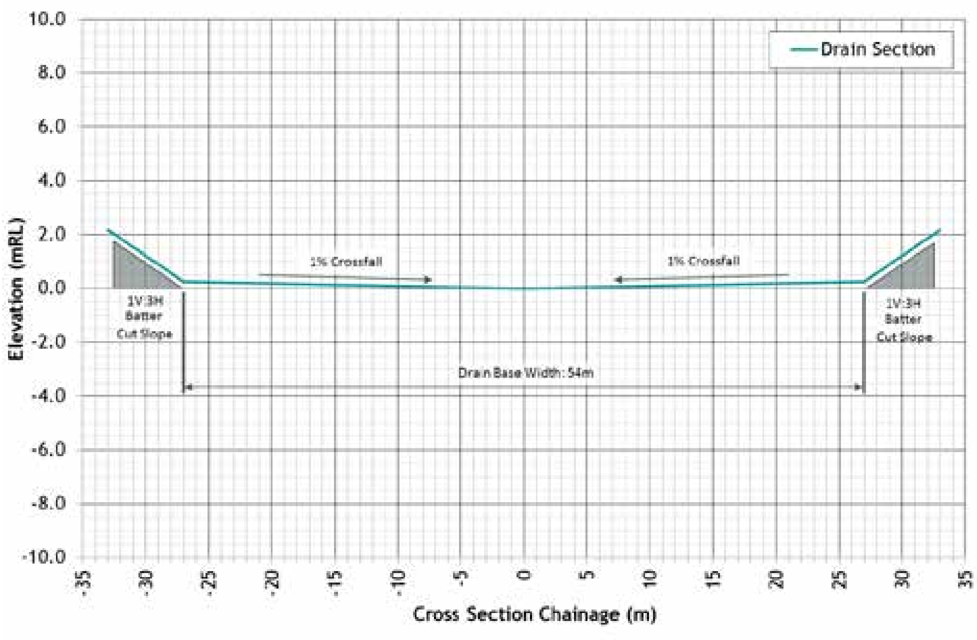

Figure 7.6 Northern highwall drain cross section, chainage 3,512m to 5,182m

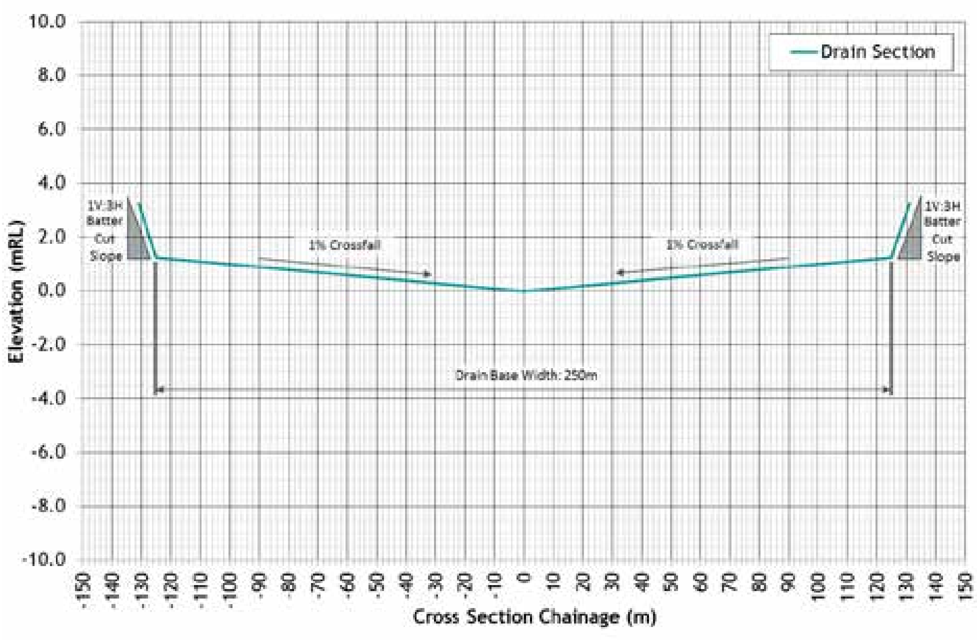

Figure 7.7 Northern highwall drain cross section, chainage 5,182m to 7,435m

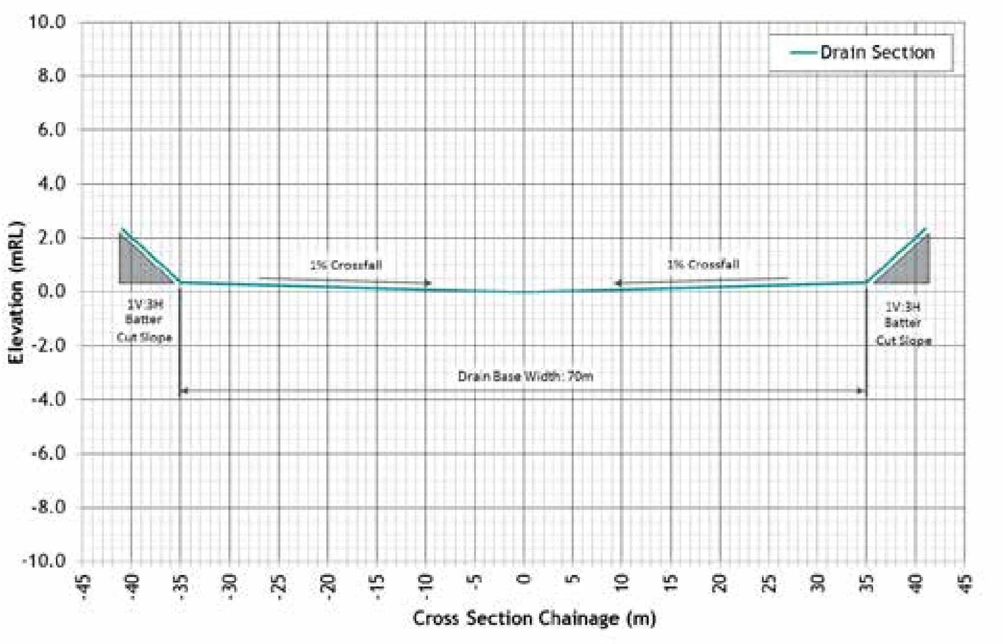

Figure 7.8 Southern highwall drain cross section, chainage 0m to 5,737m

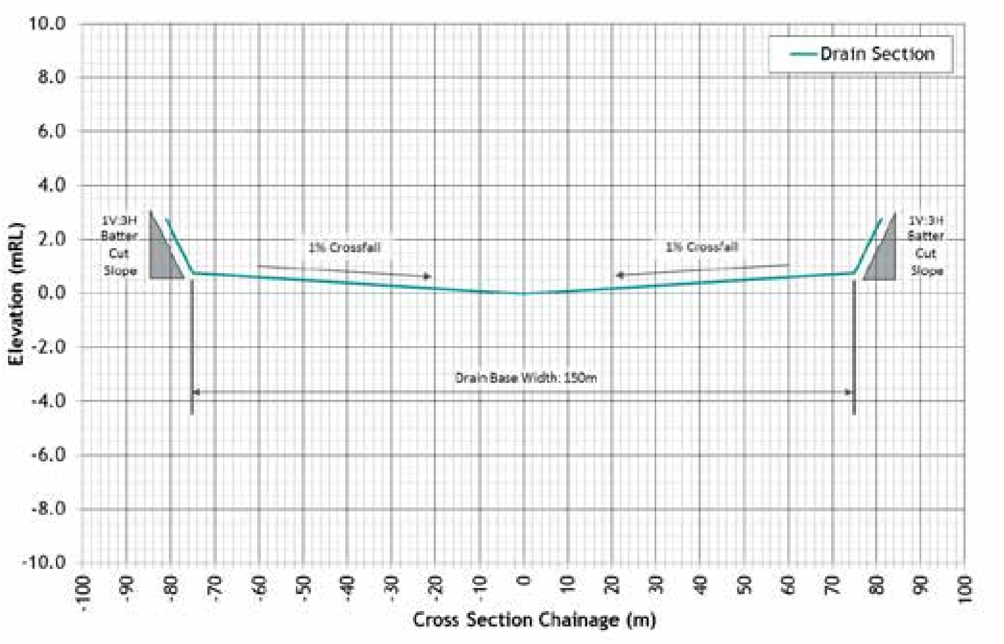

Figure 7.9 Southern highwall drain cross section, chainage 5,737m to 13,113m

7.5.4 Vegetation

The base of the highwall drains will be planted with grasses in a similar manner to the existing channels. A vegetation plan will be developed as part of the detailed design of the drains. The vegetation plan will include inspections and maintenance of the drains to ensure the establishment of adequate grass cover in the bed of the drains. A Manning's 'n' value of 0.035 has been adopted for hydraulic modelling of the highwall drains.

Due to the significant cuts required in some places to construct the highwall drains, a number of embankments will be created with slopes of up to 1V:3H and lengths of over 50m. There is potential for rill erosion to occur down the embankment faces where local stormwater runoff collects and drains down to the base of the drain. In order to limit rill erosion and protect the stability of the embankments, they will be revegetated with grasses. During the first few years of mine life particular attention will be paid to identifying locations where rill erosion is occurring and undertaking rehabilitation and maintenance works to resolve this. If necessary additional erosion protection measures including rock-lined chutes and geofabric linings may be implemented.

7.5.5 Flood protection bunds

Bunds will be required to confine floodwater within the highwall drains at locations where the drain is completely constructed in cut (i.e. where the drain intercepts an existing drainage feature, or passes along a floodplain).

Table 7.4 and Table 7.5 summarise the bund extents and heights for each highwall drain to confine all floodwater within the drain for all events up to and including the PMF, based on the post-mining final landform PMF modelling described in Section 10.4.

Table 7.4 Northern highwall drain bund requirements

Bund ID	Upstream Chainage (m)	Downstream Chainage (m)	Upstream PMF Flood Level (mAHD)	Downstream PMF Flood Level (mAHD)
1	427	580	373.37	373.25
2	1032	1122	373.02	372.72
3	1966	2069	371.28	370.23
4	2164	2758	369.47	365.65
5	3171	3202	361.96	361.93
6	3257	3292	361.91	361.90
7	3327	3692	361.90	361.09
8	6164	6562	347.66	343.54

Table 7.5 Southern highwall drain bund requirements

Bund ID	Upstream Chainage (m)	Downstream Chainage (m)	Upstream PMF Flood Level (mAHD)	Downstream PMF Flood Level (mAHD)
1	1581	2002	373.83	372.00
2	2272	2742	370.30	368.26
3	3032	3491	367.47	366.66
4	3934	3996	366.03	365.92
5	5495	5901	363.09	360.28
6	7451	8172	347.36	345.06

8 Operations and closure phase flooding

8.1 OVERVIEW

This section presents the details and results of the Year 5, Year 30 and post-mine TUFLOW models developed for the assessment.

8.2 MODEL DEVELOPMENT

The existing conditions TUFLOW models (north and south) described in Section 8 were modified to represent three scenarios:

- Project Year 5;
- Project Year 30; and
- Post-mining final landform.

8.2.1 Year 5 TUFLOW models configuration

The Year 5 TUFLOW models were used to simulate water surface elevation, depth and velocity for the 1 in 50 AEP event, for guidance in setting flood immunity levels for mine infrastructure.

The Year 5 XP-RAFTS model described in Section 7 was used to produce inflow hydrographs for the Year 5 TUFLOW models. Both the northern and southern Year 5 TUFLOW models were run for the critical 60 and 180 minute durations storm events, as estimated by the Year 5 XP-RAFTS model.

Year 5 overburden emplacements and mine infrastructure (including the airstrip and TSF) were blocked out in the Year 5 TUFLOW models. The northern and southern highwall drains were incorporated into the models. The proposed rail loop and access road road alignment along the eastern boundary of the project site was included in the Year 5 TUFLOW model, with the rail embankment level set at a sufficient height to provide immunity from the 1 in 50 AEP flood event. Openings for flow conveyance culverts were provided in the rail embankment. Concept designs for the rail embankment culverts are provide in Section 4.

Predicted Year 5 subsidence due to project longwall mining was incorporated into the Year 5 TUFLOW model topographies. Hansen Bailey also provided predicted subsidence contours for the project longwall mining operations.

8.2.2 Year 30 TUFLOW models configuration

The Year 30 TUFLOW models were used to simulate water surface elevation, depth, velocity, bed shear stress and stream power for the 1 in 2, 1 in 50 and 1 in 1000 AEP events, for guidance in setting flood immunity levels for mine infrastructure, and determining impacts of the project on surrounding properties and stream geomorphological processes.

The Year 30 XP-RAFTS model described in Section 7 was used to produce inflow hydrographs for the Year 30 TUFLOW models. The northern Year 30 TUFLOW model was run for the critical 60 minute and 180 minute storm durations for all AEPs, and the southern Year 30 TUFLOW model were run for the critical 60 and 120 minute durations storm events for all AEPs.

The Year 30 TUFLOW models included the overburden emplacements, open cut pits, TSF and southern highwall drain, with all other infrastructure as per the Year 5 TUFLOW models. Predicted Year 30 subsidence due to project longwall mining was incorporated into the Year 30 TUFLOW model topographies.

8.2.3 Post-mining final landform

The post-mining final landform TUFLOW model was used to simulate water surface elevation, depth and velocity for the PMF event, to confirm that the final voids are not inundated for any event up to and including the PMF.

The Year 30 XP-RAFTS model described in Section 7 was used to produce inflow hydrographs for the post-mining final landform models as drainage catchments within the study area are effectively unchanged between Year 30 and the final landform. The post-mining final landform TUFLOW model was run for the critical 60 minute (northern and southern TUFLOW models), 120 minute (southern TUFLOW model) and 240 minute (northern TUFLOW model) storm durations, based on the critical PMF durations estimated by the XP-RAFTS model.

The post-mining final landform TUFLOW model included the final rehabilitated landform, including open cut pit final voids, rehabilitated overburden emplacements and TSF. All mine infrastructure areas, rail loops and haul roads were removed.

8.3 YEAR 5 FLOODING

Figure A5, Appendix A shows the predicted extent and depth of flooding at the project site for the 1 in 50 AEP event during Year 5 of mine life. Peak water surface contours are also shown. The following is of note:

- All mine infrastructure (including the rail loop and all open cut pits) is free from flooding during Year 5 for the 1 in 50 AEP event.
- Peak flood depths along the Year 5 TSF embankment during a 1 in 50 AEP event are about 0.7m above existing ground levels at the lowest point. Peak 1 in 50 AEP flood levels along the Year 5 TSF embankment range from 315.75mAHD to 311.3mAHD.
- A small temporary drain and bund will be constructed to ensure the south-western corner of the overburden emplacement downstream of the southern highwall drain is not inundated during a 1 in 50 AEP event.
- Peak 1 in 50 AEP flood depths along the haul road located to the south and west of the airstrip during Year 5 are about 0.9m above existing ground levels (at the lowest point along the haul road alignment). Peak 1 in 50 AEP flood levels along the haul road range from 295.6mAHD to 287mAHD. The road embankment will be constructed at levels in this area to prevent flooding of the airstrip.
- Peak 1 in 50 AEP flood levels in the drain at the northern end of the airstrip are about 280mAHD. Flood levels in this area are due to drainage from the mine infrastructure area and overburden emplacements, and are not influenced by runoff from the undisturbed upper catchments or highwall drains.

8.4 YEAR 30 FLOODING

8.4.1 Flood levels, depths and extents

Figure A6, A7 & A8, Appendix A show the predicted extent and depth of flooding at the project site for the 1 in 2, 1 in 50 AEP and 1 in 1000 AEP events during Year 30 of mine life. Peak water surface contours are also shown. The following is of note:

- All mine infrastructure (including the rail loop and all open cut pits) is free from flooding during Year 30 for up to the 1 in 50 AEP event.
- The railway embankment is likely to be overtopped at multiple locations during a 1 in 1000 AEP event.
- The northern and southern highwall drains prevent any runoff from the upstream undisturbed catchment from draining to the open cut pits for up to and including the 1 in 1000 AEP event.
- Peak flood depths along the Year 30 TSF embankment during a 1 in 1000 AEP event are about 1.4m above existing ground levels at the lowest point. Peak 1 in 1000 AEP

flood levels along the Year 30 TSF embankment range from 326.2mAHD along the northern edge of the TSF to 311.5mAHD about midway along the eastern edge of the embankment.

- Peak flood depths along the Power Station Waste Storage Facility embankment during a 1 in 1000 AEP event are about 1.0m above existing ground levels at the lowest point. Peak 1 in 1000 AEP flood levels along the Power Station Waste Storage Facility Embankment range from 298.7mAHD to 297.5mAHD.
- Peak flood depths along the northern edge of the topsoil storage area during 1 in 2 and 1 in 50 AEP events are about 0.4m and 0.7m (respectively) above existing ground levels at the lowest point. If necessary a low bund could be constructed along the northern edge of the topsoil storage area to prevent any erosion of the topsoil stockpile during flooding.

8.4.2 Flow velocity

Figure A9, A10 & A11, Appendix A show the peak flow velocity grids for the 1 in 2, 1 in 50 and 1 in 1000AEP events during Year 30. The following is of note:

- Peak flow velocities adjacent to the TSF embankment are less than 1.5m/s during a 1 in 1000 AEP event. Scour protection will be placed at the toe of the embankment to reduce the erosion potential.
- Peak flow velocities along the toe of the Power Station Waste Storage Facility embankment during a 1 in 1000 AEP event are less than 0.5m/s.
- Peak flow velocities along the northern edge of the topsoil storage area during 1 in 2 and 1 in 50 AEP events are about 0.4m/s and 0.9m/s (respectively).
- Localised areas of high velocity are predicted within the subsided areas downstream of the southern highwall drain, and at the outlets of the rail embankment culverts.

8.5 POST-MINING FINAL LANDFORM

Figure A12, Appendix A shows the predicted extent and depth of flooding at the project site for the PMF event for the post-mining final landform. Peak water surface contours are also shown. The following is of note:

- The highwall drains prevent runoff from the upstream catchment from draining into the final voids for events up to and including the PMF.
- Floodwater from the northern and southern drainage corridors cannot enter the final void during a PMF event.

8.6 HIGHWALL DRAINS

8.6.1 Flow velocity

Figure 8.1 and Figure 8.2 show the predicted flow velocity in the northern and southern highwall drains during 1 in 2 and 1 in 50 AEP events. Table 8.1 summarises the results of the longitudinal plots. The following is of note:

- Maximum flow velocities in the northern and southern highwall drains do not exceed the upper limits recommended in DNRM (2014b).
- Predicted average flow velocities in the highwall drains during 1 in 2 and 1 in 50 AEP events are generally higher than the average velocities of the existing drainage features intercepted by the drains. The maximum velocities in the drains are however, consistent with the maximum velocities recorded in the existing drainage features.

Table 8.1 Summary of flow velocity in highwall drains

Highwall drain	1 in 2 AEP Channel Velocity (m/s)			1 in 50 AEP Channel Velocity (m/s)		
	Min.	Mean	Max.	Min.	Mean	Max.
Northern	0.1	0.9	1.5	0.2	1.5	2.4
Southern	0.4	0.7	1.1	0.7	1.1	1.7

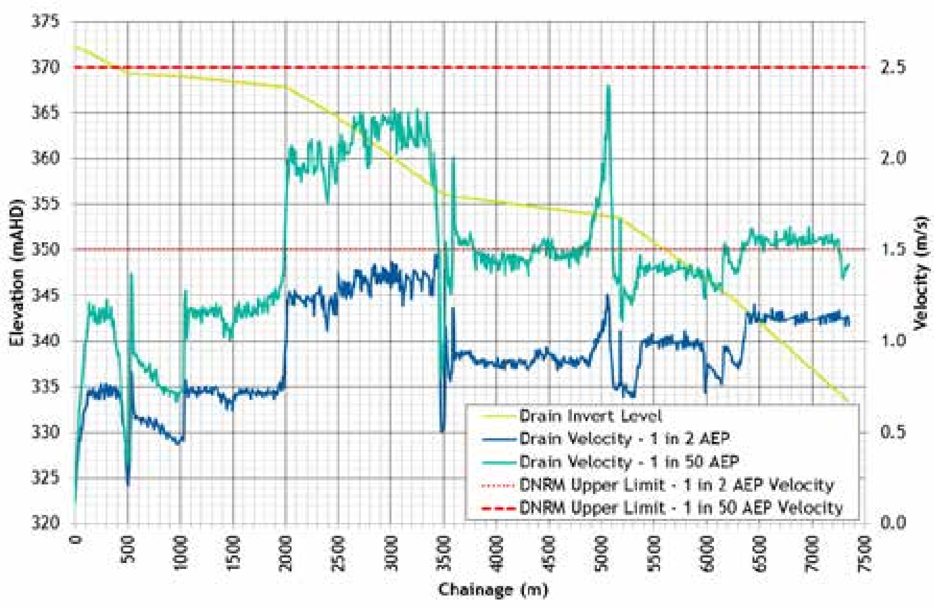

Figure 8.1 Northern highwall drain flow velocity, longitudinal profile

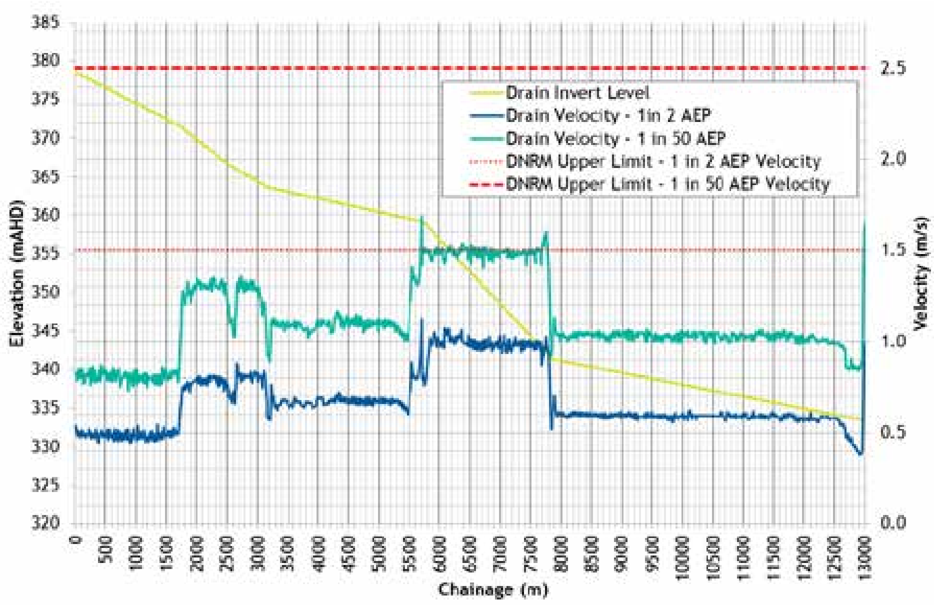

Figure 8.2 Southern highwall drain flow velocity, longitudinal profile

8.6.2 Bed shear stress

Figure 8.3 and Figure 8.4 show the predicted bed shear stresses in the northern and southern highwall drains during 1 in 2 and 1 in 50 AEP events. Table 8.2 summarises the results of the longitudinal plots. The following is of note:

- Maximum bed shear stresses in the northern and southern highwall drains do not exceed the upper limits recommended in DNRM (2014b).
- Predicted average bed shear stresses in the highwall drains during 1 in 2 and 1 in 50 AEP events are significantly less than existing bed shear stresses in the drainage features intercepted by the drains.
- Predicted average bed shear stresses in the reach between chainage 2000 and 3500 and downstream of chainage 6400 (see Figure 7.1) of the northern drain are similar to the existing average bed shear stresses in the intercepted drainage features. These reaches have the highest predicted bed shear in both drains.

Table 8.2 Summary of bed shear stress in highwall drains

Highwall drain	1 in 2 AEP Bed Shear Stress (N/m^2)			1 in 50 AEP Bed Shear Stress (N/m^2)		
	Min.	Mean	Max.	Min.	Mean	Max.
Northern	1.1	16.9	34.2	1.5	32.2	69.3
Southern	2.0	9.4	40.1	8.9	18.7	74.1

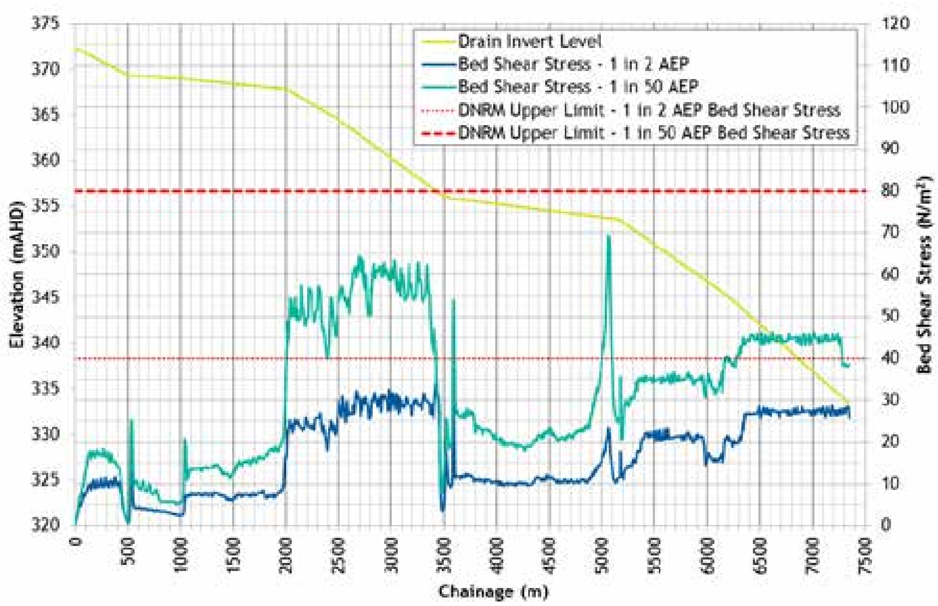

Figure 8.3 Northern highwall drain bed shear stress, longitudinal profile

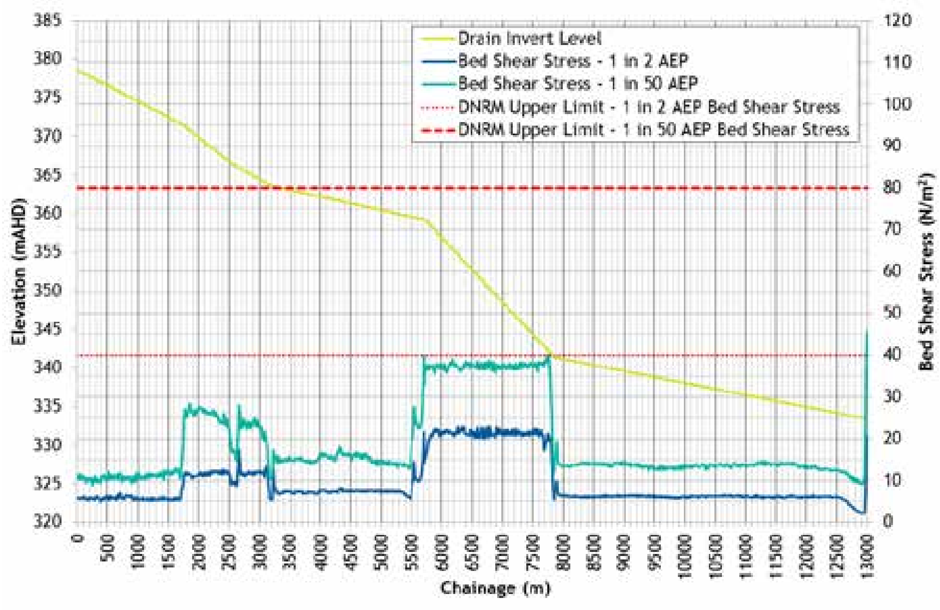

Figure 8.4 Southern highwall drain bed shear stress, longitudinal profile

8.6.3 Stream power

Figure 8.5 and Figure 8.6 show the predicted stream power in the northern and southern highwall drains during 1 in 2 and 1 in 50 AEP events. Table 8.3 summarises the results of the longitudinal plots. The following is of note:

- Maximum stream power in the northern and southern drains does not exceed the upper limits recommended in DNRM (2014b).
- Predicted average and maximum stream power in the highwall drains during 1 in 2 and 1 in 50 AEP events are generally equal to or less than existing bed shear stresses in the drainage features intercepted by the drains.
- Predicted average stream power in the reach between chainage 2000 and 3500 and downstream of chainage 6400 (see Figure 7.1) of the northern drain are marginally higher than the existing average stream power in the intercepted drainage features. These reaches have the highest predicted steam power in both drains. The maximum values in these reaches are well below the maximum stream power in the intercepted reaches.

Table 8.3 Summary of stream power in highwall drains

Highwall drain	1 in 2 AEP Stream power (W/m²)			1 in 50 AEP Stream power (W/m²)		
	Min.	Mean	Max.	Min.	Mean	Max.
Northern	0.4	21.3	51.0	0.5	60.0	165.7
Southern	0.8	8.8	38.5	7.1	24.6	125.7

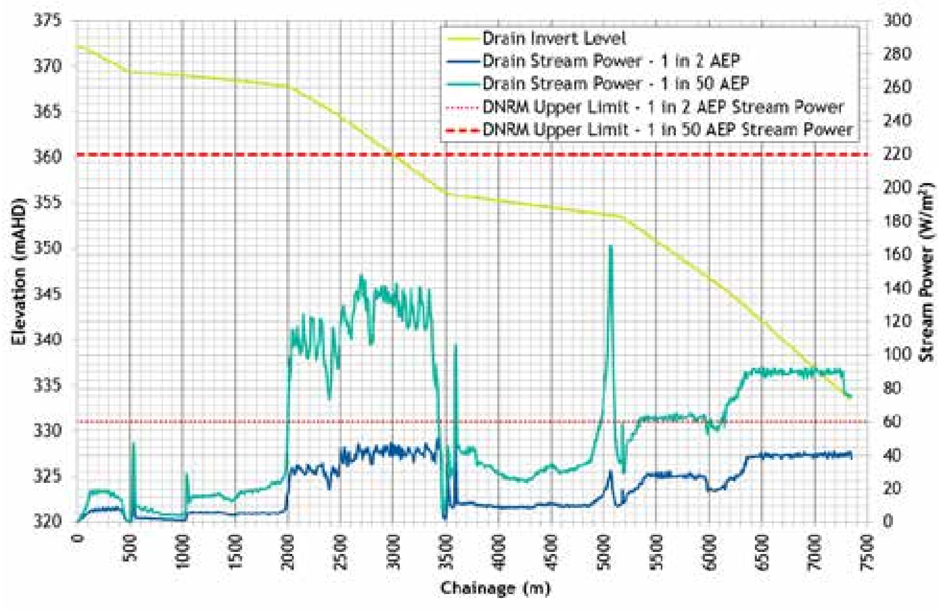

Figure 8.5 Northern highwall drain stream power, longitudinal profile

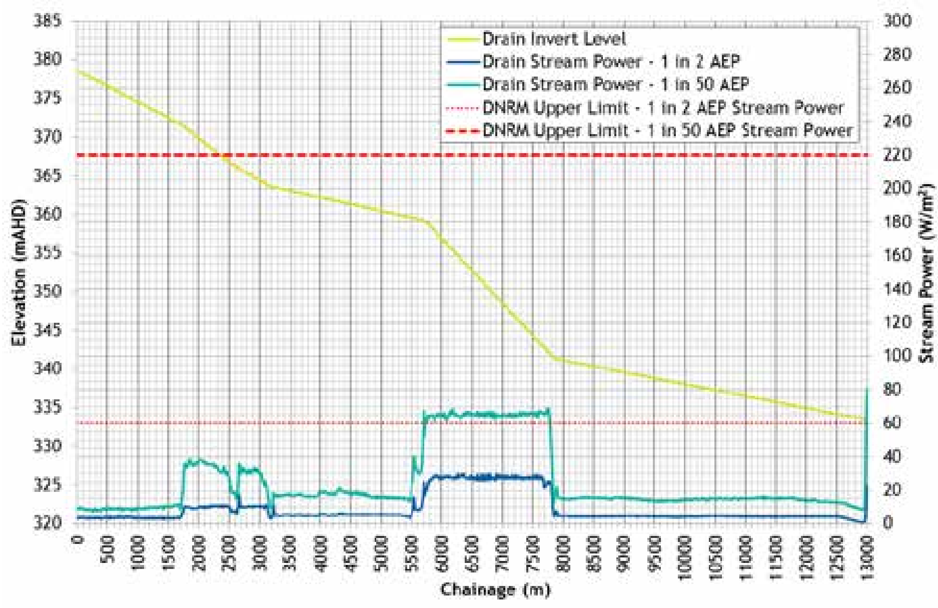

Figure 8.6 Southern highwall drain stream power, longitudinal profile

8.7 DISCUSSION OF HIGHWALL DRAIN RESULTS

8.7.1 Northern highwall drain

The northern highwall drain generally replicates the hydraulic characteristics of the drainage features it intercepts, with the exception of the reach located between chainage 2000 and 3500 and downstream of chainage 6400. Average flow velocities and stream powers within this reach are marginally higher than those experienced by the intercepted drainage features, but well below the existing conditions drainage feature maximums. Bed shear stresses within this reach of the drain are similar to those in the intercepted drainage features under existing conditions.

Provided the northern drain is geotechnically stable, sufficiently vegetated, and protected against rill erosion, the drains should perform adequately during runoff events and be stable in the long term. Particular care during vegetation establishment will be paid to the reach of the drain between chainage 2000 and 3500 and downstream of chainage 6400, which are located nearby to the mine infrastructure area. These reaches will be monitored regularly for signs of erosion, and rehabilitation works undertaken if required.

8.7.2 Southern highwall drain

The southern highwall drain generally replicates the hydraulic characteristics of the drainage features it intercepts. Provided the southern drain is geotechnically stable, sufficiently vegetated, and protected against rill erosion, the drains should perform adequately during runoff events and be stable in the long term.

9 Downstream impacts

9.1 OVERVIEW

This section presents the assessment of the impacts of the project on drainage and flooding on downstream properties and the local drainage features.

The TUFLOW model results for Year 30 were compared with existing conditions TUFLOW model results for 1 in 2 and 1 in 50 AEP events in order to quantify impacts on surrounding properties and stream geomorphology. Impacts on peak water levels, flow velocities, bed shear stress and stream power are discussed in the following sections.

9.2 FLOOD LEVELS

Figures A13 and A14, Appendix A show the predicted impact of the project on 1 in 2 and 1 in 50 AEP flood levels. The following is of note:

- Minor increases in 1 in 2 AEP flood levels are predicted downstream of the eastern boundary of the project site. In most cases the predicted increase in peak level is less than 0.05m (50mm), however in some localised areas increases of up to 0.1m (100mm) are predicted.
- Localised increases in 1 in 2 AEP flood levels of up to about 0.3m (300mm) are predicted along the northern boundary of the project site in the study area, downstream of the outlet of the northern highwall drain.
- Increases in 1 in 50 AEP flood levels are greater and more extensive than those predicted for the 1 in 2 AEP event.
- Peak 1 in 2 AEP flood levels are predicted to increase by between 0.05 and 0.1m in some drainage features downstream of the eastern project site boundary, with some localised increases of up to 0.15m (150mm) occurring.
- Peak 1 in 50 AEP flood levels at the northern boundary of the project site in the study area downstream of the northern highwall drain are predicted to increase by between 0.3m and 0.5m. The increase is localised and dissipates less than 200m north of the project site boundary.
- Reductions in flood levels are also predicted in numerous drainage features downstream of the project site boundary. This is mainly due to the redistribution of flow that will occur due to the project, resulting in some drainage features carrying more water and flow in others being reduced.
- The predicted increases in flood level will not impact on any structures or property, and in most cases will be indiscernible when compared to existing conditions due to the wide shallow nature of the floodplain.

9.3 FLOW VELOCITIES

Figures A15 and A16, Appendix A show the predicted impact of the project on 1 in 2 and 1 in 50 AEP peak flow velocities. The following is of note:

- Minor increases in 1 in 2 AEP flow velocities are predicted at and downstream of the eastern boundary of the project site. In most cases the predicted increases are less than 0.1m/s. Existing peak 1 in 2 AEP flow velocities in this area are between 0.25m/s and 0.5m/s.
- Localised increases in 1 in 2 AEP flow velocity of up to about 0.3m/s are predicted along the northern boundary of the project site within the study area, downstream of the outlet of the northern highwall drain. Existing peak 1 in 2 AEP velocities within the drainage feature channels in this area are greater than 1.5m/s.

- Peak 1 in 2 AEP flow velocities in the drainage feature 2 channel downstream of the northern highwall drain outlet are predicted to increase by up to 0.1m/s. Existing 1 in 2 AEP velocities in the channel over this reach are between 1m/s and 2m/s.
- Peak 1 in 2 AEP flow velocities are predicted to significantly increase through the subsided area of the southern drainage corridor above the Southern Underground, located downstream of the southern highwall drain. Peak 1 in 2 AEP flow velocities are predicted to increase by up to 0.5m/s within the subsided area. Existing peak 1 in 2 AEP flow velocities are typically less than 0.5m/s in this area.
- Peak 1 in 50 AEP flow velocities in drainage features downstream of the eastern boundary of the project site are predicted to increase by up to 0.1m/s, with some localised increases of up to 0.2m/s, particularly adjacent to the northern topsoil stockpile and at the outlets of the railway embankment culverts. If necessary, a small earth bund will be constructed at the northern end of the topsoil stockpile area to prevent erosion of the stockpile during flooding.
- Peak 1 in 50 AEP flow velocities along the northern boundary of the project site within the study area downstream of the northern highwall drain are predicted to increase by up to 0.4m/s. Similar increases are predicted along sections of drainage feature 1 downstream of this location. Existing conditions 1 in 50 AEP velocities in the drainage features in these areas are between 1.5m/s and 2.5m/s. Existing overbank velocities are significantly less than those in the existing channels.
- Significant increases in 1 in 50 AEP flow velocities are predicted through the subsided area of the southern drainage corridor above the Southern Underground, downstream of the southern highwall drain. Peak 1 in 50 AEP flow velocities are predicted to increase by up to 1m/s within the subsided area. Increased velocities appear to be limited to within the subsided area.
- In contrast to the predicted increases in velocities there are a number of drainage features, both within the project site boundary and downstream of the eastern boundary which are predicted to experience reduced peak flow velocities due to redistribution of flows.

9.4 BED SHEAR STRESS

Figures A17 and A18, Appendix A show the predicted impact of the project on 1 in 2 and 1 in 50 AEP bed shear stresses. The following is of note:

- There is little change to 1 in 2 and 1 in 50 AEP bed shear stresses downstream of the eastern boundary of the project site. This is due to the shallow, low velocity nature of flooding in this area and the relative small changes to flow behaviour predicted.
- Localised increases to 1 in 50 and 1 in 2 AEP bed shear stress are predicted downstream of the northern topsoil stockpile, and downstream of the railway embankment culverts. Bed shear stresses in these areas are predicted to increase by up to $5N/m^2$ for the 1 in 2 AEP event, and up to $15N/m^2$ for the 1 in 50 AEP event.
- Localised increases in 1 in 50 and 1 in 2 AEP bed shear stress are predicted along the northern boundary of the project site within the study area, downstream of the northern highwall drain outlet. Bed shear stresses in these areas are predicted to increase by up to $10N/m^2$ for the 1 in 2 AEP event, and up to $25N/m^2$ for the 1 in 50 AEP event. There are very small and confined areas which may experience increases of up to $20N/m^2$ for the 1 in 2 AEP event and up to 50N/m2 during a 1 in 50 AEP event. Existing bed shear stresses in the drainage features in this area are about $45N/m^2$ during a 1 in 2 AEP event and greater than $70N/m^2$ during a 1 in 50 AEP event. Existing overbank shear stresses are significantly less than those in the existing channels.
- Minor increases in 1 in 50 and 1 in 2 AEP bed shear stress are predicted in drainage feature 2 downstream of the northern highwall drain outlet, extending through the project site towards the eastern boundary. Increases in bed shears stress in this

region are typically less than 10N/m^2 for both the 1 in 2 and 1 in 50 AEP events. Existing bed shear stresses in this region are up to 80N/m^2 during a 1 in 2 AEP event, and up to 170N/m^2 during a 1 in 50 AEP event.

- Significant increases in 1 in 50 and 1 in 2 AEP bed shear stress are predicted through the subsided area of the southern drainage corridor above the Southern Underground, downstream of the southern highwall drain. Bed shear stress is predicted to increase within the subsidence zone by up to 55N/m^2 for the 1 in 2 AEP event and up to 150N/m^2 for the 1 in 50 AEP event. The increased shear stresses are typically confined within the subsidence zones
- Bed shear stresses are predicted to be reduced in a number of drainage paths both within and downstream of the project site due to redistribution of flows.

9.5 STREAM POWER

Figures A19 and A20, Appendix A show the predicted impact of the project on 1 in 2 and 1 in 50 AEP stream power. The following is of note:

- There is little change to 1 in 2 and 1 in 50 AEP stream power downstream of the eastern boundary of the project site. This is due to the shallow, low velocity nature of flooding in this area and the relative small changes to flow behaviour predicted.
- Localised increases to 1 in 50 and 1 in 2 AEP stream power are predicted downstream of the northern topsoil stockpile, and downstream of the railway embankment culverts. Stream power in these areas are predicted to increase by up to 5W/m^2 for the 1 in 2 AEP event, and up to 15W/m^2 for the 1 in 50 AEP event.
- Localised increases in 1 in 50 and 1 in 2 AEP stream power are predicted along the northern boundary of the project site within the study area, downstream of the northern highwall n drain outlet. Stream power in these areas is predicted to increase by up to 10W/m^2 for the 1 in 2 AEP event, and up to 25W/m^2 for the 1 in 50 AEP event. There are very small and confined areas which may experience increases of up to 20W/m2 for the 1 in 2 AEP event and up to 70W/m2 during a 1 in 50 AEP event. Existing stream power in the drainage features in this area are about 50W/m^2 during a 1 in 2 AEP event and greater than 100W/m^2 during a 1 in 50 AEP event. Existing overbank stream power is significantly less than those in the existing channels.
- Minor increases in 1 in 50 and 1 in 2 AEP stream power are predicted in drainage feature 2 downstream of the northern highwall drain outlet, extending through the project site towards the eastern boundary. Increases in bed shears stress in this region are typically less than 10W/m^2 for both the 1 in 2 and 1 in 50 AEP events. Existing stream power in this region is up to 80W/m^2 during a 1 in 2 AEP event, and 170W/m^2 during a 1 in 50 AEP event.
- Significant increases in 1 in 50 and 1 in 2 AEP stream power are predicted through the subsided area of the southern drainage corridor above the Southern Underground, downstream of the southern highwall drain. Stream power is predicted to increase within the subsidence zone by up to 60W/m^2 for the 1 in 2 AEP event and up to 200W/m^2 for the 1 in 50 AEP event. The increased stream power is typically confined within the subsidence zone
- Stream power is predicted to be reduced in a number of drainage paths both within and downstream of the project site due to redistribution of flows.

9.6 MITIGATION AND MANAGEMENT MEASURES

9.6.1 Eastern boundary of project site

The floodplain and drainage features downstream of the eastern boundary of the project site are characterised by low depth and low velocity flows, typical of low energy flowpaths.

The project is not predicted to have significant impacts on these drainage features, with very minor increases in velocities, flood depths, bed shear stresses and stream power. No management or mitigation measures are proposed for these drainage features.

9.6.2 Railway embankment culverts

The culverts beneath the railway embankment will be subject to detailed design, to be undertaken at a later stage. Detailed design of these culverts will include selection of appropriate culvert headwall and apron structures (including concrete or rock gabion erosion protection and energy dissipation areas) to minimise erosion at culvert inlets and outlets.

Detailed design of the railway embankment culverts will also include works downstream of culvert outlets to mitigate the concentration of flow by returning culvert discharges to a wide shallow flowpath before they pass across the eastern boundary of the project site.

9.6.3 Northern boundary of project site

A limited area along and immediately downstream of the northern boundary of the project site within the study area (downstream of the outlet of the northern highwall drain) will be exposed to increased flood levels, flow velocities, bed shear stresses and stream power. It is of note that this area experiences reasonably high velocities, bed shears and stream power under existing conditions, and existing erosion is evident in drainage feature 2 downstream of this area.

It is possible that this area will experience increased erosion in both channels and overbank areas. Erosion protection and energy dissipation measures for the drainage features downstream of the northern highwall drain will be considered during detailed design. Measures to be considered may include, but are not limited to:

- Rock erosion protection around areas of high velocity and bed shear stress;
- Energy dissipation structures including flow spreaders;
- Geofabric protection and extensive planting and revegetation of overbank and floodplain areas.

The proponent will monitor the potentially affected area to the north of the project site boundary, particularly during and following significant rainfall and flow events. If necessary erosion protection and remediation works may be extended outside of the project site boundary.

9.6.4 Southern underground subsidence zone

Geomorphological impacts due to the subsidence above the Southern Underground will be limited to areas within the project site boundary. Hydraulic model results indicate that the drainage features within the subsidence zone could potentially experience significant erosion over the life of the project. This area will be monitored for erosion after flow events and erosion control measures will be installed if necessary.

10 Summary

10.1 OVERVIEW

WRM Water & Environment were engaged by Hansen Bailey to undertake a drainage assessment for the open cut mine and mine infrastructure area for the Project China Stone EIS. As part of this study, WRM have investigated the flooding and geomorphological characteristics of the existing watercourses and drainage features in the study area, and assessed flooding and geomorphological behaviour for Year 5, Year 30 and post mining scenarios.

Concept designs were developed for the highwall drains proposed as part of the project, and culverts beneath the proposed railway embankment.

10.2 EXISTING DRAINAGE FEATURES

There are no watercourses, as defined by the Water Act 2000, on the project site (DNRM, 2014a). The characteristics of drainage features within the study area differ substantially from the steep upper catchment adjacent to Darkies Range, to the wide flat overland flowpaths evident across the majority of the study area. The project site is located within the upper catchments of Tomahawk Creek and North Creek. Tomahawk Creek and North Creek become watercourses approximately 20 km and 8km downstream of the project site, respectively.

A number of unnamed drainage features traverse the project site. The majority of the drainage features originate along Darkies Range, via a number of confined, steep and rocky channels. The drainage features transition to wide, shallow overland flowpaths in the flatter regions of the project site, typically with no defined channels. Channel bed slopes in the upper reaches of the drainage features at the project site range from 1% to 2.5%. Bed slopes along the drainage features in the flatter portion of the project site range from 0.2% to 0.5%.

The following is of note with regards to existing conditions flooding at the project site:

- Flooding at the project site is typically via wide shallow sheet flow in areas where there are no defined channels, with more than 80% of the flood extent inundated to a depth of 0.5m or less.
- Flooding is more confined in the upper reaches of the drainage features traversing the project site, with flood depths ranging from 1m to 2m. Flood depths of between 2m and 3m occur in isolated regions where the drainage feature channels are well defined.
- Peak flow velocities are less than 0.5m/s for the vast majority of the inundated area.
- Higher velocities tend to be limited to the upper reaches of the drainage features adjacent to Darkies Range and within drainage features with well defined channels. These areas are predicted to experience peak velocities of greater than 1.5m/s.
- Bed shear stresses and stream power are high in the upper reaches of the drainage features traversing the project site, and in drainage features with well defined channels. Bed shear stresses and stream power are significantly lower in the flatter parts of the project site due to the wide, shallow nature of the floodplain and small main channel.

10.3 MINE LIFE AND POST-MINING DRAINAGE

Two highwall drains will be constructed as part of the project, as well as mine infrastructure, including a tailing storage facility (TSF) and rail loop and embankment. The TSF embankment will extend into the floodplain of the northern drainage corridor, whilst the railway embankment crosses the floodplain in the southern drainage corridor downstream of the southern highwall drain.

The TSF embankment will be designed and constructed by a suitably qualified person to ensure it will remain stable during flood events in the northern drainage corridor, and remain as a sustainable landform post-mining. Erosion protection along the toe of the embankment will be provided. The railway embankment will have a number of culvert openings to pass runoff from the upstream catchments across the eastern boundary of the project site.

The following key points are of note with regards to flooding at the project site during the operations phase and post-mining:

- All mine infrastructure (including the rail loop) is free from flooding for up to the 1 in 50 AEP event.
- A small temporary drain and bund will be constructed to ensure the south-western corner of the Year 5 overburden emplacement downstream of the southern highwall drain is not inundated.
- The railway embankment is likely to be overtopped at multiple locations during a 1 in 1000 AEP event.
- Localised high velocities are predicted to occur at the base of the TSF embankment during large flood events. Erosion protection measures will be placed at the toe of the embankment to reduce erosion potential.
- The northern and southern highwall drains prevent any runoff from the upstream undisturbed catchment from draining to the open cut pits for up to and including the PMF event.
- Localised areas of high velocity are predicted within the subsided areas above the Southern Underground downstream of the southern highwall drain, and at the outlets of the rail embankment culverts.
- Floodwater from the northern and southern drainage corridors downstream of the highwall drains cannot enter the final void during a PMF event.
- The northern highwall drain generally replicates the hydraulic characteristics of the drainage features it intercepts, with the exception of the reach located between chainage 2000 and 3500 and downstream of chainage 6400. Average flow velocities and stream powers within these reaches are marginally higher than those experienced by the intercepted drainage features, but well below the existing conditions drainage feature maximums. Bed shear stresses within these reaches of the drain are similar to those in the intercepted drainage features under existing conditions.
- The southern highwall drain generally replicates the hydraulic characteristics of the drainage features it intercepts.
- Provided the highwall drains are geotechnically stable, sufficiently vegetated, and protected against rill erosion, the drains should perform adequately during runoff events and be stable in the long term. Particular care during vegetation establishment will be paid to the reaches of the northern highwall drain between chainage 2000 and 3500 and downstream of chainage 6400, due to average flow velocities and stream power in this reach marginally exceeding those in the existing drainage features. These reaches will be monitored regularly for signs of erosion, and rehabilitation works undertaken if required.

10.4 IMPACT ASSESSMENT

The impact of the project on flood levels, flow velocities and geomorphology was assessed. The following key findings are of note:

- Minor increases in peak flood level are predicted to occur at several locations along the northern and eastern boundary of the project site.

- The predicted increases in flood level will not impact significantly on any structures or property, and in most cases will be indiscernible when compared to existing conditions due to the wide shallow nature of the floodplain.
- Peak flow velocities are predicted to increase at a number of locations at the northern and eastern boundary of the project site, particularly downstream of the northern highwall drain and at the outlets of the railway embankment culverts. Significant increases in peak flow velocities are also predicted within the subsidence zone above the Southern Underground, which is located downstream of the southern highwall drain.
- Increases in bed shear stress and stream power are also predicted at the locations that are likely to experience increased flow velocities.

10.5 MANAGEMENT AND MITIGATION

Detailed design of the railway embankment culverts will include selection of appropriate culvert headwall and apron structures (including concrete or rock gabion erosion protection and energy dissipation areas) to minimise erosion at culvert inlets and outlets. Detailed design of the railway embankment culverts will also include works downstream of culvert outlets to mitigate the concentration of flow by returning culvert discharges to a wide shallow flowpath before they pass across the eastern boundary of the project site.

Erosion protection and energy dissipation measures will be subject to further evaluation during detailed design for the drainage features downstream of the northern highwall drain. Measures to be considered may include, but are not limited to:

- Rock erosion protection around areas of high velocity and bed shear stress;
- Energy dissipation structures including flow spreaders;
- Geofabric protection and extensive planting and revegetation of overbank and floodplain areas.

The proponent will monitor the potentially affected area downstream of the northern highwall drain to the north of the project site boundary, particularly during and following significant rainfall and flow events. If necessary erosion protection and remediation works may be extended outside of the project site boundary.

The proponent will monitor the subsidence zone above the Southern Underground for erosion after flow events and erosion control measures will be installed in this area, if necessary.

11 References

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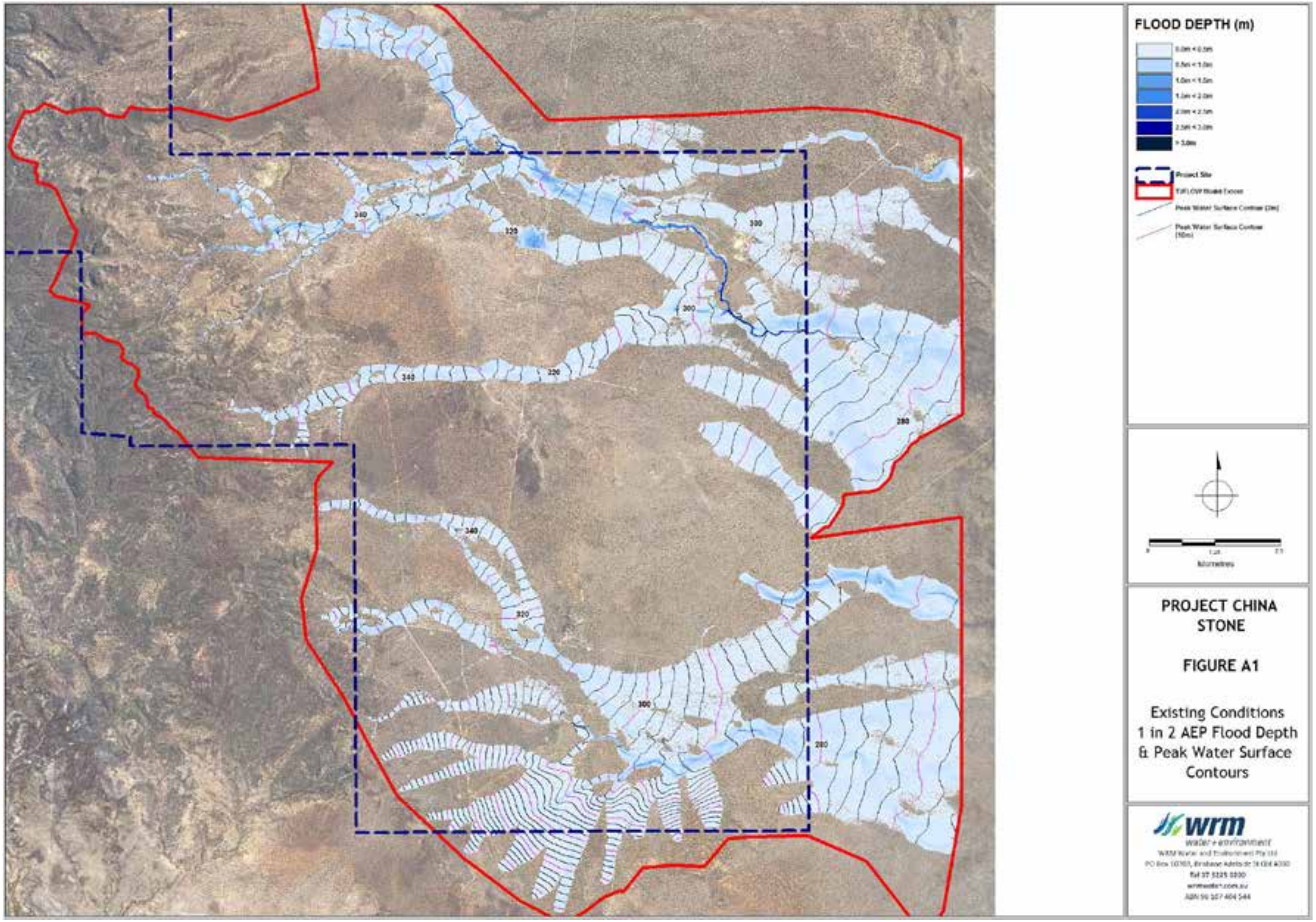
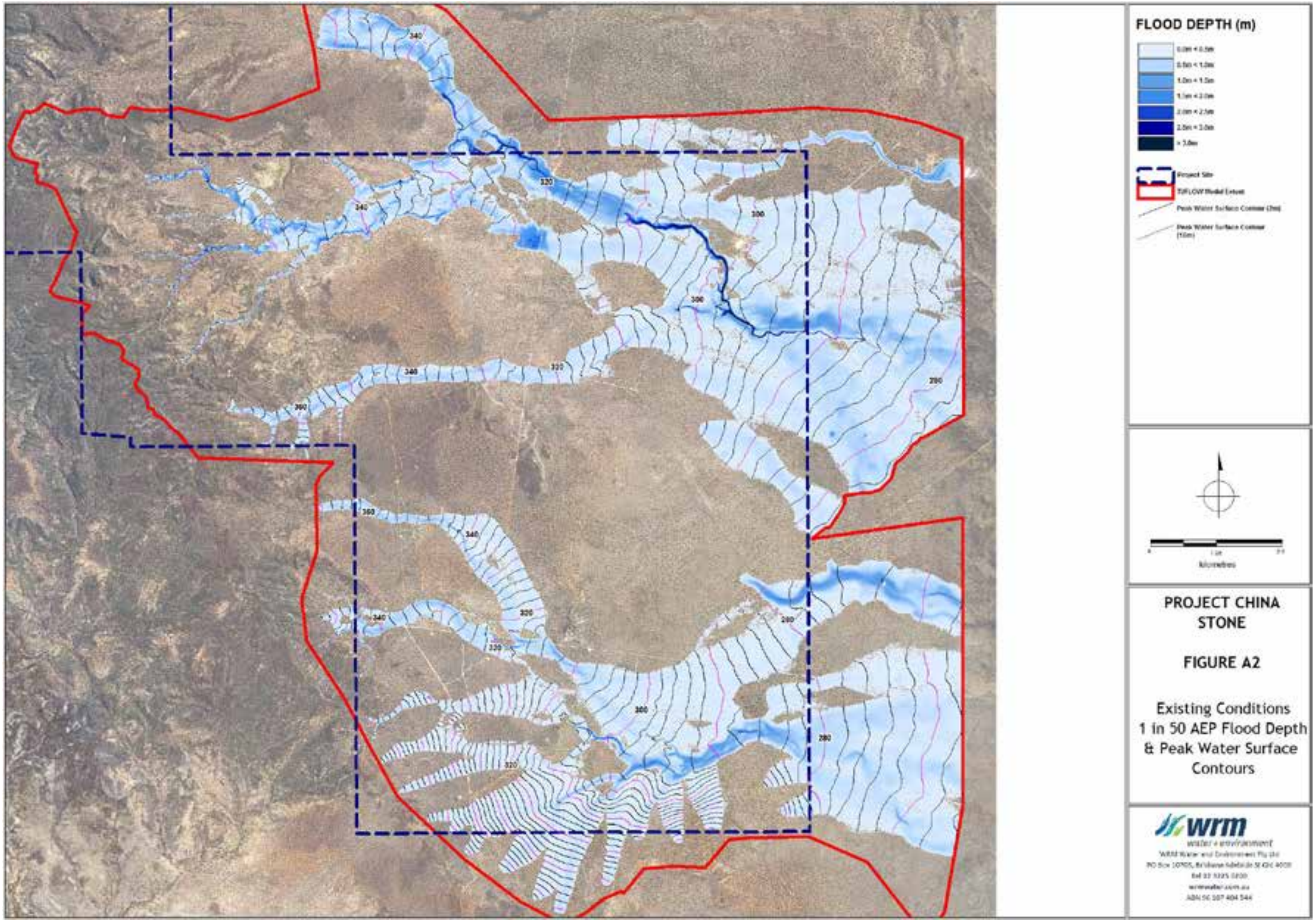
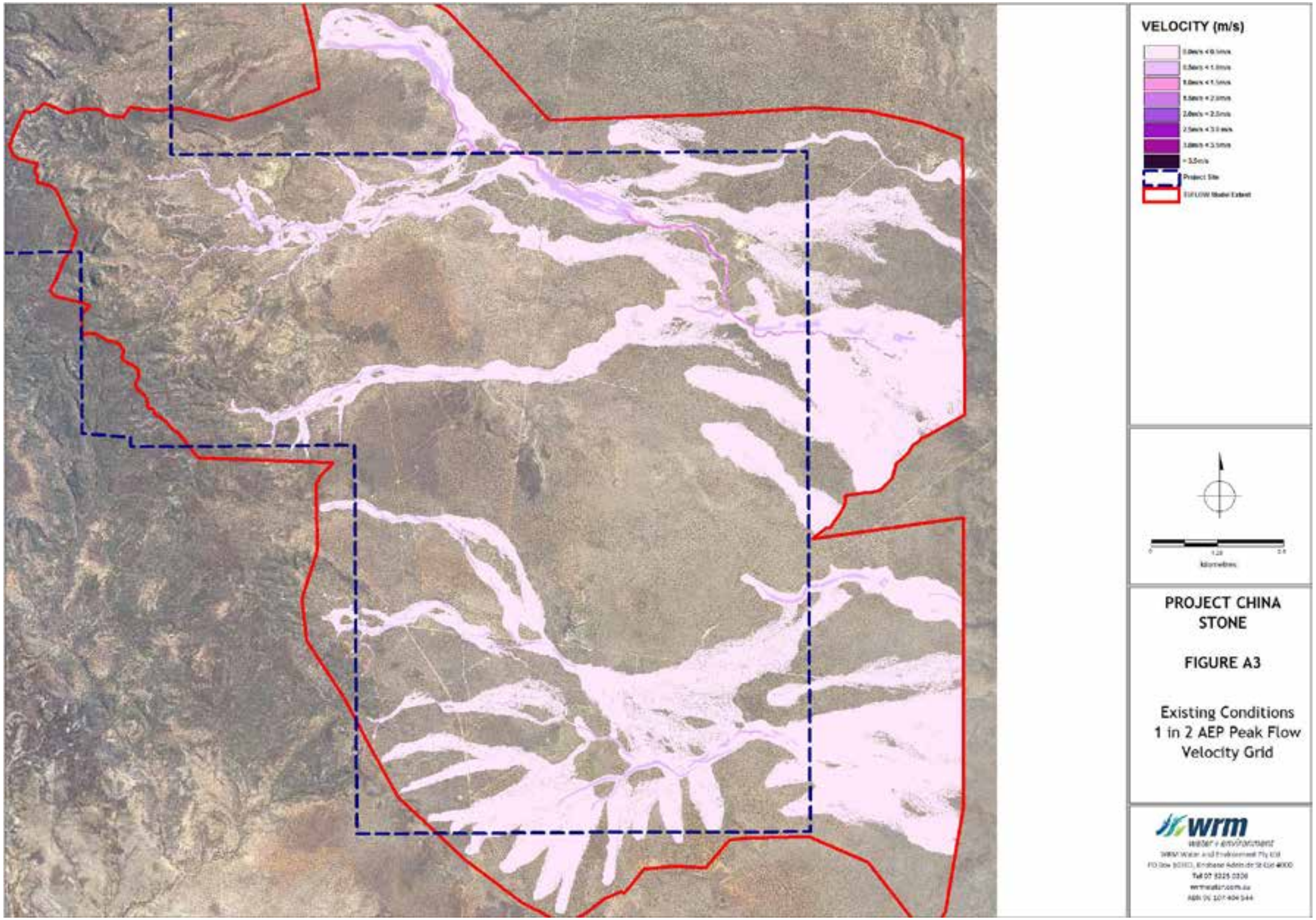
Appendix A Flood mapping


FLOOD DEPTH (m)
Project Site
PROJECT CHINA STONE
FIGURE A1
Existing Conditions
1 in 2 AEP Flood Depth
& Peak Water Surface
Contours
wrm

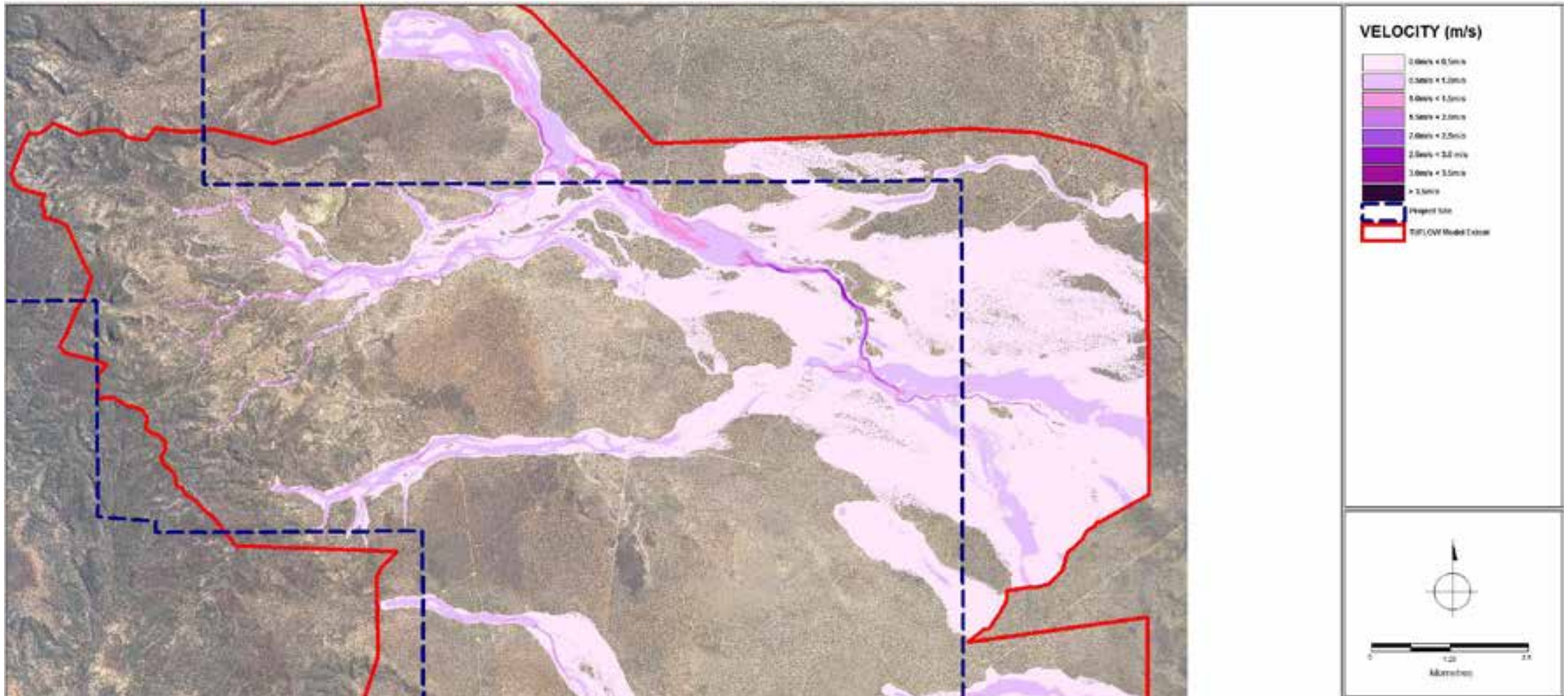

FLOOD DEPTH (m)
Project Site
340
320
300
280
PROJECT CHINA STONE
FIGURE A2
Existing Conditions
1 in 50 AEP Flood Depth
& Peak Water Surface
Contours
wrm



PROJECT CHINA STONE

FIGURE A3

Existing Conditions
1 in 2 AEP Peak Flow
Velocity Grid

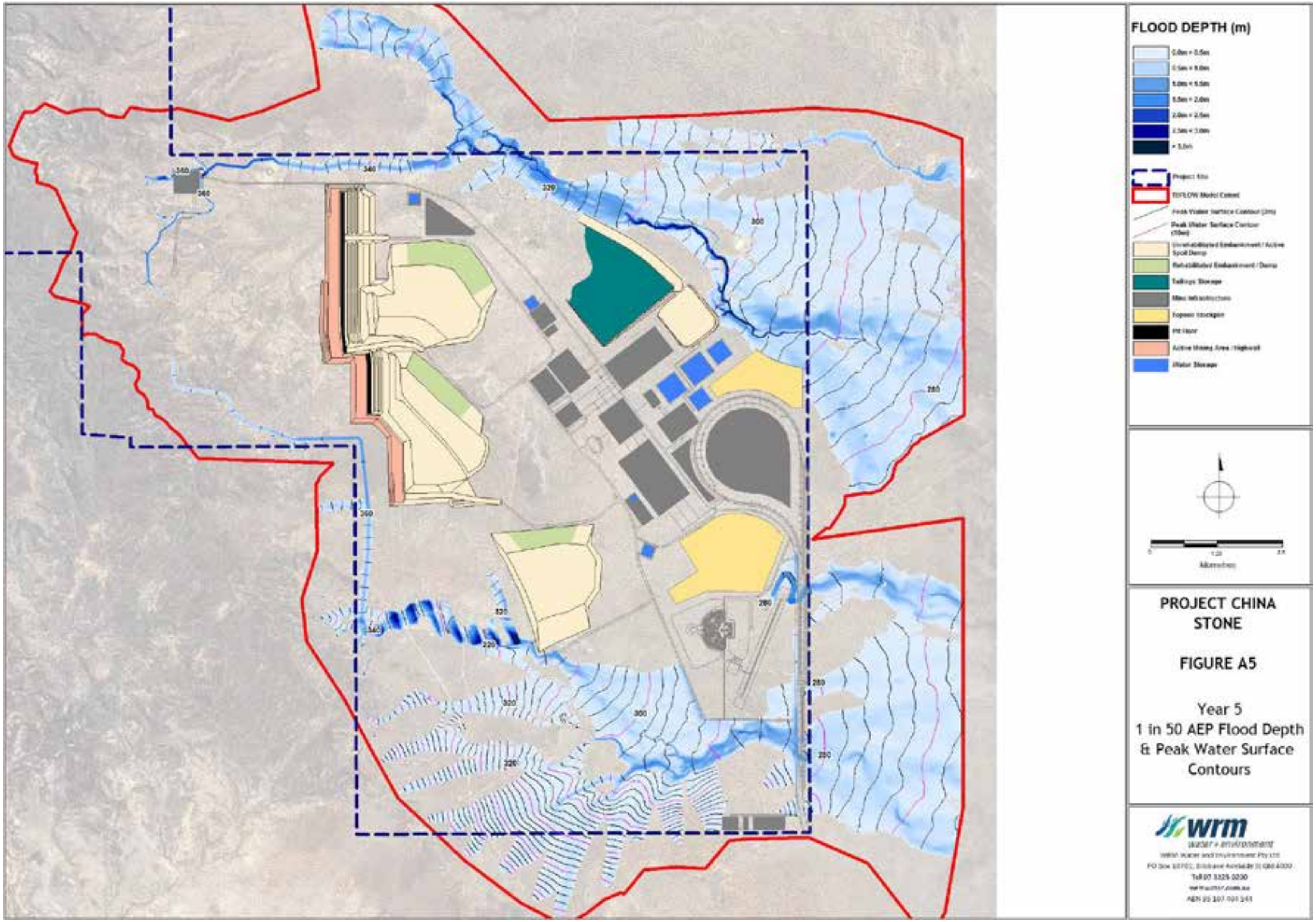


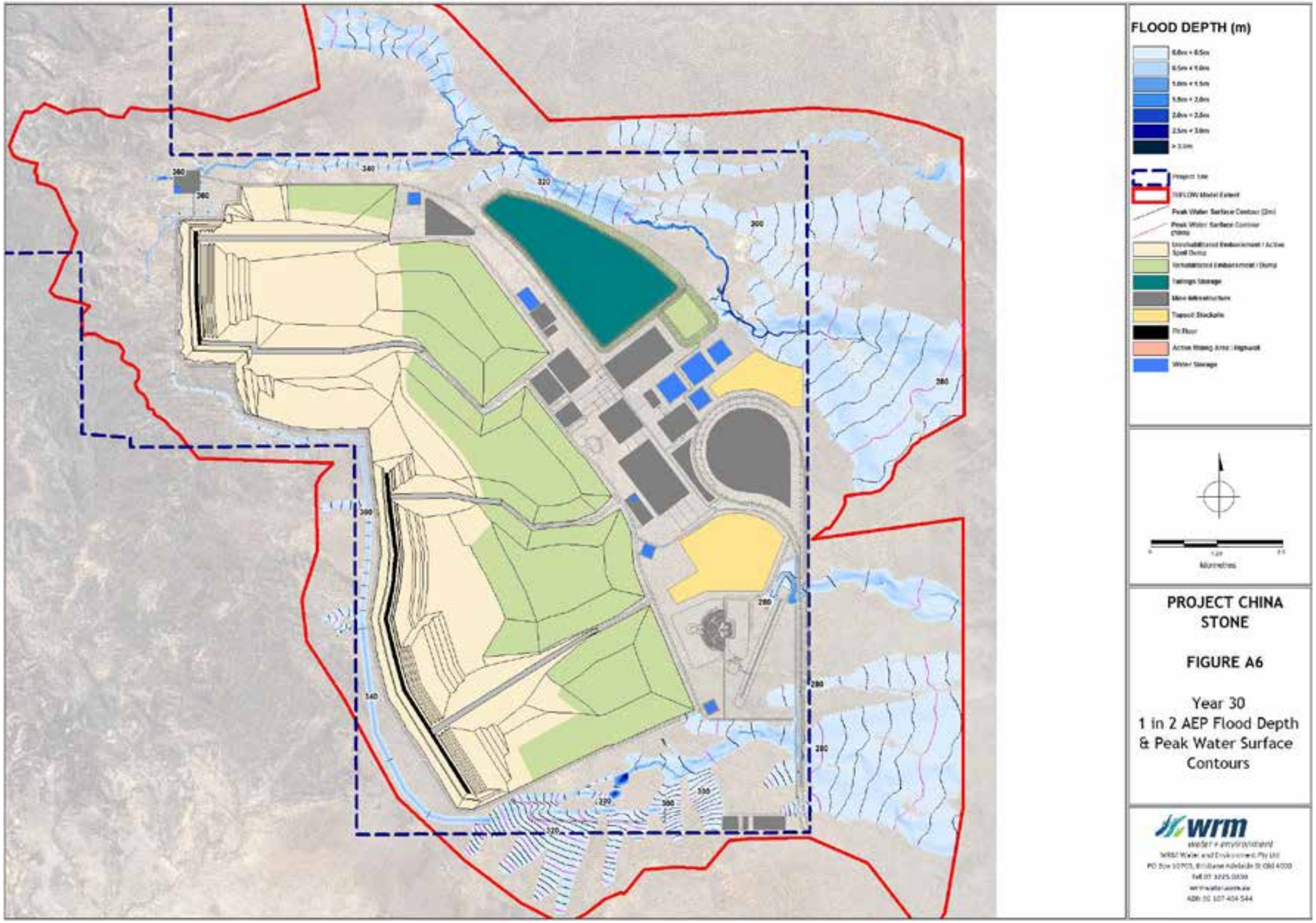
PROJECT CHINA STONE

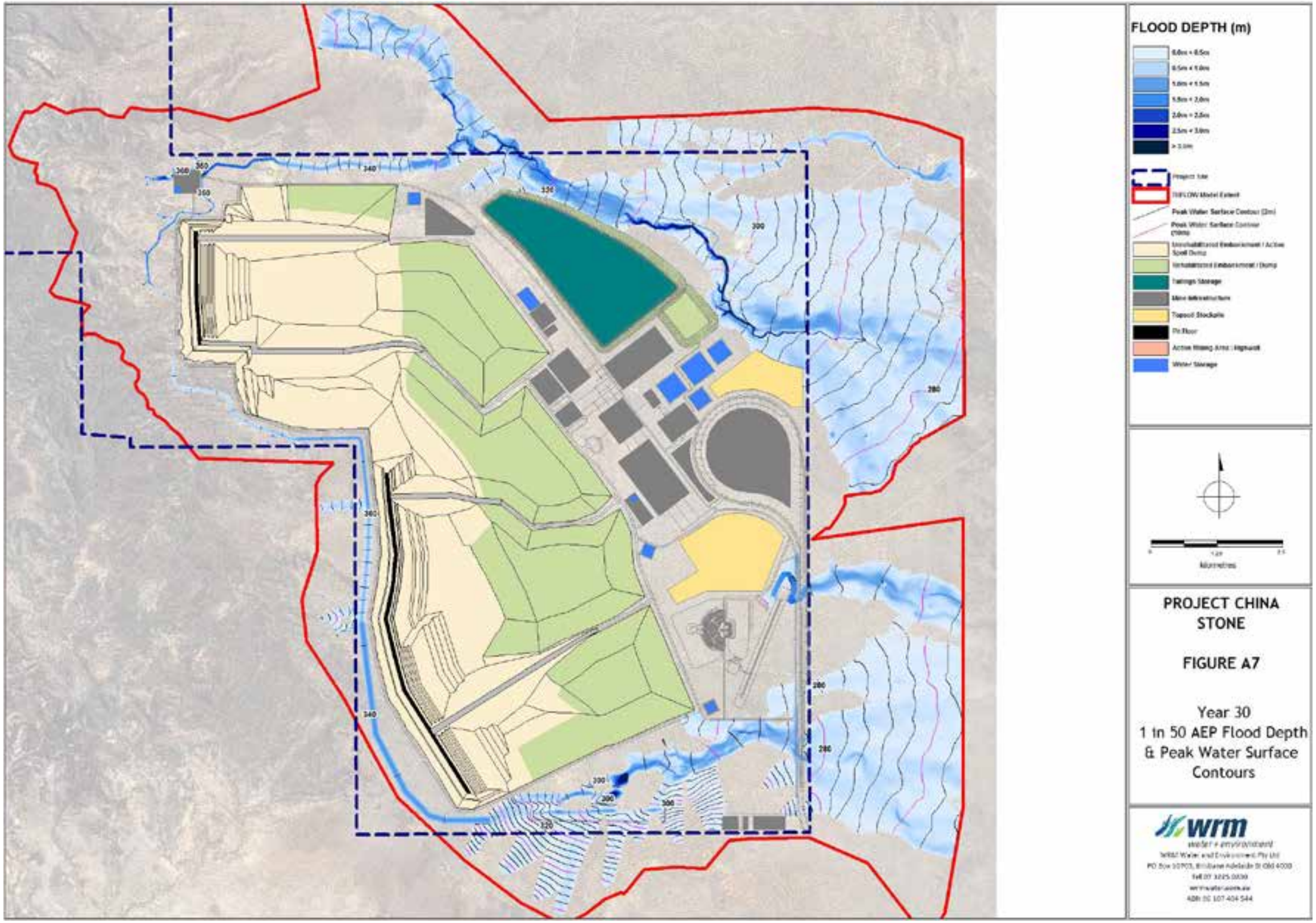
FIGURE A4

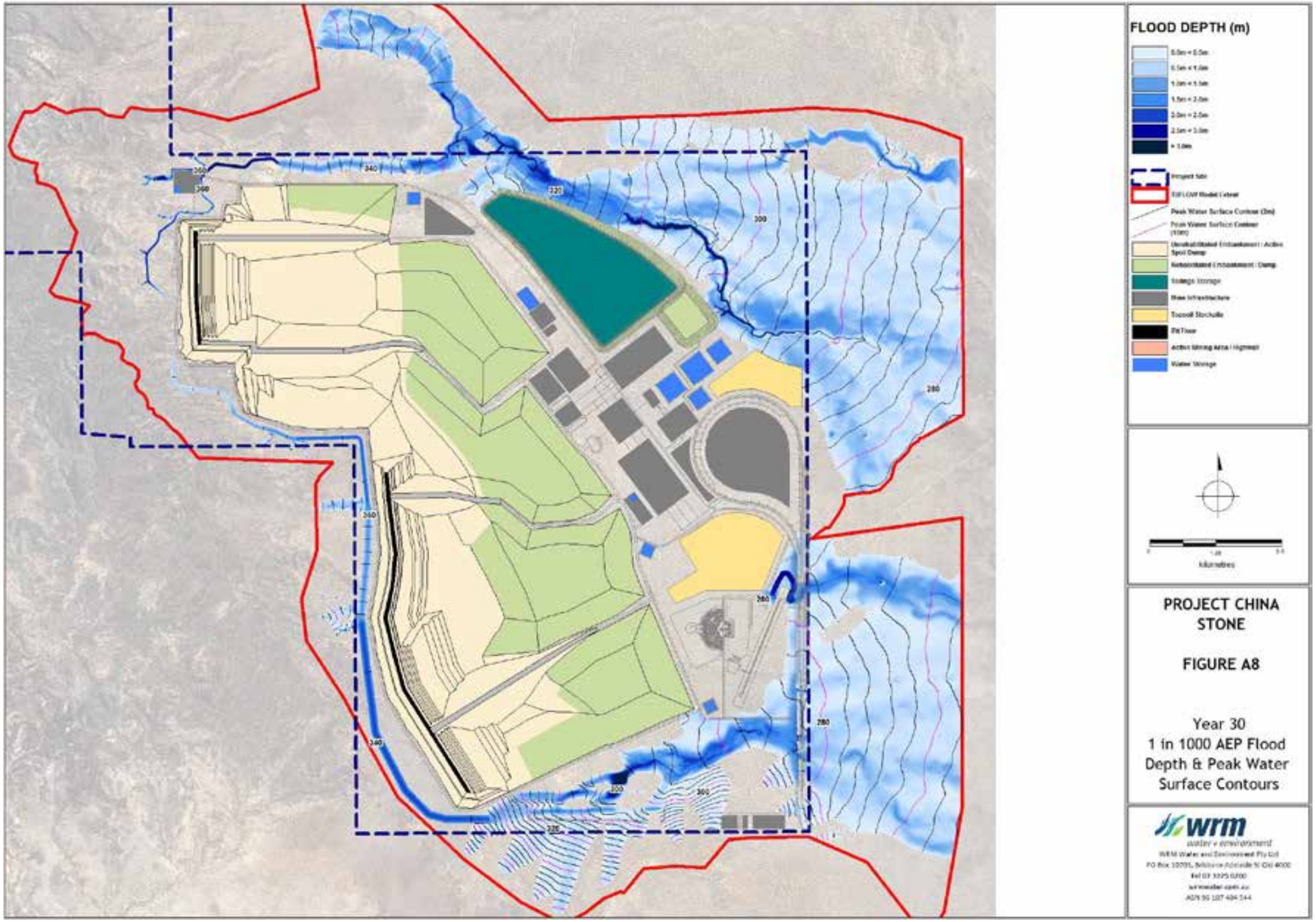
Existing Conditions
1 in 50 AEP Peak Flow
Velocity Grid

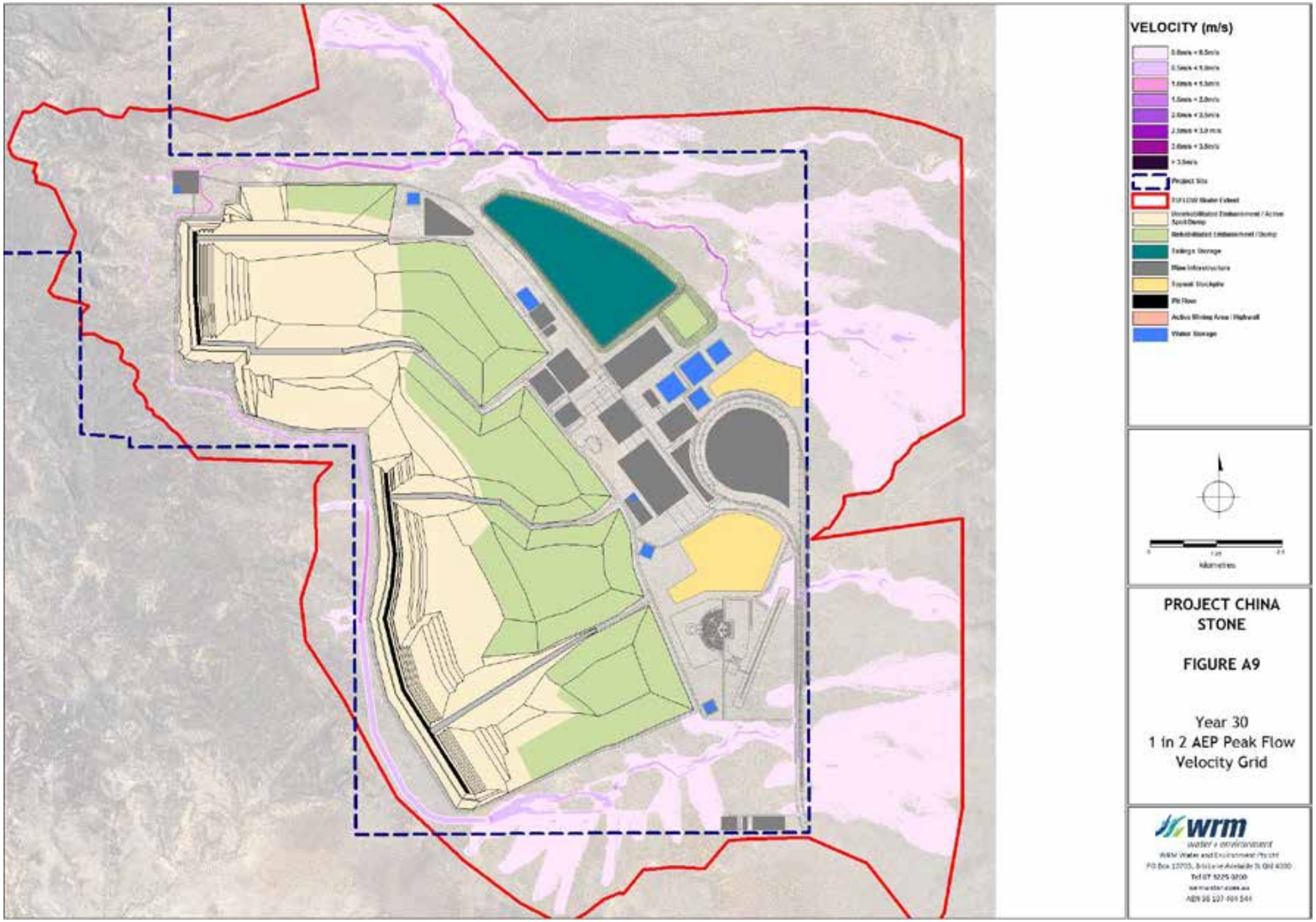
wrm water + environment

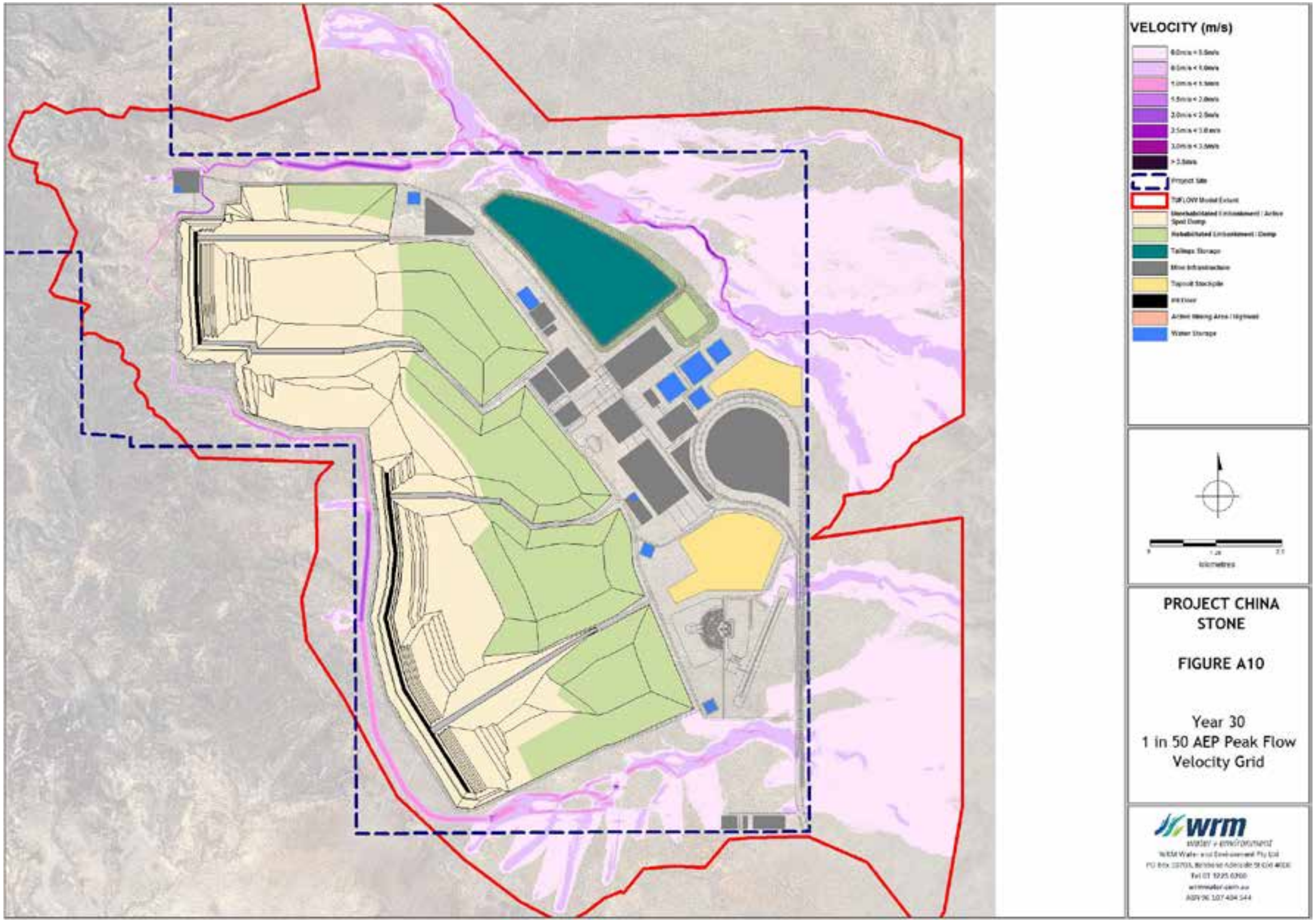

FLOOD DEPTH (m)
PROJECT CHINA STONE
FIGURE A5
Year 5
1 in 50 AEP Flood Depth & Peak Water Surface Contours
wrm
water + environment

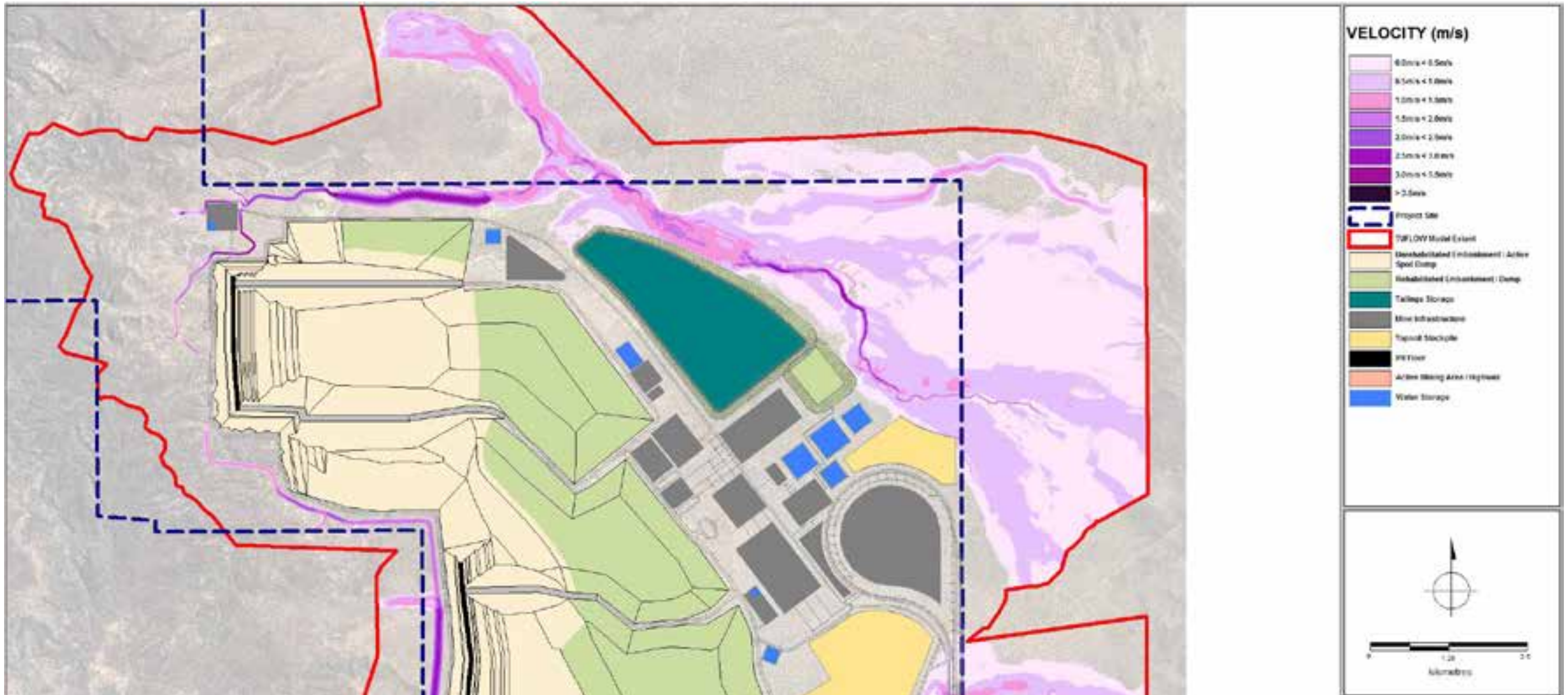

FLOOD DEPTH (m)
PROJECT CHINA STONE
FIGURE A6
Year 30
1 in 2 AEP Flood Depth & Peak Water Surface Contours
wrm


FLOOD DEPTH (m)
PROJECT CHINA STONE
FIGURE A7
Year 30
1 in 50 AEP Flood Depth & Peak Water Surface Contours
wrm


FLOOD DEPTH (m)
Peak Water Surface Contour (2m)
Topsoil Stockpile
PROJECT CHINA STONE
FIGURE A8
Year 30
1 in 1000 AEP Flood Depth & Peak Water Surface Contours
wrm
water + environment


VELOCITY (m/s)
Project Site
Pit Floor
Water Storage
Kilometres
PROJECT CHINA STONE
FIGURE A9
Year 30
1 in 2 AEP Peak Flow
Velocity Grid
wrm
water + environment


VELOCITY (m/s)
Project Site
Tailings Storage
Mine Infrastructure
Topsoil Stockpile
Water Storage
PROJECT CHINA STONE
FIGURE A10
Year 30
1 in 50 AEP Peak Flow
Velocity Grid
wrm
water + environment

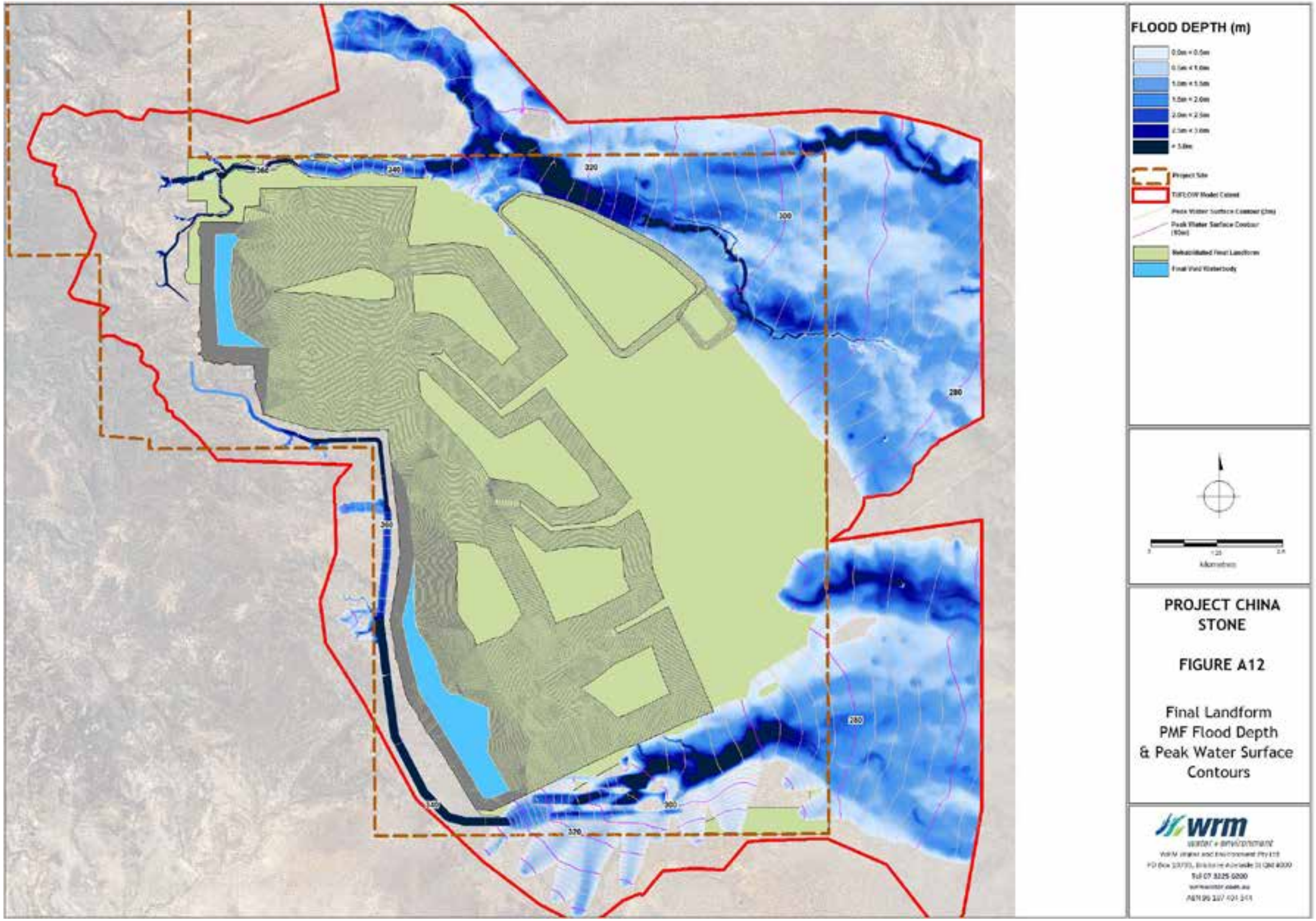


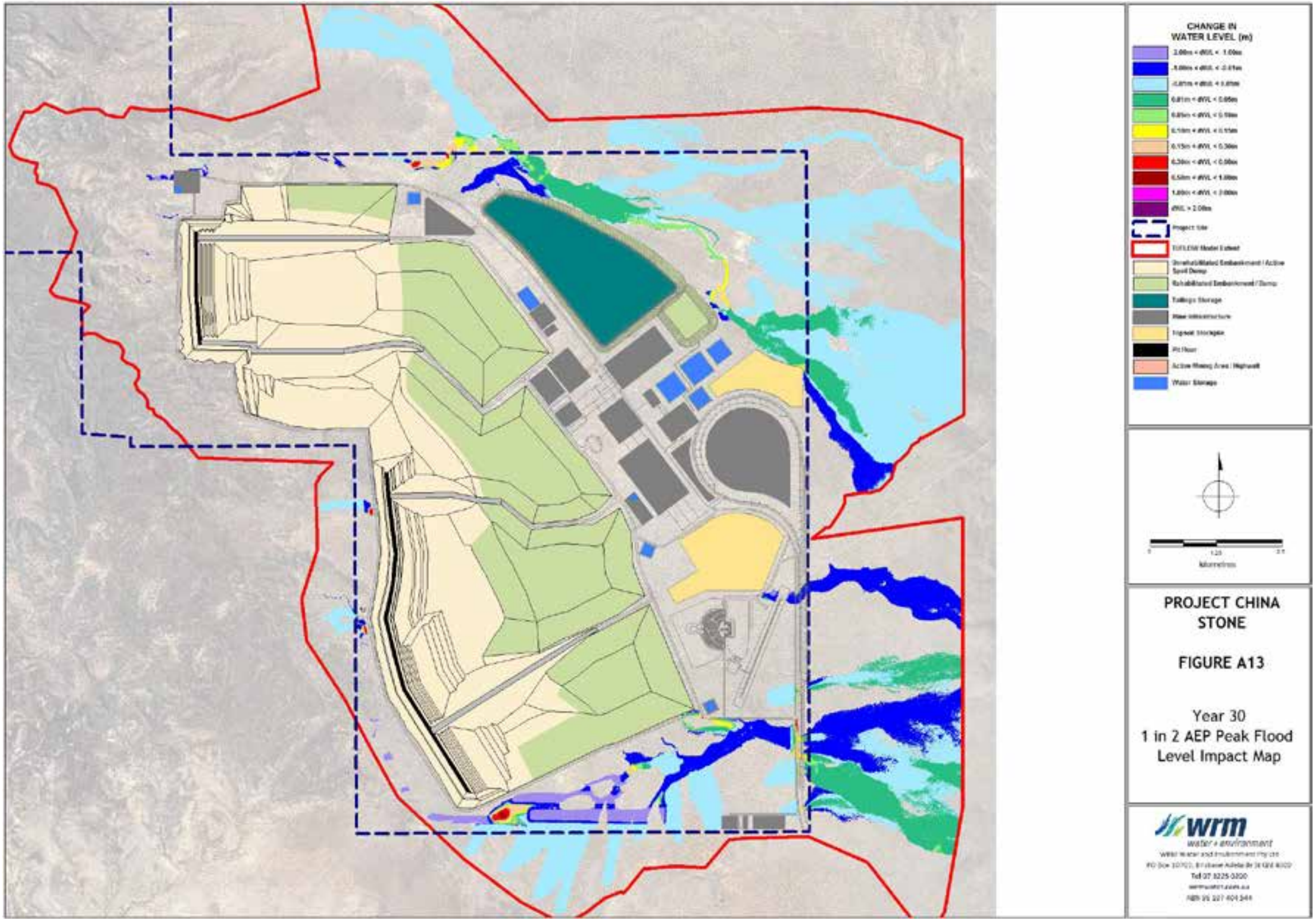
PROJECT CHINA STONE

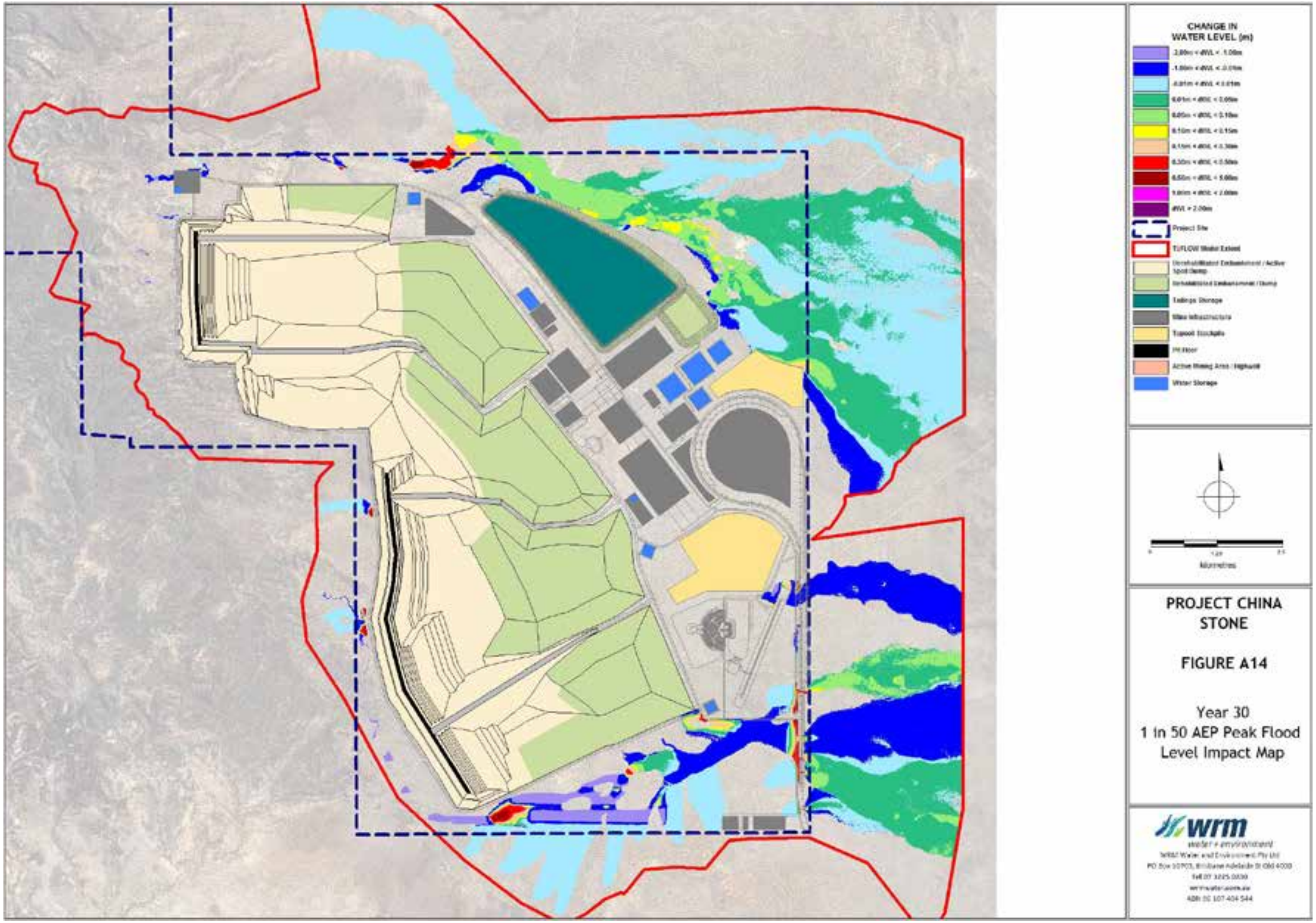
FIGURE A11

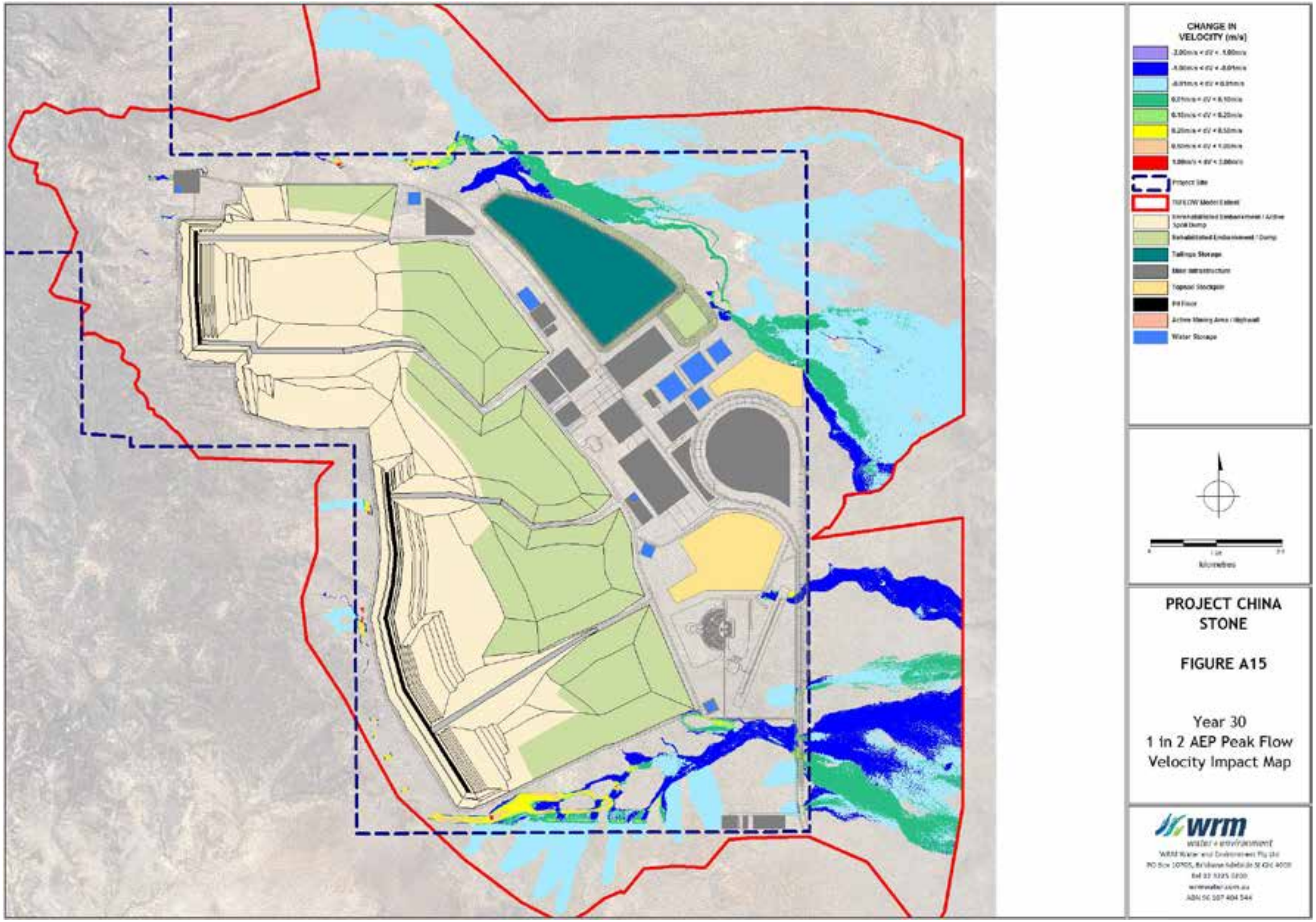
Year 30
1 In 1000 AEP Peak Flow
Velocity Grid

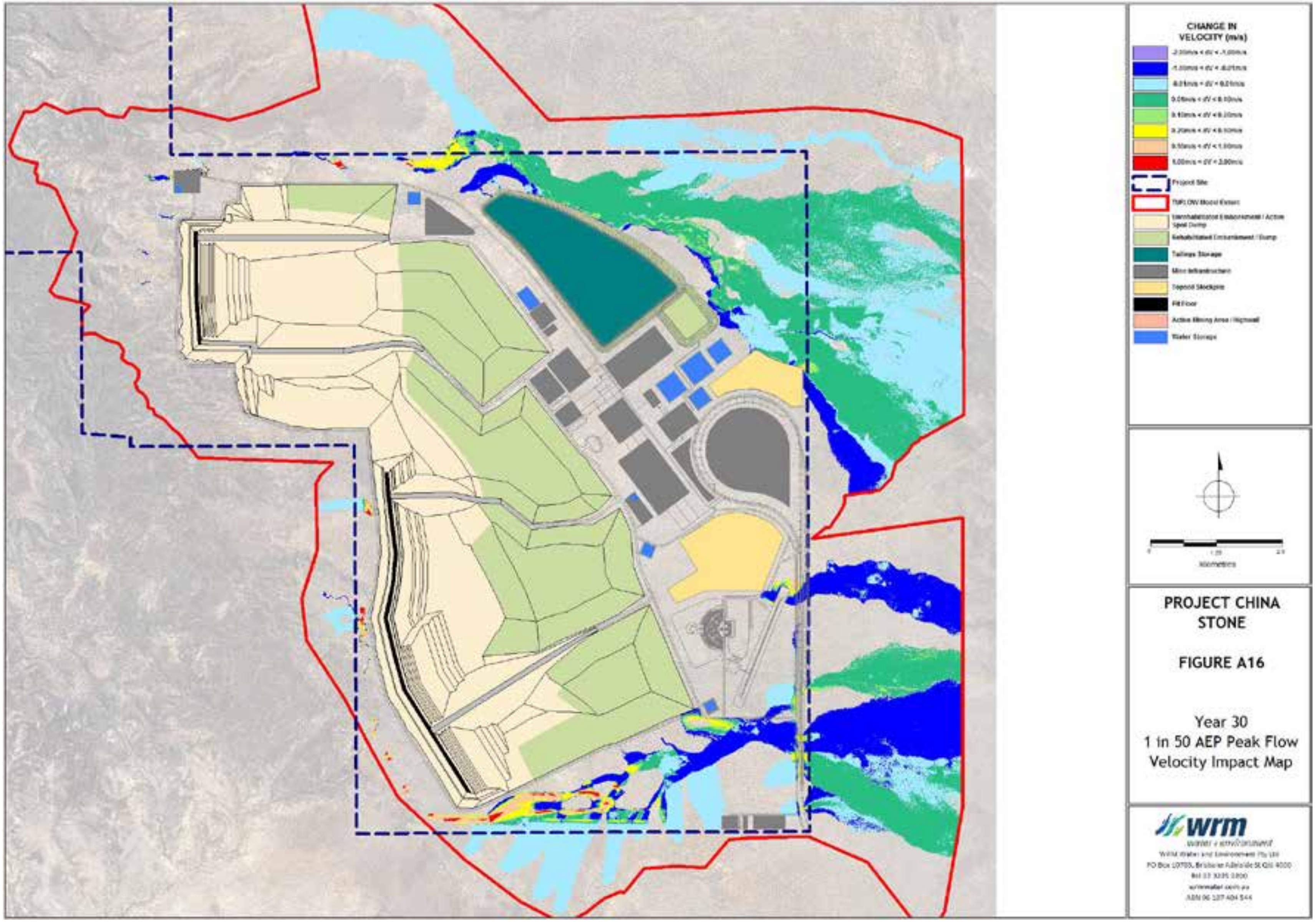
wrm water + environment

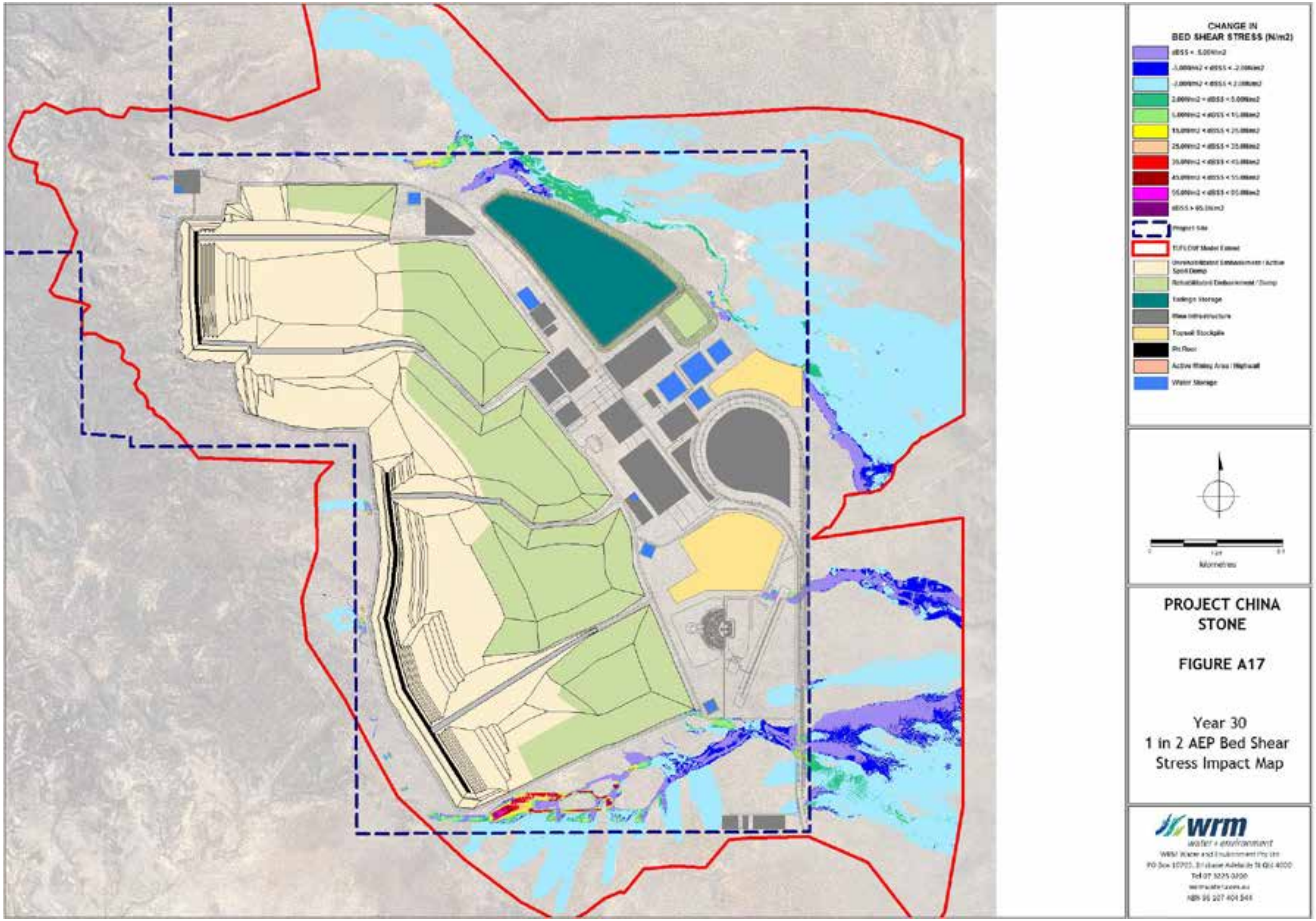

FLOOD DEPTH (m)
PROJECT CHINA STONE
FIGURE A12
Final Landform
PMF Flood Depth
& Peak Water Surface
Contours
wrm

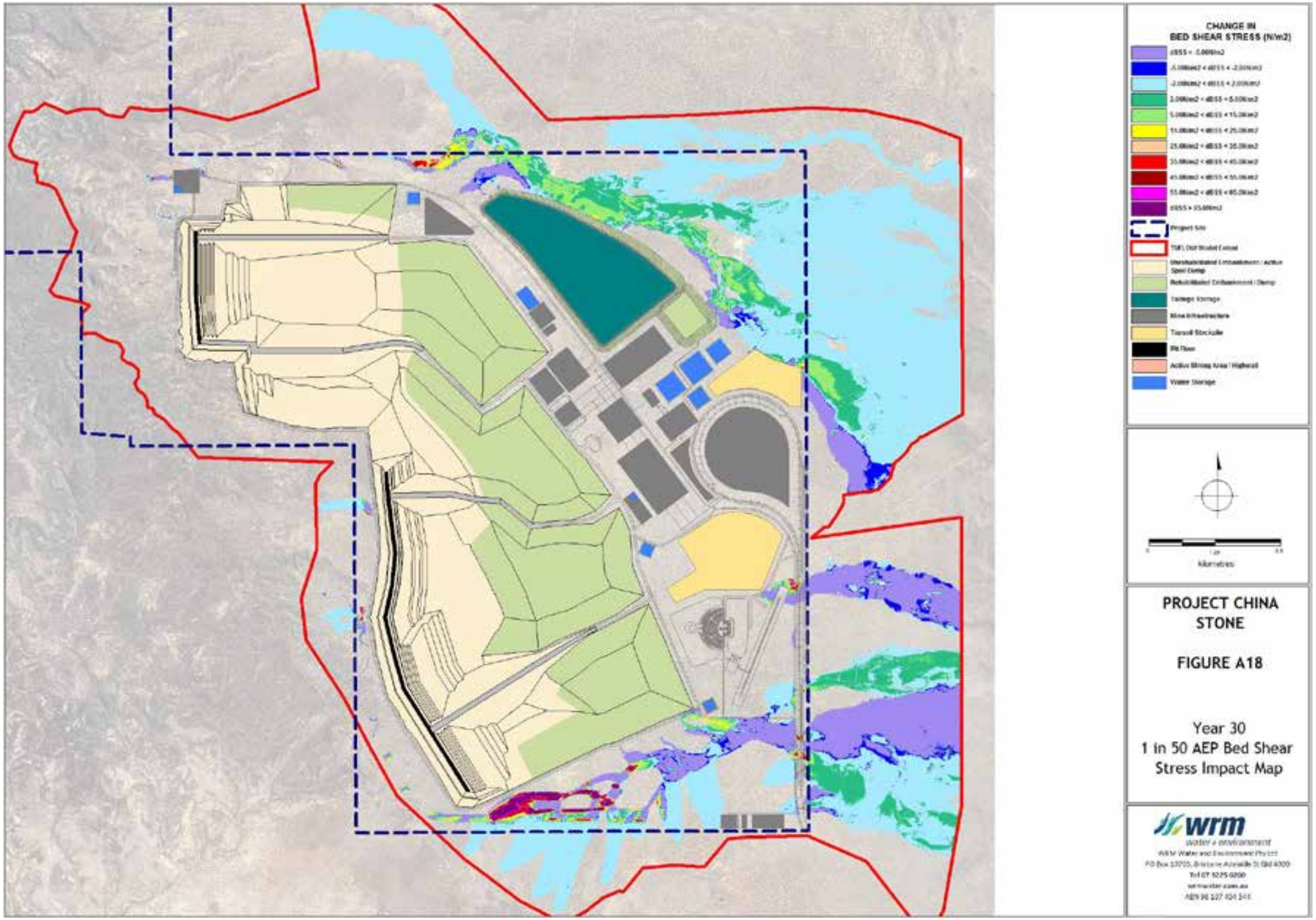

CHANGE IN
WATER LEVEL (m)
PROJECT CHINA
STONE
FIGURE A13
Year 30
1 in 2 AEP Peak Flood
Level Impact Map
wrm
water + environment

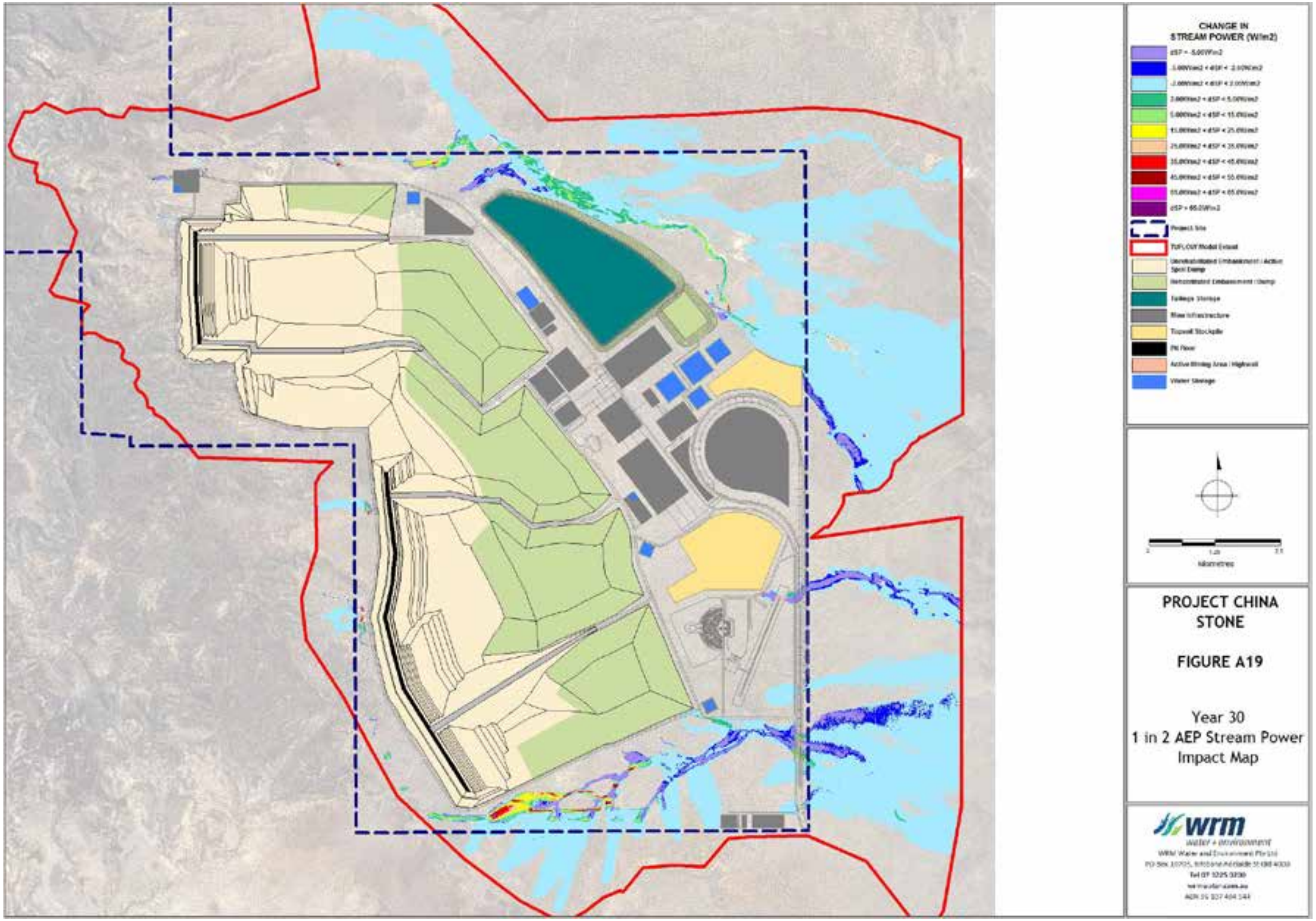

CHANGE IN WATER LEVEL (m)
Project Site
TUFLOW Model Extent
Tailings Storage
Water Storage
PROJECT CHINA STONE
FIGURE A14
Year 30
1 in 50 AEP Peak Flood Level Impact Map
wrm
water + environment

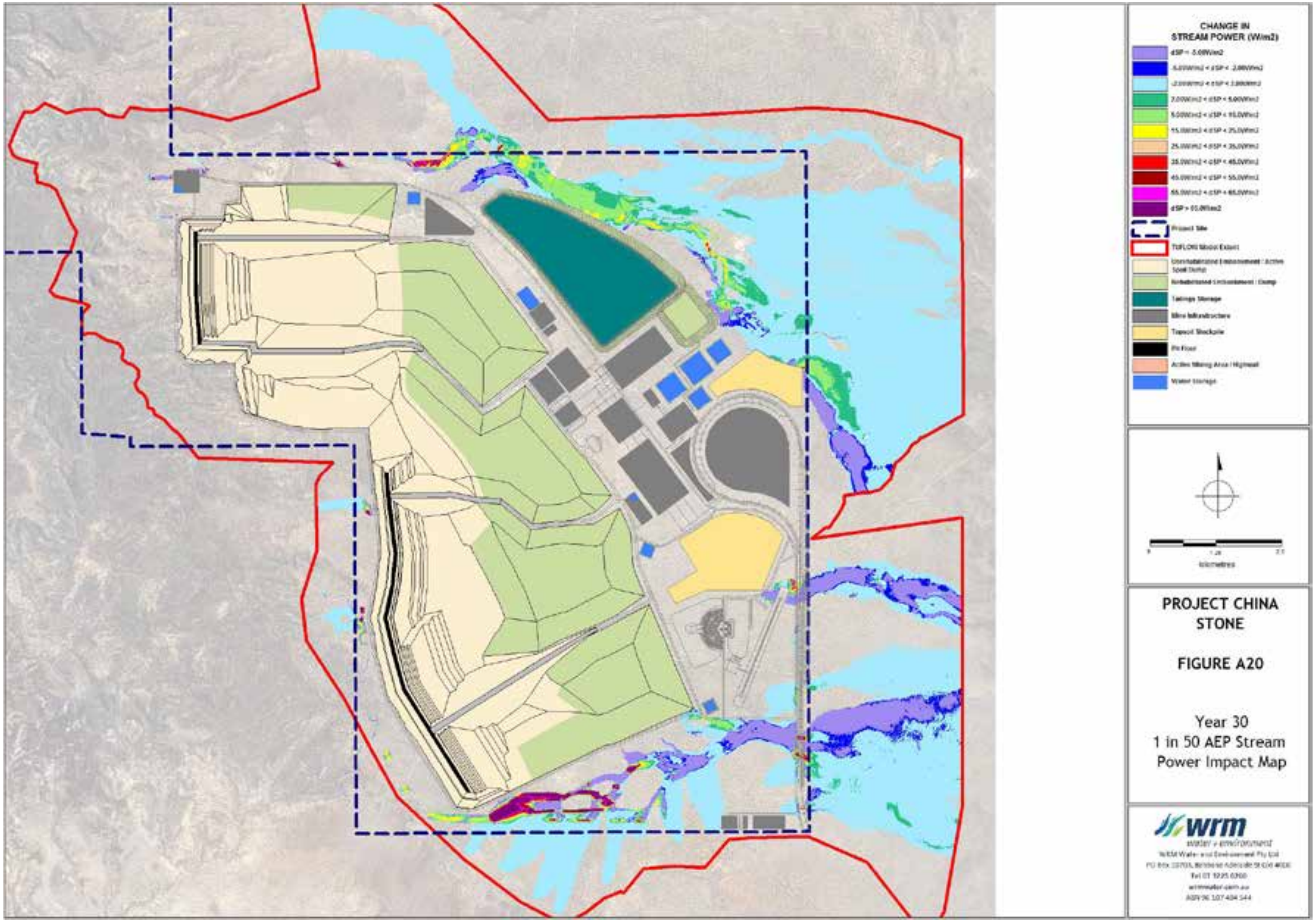

CHANGE IN
VELOCITY (m/s)
Project Site
Tailings Storage
Water Storage
PROJECT CHINA
STONE
FIGURE A15
Year 30
1 in 2 AEP Peak Flow
Velocity Impact Map
wrm


CHANGE IN
VELOCITY (m/s)
Project Site
Tailings Storage
Water Storage
PROJECT CHINA STONE
FIGURE A16
Year 30
1 in 50 AEP Peak Flow
Velocity Impact Map
wrm


CHANGE IN
BED SHEAR STRESS (N/m2)
Topsoil Stockpile
Pit Floor
Water Storage
Kilometres
PROJECT CHINA STONE
FIGURE A17
Year 30
1 in 2 AEP Bed Shear Stress Impact Map
wrm
water + environment


CHANGE IN
BED SHEAR STRESS (N/m2)
Project Site
Tailings Storage
Topsoil Stockpile
Water Storage
Kilometres
PROJECT CHINA STONE
FIGURE A18
Year 30
1 in 50 AEP Bed Shear Stress Impact Map
wrm
water + environment


CHANGE IN
STREAM POWER (W/m2)
Project Site
TUFLOW Model Extent
Tailings Storage
Mine Infrastructure
Topsoil Stockpile
Pit Floor
Active Mining Area / Highwall
Water Storage
PROJECT CHINA STONE
FIGURE A19
Year 30
1 in 2 AEP Stream Power Impact Map
wrm
water + environment


CHANGE IN
STREAM POWER (W/m2)
PROJECT CHINA
STONE
FIGURE A20
Year 30
1 in 50 AEP Stream
Power Impact Map
wrm