SOUTH GALILEE COAL PROJECT SURFACE WATER ASSESSMENT

AMCI Investments Pty Ltd
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For and on behalf of
WRM Water & Environment Pty Ltd

Michael Batchelor
Principal Engineer

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1 INTRODUCTION

1.1 GENERAL

The South Galilee Coal Project (SGCP) is a proposed coal mine to be located approximately 12km southwest of Alpha, as shown in Figure 1.1. Alpha is situated approximately 170km west of Emerald and 450km west of Rockhampton in Central Queensland.

The project is to be established by the Proponent, AMCI (Alpha) Pty Ltd (AMCI) and Alpha Coal Pty Ltd (Bandanna), and will produce up to 17 million tons per annum (Mtpa) of high volatile, low sulphur steaming coal for export to international markets. The mine will comprise both open cut and underground components.

Construction activities are expected to commence in 2013, following granting of the required Environmental Authority. Operations are expected to commence in 2015 with a scheduled mine life of 33 years until 2047. However, it is possible that there will be sufficient economic coal reserves to extend the operational life of the Project beyond the currently planned 33 years.

The Project's development timeframe will be dependent on the completion and access to third party rail and port infrastructure and the availability of electricity and water supplies. As a result, some variation to the proposed development timeframe may occur.

1.2 SCOPE OF THIS STUDY

This report, prepared by WRM Water & Environment Pty Ltd, presents the methodology and results of the surface water investigations undertaken to assess the potential impacts of the proposed SGCP on local surface hydrology and the existing mine water management system. The report provides the basis for the surface water component of the Project Environmental Impact Study (EIS) for the SGCP. The key components of the surface water study include the following:

- A description of the existing environment including local and regional catchments, drainage lines, geomorphology, meteorology, water features of interest, water quality and downstream water users;
- An assessment of potential constraints with respect to flooding in Tallarenha Creek, Sapling Creek and Dead Horse Creek which drain through the SGCP, including;
    - An assessment of the impact of surface subsidence due to underground mining on flood flow paths, flood levels, flood extents, flood frequency, stream stability and erosion;

  - An assessment of the impact of the proposed SGCP on the depth, extent and frequency of flooding, and the impact on stream stability and erosion in waterways found within the mining lease; and
  - An assessment of the potential impacts of the proposed mining operations on downstream surface water quality and quantity.
- The identification and development of appropriate safeguards and surface water control measures to minimise or mitigate the potential impacts of current and proposed (different stages of) mining in accordance with the objectives of local and regional catchment management and water resource plans;
- The development of an integrated water management plan, including an annual mine site water balance, for the SGCP mining operations;
- The design of a surface water monitoring program to monitor the impact of mining activities on the receiving environment;
- An assessment of the potential impact of the proposed infrastructure corridor on the nearby surface water features; and
- An assessment of the long-term (hydrologic) storage behaviour of the final voids.

1.3 REPORT STRUCTURE

This report is structured as follows:

- Section 2 describes the existing environment with respect to surface water resources;
- Section 3 describes the applicable legislation relating to surface water for the site;
- Section 4 outlines the potential impacts of the proposed SGCP on surface water resources and identifies the proposed measures to mitigate the impacts;
- Section 5 describes the proposed SGCP surface water monitoring system;
- Section 6 is a list of references;
- Appendix A details the site water balance modelling undertaken to assess the performance of the site water management system, and changes in receiving water flows due to the proposed SGCP;
- Appendix B describes the catchments contributing runoff to each of the proposed site water management structures, including description of the land use;
- Appendix C describes the water balance modelling carried out for assessment of the final void storage behaviour;
- Appendix D details the hydrologic and hydraulic models used to assess the impact of the proposed project on flood behaviour in the streams crossing the SGCP, including the infrastructure corridor. It also describes the basis of the conceptual design of a diversion of Sapling Creek;
- Appendix E provides mapping of the extent of flooding under both existing conditions and following the proposed mine development.

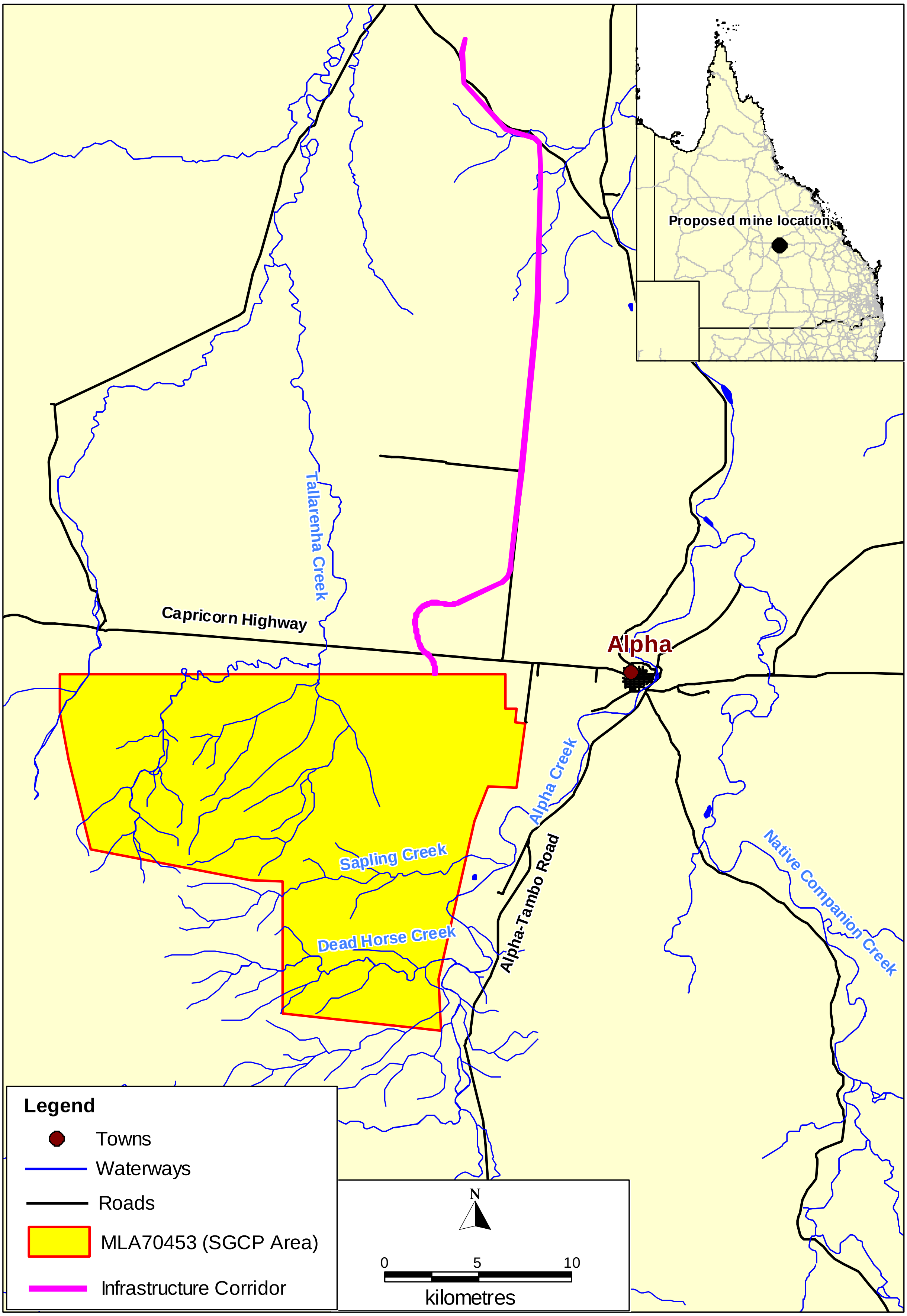


Figure 1.1 South Galilee Coal Project Locality Plan

2 EXISTING ENVIRONMENT

2.1 REGIONAL DRAINAGE BASIN CHARACTERISTICS

As shown in Figure 2.1, the proposed project is located in the upper catchment of the Burdekin River Basin. With a catchment area of approximately 130,500km^2, the Burdekin River Basin is one of Queensland's largest basins. The river flows into the sea near the Great Barrier Reef.

Land use in the Burdekin River Basin varies, ranging from beef cattle production and mining in the inland areas, to irrigated sugarcane and crop cultivation on the coastal delta and floodplains including the Burdekin River Irrigation Area (BRIA). The BRIA is supplied from Lake Dalrymple, Queensland's largest reservoir, which has a capacity of 1,860GL and is formed by Burdekin Falls Dam.

The proposed SGCP Mine Lease Application (MLA) crosses the upper tributaries of Sandy Creek and Native Companion Creek, which are both tributaries of the Belyando River. The Belyando River is part of the Suttor River sub-basin, which has a catchment area of approximately 52,550km^2. The proposed SGCP MLA covers an area of approximately 310 km^2, which is approximately 0.6% and 0.2% of the Suttor River and Burdekin River catchments respectively.

The mine lease area is located approximately 350km upstream of the Burdekin Falls Dam and 510km upstream of the river mouth.

The Galilee Basin has a complex geology and geomorphology and soils found in the region include shallow loam, duplex and deep alluvium soils. The soils of the SGCP tenements typically consist of loams interspersed with pockets of cracking clay, Gilgai clay soils and sandy duplex.

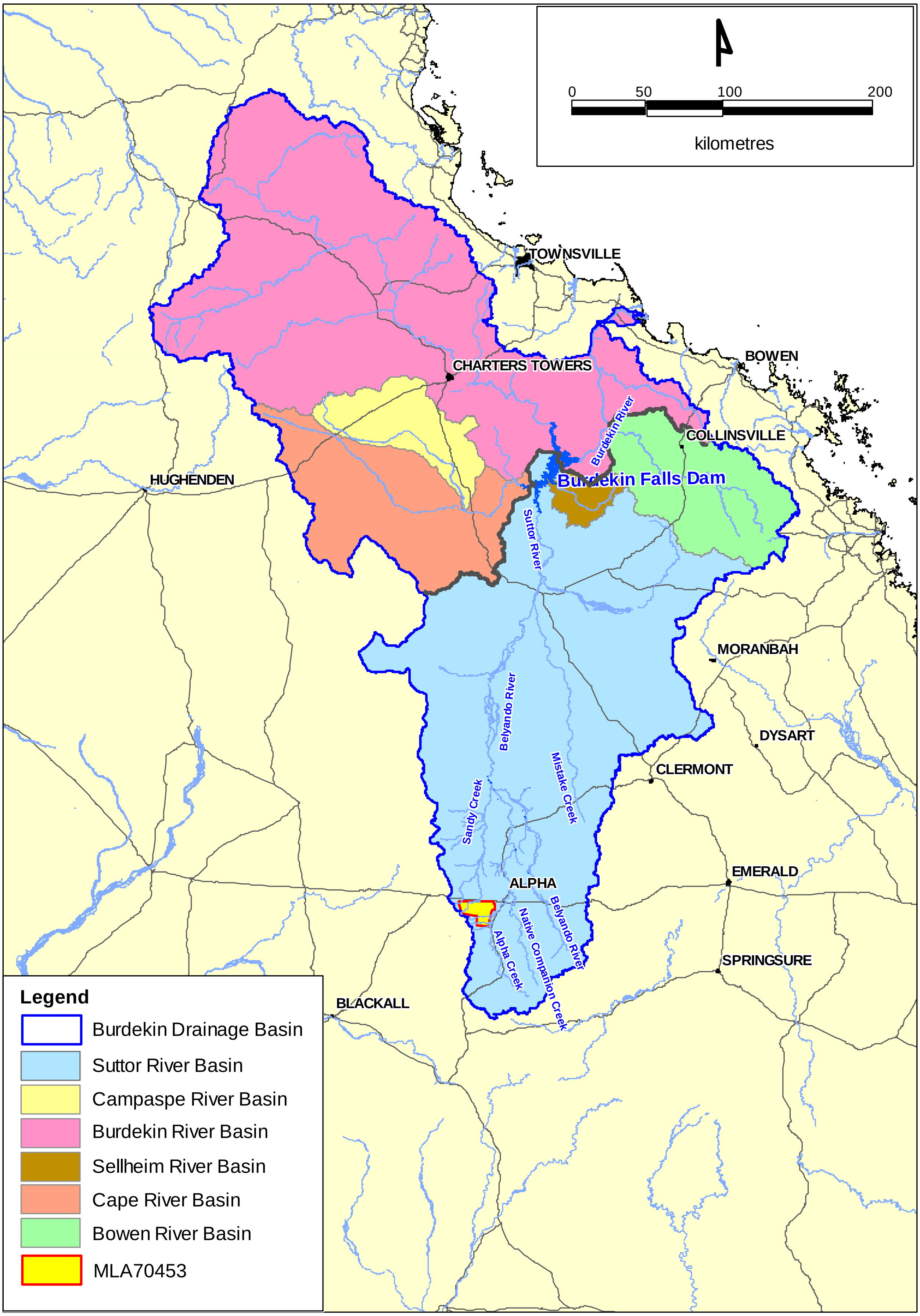


Figure 2.1 Burdekin River Basin

2.2 LOCAL CATCHMENTS

2.2.1 MLA Area

The proposed SGCP mining operations cross the catchments of Tallarenha Creek in the north, and Sapling Creek and Dead Horse Creek in the south. Figure 2.2 shows the location of these three (3) catchments in relation to the proposed SGCP operation.

As shown in Figure 2.2, Tallarenha Creek flows north to Lagoon Creek and then to Sandy Creek before joining the Belyando River some 120km downstream (north). Tallarenha Creek has a catchment area of approximately 209.5km^2, and falls from elevations of approximately 530m AHD at the catchment ridge to approximately 365m AHD at the Capricorn Highway crossing just downstream of the MLA boundary. Almost all of the Tallarenha Creek catchment to this location is within the MLA.

An unnamed north-flowing tributary of Tallarenha Creek flows west of the proposed CHPP area and through the northern section of the proposed open cut. This tributary crosses the northern MLA boundary and joins Tallarenha Creek 900m downstream of the boundary. This tributary has no well-defined bed and banks, and is fed by a number of smaller poorly defined tributaries which cross the proposed mining area in a generally north-easterly direction. The Tallarenha Creek channel itself is located well to the west and north of the proposed open cut, but would be undermined by the proposed underground operation.

Sapling Creek and Dead Horse Creek are east-flowing tributaries of Alpha Creek, which flows to the north through the township of Alpha before joining Native Companion Creek. Native Companion Creek also flows north before joining the Belyando River near the Sandy Creek confluence.

Sapling Creek and Dead Horse Creek have catchment areas of approximately 63.5km^2 and 65.5km^2 respectively. The upper catchments of these creeks are located to the south and southwest of the MLA, where ground surface elevations are up to 550m AHD. The lower catchment elevations, on the Alpha Creek floodplain are approximately 360m AHD.

Alpha Creek has a catchment of 2,429km^2 to the Dead Horse Creek confluence. A short (1.6km) reach of Alpha Creek flows through the MLA between Dead Horse Creek and Sapling Creek, however, no disturbance associated with the project would directly impact the bed or banks of Alpha Creek.

Sapling Creek will need to be diverted away from the proposed open cut operation, and parts of the upper catchment will be undermined by the underground mine. Dead Horse Creek will receive additional flows from the Sapling Creek diversion, but otherwise will not be affected directly.

Land use within the MLA catchments is predominantly cleared farmland used for low intensity cattle grazing. There is little to no development within the MLA. Figure 2.3 shows a photograph of the typical upper catchment areas, taken from location A in Figure 2.5.

There are a number of small farm dams in the vicinity of the project area. The most significant dams are shown in Figure 2.5. The largest (labelled 1), creates a large shallow lake in the area of the proposed open cut when full. The inundated area is approximately 800m wide by 1000m long, with a dam wall approximately 3.5m high. Upstream of the project area (at location B), a historic concrete structure intercepts runoff from a small tributary of Sapling Creek.

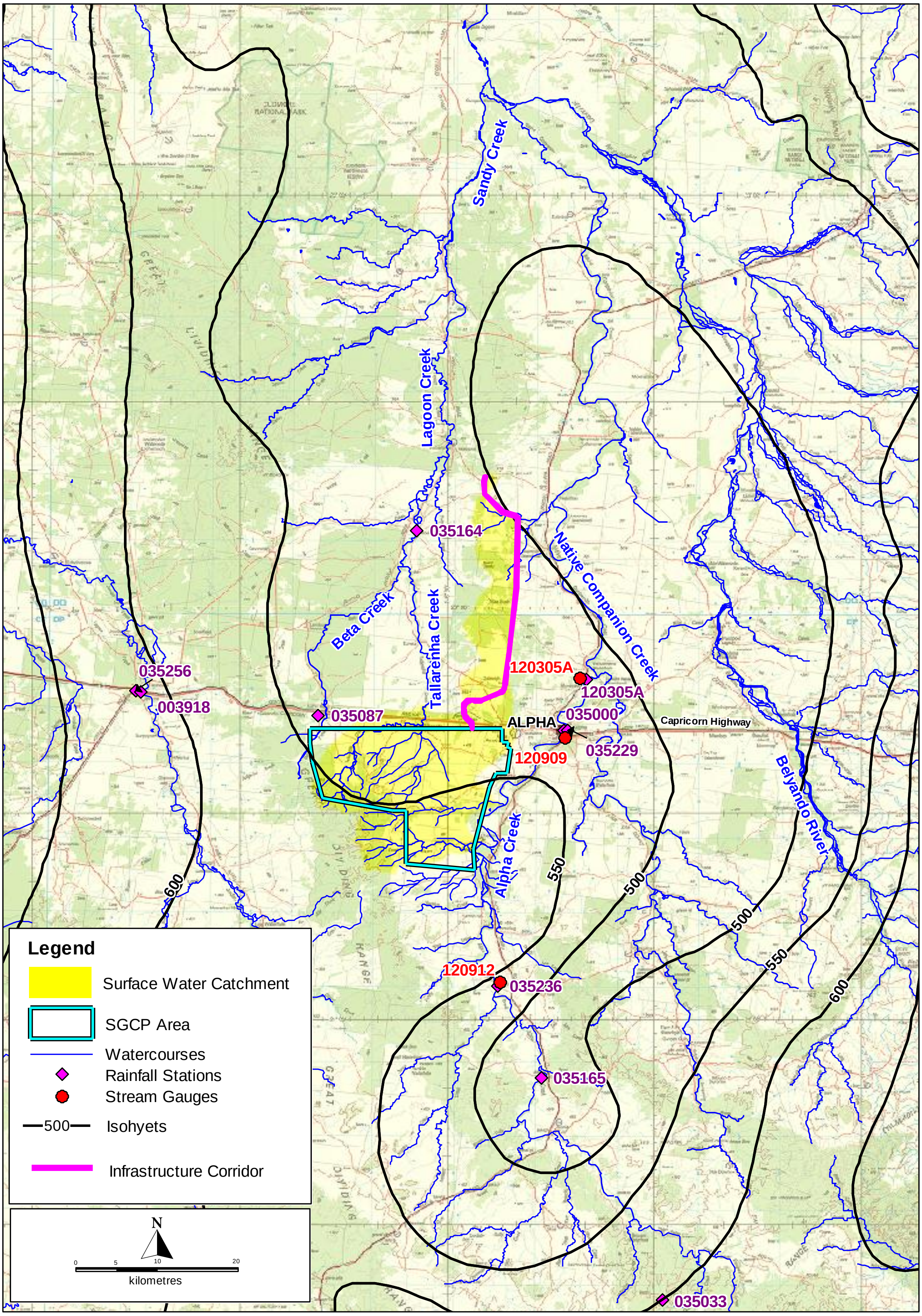


Figure 2.2 Mean Annual Rainfall Isohyets and Rainfall and Water Level Recording Stations

Figure 2.3 Upstream catchment of Sapling and Tallarenha Creeks – Location A

2.2.2 Infrastructure Corridor

As shown in Figure 2.4, the proposed infrastructure corridor runs in a generally north-south direction. With the exception of the northernmost 3km, which drains west to Saltbush Creek (a tributary of Lagoon Creek), the corridor drains generally north and north-east to Native Companion Creek. The channel of Native Companion Creek is generally located between 4km and 11km to the east of the infrastructure corridor. Figure 2.4 shows the approximate maximum flood extent of the Native Companion Creek floodplain mapped by the Queensland Reconstruction Authority (QRA). Figure 2.4 shows that the proposed corridor is outside the extent of the mapped floodplain.

Catchments crossing the proposed infrastructure corridor are shown in Figure 2.4 and these areas are summarised in Table 2.1. The catchment boundaries were defined using a combination of data from airborne laser scanning (ALS) prepared for the proponent, and data from the Shuttle Radar Topographic Mission (SRTM) where ALS data was unavailable. The local topography is relatively flat, and drainage paths from these catchments are not well defined. Channels with defined bed and banks are only discernible in the available terrain data for catchments 3, 4 and 6. Runoff from catchment 6, the largest crossing the corridor generally flows north.

Ground surface slopes in the vicinity of the catchment 12 crossing are less than 0.2%, and likely flow directions are therefore difficult to determine. The results of the 2-dimensional hydrodynamic flood modelling described in Section 2.8.2, show that runoff from these areas, flows in generally northerly direction along the corridor alignment.

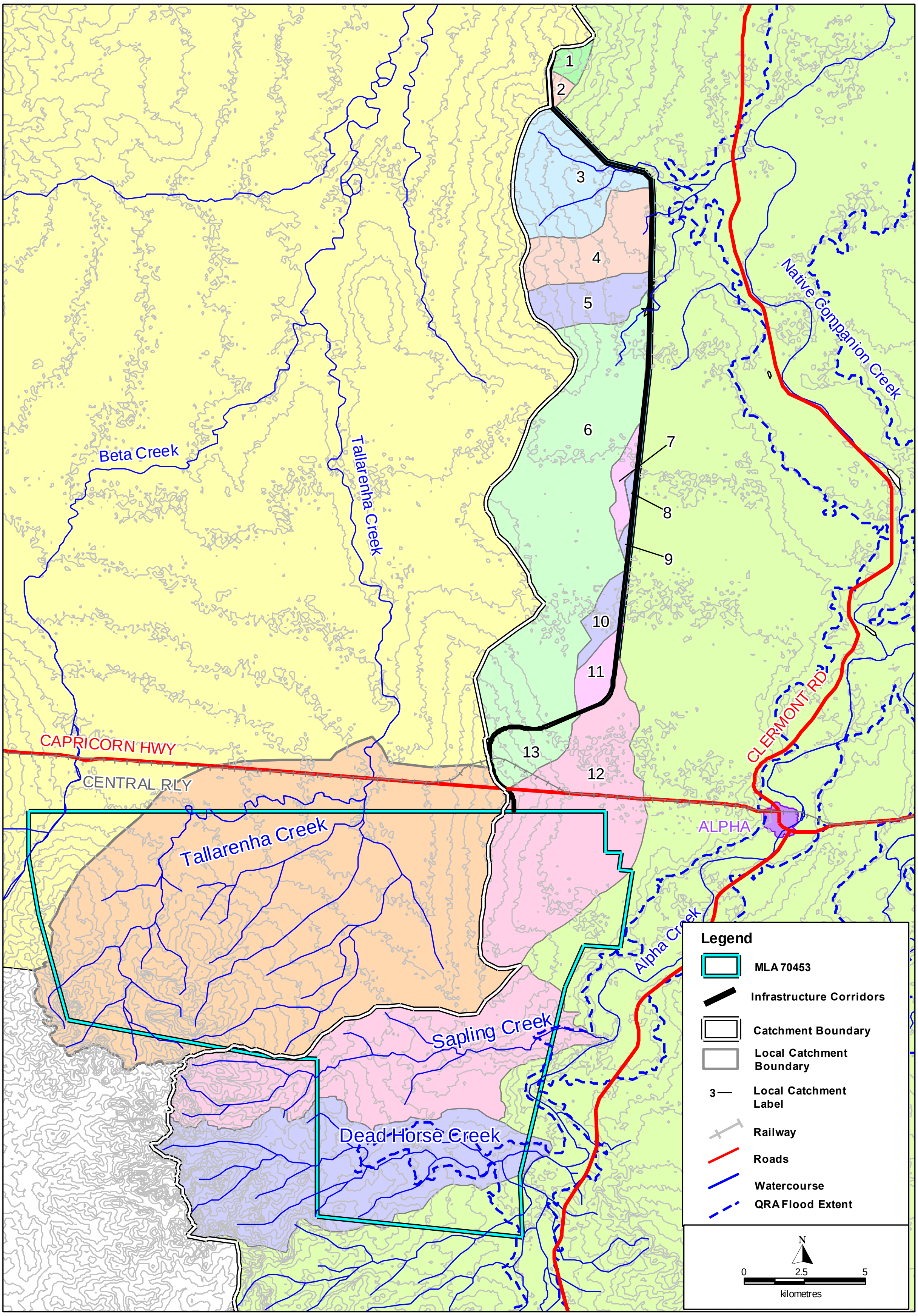


Figure 2.4 Catchments potentially impacted by the project

Table 2.1 Catchment Areas Crossing the Infrastructure Corridor

Catchment Label	Catchment Area (ha)
1	207
2	108
3	1,912
4	1,417
5	886
6	7,091
7	289
8	56
9	48
10	371
11	382
12	5,127
13	703

2.3 LOCAL STREAM MORPHOLOGY

The alignments of the streams crossing the MLA are shown in Figure 2.5. The near-surface rock strata are consolidated deposits of siltstone and sandstone. These deposits are thickest in the northern and central region of the SGCP. In the eastern part of the SGCP, there are alluvial deposits of gravel, sand and poorly consolidated clayey sandstone.

Sapling Creek and Dead Horse Creek have similar geomorphological characteristics. They have similar catchment areas, flow in a similar direction and cross similar geological features. Areas of the upper (western) parts of theses catchments are drained by a closely spaced network of well-defined, relatively steep gullies. The creek channels themselves are also relatively steep, straight and well incised compared to other streams in the area.

In the lower eastern reaches, the channels widen, and the floodplains become better defined. Sapling Creek crosses a relatively steep escarpment at the edge of the Alpha Creek floodplain. Photographs of the Sapling Creek channel at the locations indicated in Figure 2.5 are shown in Figure 2.6 to Figure 2.9. In the upper reaches, the channel is relatively deep, the banks are well vegetated and apparently stable, with a coarse sandy bed. Further downstream, the channel becomes wider. At Location C, the channel is draped in fine sediment, and waterholes persist after rainfall. However, at location D, the channel banks are high and unstable – with significant erosion occurring through dispersive clay.

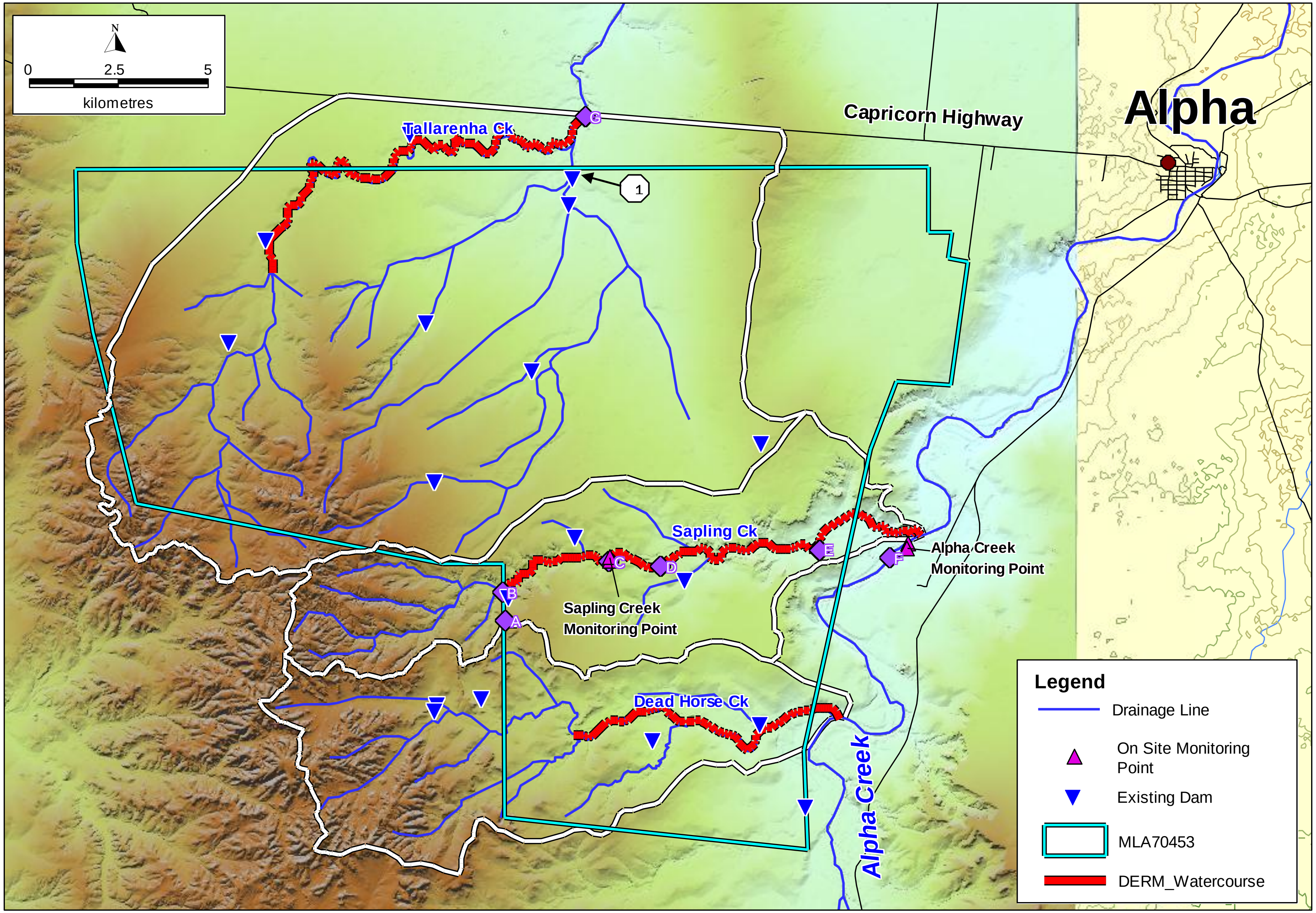


Figure 2.5 Local Drainage Paths and Watercourses – Main Lease Area

Figure 2.6 Sapling Creek – Location B

Figure 2.7 Sapling Creek – Location C

Figure 2.8 Sapling Creek – Location D

Figure 2.9 Sapling Creek – Location E

Figure 2.10 Alpha Creek – Location F

Figure 2.11 Tallarenha Creek – Location G

2.3.1 Tallarenha Creek Sections

Existing and post-subsidence longitudinal profiles of Tallarenha Creek are shown in Figure 2.12. Figure 2.13 shows the location of points plotted in Figure 2.12. Channel slopes are typically in the range 0.2% (1V in 500H) to 0.6% (1V in 170H).

Cross-sections at various locations along Tallarenha Creek are also shown in Figure 2.14, Figure 2.15 and Figure 2.16.

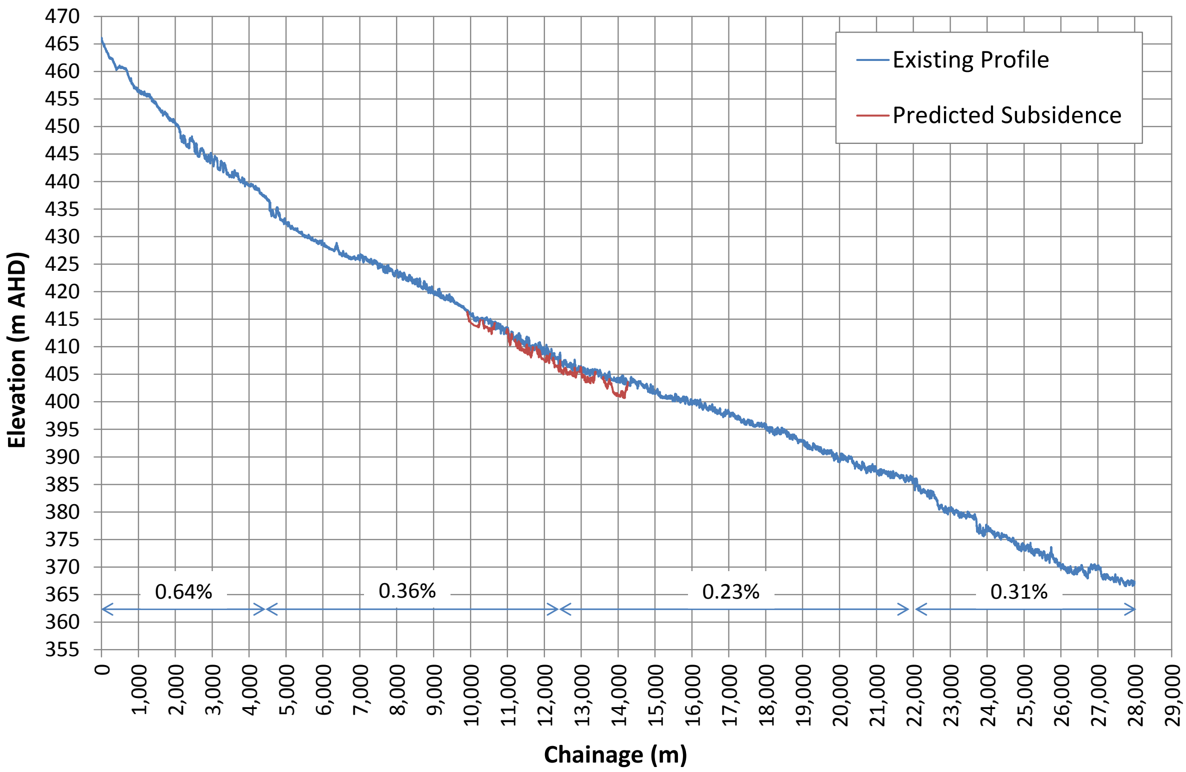


Figure 2.12 Tallarenha Creek – Longitudinal Profile

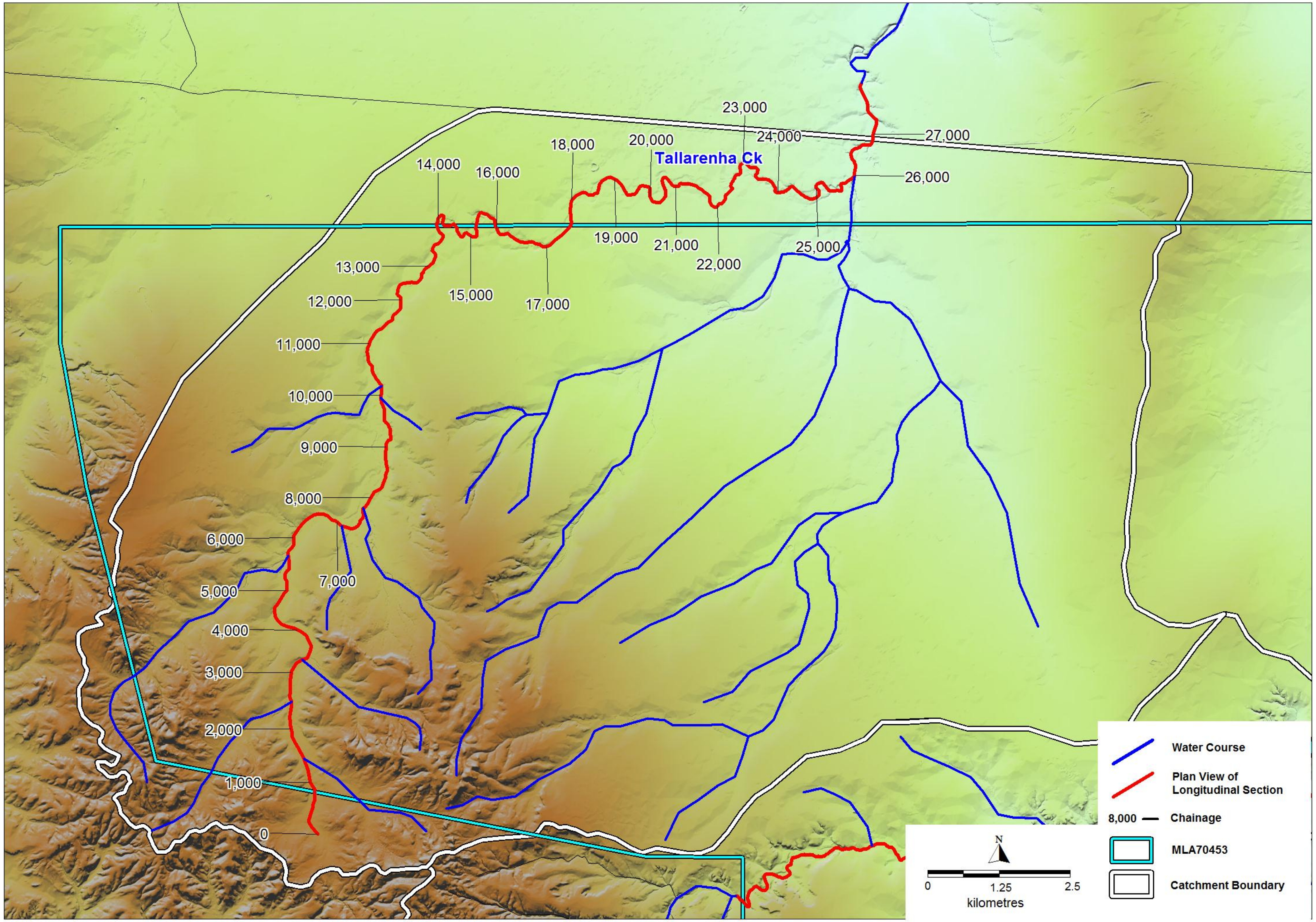


Figure 2.13 Tallarenha Creek – Key Plan for Sections

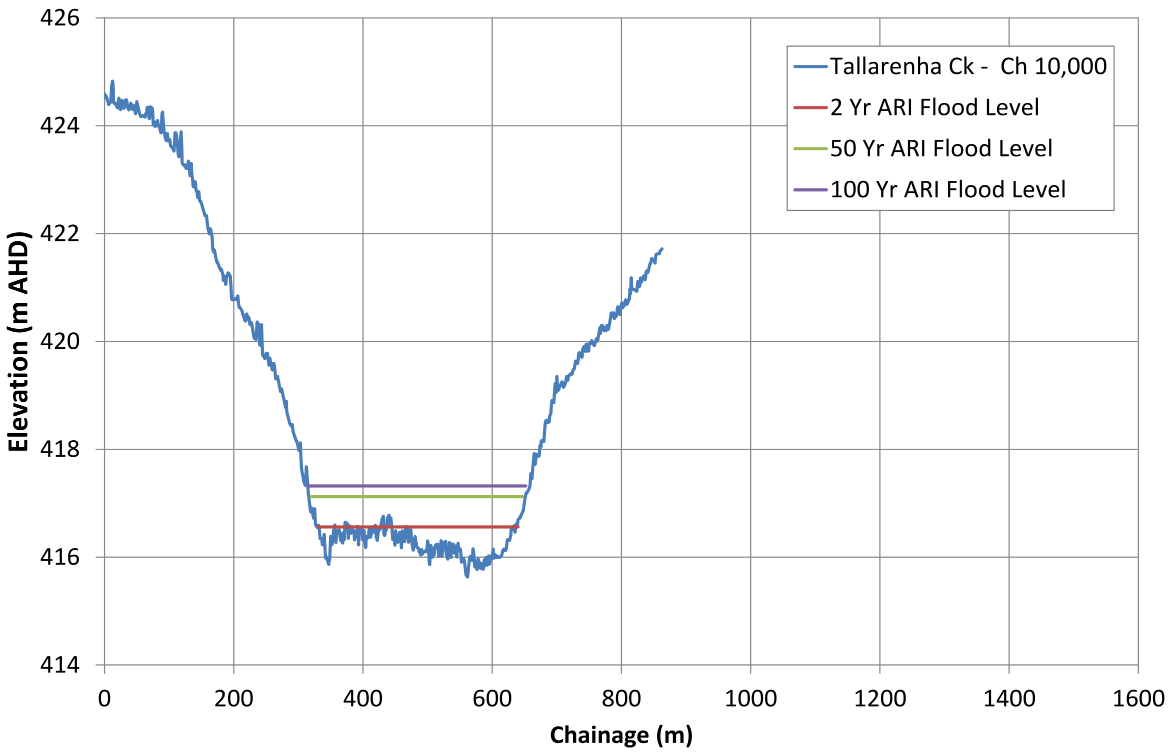


Figure 2.14 Tallarenha Creek – Cross Section CH 10,000

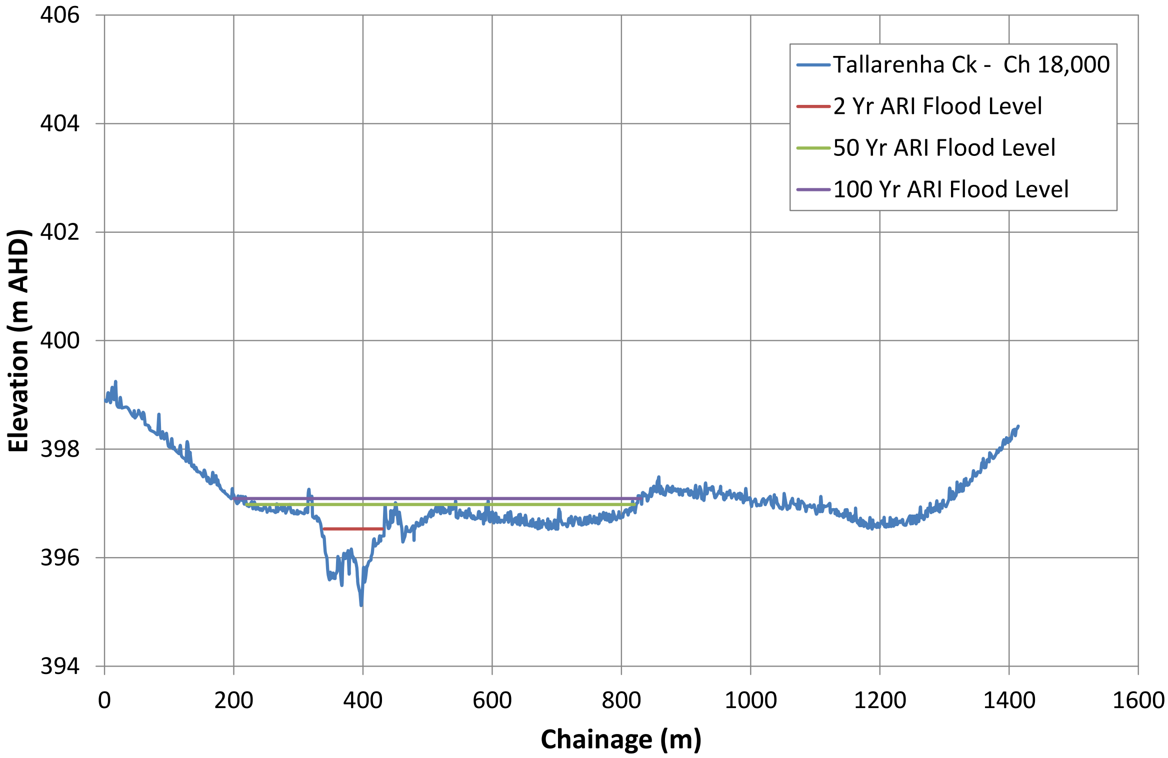


Figure 2.15 Tallarenha Creek – Cross Section CH 18,000

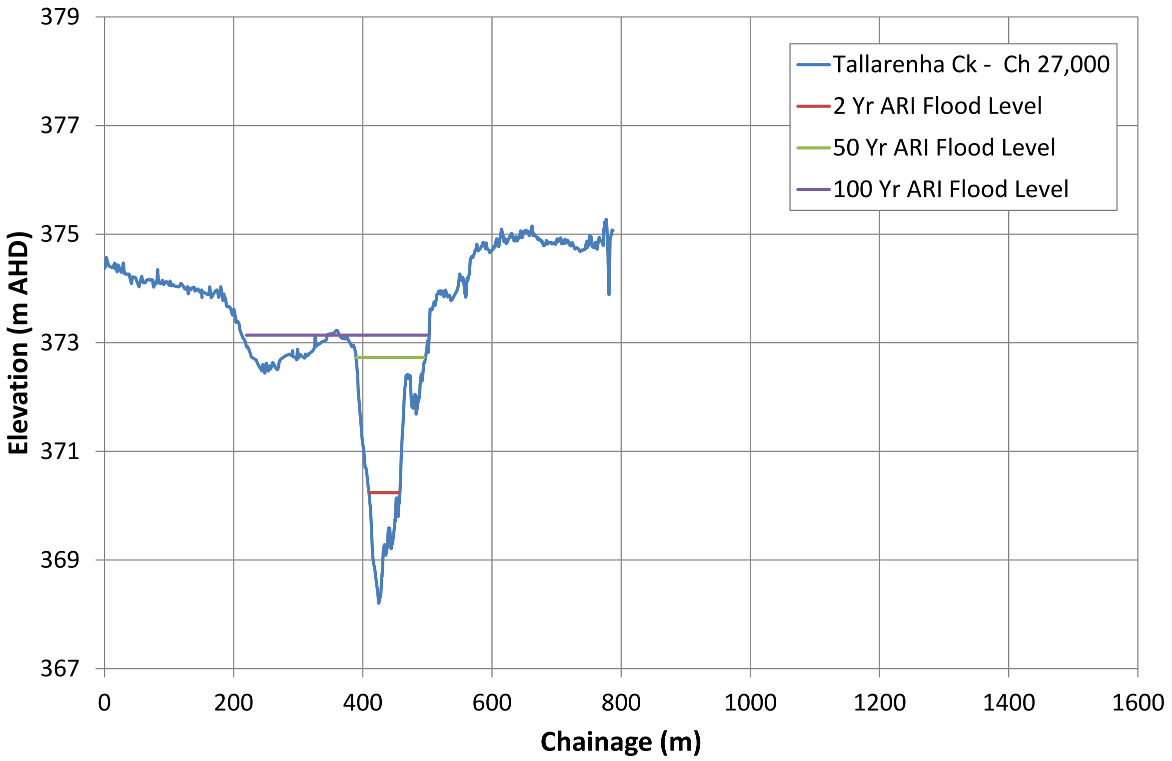


Figure 2.16 Tallarenha Creek – Cross Section CH 27,000

2.3.2 Sapling Creek Sections

The longitudinal profile of Sapling Creek is shown in Figure 2.17. Figure 2.18 shows the location of points plotted in Figure 2.17. Channel slopes are typically in the range 0.2% (1V in 500H) to 0.5% (1V in 200H).

Cross-sections at various locations along Sapling Creek are also shown in Figure 2.19, Figure 2.20 and Figure 2.21.

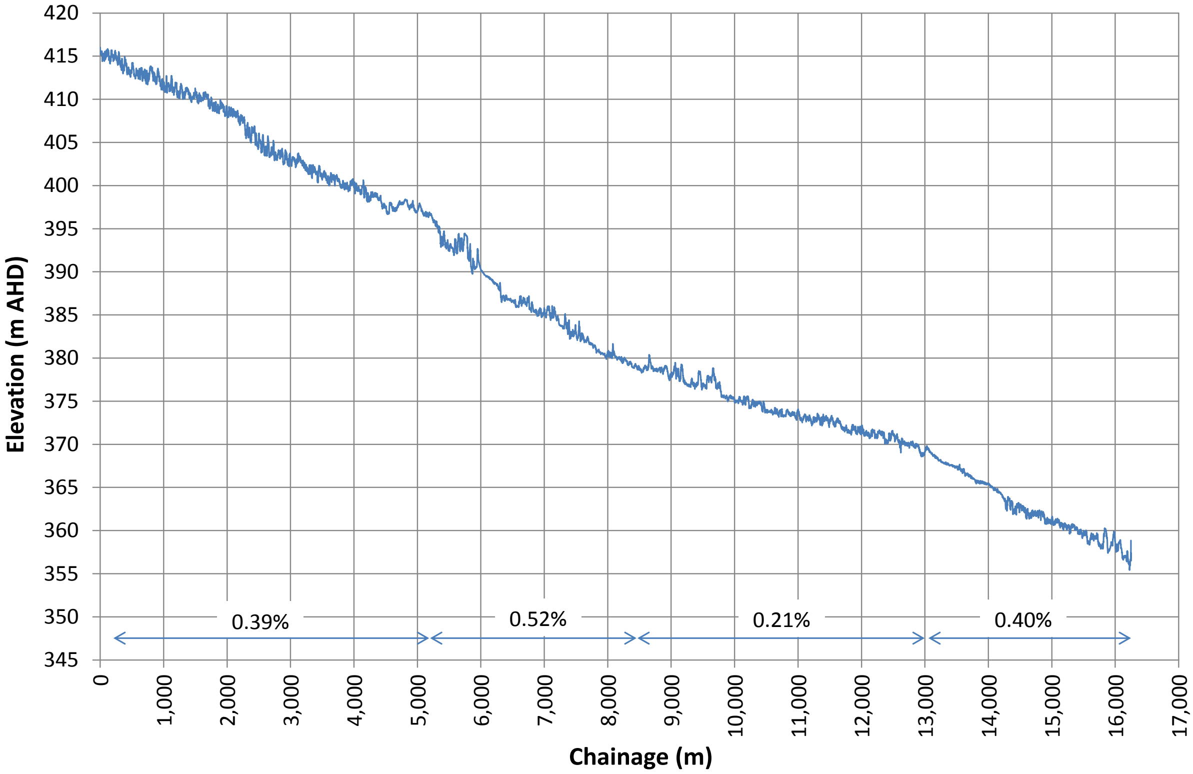


Figure 2.17 Sapling Creek – Longitudinal Profile

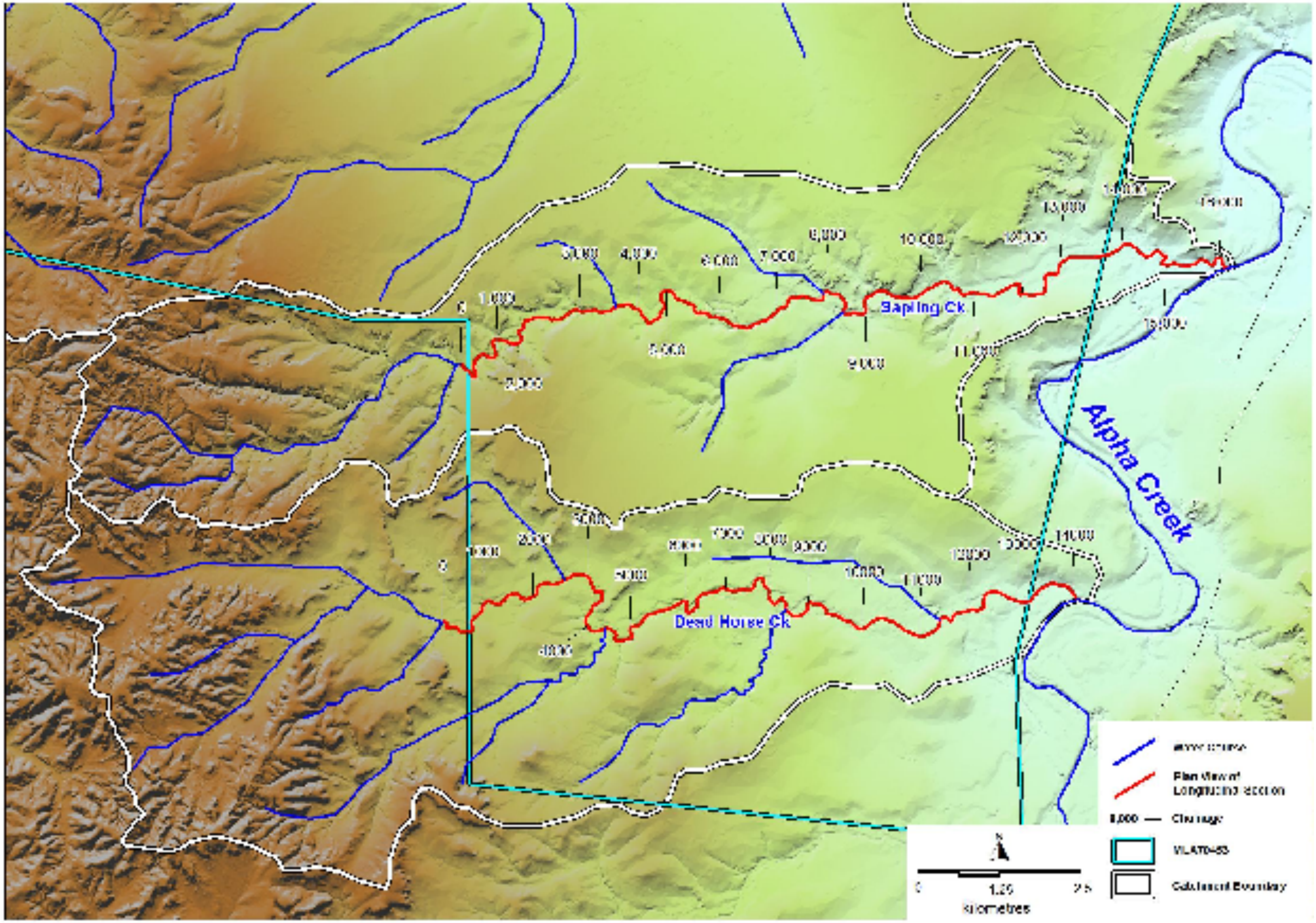


Figure 2.18 Sapling Creek and Dead Horse Creek – Key Plan for Sections

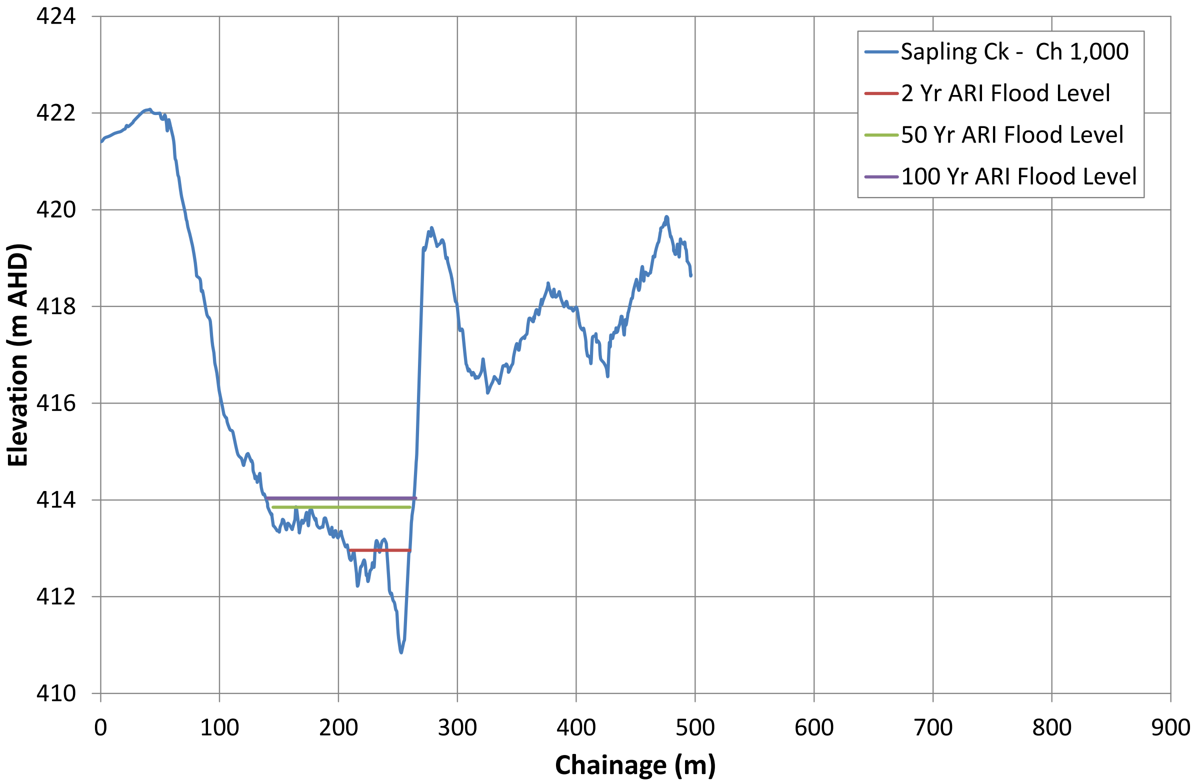


Figure 2.19 Sapling Creek – Cross Section at CH 1,000

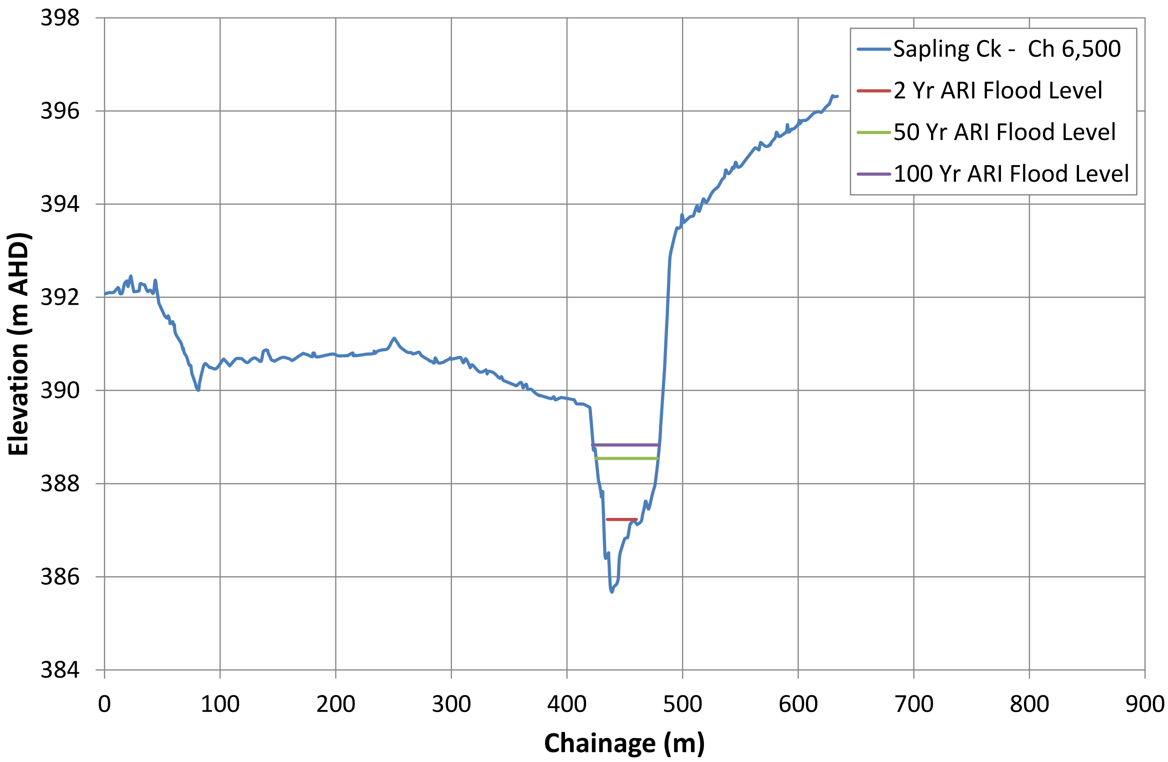


Figure 2.20 Sapling Creek – Cross Section at CH 6,500

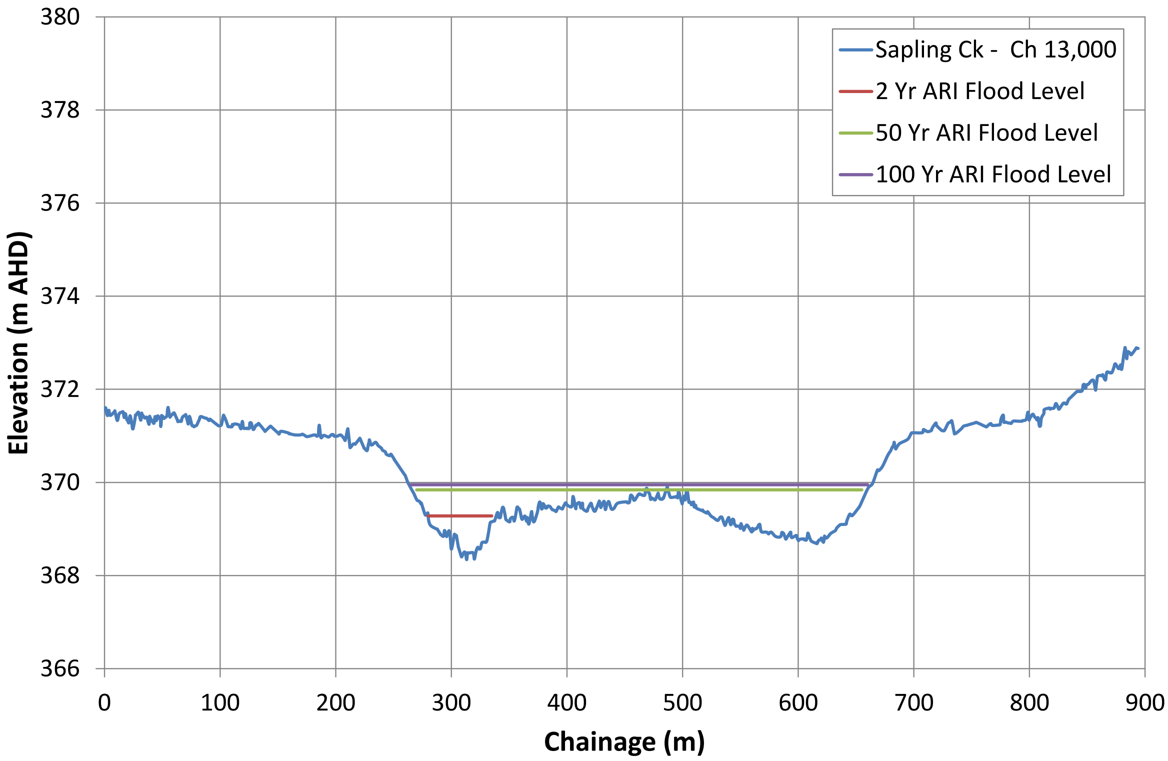


Figure 2.21 Sapling Creek – Cross Section at CH 13,000

2.3.3 Dead Horse Creek Sections

The longitudinal profile of Dead Horse Creek is shown in Figure 2.22. Figure 2.18 shows the location of points plotted in Figure 2.22. Channel slopes are typically in the range 0.3% (1V in 330H) to 0.5% (1V in 200H).

Cross-sections at various locations along Dead Horse Creek are also shown in Figure 2.23, Figure 2.24 and Figure 2.25.

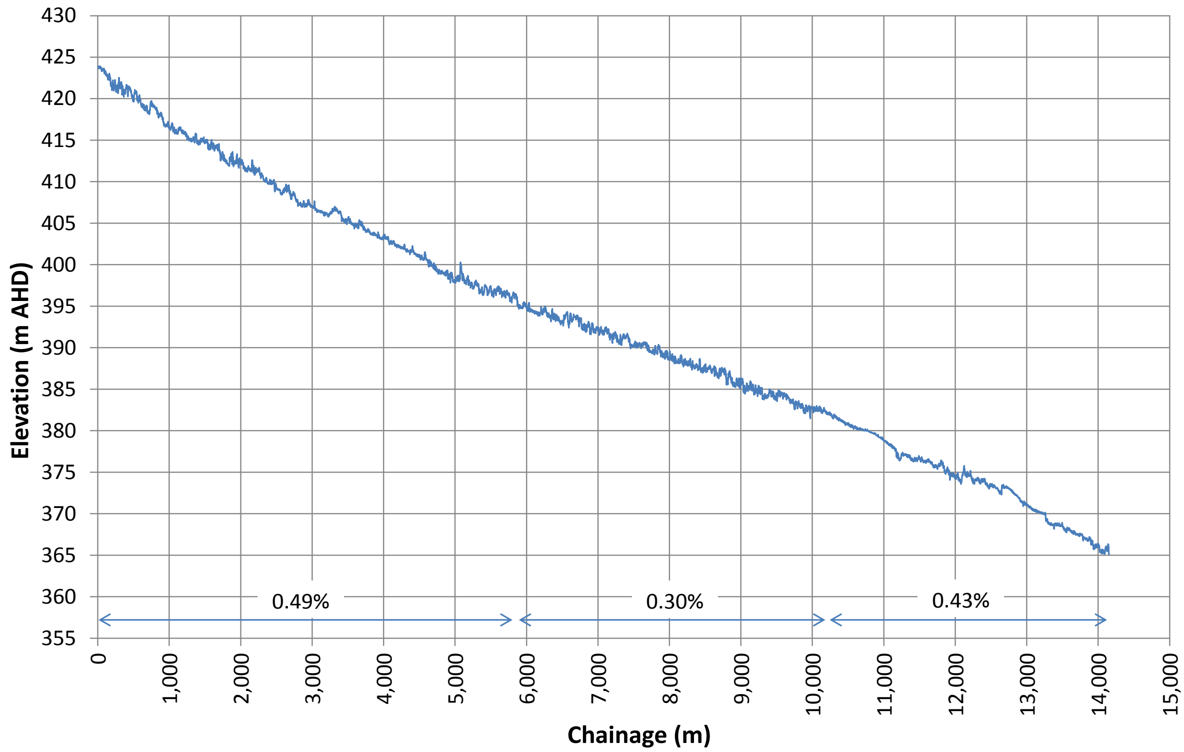


Figure 2.22 Dead Horse Creek – Longitudinal Profile

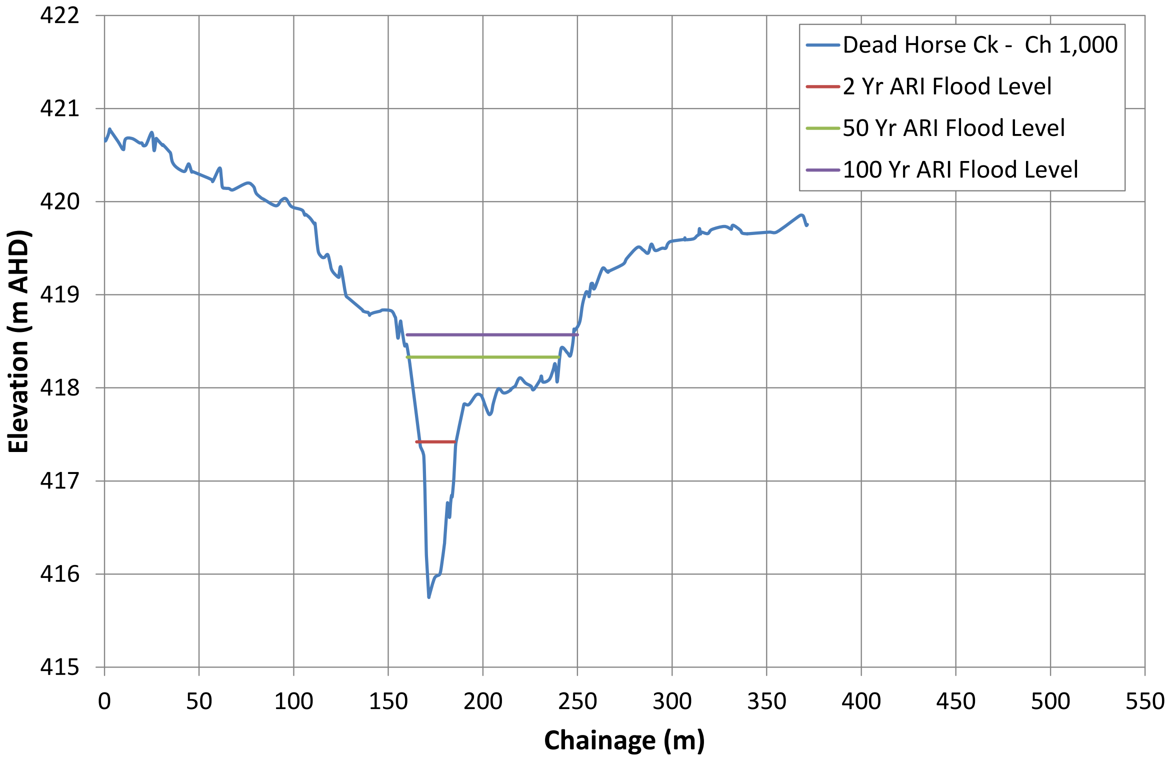


Figure 2.23 Dead Horse Creek – Cross Section at CH 1,000

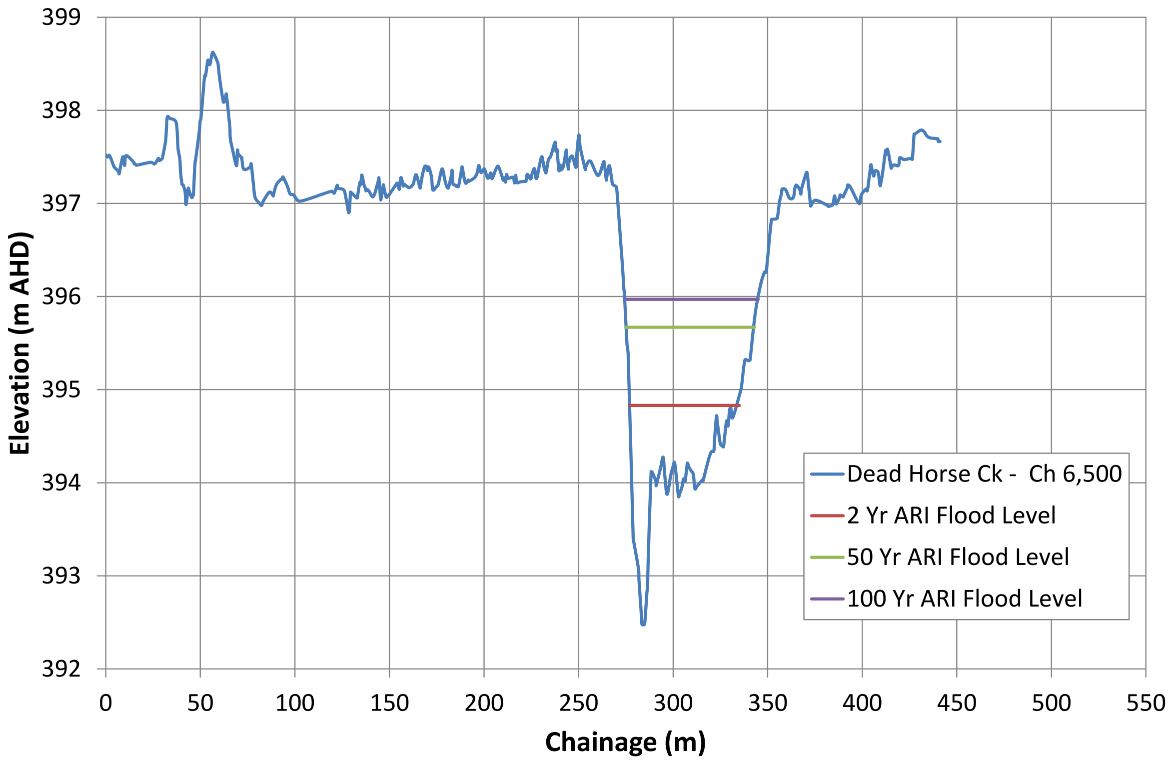


Figure 2.24 Dead Horse Creek – Cross Section at CH 6,500

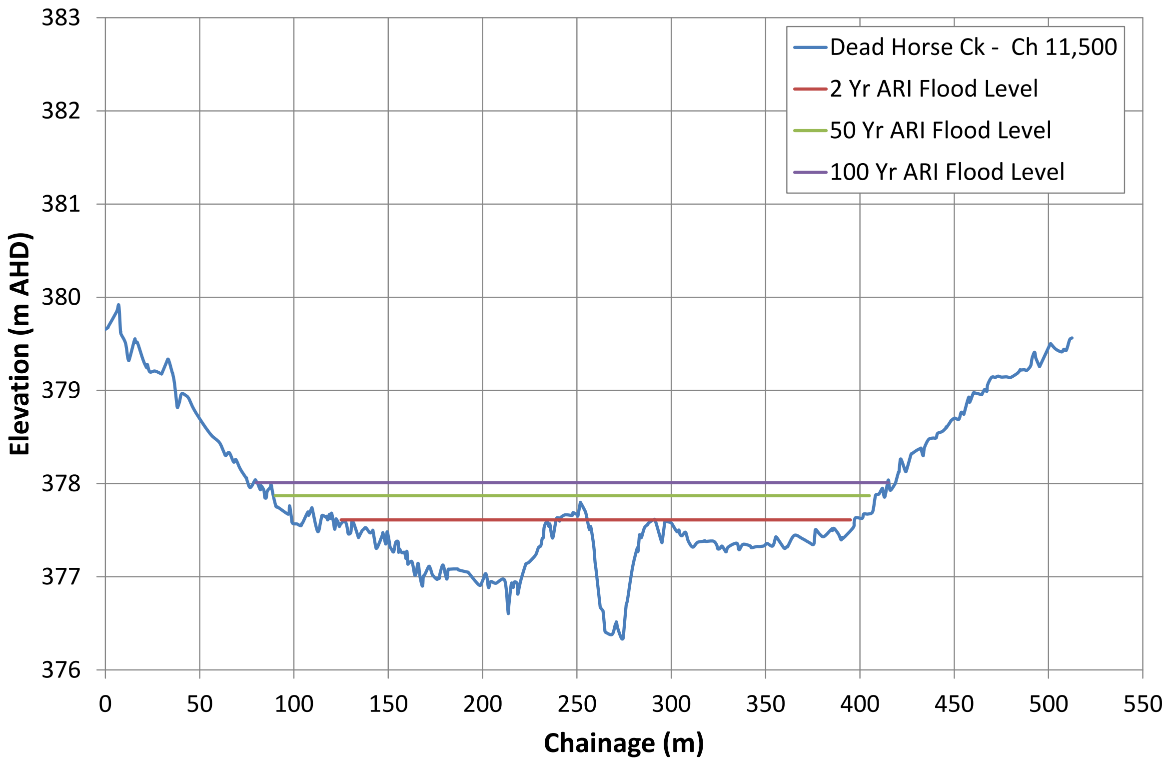


Figure 2.25 Dead Horse Creek – Cross Section at CH 11,500

2.4 RAINFALL AND EVAPORATION

Long-term rainfall data has not been recorded at the SGCP site, however records have been kept at a number of nearby Bureau of Meteorology rainfall stations, as summarised in Table 2.2 and in Figure 2.26.

Table 2.2 Details of Rainfall Stations in the Area of Interest

Station No.	Station Name	Type	Elevation (m)	Distance from Site (km)	Opened	Closed	Owner
035229	Alpha	Daily	341.0	12.9	1992	-	BOM (AUS)
035000	Alpha Post Office	Daily	368.0	12.7	1886	-	BOM (AUS)
120305A	Native Companion Creek	Continuous		17.1	1967	-	NRW (QLD)
035087	Betanga	Daily	400.0	33.0	1994	1999	BOM (AUS)
035236	Rivington	Daily	350.0	19.8	1968	1997	BOM (AUS)
003918	Jericho	Daily	347.0	39.9	1992	-	BOMQ (QLD)
035256	Jericho Post Office	Daily	355.0	40.4	1896	-	BOM (AUS))
035164	Monklands	Daily	396.0	30.1	1971	-	BOM (AUS)
035165	Durrandella	Daily	405.4	46.1	1958	-	BOM (AUS)
035033	Harden Park	Daily	370	78.9	1917	2001	BOM (AUS)

The nearest long-term station is the Alpha Post Office rainfall station, located about 7km north east of the MLA, where records have been kept since 1886. In order to infill gaps in the record, the Patched Point Dataset for the Alpha Post Office station was obtained from DERM. The Patched Point Dataset uses original Bureau of Meteorology measurements for a particular meteorological station, but missing data are filled ("patched") with interpolated values. DERM's interpolations are calculated by splining and kriging techniques. Details are provided in Jeffrey et al, 2001. Mean annual rainfall from this dataset over the 123 year period from 1889 to 2011 for which patched data is available, is 562mm.

2.4.1 Spatial Variability

Figure 2.2 shows mean annual rainfall isohyets provided by the Bureau of Meteorology derived from recordings for the period 1969 to 1990. The figure shows that mean annual rainfall does not vary significantly in the immediate vicinity of the project site, and as a result, the Alpha Post Office Patched Point Dataset has been used throughout this assessment to represent rainfalls in the study area.

2.4.2 Temporal Variability

As shown in Figure 2.26, annual rainfall at Alpha has been highly variable, ranging from 205mm in 2002 to 1,577mm in 1956.

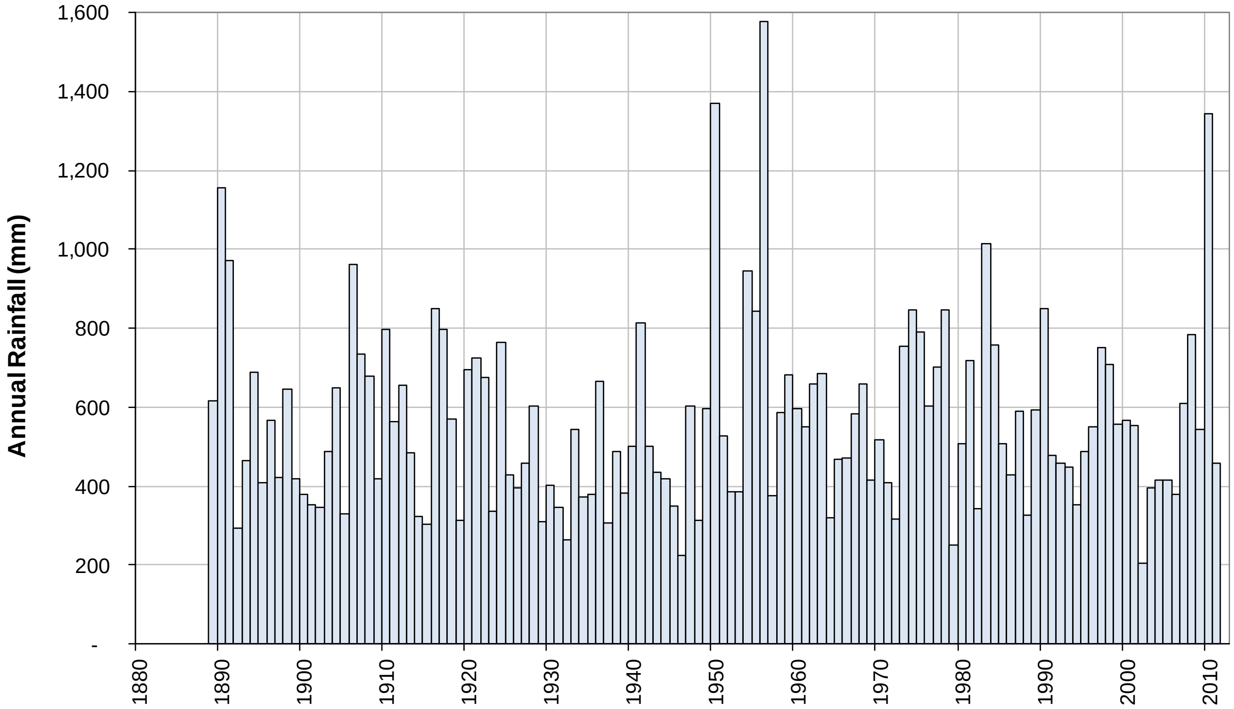


Figure 2.26 Annual Rainfall at Alpha Post Office

Mean monthly rainfall is highest between December and February. This is illustrated in Figure 2.27 along with mean monthly pan evaporation, which is highest between October and March. Mean annual pan evaporation is estimated to be 2,246 mm (with annual totals ranging between 1,677mm and 2,614mm).

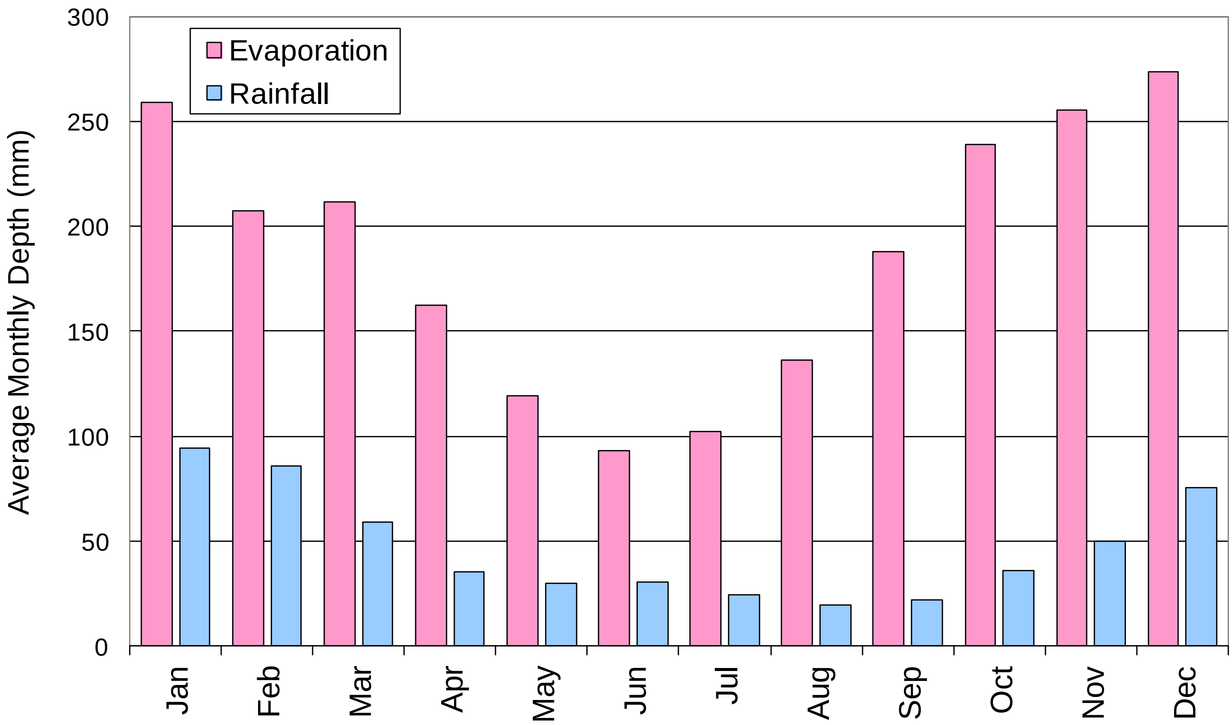


Figure 2.27 Average Monthly Rainfall and Pan Evaporation at Alpha Post Office

2.5 STREAMFLOW

As shown in Figure 2.2, streamflow has been recorded at several stream gauging stations downstream of the MLA.

The nearest DERM gauge providing long-term streamflow data is on Native Companion Creek at Violet Grove, approximately 13km north-east of the site and approximately 30km downstream of the MLA boundary.

This gauge has measured streamflow from a catchment of 4,065km² since 1967. Figure 2.28 shows that mean streamflow is greatest between December and February, though average streamflow is also high in April (due to a very large flood which occurred in April 1983).

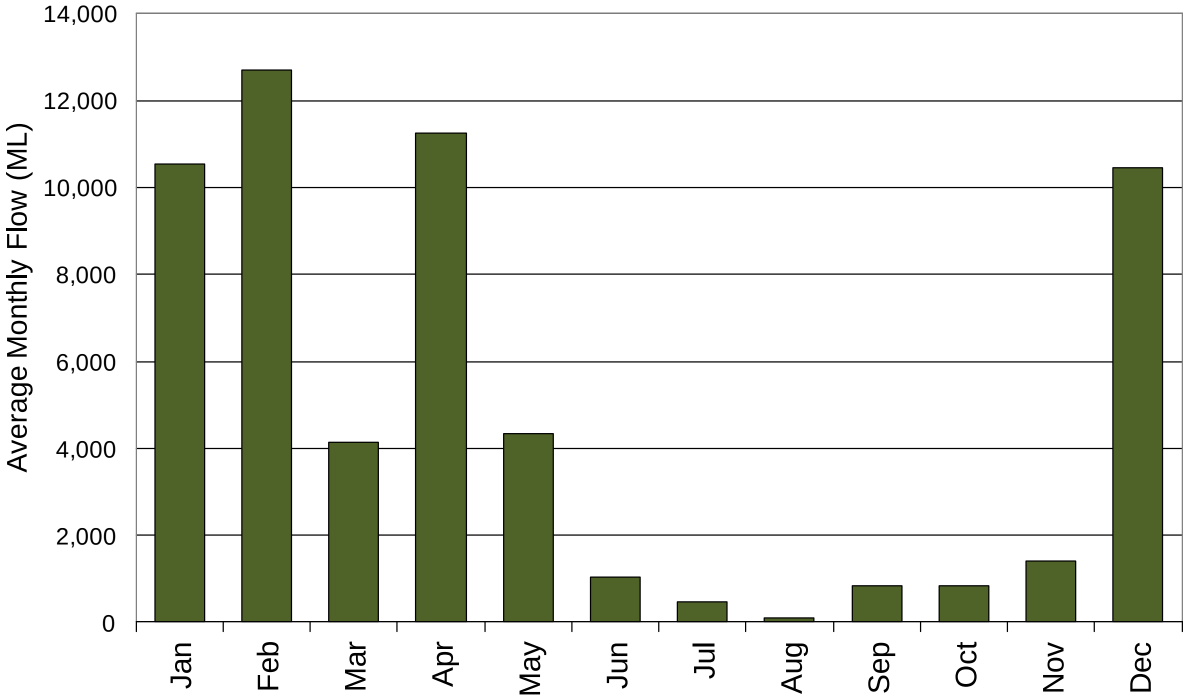


Figure 2.28 Mean Monthly Streamflow in Native Companion Creek at Violet Grove

The flow duration curve in Figure 2.29 indicates that the local streams are ephemeral, with long periods of low or no flow. Flows greater than 1ML/d have been observed only 27% of the time.

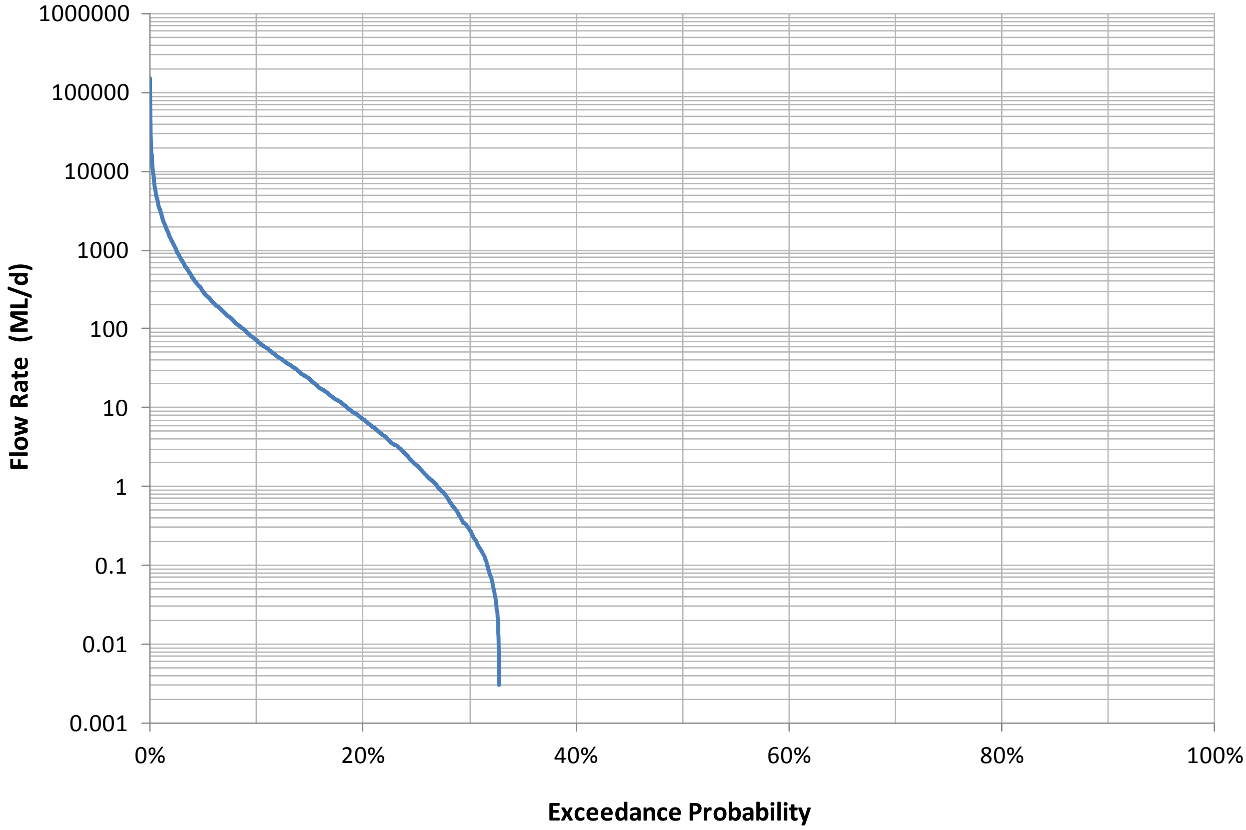


Figure 2.29 Flow Duration Curve – Native Companion Creek at Violet Grove

Recording of local surface water levels has been undertaken by Met Serve at locations in Alpha Creek and Sapling Creek as shown in Figure 2.5.

The runoff response in local watercourses is characterised by long periods of no flow interspersed with short periods of streamflow. The data recorded during May 2010 to September 2011 is shown in Figure 2.30. During this period, the gauges recorded elevated water levels over a period of more than 1 year.

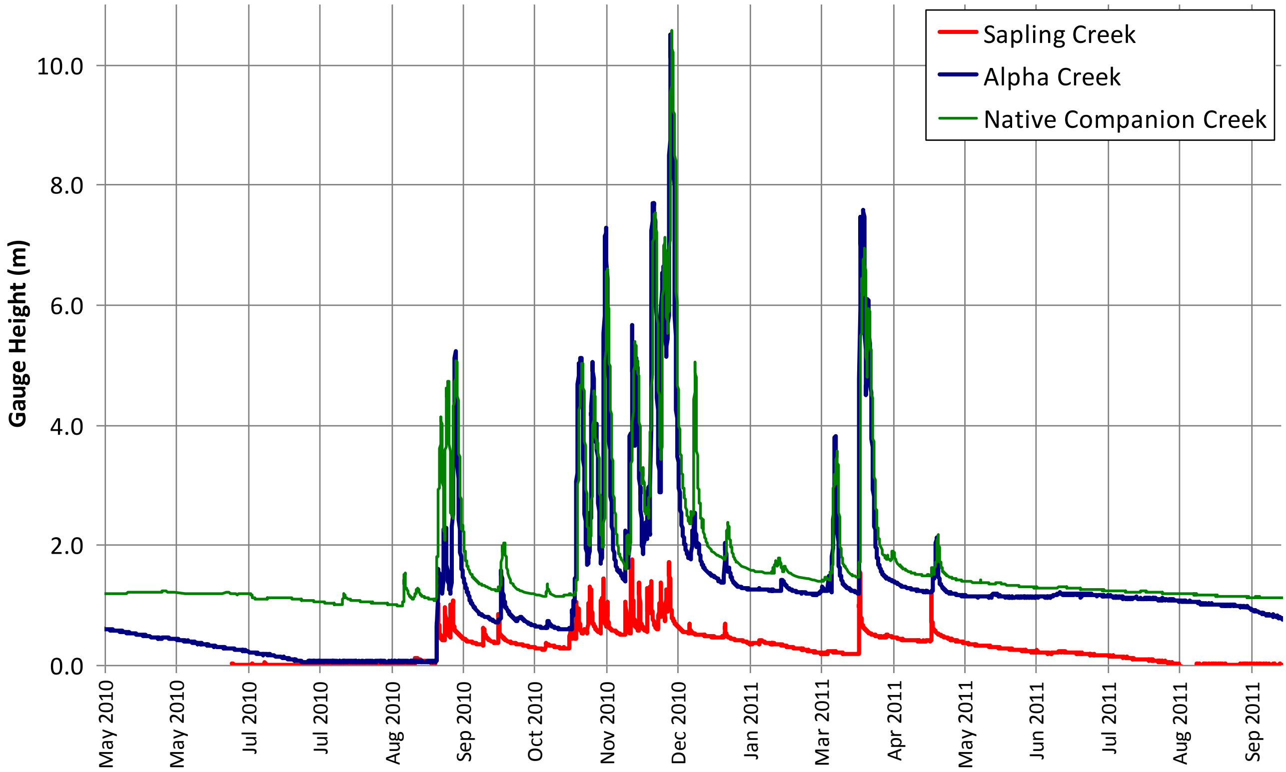


Figure 2.30 Sample Receiving Water Streamflow Response

2.6 ENVIRONMENTAL VALUES OF RECEIVING WATERS

The *Australian and New Zealand Guidelines for Fresh and Marine Water Quality* (2000) (ANZECC Guidelines) define the 'Environmental Values' of receiving waters as those values or uses of water that the community believes are important for a healthy ecosystem.

Under Section 7 of the Queensland Environmental *Protection Policy (Water) 2009* (EPP Water), there are no particular environmental values attributed to the specific waterways located within the SGCP as they are not listed in Schedule 1. Section 7 (2) however, assigns the environmental values in the receiving water to be protected under the category 'other waters' as:

- ecosystem protection (Level 2 – disturbed ecosystems, Queensland Water Quality Guidelines (QWQG) 2009); and
- agricultural uses (Irrigation and Stock Watering).

The ANZECC guidelines specify levels of protection corresponding to each of the following measures of the receiving water ecosystem condition:

- of high conservation value;
- slightly to moderately disturbed; or
- highly disturbed.

The receiving waterways adjacent to the project area are slightly to moderately disturbed.

2.7 SURFACE WATER QUALITY

2.7.1 Regional Sediment Transport Characteristics

Annual total suspended solid loads to the Great Barrier Reef Lagoon are estimated to be 17 million tonnes. The Burdekin River catchment is estimated to supply approximately 30% of this total (4.7 million tonnes per year), of which 4.1 million tonnes are from human activity derived from extensive areas under grazing (Reef Water Quality Protection Plan, 2011).

Sediment transport rates from the Native Companion Creek catchment (which has an area of 5,460km^2 including the project area of 310km^2) have been estimated in SedNet modelling studies by the CSIRO (Kinsey-Henderson, Sherman and Bartley, 2007). These studies concluded the creek conveys relatively high mean event concentrations of suspended sediments and nutrients (629 mg/L) attributable mainly to hillslope erosion (62%). However, due to its relatively low contribution to catchment runoff, Native Companion Creek contributes only a small proportion (less than 0.2 million tonnes per annum) of total sediment load (Dight, 2009).

The Burdekin Falls Dam significantly reduces sediment transport rates to the Great Barrier Reef lagoon. The sediment trapping efficiency has been estimated as approximately 60-70% of suspended sediment in moderate to large flow events, and up to 80-90% in smaller events (Lewis et al,2009).

2.7.2 Local Water Quality

Background water quality data has been collected by AMCI Pty Ltd at various locations across the Project area.

The following sections present water quality assessment results for parameters of relevance to the SGCP.

2.7.3 Alpha Creek Water Quality

Met Serve has recorded water level and salinity (electrical conductivity (EC)) on Alpha Creek and Sapling Creek at the locations shown in Figure 2.5. Figure 2.31 and Figure 2.32 show the relationship between water level and salinity and pH in Alpha Creek (when water level exceeds 0.5m gauge height) for recordings made between January 2010 and December 2011:

- salinity reduces with flow rate, and is less that 100μS/cm when flow depths exceed 8.5m, and
- pH varies between 6.5 and 10.7, with the high values occurring during low or zero flow. During low flow, pH is typically close to neutral. The cease to flow level is probably around 1.0m;
- when flow depth exceeds 1m, EC is typically below 300μS/cm;
- when flow depth exceeds 0.5m EC is less than 450μS/cm.

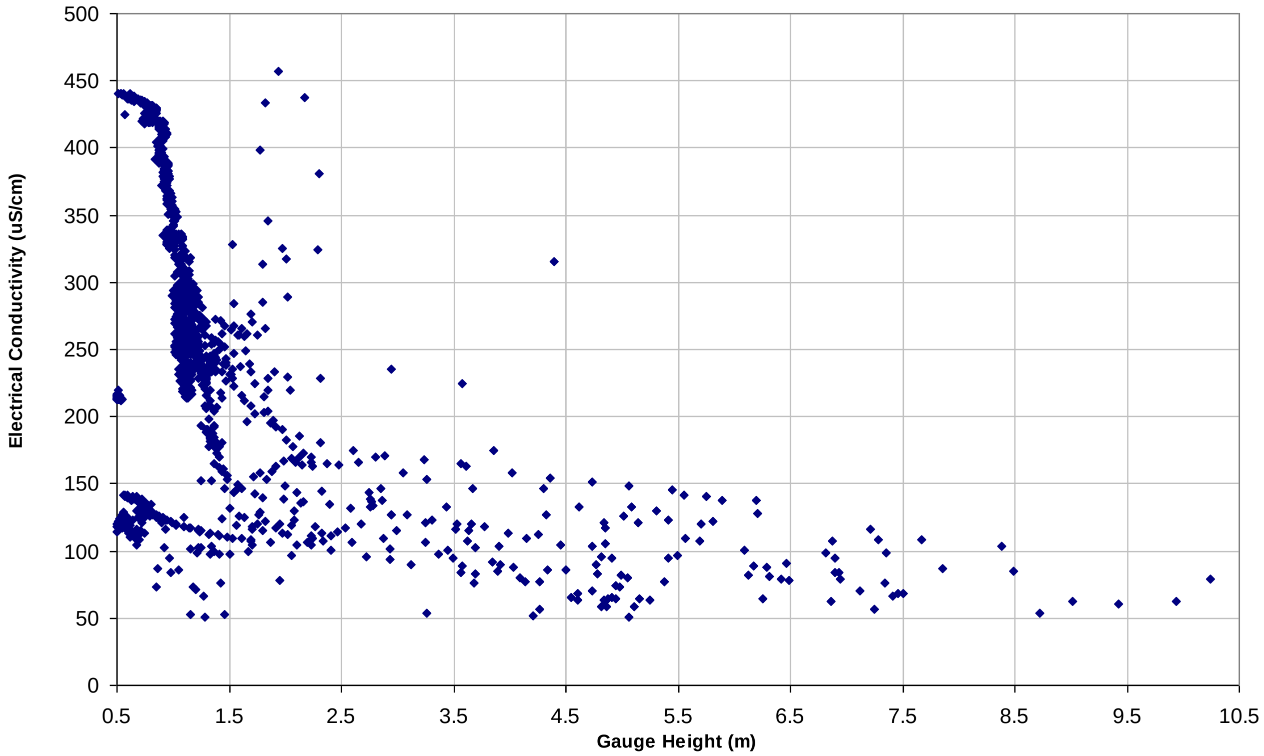


Figure 2.31 Salinity vs Flow Depth Relationship for Alpha Creek

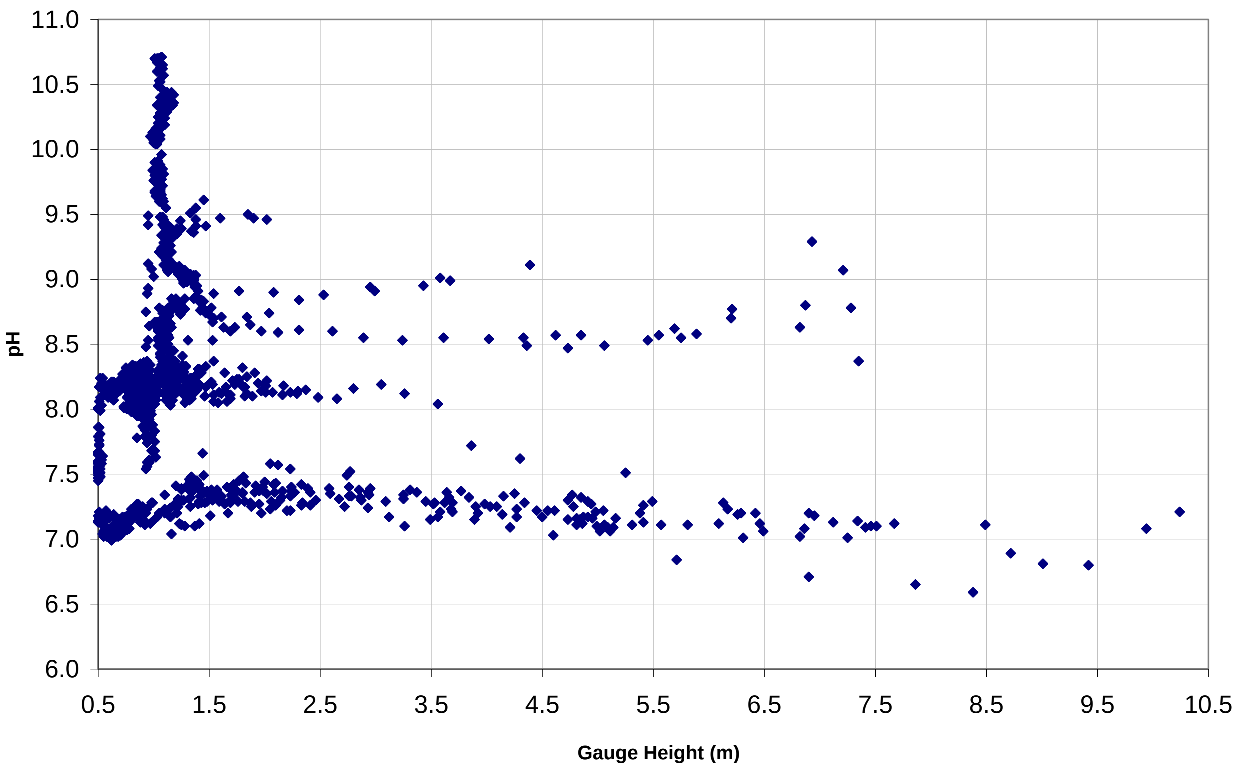


Figure 2.32 pH vs Flow Depth Relationship for Alpha Creek

2.7.4 Results of Full Water Quality Analysis at Nearby Sampling Locations

Full laboratory analysis has been undertaken for a small number of water samples collected on and downstream of the SGCP site. The results of this analysis are summarised in Table 2.3.

The variability of flow in ephemeral streams can lead to changes in the physical and chemical properties of flow compared to perennial streams. The current ANZECC Guidelines therefore may not be well suited to assess the characteristics of water quality for ephemeral streams. Site specific trigger values should be established for the receiving waters in the vicinity of the project site once sufficient background water quality data has been collected.

wrm water+environment

Table 2.3 Summary of Available Water Quality Data in the Vicinity of the SGCP

	LOR#	DERM Sampling Site (Native Companion Ck at Violet Grove)			SGCP Sampling Site																				
		Count	Median	90th %ile	AC1	AC2	AC2	AC2	AC3	AC4	DH1	DH1	SC1	SC1	SC1	SC1	SC1	SC2	T1	T1	T1	TA1	T2	T2	T2
Conductivity (µS/cm)	1	60	142	300	422	345	287	172	268	287	305	270	59	202	145	324	144	208	587	599	288	71	223	168	275
pH	0.01	61	7	8	7.88	7.95	7.95	6.54	7.59	6.99	7.36	7.52	6.59	7.14	6.63	6.45	6.2	7.41	7.96	7.65	7.65	7.26	7.85	6.79	7.87
Turbidity	0.1	35	200	597	8.8	10.0	22.2	94	16.5		>1000	120	39.0	220	197	832	854	110	13.0	8.43	300	26.1	80.4		75.0
TDS (mg/L)	5	59	90	170	240	204				186.2		188	37	283				181	347		493			106.7	220
TSS (mg/L)	5	55	110	724	7	10						LOR	6	141				36	16		202				23
Sulphate (mg/L)	1	43	2	7	LOR	LOR				2		1	LOR	2				LOR	2		2			2	4
Aluminium (µg/L)	100	30	50	415	LOR	LOR				30*		1150	LOR	450				230	170		480			40*	1440
Antimony (µg/L)	10				LOR	LOR						LOR	LOR	LOR				LOR	LOR		LOR				10
Arsenic (As III) (µg/L)	10				LOR	LOR				LOR		LOR	LOR	LOR				LOR	20		LOR			LOR	LOR
Cadmium (µg/L)	5				LOR	LOR				LOR		LOR	LOR	LOR				LOR	LOR		LOR			0.3*	LOR
Calcium (Ca) (mg/L)	1	60	11	26	43	34						15	2	6				10	33		19				16
Chromium (CrVI) (µg/L)	10				LOR	LOR				LOR		LOR	LOR	LOR				LOR	LOR		LOR			LOR	LOR
Copper (µg/L)	10	32	25	50	LOR	LOR				2*		LOR	LOR	LOR				LOR	LOR		LOR			3*	LOR
Iron (µg/L)	5	42	75	1227	450	360				140		1150	1500	4170				1160	420		290			190	1250
Lead (µg/L)	10				LOR	LOR				LOR		LOR	LOR	LOR				LOR	LOR		LOR			LOR	LOR
Mercury (inorganic) (µg/L)	0.1				LOR	LOR				LOR		LOR	LOR	LOR				LOR	LOR		LOR			LOR	LOR
Nickel# (µg/L)	10				LOR	LOR				2*		LOR	LOR	LOR				LOR	LOR		LOR			2*	LOR
Potassium (K) (mg/L)	1	55	6	9	7	7						27	2	8				7	10		10				10
Zinc (µg/L)	10	31	10	60	LOR	LOR				LOR		LOR	LOR	LOR				LOR	LOR		20			7*	20
Boron (µg/L)	100	38	50	100	LOR	LOR				LOR		100	LOR	LOR				LOR	200		100			50*	200
Cobalt (µg/L)	10				LOR	LOR				LOR		LOR	LOR	LOR				LOR	LOR		LOR			LOR	LOR
Magnesium (Mg) (mg/L)	1	60	5	11	16	13						9	2	6				8	27		9				11
Manganese# (µg/L)	10	34	5	27	210	60				9*		260	40	200				80	270		10			95	40
Molybdenum (µg/L)	10				LOR	LOR				LOR		LOR	LOR	LOR				LOR	LOR		LOR			LOR	LOR
Selenium (µg/L)	10				LOR	10				LOR		LOR	LOR	LOR				LOR	20		LOR			LOR	20
Silver (µg/L)	10				LOR	LOR				LOR		LOR	LOR	LOR				LOR	LOR		LOR			LOR	LOR
Sodium (Na) (mg/L)	1	59	9	24	21	17						14	6	21				16	43		23				25
Uranium (µg/L)																									
Vanadium (µg/L)	10				LOR	LOR						LOR	LOR	10				LOR	LOR		LOR				LOR
Ammonia (as N) (µg/L)	10	28	37	86	20	20				20		90	160	30				40	120		40			50	10
Nitrate (µg/L)	10	47	1350	3132	40	20				LOR		60	20	80				130	20		10			LOR	10
Nitrite (µg/L)	10				LOR	LOR				LOR		LOR	LOR	LOR				LOR	LOR		LOR			LOR	LOR
Total Nitrogen (µg/L)	10	12	948	1490	200	100						3400	600	1600				700	1000		1400				700
Pet. hydrocarbon (C6-C9)																									
Pet. hydrocarbon (C10-C36)																									
Fluoride (total) (µg/L)	100	60	185	300	200	200				100		LOR	LOR	100				100	200		200			LOR	100

#LOR = Limit of reporting

*Lower LOR used for this test

2.8 FLOODING

2.8.1 MLA Areas

A flood study was undertaken to estimate design flood levels and the flood extent for existing conditions along the streams crossing the MLA. Full details of the methodology and results of the flood study are provided in Appendix D, and mapping of flood depths, velocities and extents for a range of flood events are provided in Appendix E. Design flood flows for the affected streams are summarised in Table 2.4.

Table 2.4 Design Flow Rates Downstream of SGCP Boundary (m^3/s)

Average Recurrence Interval (years)	Tallarenha Creek	Sapling Creek	Dead Horse Creek
2	125	53	76
50	535	228	327
100	686	295	420
1,000	957	454	586
3,000	1,152	546	704
Probable Maximum Flood (PMF)	3,005	1,421	1,833

Figure 2.33 shows the estimated extent and depths of inundation across the MLA for the 100 year Average Recurrence Interval (ARI) flood event. Mapping of flooding in Alpha Creek had been carried out previously as part of a model study by Aurecon (Aurecon, 2010). The results of this modelling are also included in Figure 2.33.

Figure 2.33 shows that the tenement boundary encroaches onto the floodplain of Alpha Creek at two localised locations in the south-east. Alpha Creek flooding will otherwise have little effect on the project.

Flood flows in the north and north-east flowing tributaries of Tallarenha Creek cross the proposed mining area. Figure 2.33 shows the inundation is shallow (generally less than 500mm in the 100 year ARI flood) and covers a broad area. As shown in Figure 2.34 flood velocities are generally less than 1m/s, except in the channels and localised areas, where velocities can exceed 2m/s.

In the upper reaches of Tallarenha Creek, flood flows are well confined to a floodplain less than 500m wide. However, as it turns east, a portion of flow breaks out and flows north to the Capricorn Highway. In small floods, this flow heads east along the highway. In larger floods, it crosses the highway and railway and flows north-east to the downstream reaches of Tallarenha Creek.

Flooding in Dead Horse Creek will not directly affect the proposed project, while Sapling Creek crosses the southern end of the proposed mining area. Sapling Creek and Dead Horse Creek have less significant floodplains, and the extent of flood inundation is typically less than 100m wide, except in broader, flatter areas near the confluence with Alpha Creek.

Design flow conditions in Sapling Creek and Dead Horse Creek in the vicinity of the site are summarised for a range of ARIs in Table 2.5 and Table 2.6.

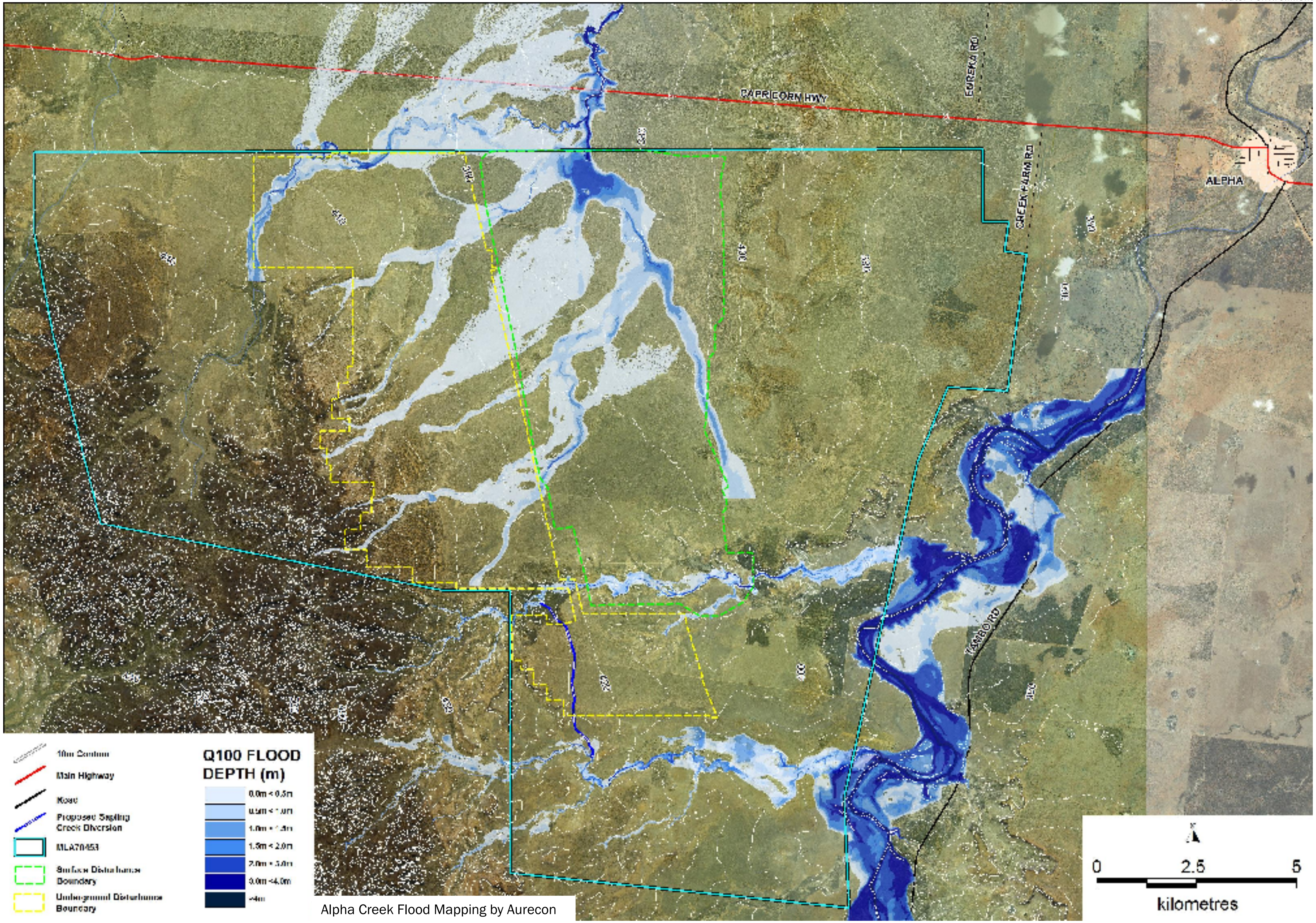


Figure 2.33 Extent and depths of Flood Inundation - 100 year ARI

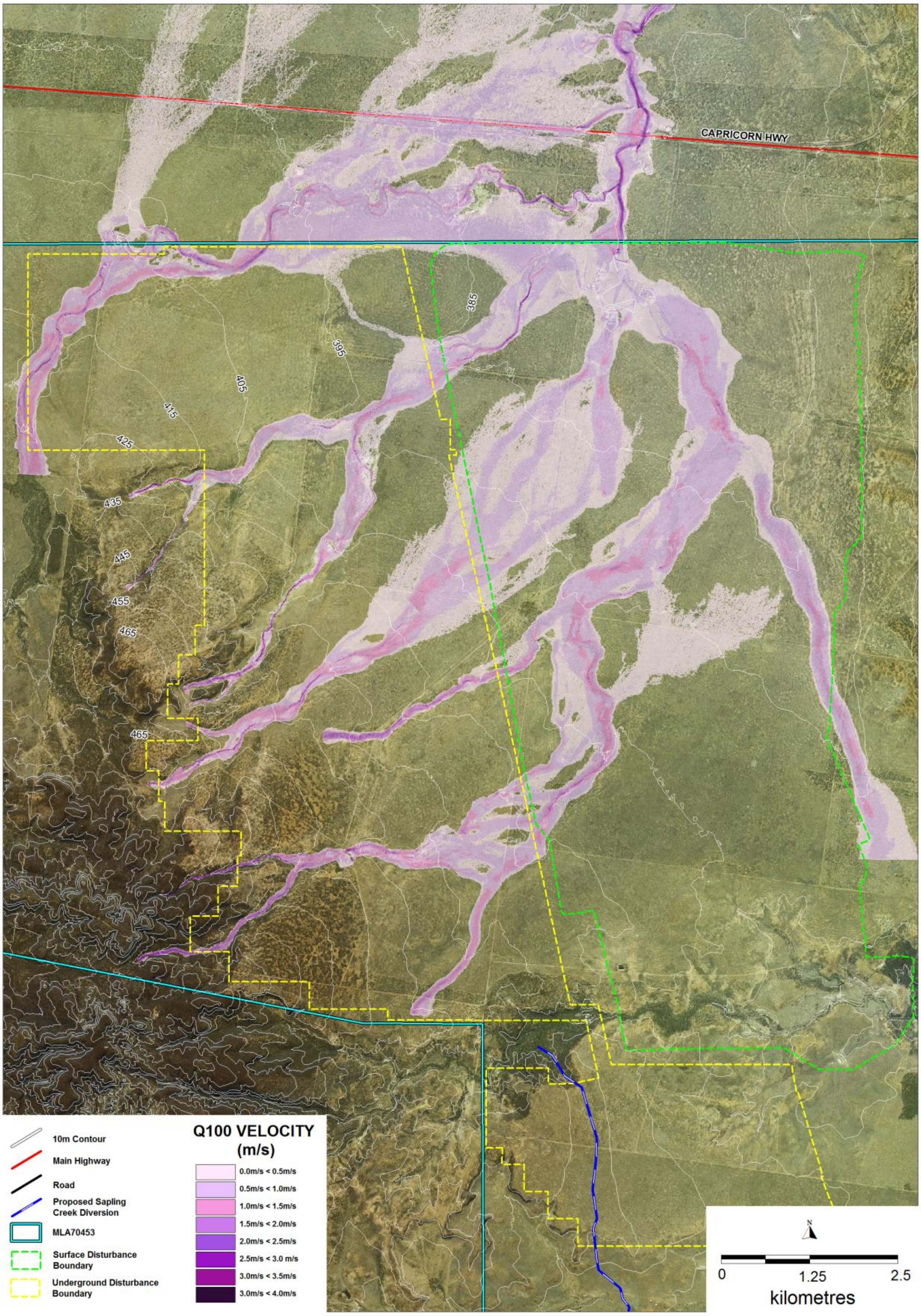


Figure 2.34 Flood Velocity - 100 year ARI

Table 2.5 Sapling Creek Design Flow Conditions

Average Recurrence Interval y	Flow Rate u/s-median-d/s m/s	Velocity min-median-max m/s	Total Section Stream Power min-median-max N/m²	Total Section Shear Stress min-median-max N/ms
2	30 - 54 - 63	0.5 - 1.4 - 2.7	1.1 - 22 - 271	3 - 18 - 99
50	137 - 246 - 275	0.8 - 2.1 - 5.3	2.3 - 37 - 614	4.8 - 29 - 186
100	176 - 315 - 355	0.9 - 2.2 - 4.8	2.9 - 43 - 628	5.5 - 26 - 160

Table 2.6 Dead Horse Creek Design Flow Conditions

Average Recurrence Interval y	Flow Rate u/s-median-d/s m/s	Velocity min-median-max m/s	Total Section Stream Power min-median-max N/m²	Total Section Shear Stress min-median-max N/ms
2	35 - 55 - 59	0.1 - 1.6 - 3.1	0 - 34 - 355	0.2 - 24 - 116
50	125 - 209 - 248	0.5 - 2.3 - 3.7	3.3 - 68 - 296	6 - 34 - 123
100	183 - 292 - 322	0.6 - 2.5 - 4.0	5 - 39 - 448	7 - 30 - 163

2.8.2 Infrastructure Corridor

A flood study was undertaken to estimate design flood levels and the extent of inundation under existing conditions along the infrastructure corridor. Full details of the methodology and results of the flood study are provided in Appendix D, and mapping of flood depths, velocities and extents for a range of flood events are provided in Appendix E.

Design flood flows in the unnamed tributaries of Native Companion Creek at the outlet of the model area are summarised in Table 2.7.

Table 2.7 Infrastructure Catchment Design Discharges

Average Recurrence Interval (Years)	Flow Rate (m^3/s)
2	421
50	710
PMF	4,239

Figure 2.35 shows the estimated extent and depths of inundation along the infrastructure corridor for the 50 year Average Recurrence Interval (ARI) flood event. The figure shows that flow is in a generally northerly direction via two broad connected flow paths, in which depths are generally less than 0.5m.

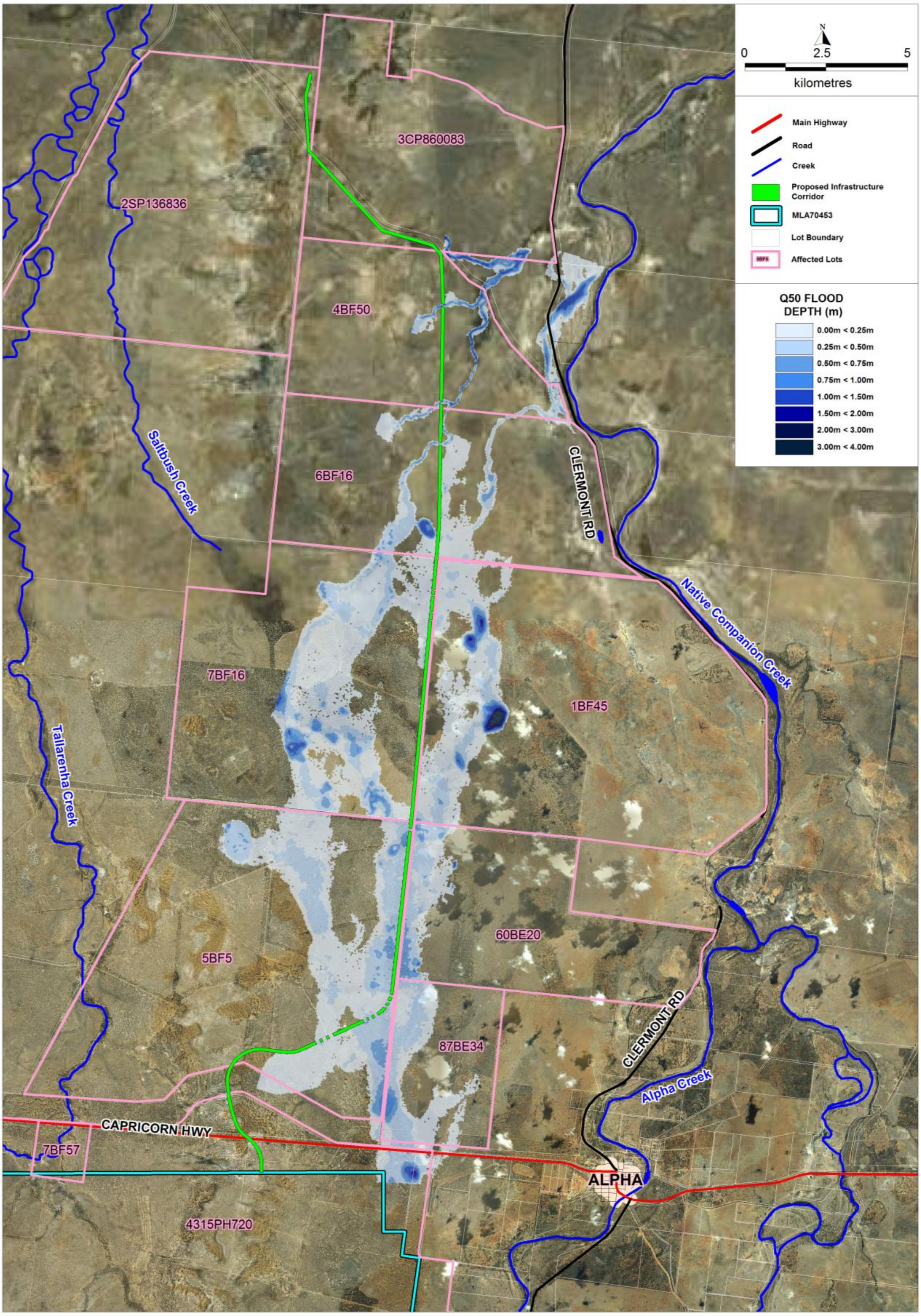


Figure 2.35 Infrastructure Corridor Flood Extent - 50 year ARI

3 APPLICABLE LEGISLATION

3.1 OVERVIEW

In undertaking these assessments, the key relevant Acts include:

- *Water Act 2000 (Water Act);*
- *Water Resource (Burdekin Basin) Plan 2007;*
- *Burdekin Basin Resource Operations Plan 2009;*
- *Water Supply (Safety and Reliability) Act 2008.*
- *Water Regulation 2002;*
- *Sustainable Planning and Other Legislation Amendment Act 2012 (SPOLAA).*
- *Environmental Protection Act 1994 (EP Act);*
- *Environmental Protection Policy (Water) 1997;*

3.2 WATER ACT 2000

In Queensland, the Water Act 2000 is the primary statutory document that establishes a system for the planning, allocating and using of non-tidal water. The Act is administered by the Department of Environment and Resource Management (DERM).

The Water Act prescribes the process for preparing Water Resource Plans (WRPs) and Resource Operation Plans (ROPs) for specific catchments within Queensland. Under this process, WRPs are prepared to identify a balance between waterway health and community needs, and to set allocation and management objectives. The ROPs provide the operational details on how this balance can be achieved.

The WRPs and ROPs determine conditions for granting water allocation licences, permits and other authorities, as well rules for water trading and sharing. The WRP sets Environmental Flow Objectives (EFOs) to protect waterway health, and Water Allocation Security Objectives (WASOs) to maintain community water supplies. The nearest WRP node of relevance to the SGCP for assessing EFOs and WASOs is well downstream of the site (at the Suttor R-Belyando R confluence).

A water licence is for the taking of and using water or interfering with the flow of water. Water licences are tied to the land, and are not tradable.

Under the Water Act, the preparation of land and water management plans may be required in specific areas. DERM has advised that there are no land and water management plans in place in the vicinity of the project.

3.2.1 Water Resource (Burdekin Basin) Plan 2007

The WRP provides a framework for managing and taking the water, and establishing water allocations. The plan WRP applies to:

1. water in a watercourse or lake;
2. water in springs not connected to:
    a. artesian water;
    b. subartesian water connected to artesian water.
3. overland flow water, other than water in springs connected to:
    a. artesian water;
    b. subartesian water connected to artesian water.

The project site is within the Belyando-Suttor sub-catchment area of the WRP. The site is not part of declared Water Management Areas.

The WRP identifies over 543,000ML of unallocated water that may be made available in the plan area. In addition to water that may be granted from the unallocated water reserves, permits may be issued for water required for short-term projects (such as the construction and maintenance of roads and bridges).

3.2.2 Overland Flow

Overland flow (OLF) is defined in the Water Act as follows:

..water, including floodwater, flowing over land, otherwise than in a watercourse or lake —
(a) after having fallen as rain or in any other way; or
(b) after rising to the surface naturally from underground.

It excludes:

(a) water that has naturally infiltrated the soil in normal farming operations, including infiltration that has occurred in farming activity such as clearing, replanting and broadacre ploughing; or
(b) tailwater from irrigation if the tailwater recycling meets best practice requirements; or
(c) water collected from roofs for rainwater tanks.

There is provision in the WRP for the taking of overland flow water to satisfy the requirements of an Environmental Authority issued under the Environmental Protection Act 1994. This provision is likely to apply to surface water intercepted by the site water management system to protect downstream water quality.

3.2.3 Burdekin Basin ROP

The Water Act states that all rights to the use, flow and control of all water in Queensland are vested in the State. Water cannot be legally taken or used unless it is authorised under a water entitlement (a water allocation or licence). The Burdekin Basin ROP sets down the rules by which water allocations and licences may be granted. A water allocation is defined under the Act as an authority to take water granted under Section 121 or 122 of the Water Act 2000. A water allocation can only be issued under an approved resource operations plan.

Unsupplemented water is water taken under a water allocation or water licence that is not managed under a Resource Operations License (ROL) or Interim Resource Operations License (IROL). All potential surface water supplies on watercourses in the immediate vicinity of the Project site are unsupplemented. Unsupplemented water management relates to:

- taking water under high stream flow conditions (water harvesting) within the bounds of a water supply scheme;
- taking water under any flow conditions outside of the bounds of a water supply scheme.

For unsupplemented water, a water allocation may be specified in terms of:

- the nominal volume of water for the allocation;
- the volumetric limit for the allocation;
- the location from which the water may be taken under the allocation;
- the purpose for which water may be taken under the allocation;
- the maximum rate for taking water;
- the flow conditions under which water may be taken;
- the water allocation group to which the allocation belongs.

There is no proposal to take water for the SGCP from a watercourse. However, there is potential for the project to impact on the reliability of supplies for licence holders downstream of the project. Figure 3.1 below shows the locations of water licence holders in the vicinity of the proposed project. There may also be other users who take unsupplemented water for stock or domestic purposes.

3.2.4 Structures and activities requiring approval under the Water Act

Where the bed and banks of watercourses will be disturbed by proposed works, licensing will be required under the Water Act. Once design of these structures is finalised, they must be submitted to DERM with an application for a Riverine Protection Permit and/or Water Licence application. Streams determined by DERM to be watercourses are shown in Figure 2.5.

The EIS identifies a number of proposed levees for flood protection. The authorisation of levee banks on mining tenements falls under the jurisdiction of the *Environmental Protection Act 1994*. However where they form plugs for the existing watercourses, some levees may be incorporated into the licensing of the watercourse diversions, and would be assessed under the Water Act 2000, in negotiation with DERM.

3.3 REFERABLE DAMS

Referable dams are legislated under the *Water Supply (Safety and Reliability) Act* 2008. The exact number and design details of referable dams (including levees) will not be finalised until the detailed design stage and during operations of the Project. An assessment of the population at risk (PAR) will be carried out for each dam (which does not contain hazardous waste) to determine if it meets the criteria for referable dams.

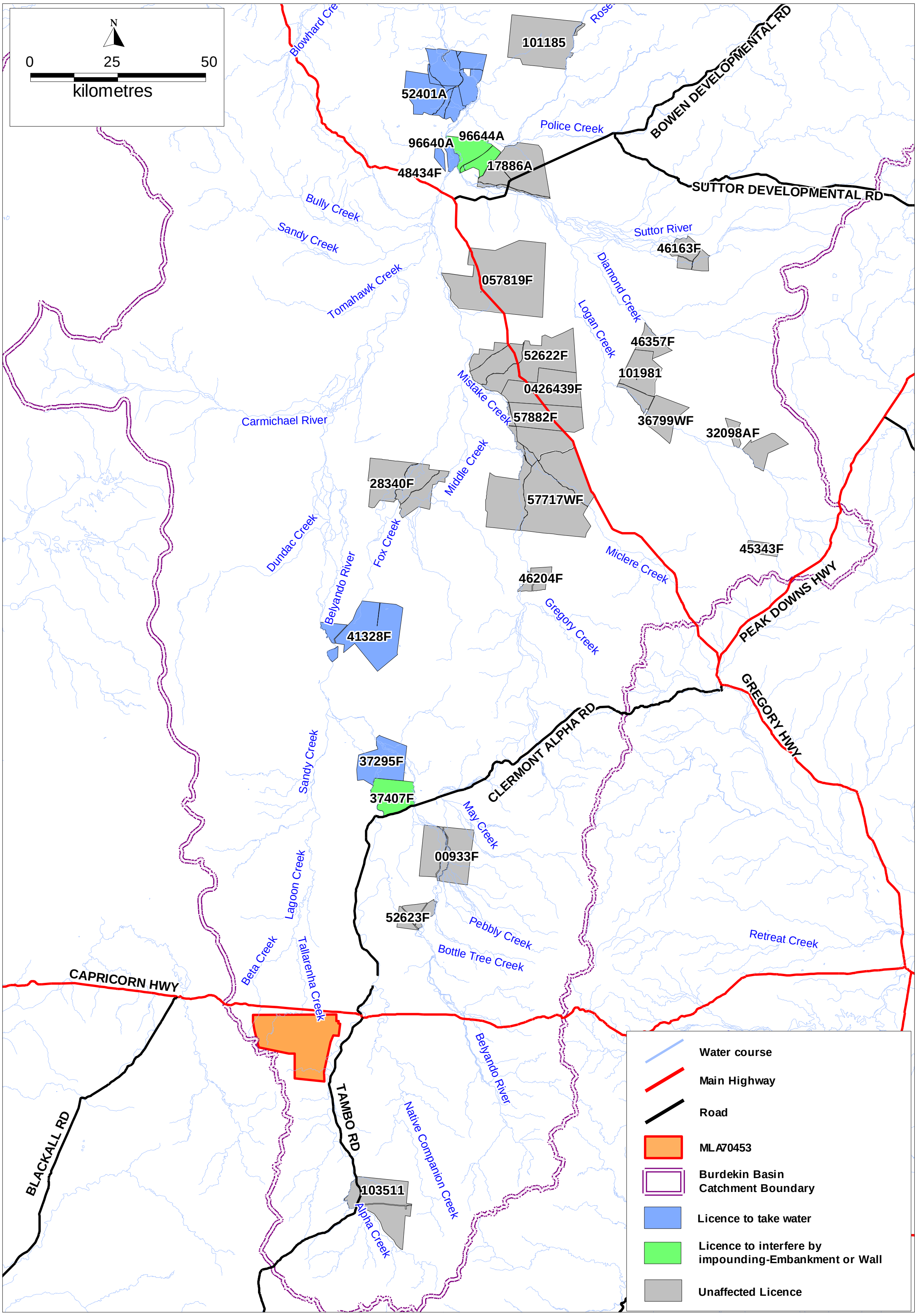


Figure 3.1 Downstream water users

3.4 ENVIRONMENTAL PROTECTION ACT (1994)

The *Environmental Protection Act* (EP Act) 1994 requires that an Environmental Management Plan (EM Plan) is prepared for mining activities and that an Environmental Authority (EA) issued by DERM is required for operations to proceed. Surface water management is regulated through these documents.

The EA conditions regulate dams containing hazardous waste. Surface water discharges, and associated monitoring are authorized and regulated through the EA conditions.

3.4.1 EPP Water

The Environmental Protection (Water) Policy is subordinate legislation under the Environmental Protection Act 1994.

The EPP Water seeks to protect Queensland's waters while allowing for development that is ecologically sustainable. Queensland waters include water in rivers, streams, wetlands, lakes, aquifers, estuaries and coastal areas.

This purpose is achieved within a framework that includes:

- identifying environmental values (EVs) for aquatic ecosystems and for human uses (e.g. water for drinking, farm supply, agriculture, industry and recreational use)
- determining water quality guidelines (WQGs) and water quality objectives (WQOs) to enhance or protect the environmental values.

The processes to identify EVs and to determine WQGs and WQOs are based on the National Water Quality Management Strategy (NWQMS, 2000), Implementation Guidelines (1998) and further outlined in the Australian and New Zealand Guidelines for Fresh and Marine Water Quality (2000). EVs and WQOs that are adopted by the Government for particular waters are included in Schedule 1 of the EPP Water. However, streams in the Burdekin Basin will not be scheduled until late 2013.

In the absence of scheduled EVs, the following have been adopted for the area within and downstream of the SGCP:

- Protection of aquatic ecosystems;
- Suitability for recreational use and aesthetics, including fishing activities;
- Cultural and spiritual values; and
- Suitability for primary industrial uses, including irrigation and stock drinking water.

4 POTENTIAL IMPACTS AND MITIGATION MEASURES

4.1 DESCRIPTION OF OPERATIONS

4.1.1 Open Cut Mining

The project will feature four open cut pits mined using strip mining methods along a total strike length of approximately 14 km. The waste rock and coal will be extracted in a series of parallel north-south ‘strips’. Mining will commence in the south-east of the mining area and proceed from east to west.

Overburden will be removed using draglines. During establishment of the open-cut pits, waste rock initially will be stockpiled in waste rock emplacements immediately to the east of the pits. Waste rock will then be spoiled in previous strips. Reject material from washing the coal will also be dumped within the dragline spoil piles. Rehabilitation of the waste rock dumps will be undertaken progressively.

The mine layout and water management system at Year 33 of the mine operation are shown in Figure 4.1.

4.1.2 Underground Mining

The underground mining operations will commence in 2017 and will continue for the life of the SGCP. Underground operations will utilise the longwall mining method.

The underground mine will be a multi-seam operation, with the top seam (D1) being mined first, followed by the lower D2 seam. The minimum depth of cover will be 140m.

The southernmost open pit has been designed to facilitate access to the underground mine area via a boxcut. Seven headings have been designed from the boxcut to the D1 seam. Access to D2 seam will be by short inter-seam drifts from D1 to D2, and subsequently seven heading mains will be developed in the D2 seam. The separation distance between the seams is approximately 9m to 17m. Coal will be extracted in panels 350m wide, and up to 5,000m in length.

A series of pillars will be left in place to support the overlying strata and protect the roadways as mining proceeds, with mains pillars being approximately 60m x 30m and gateroad pillars approximately 125m x 25m.

Underground operations will result in subsidence of the overlying ground surface. The predicted subsidence impacts are illustrated in Figure 4.2

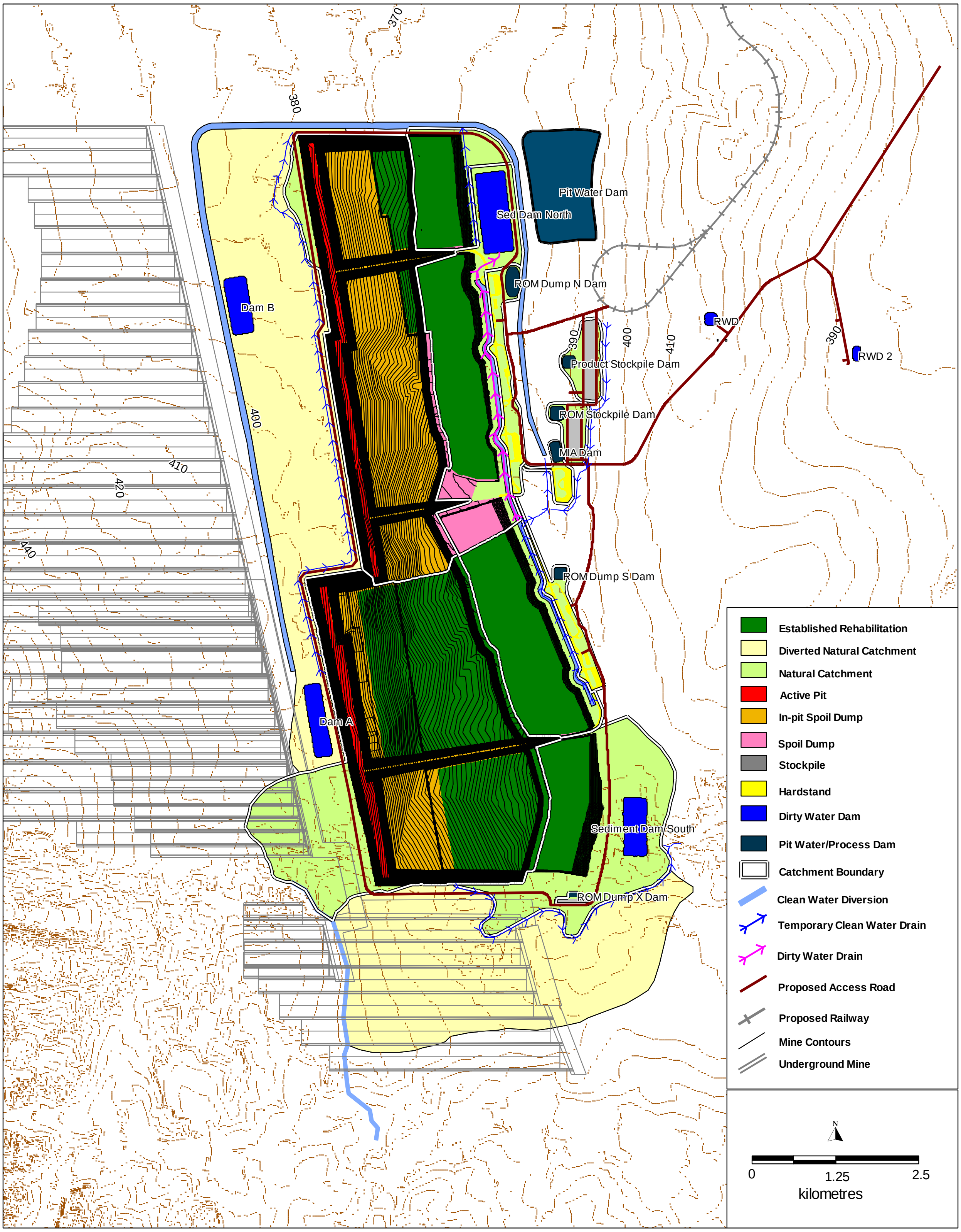


Figure 4.1 Year 33 Mine Layout and Water Management System

4.1.3 Coal Handling

Run of Mine (ROM) coal from open cut mining will be hauled by truck to one of two main ROM dump stations. Transfer conveyors will transport coal to the sizing station, before being transported by overland conveyor to the raw coal stockpiles located near the Coal Handling and Preparation Plant (CHPP). The raw coal stockpile area will receive both open cut and underground ROM coal and will consist of four separate stockpiles.

ROM coal from the underground mining operations will be transferred via drift conveyors to a centralised underground ROM stockpile located in the boxcut area. Coal will then be conveyed to the raw coal stockpile area located on the surface, near the CHPP.

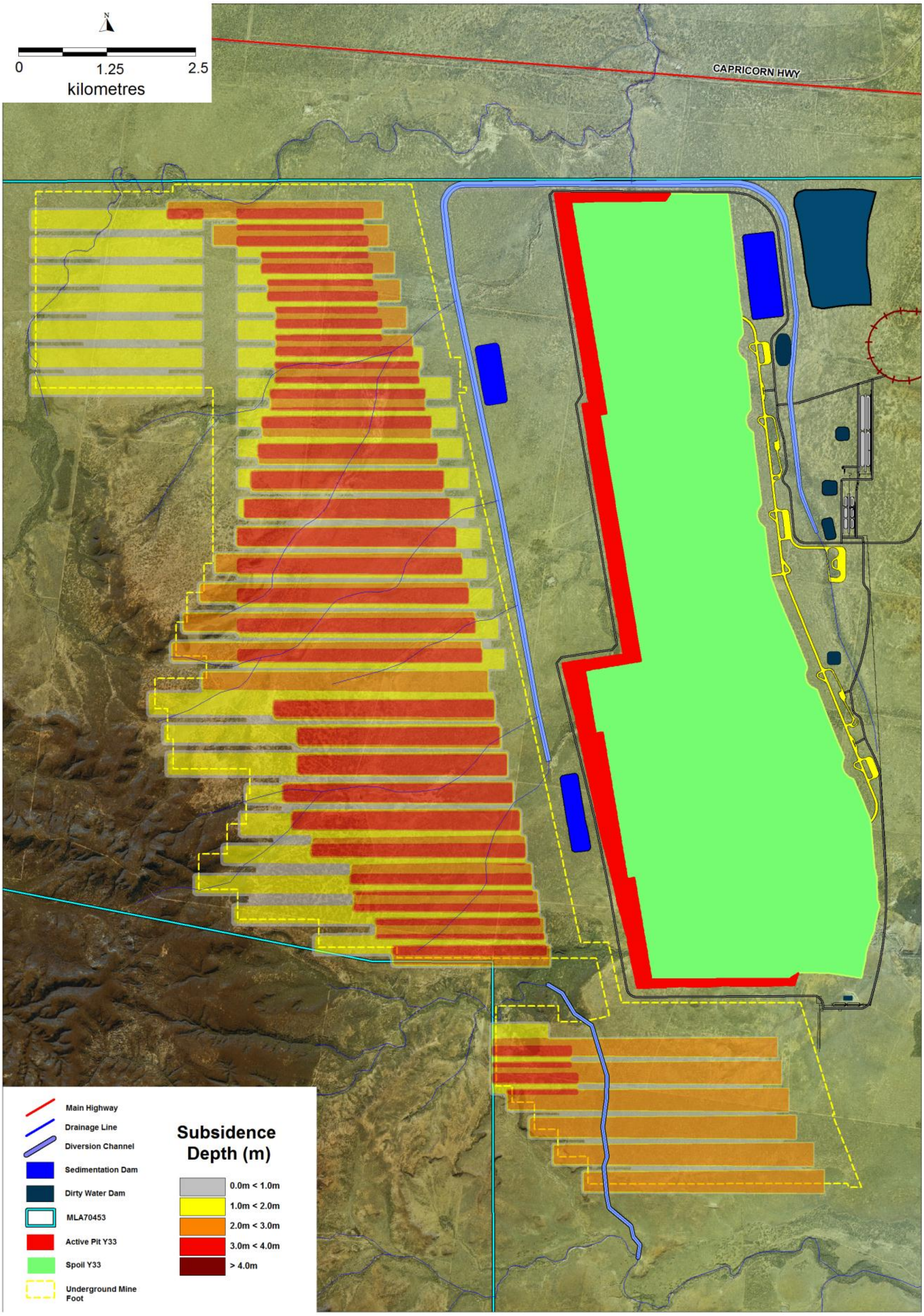


Figure 4.2 Year 33 Predicted Subsidence Impacts

4.2 WATER DEMANDS

Estimated water demands over the project life are summarised in Table 4.1. The demand peaks around Year 10 at 5,172 ML/a. Further details are provided in the following sections, and in Appendix A.

Table 4.1 Mine Site Water Demand Summary

	Average Annual Demand (ML)				
Demand	**Year 1**	**Year 4**	**Year 10**	**Year 20**	**Year 33**
CHPP	656	2,103	3,298	3,037	1,846
Haul Road Dust Suppression	336	456	889	917	1,051
Stockpile Dust Suppression	300	300	300	300	300
Potable Demand	62	84	84	84	84
Underground (Potable)	0	470	470	470	470
Misc. + Vehicle Wash	131	131	131	131	131
Total	**1,485**	**3,544**	**5,172**	**4,939**	**3,882**

4.2.1 Raw Water

Construction

Raw water for construction activities will be sourced from groundwater bores located within MLA 70453. On-site raw water dams will be constructed to store water from these approved bores in order to maintain 7-day supply.

Operation

The CHPP will be operated as a dry tailings system until the end of Year 3 and as a wet tailings system from Year 4 onwards. The estimated CHPP demand during the dry tailings is estimated to be 117L/tonne of ROM (dry) and during the wet tailings phase will be 177L/tonne of ROM (dry).

Dust suppression requirements at South Galilee Coal comprise haul road dust suppression and stockpile dust suppression. The adopted stockpile dust suppression is 300ML/a over each modelled stage. This is based on an assumed demand of 100ML/a per coal stockpile, of which there are three in each stage.

Operational raw water demands will be supplied using water captured in the site water management system as a priority. Any shortfall will be supplied from the raw water pipeline (from an external water source).

A raw water dam and associated pipelines will be constructed on-site to store and supply raw water during operations.

4.2.2 Potable Water

Construction

An on-site water treatment plant will be constructed to treat groundwater to supply up to approximately 225 ML/a of potable water for the construction workforce and accommodation facilities.

Operation

Up to approximately 84 ML/a of potable water will be required for domestic and underground mining activities. A water treatment plant will be constructed near the Raw Water Dam to supply potable water. Potable water will be stored in two water tanks, one to supply the accommodation village and one to supply the mine site.

Water demand for underground mining purposes is likely to be approximately 470ML/a from Year 3. Underground mining equipment requires high quality water, and it has been assumed that it would be treated to potable standards. Should water for underground mining activities not be required to meet the same standards as potable water, a separate water treatment system may be constructed.

4.2.3 Waste Water

A waste water treatment plant (WWTP) will be located on-site. Waste water and sewage (from the Mine Industrial Area (MIA), CHPP and accommodation village) will report to the WWTP for treatment. Facilities isolated from the sewage network (e.g. underground mine receiving centre) will operate on septic systems which will be collected periodically and transported by tanker to the WWTP. Treated waste water will be piped to an on-site storage dam for reuse where approved.

The potential impacts on surface water during the life of the project are discussed in Section 4.3.

4.3 POTENTIAL SURFACE WATER IMPACTS

4.3.1 Changes in Runoff Water Quality

Land disturbance associated with mining has the potential to adversely affect the quality of surface runoff by increasing sediment loads and transporting contaminants from waste rock and coal seams.

The results of the geochemical overburden assessment (EGi, 2011) indicate that the bulk of the overburden and interburden material is likely to be non-acid forming (NAF). The roof within 5m of the upper coal seam appears to be the main potentially acid-forming (PAF) horizon of concern, with a number of other lower capacity PAF horizons associated with coal seams and interburden.

Final pit floor material is likely to be low capacity PAF. ROM coal and washery wastes are also likely to be mainly PAF.

Water extract testing indicates that once acid conditions develop, elevated concentrations of dissolved Al, Co, Cu, Fe, Mn, Ni, SO_4 and Zn are likely to occur.

All PAF material will be selectively handled where practicable to ensure that the potential for acid drainage (ARD) is limited. Once all PAF material has been placed, a 10m cover of NAF material will be applied over the entire waste rock emplacement area to ensure that the PAF waste is not exposed.

Salinity testing of the overburden material yielded EC1:5 values ranging from 40 to 3,130μS/cm with approximately half the samples falling within the non-saline to slightly saline range with an EC of 300 μS/cm or less. Eleven of the remaining samples were saline (>600 μS/cm). Results indicate a general lack of immediately available acidity and salinity in the samples except where partial oxidation of pyrite has occurred. Hence control of ARD will largely control salinity.

Contaminant concentrations in Pit Water have significant potential to adversely impact downstream environmental values if it is released into the environment.

If managed appropriately, environmental risks from the release of runoff from waste rock emplacements are lower. However, there is potential for environmental harm if contaminant concentrations increase over time

4.3.2 Changes to Downstream Water Quantity

Water captured in the open pit and the associated water management system will be reused on site. The proposed water management system will contain the bulk of these inflows on site for preferential reuse. As a result, streamflow in the receiving waters (Tallarenha Creek, Sapling Creek and Alpha Creek) will be reduced.

Mine-induced subsidence will potentially result in the formation of pools within the channels of Tallarenha Creek and its tributaries, as shown in Figure 4.3. The potential evaporation and seepage from these pools could potentially reduce streamflow in the downstream reaches of Tallarenha Creek.

Subsidence-induced cracking of the Tallarenha Creek and Sapling Creek catchments could also potentially result in enhanced infiltration and subsequent loss of streamflow in the receiving waters.

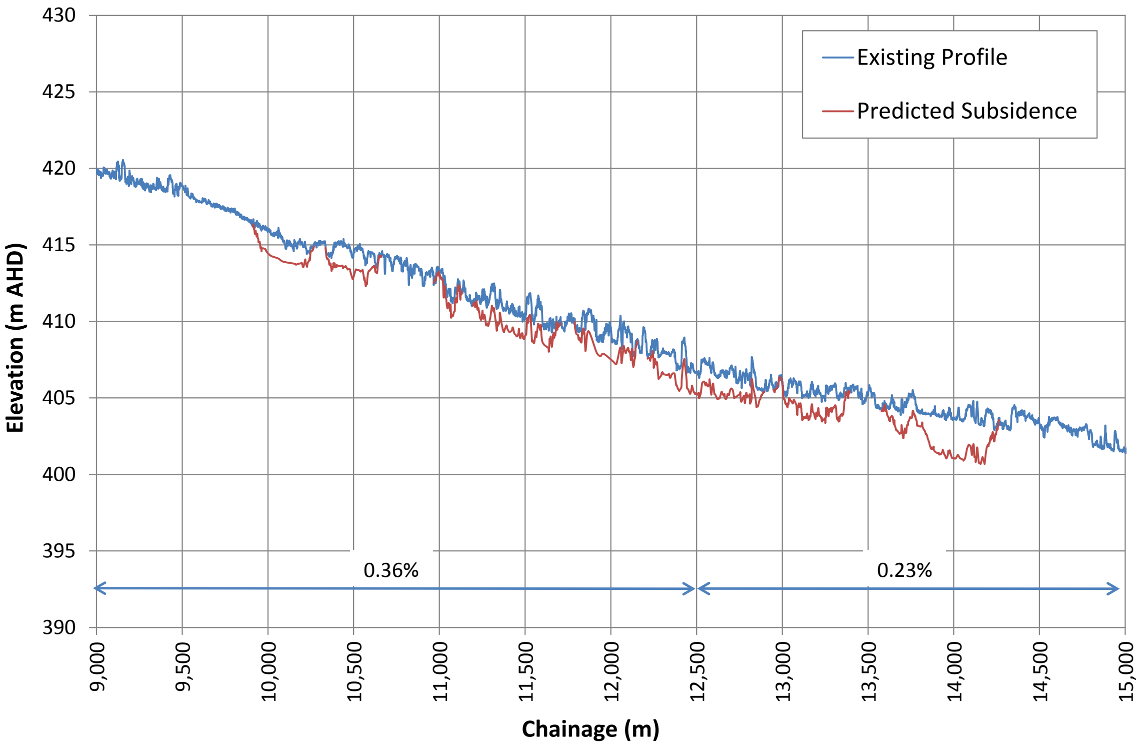


Figure 4.3 Tallarenha Creek Channel– Subsidence Impact

4.3.3 Changes to Flooding Conditions

The proposed open cut pit will intersect a number of tributaries of Tallarenha Creek. To minimise the potential volumes of water coming into contact with disturbed areas, a channel and levee system will be constructed to the west of the proposed highwall to protect the open cut from flooding. Another channel will be required to divert an eastern tributary of Tallarenha Creek east around the mine workings. These changes will result in significant local changes to the pattern of flooding in these tributaries.

The southern end of the proposed open cut crosses Sapling Creek, and as a result, the upper reach is to be diverted south into Dead Horse Creek. The diversion will comprise a diversion plug or levee, which will direct streamflow into the diversion. Peak flood flows in Dead Horse Creek, and consequently peak velocities will be increased. This will result in higher flood levels and an increased potential for erosion in Dead Horse Creek.

Subsidence induced by the proposed underground operations will impact on the channel and floodplain of Tallarenha Creek and its tributaries, this will impact on the pattern of flooding in this area (as shown in Figure 4.4). The Sapling Creek diversion will also be undermined by the proposed underground workings. The resultant subsidence will potentially affect flood flows in the diversion.

4.3.4 Changes to Sediment Movement

A Quarry Material Allocation Notice (QMAN) is held for removing material from Lagoon Creek (which is downstream of Tallarenha Creek).

The movement of sediment through watercourses could be affected by mine subsidence-induced changes to the profile of Tallarenha Creek, and the Sapling Creek diversion.

However, the works proposed to ensure that the stream channels remain free-draining following subsidence, should ensure that the movement of sediment is not significantly restricted by the expected subsidence.

4.4 MITIGATION MEASURES

4.4.1 Erosion and Sediment Control

In the operational phase, progressive rehabilitation of the waste rock emplacements will minimise the potential generation of sediment. And the site water management system will reduce the risk of discharge of sediment into the receiving waters.

An Erosion and Sediment Control Plan will be developed and implemented throughout construction and operations. A ‘best practice’ approach will be adopted which is consistent with the International Erosion Control Association (IECA) recommendations. The following broad principles will apply:

- minimise the area of disturbance;
- where possible, apply local temporary erosion control measures;
- intercept run-off from undisturbed areas and divert around disturbed areas;
- where temporary measures will be ineffective, divert run-off from disturbed areas to sedimentation basins prior to release from the site.

4.4.2 Flood Protection

Highwall and Lowwall protection works

The conceptual design of the proposed flood protection works is shown in Figure 4.4.

Levees are proposed to prevent flow down the Tallarenha Creek tributaries into the mining area, and a north-south channel collects flow and diverts it north around the pit back to Tallarenha Creek. During operations, the levees will be designed to protect the pit from flooding up to the 3000yr average recurrence interval (ARI) flood event. Before mine closure, the levees will be upgraded to protect the pit from flooding up to the Probable Maximum Flood. The channel will be sized in accordance with the hydraulic performance criteria specified in the DERM document, *Central West Water Management and Use Regional Guideline: Watercourse Diversions* (DERM 2008).

The longitudinal profiles of peak flood levels for a range of design flood events along leveed reaches of Tallarenha Creek is illustrated in Figure 4.5. Details of the flood modelling undertaken to estimate design flood levels are provided in Appendix B. The modelling results show that the loss of catchment to the open cut area will result in some reduction in design flow rates and levels downstream of the project. The duration of flooding will be essentially unchanged by the SGCP.

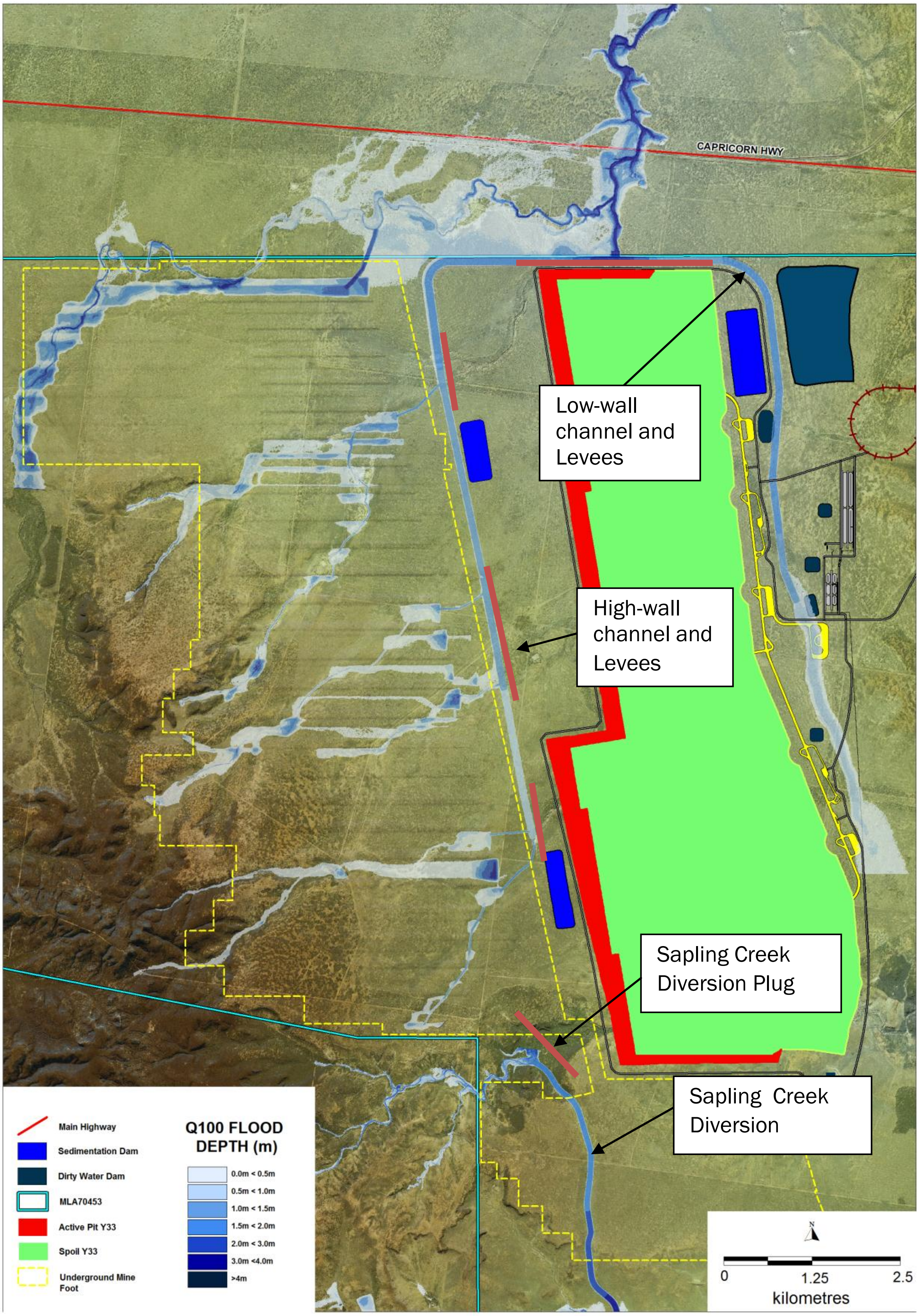


Figure 4.4 Extent and Depth of 100y ARI Flooding and Proposed Flood Protection Works

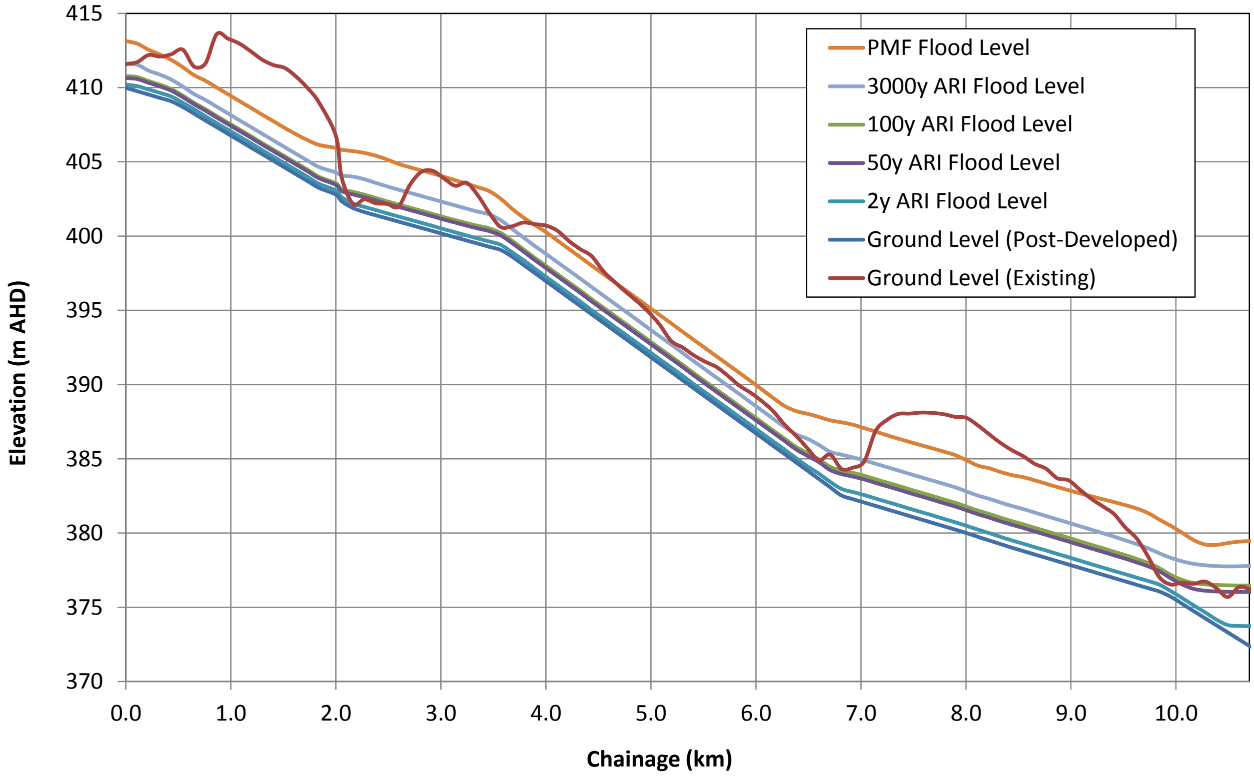


Figure 4.5 Design Longitudinal Profile – Western Highwall channel

Sapling Creek Diversion

A conceptual design of the diversion has been prepared for impact assessment purposes. The design was prepared to ensure the hydraulic design criteria set out in the DERM Guideline, *Central West Water Management and Use Regional Guideline: Watercourse Diversions* (DERM 2008) are not exceeded.

The design will be the subject of further detailed studies to be conducted as part of the Detailed Feasibility Study (DFS) and as part of the diversion licensing process under the Water Act 2000.

Potential alignment options for the proposed Sapling Creek diversion are limited due to the locations of the underlying coal resource, associated mining activities, and the relatively steep topography in the immediate vicinity of the pit. The selected preliminary alignment is shown in Figure 4.6.

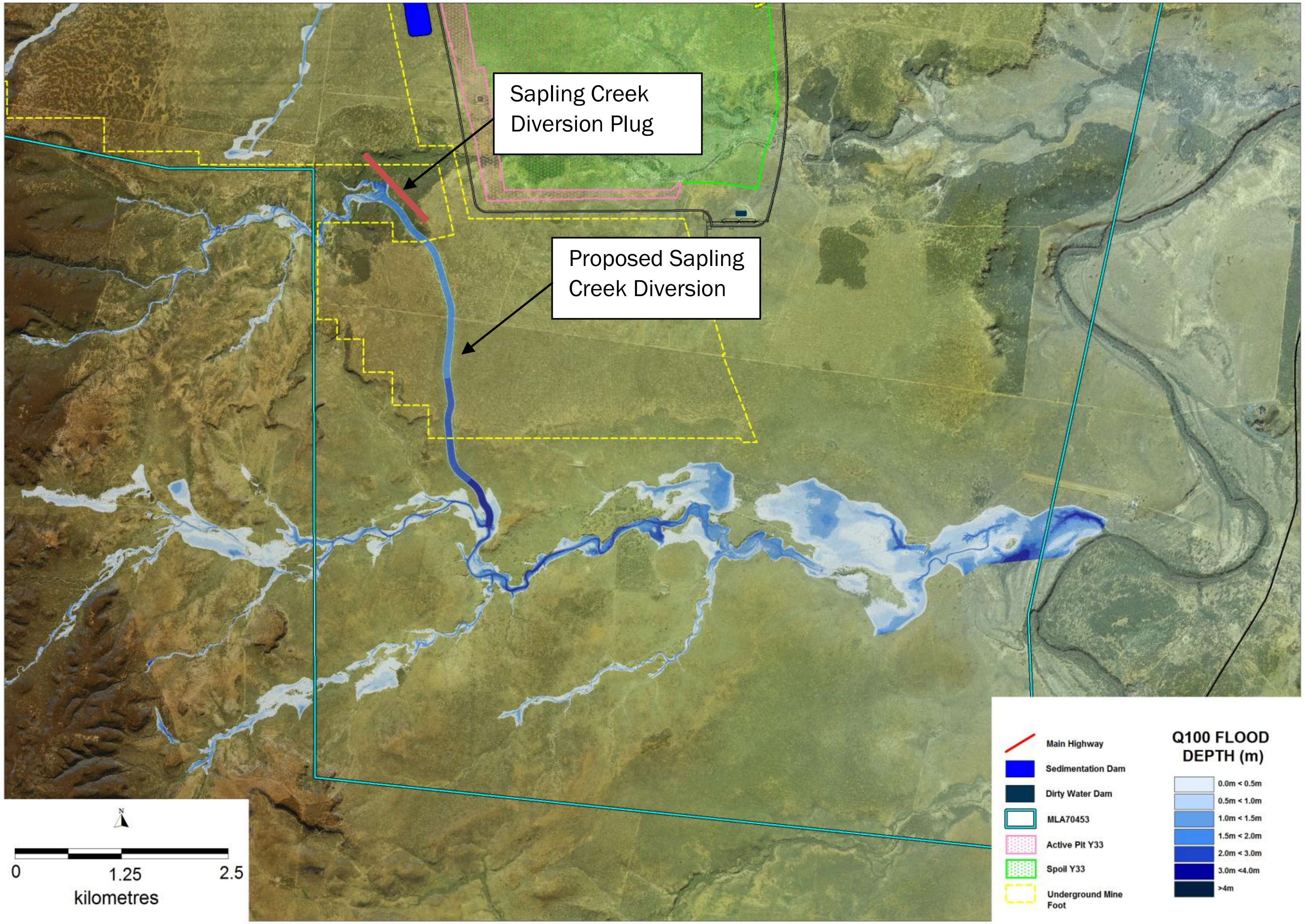


Figure 4.6 Sapling Creek Diversion – Horizontal Alignment

The adopted alignment is the shortest possible, but because the outlet is located relatively high in Dead Horse Creek, the resultant slope is significantly less than the diverted reach of Sapling Creek. This is illustrated in Figure 4.7, which shows the slope of the diversion is approximately 0.1% while the adjacent reach is at 0.39%.

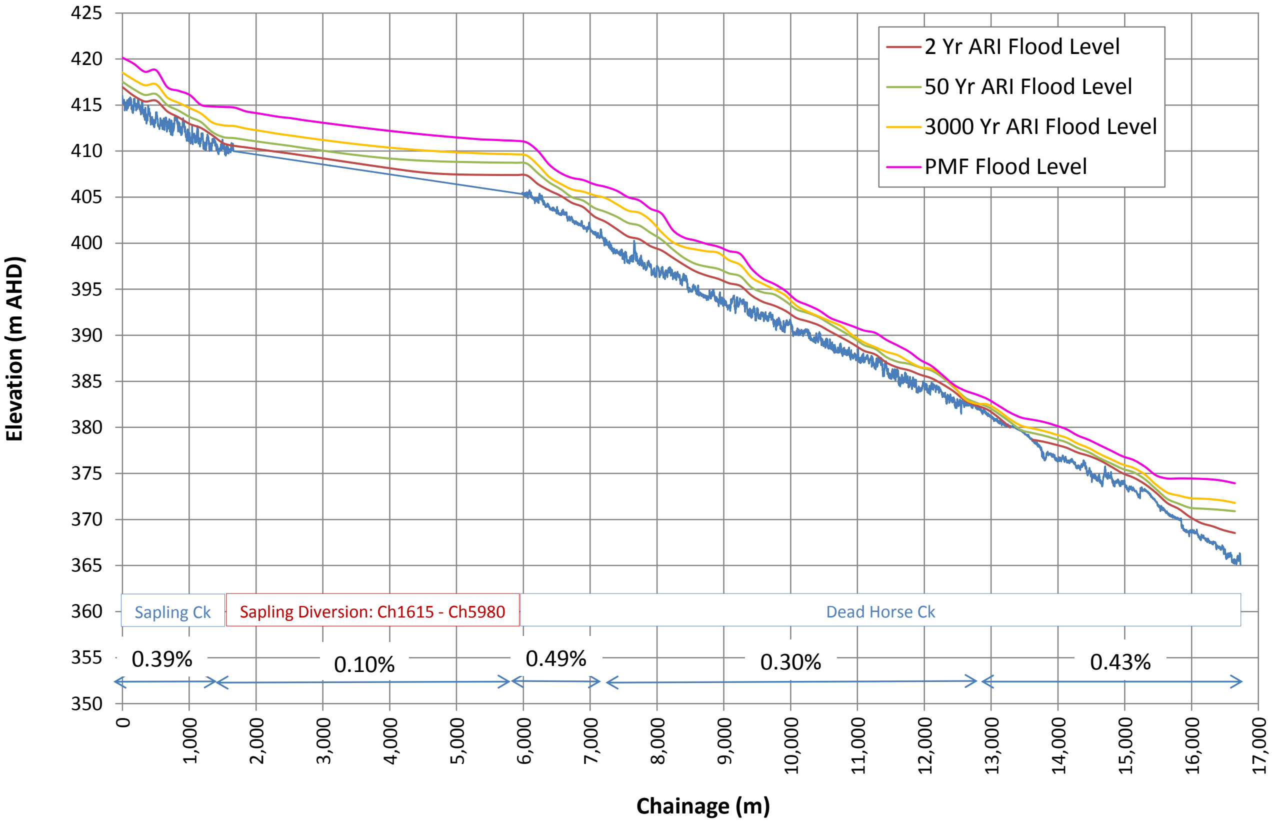


Figure 4.7 Design Invert and Flood Levels - Sapling Creek Diversion

The diversion will be constructed as a compound trapezoidal channel, with a narrow, shallow channel conveying low flows. Channel meanders will be provided if required to mimic conditions in the existing channel. Based on the channel dimensions in the adjacent reaches of Sapling Creek, the low flow channel will be approximately 4m wide at the base and 1m deep (top width 10m). The proposed channel cross-section is compared to part of the existing Sapling Creek channel geometry in Figure 4.8. In the absence of detailed geotechnical studies, the preliminary channel sideslope is 1Vertical:3Horizontal. In practice, as the proposed diversion channel is very deep (exceeding 20m for much of its length), the upper sections of the cut slope may need to be benched to achieve appropriate stability,

The design hydraulic conditions summarised in Table 4.2 for the conceptual design are well below the design guideline values, and the naturally occurring conditions. The diversion channel itself is therefore likely to be relatively stable.

Erosion of the channel will be managed through revegetation with native grasses and locally occurring trees and shrubs.

Details of the flood modelling undertaken to assess hydraulic conditions in the diversion are provided in Appendix D. Mapping of the model results are provided in Appendix E.

Table 4.2 Sapling Creek Diversion Design Flow Conditions

Average Recurrence Interval y	Flow Rate min-median-max m/s	Velocity min-median-max m/s	Total Section Stream Power min-median-max N/m²	Total Section Shear Stress min-median-max N/ms
2	29 – 31 - 31	0.3 – 1.0 – 1.1	0.1 – 3.4 – 4.5	0.6 – 5.9 – 7.2
50	121 – 127 - 127	0.7 – 1.3 – 1.6	0.9 – 9 - 15	2.3 – 11 – 16
100	154 – 162 - 162	0.7 – 1.4 – 1.7	1.2 – 11 - 18	3.0 – 13 - 18

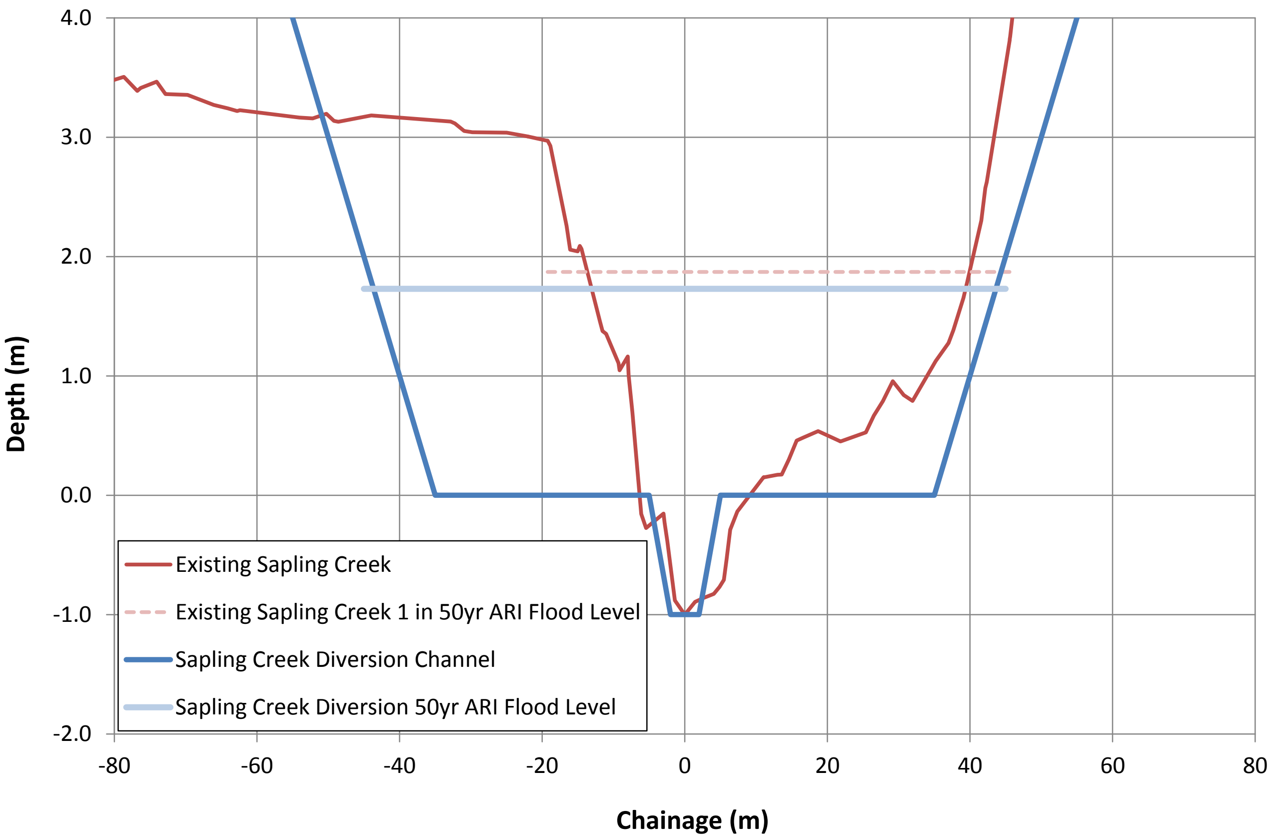


Figure 4.8 Comparison of Cross-sections of Sapling Creek Diversion Channel and Existing Channel in Diverted Reach

4.4.3 Change in Dead Horse Creek Flood Flow Conditions

The diversion of the upper catchment of Sapling Creek into Dead Horse Creek will see peak design flood flows in Dead Horse Creek increase by approximately 47%. This in turn will increase peak velocities and the potential for erosion of the downstream reaches of Dead Horse Creek.

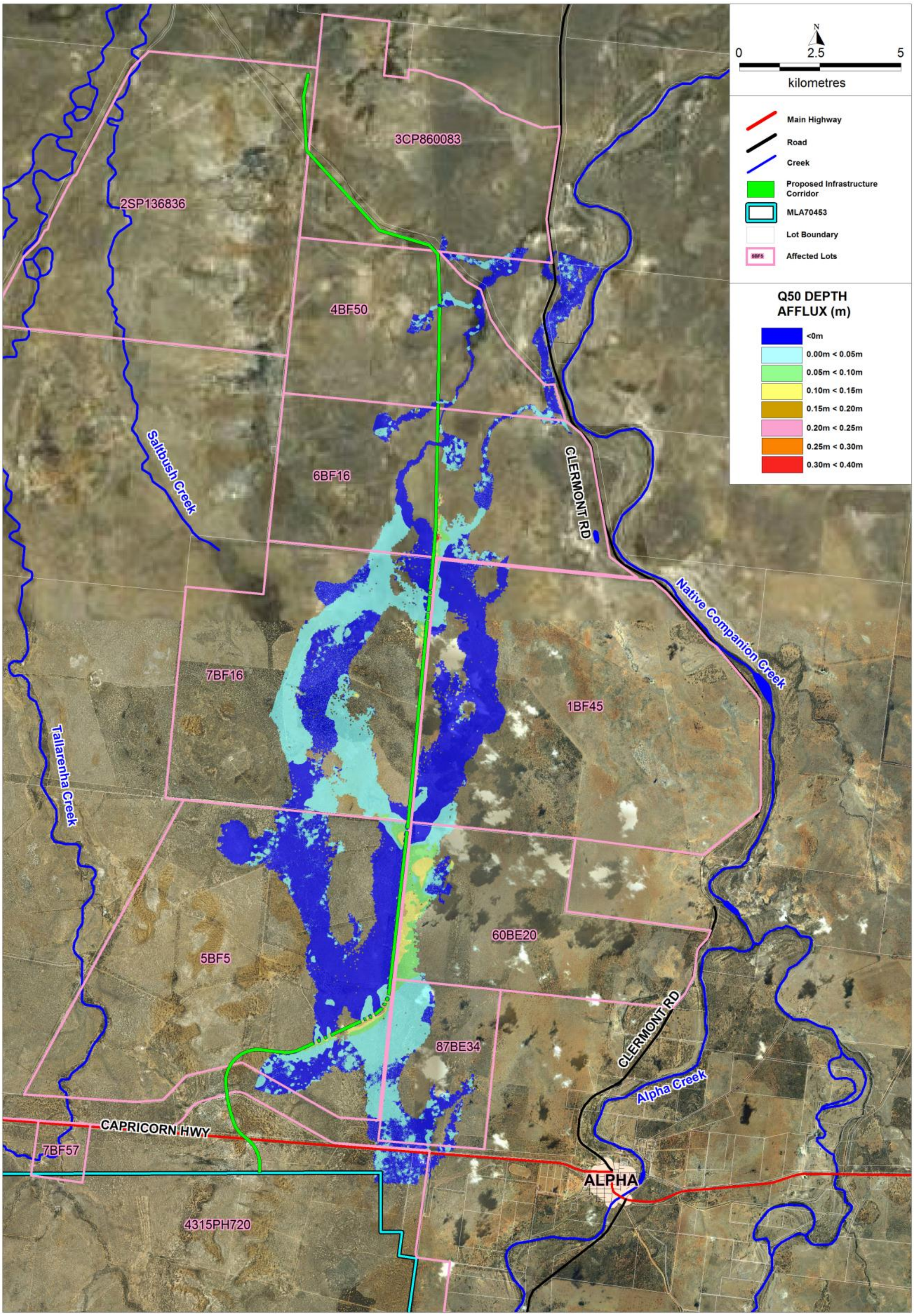
The flood modelling results presented in Appendix D show that in the 2 year ARI flow, conditions downstream of the diversion will exceed guideline values in short reaches less than 500m long, where existing conditions already exceed guidelines. In the 50 year ARI flow, a 3km reach between Chainage 5500m and Chainage 8500m will experience stream power increases of over 30%, in a reach where existing conditions already exceed guideline values significantly. The changes will result in an increased potential for erosion in Dead Horse Creek, but the impacts will depend on the erosion resistance of the underlying geology.

Further investigations will be required during the detailed feasibility design phase to ensure that the potential impacts can be managed effectively. A monitoring program will be established to measure changes to the stream geometry so that additional erosion control measures can be implemented if necessary.

4.4.4 Impacts of Infrastructure Corridor on Flood Flow Conditions

The proposed rail embankment structure along the infrastructure corridor has the potential to change the flooding characteristics in the area. These potential impacts will be mitigated by providing cross-drainage structures along the embankment to maintain existing flow conditions.

The hydraulic model of the infrastructure corridor area was modified to incorporate the proposed embankment, and openings were included at key locations such as creek crossings and high-flow areas to maintain existing flow patterns in large floods. The flood model results presented in Appendix D show that the proposed arrangement of cross-drainage structures would ensure that the impact on flood depths would be minimal. During detailed design, the cross-drainage arrangements may be further optimised while maintaining flood management outcomes if required.


0
2.5
5
kilometres
Main Highway
Road
Creek
Proposed Infrastructure Corridor
MLA70453
Lot Boundary
Affected Lots
Q50 DEPTH AFFLUX (m)
<0m
0.00m < 0.05m
0.05m < 0.10m
0.10m < 0.15m
0.15m < 0.20m
0.20m < 0.25m
0.25m < 0.30m
0.30m < 0.40m
3CP860083
2SP136836
4BF50
6BF16
7BF16
1BF45
5BF5
60BE20
87BE34
7BF57
4315PH720
Saltbush Creek
Tallarenha Creek
Native Companion Creek
Alpha Creek
CLERMONT RD
CAPRICORN HWY
ALPHA

4.4.5 Water Management System

A conceptual design of the SGCP water management system (WMS) has been prepared. Details of the system will be finalised in later stages of design, but the system will comprise the elements described in the following sub-sections and meet the design criteria set out in the Environmental Authority and the performance targets outlined in this report.

The proposed system comprises three separate sub-systems, according to the water quality characteristics of the site catchments:

Saline Water System
This system manages catchment runoff which is potentially coal-affected. Water captured in this system is expected to have high salinity, and will potentially have elevated concentrations of dissolved metals. Water in this system will be pumped to the Pit Water Dam with a goal of containing all water on site for later reuse. Dams forming part of this system are:

- Pit Water Dam;
- ROM Dump X Dam;
- ROM Dump S Dam;
- MIA Dam;
- ROM Stockpile Dam;
- Product Stockpile Dam;
- ROM Dump N Dam;
- Dam A;
- Dam B.

The locations of the above dams are shown in Figure 4.1 and Appendix B.

Waste Rock Runoff Water System
This system manages runoff from waste rock areas, which are expected to have high turbidity, and a risk of moderately elevated salinity, and a lower risk of elevated metal concentrations. Dams forming part of this system are:

- Sediment Dam South;
- Sediment Dam Central;
- Sediment Dam North;
- Dirty Water Dam.

The locations of the above dams are shown in Figure 4.1 and Appendix B.

Raw Water System
Water supplied from the raw water pipeline is expected to have low salinity levels, will be managed separately to all other waters, and stored in the Raw Water Dam.

Clean Water System
Clean water from undisturbed catchments will be diverted around the active mining areas to minimise the volume of dirty water generated and captured in the site water management system.

WMS Staging
The WMS layout will evolve over the 33 year mine life. As additional catchments are disturbed, new sediment dams will be brought on line to treat waste rock runoff. Figure 4.1 shows the site water management system at the end of the mine life. For the purposes of concept design, and impact assessment, a number of stages have been assumed to be in place as follows:

Table 4.3 Mine Stages

Stage Label	Period	Duration
Stage Y1	2013-2015	3 years
Stage Y4	2016-2016	1 years
Stage Y5	2017-2021	5 years
Stage Y10	2022-2026	5 years
Stage Y15	2027-2031	5 years
Stage Y20	2032-2036	5 years
Stage Y25	2037-2041	5 years
Stage Y30	2042-2024	5 years
Stage Y33	2045-2047	2 years

Plans showing the conceptual water management layout at each of these stages are provided in Appendix B.

For the purposes of the impact assessment, the operational catchment areas have been classified into different types based on their hydrological and geochemical characteristics. The adopted areas are summarised in Table 4.4. Further details are provided in Appendix B. The area intercepted by the planned SGCP water management system increases from 1,238ha to 3,910ha over the mine life.

Table 4.4 Changes in Catchment Areas and Land Use Types Intercepted by WMS

Stage	Land Use Classification (ha)							Total Area (ha)
	Natural (grassed)	Stockpile	Hardstand	Active Pit	In Pit Dump	Out Pit Dump	Established Rehabilitation	
Year 1	811	43	34	87	0	263	0	1,238
Year 4	685	43	34	153	273	508	0	1,696
Year 5	1,502	43	65	222	355	768	0	2,956
Year 10	1,690	43	65	312	620	561	365	3,657
Year 15	1,584	43	65	289	714	386	683	3,764
Year 20	1,409	43	65	312	808	144	1,068	3,851
Year 25	1,410	43	63	411	758	162	986	3,833
Year 30	1,122	43	63	351	880	110	1,266	3,836
Year 33	1,048	43	63	328	1,048	111	1,268	3,910

The areas draining to each of the proposed water management dams are summarised in Table 4.5. As the area draining to the mine pits increases as the mine life evolves, the storage volume requirement for the pit water dam will increase. The proposed Pit Water Dam storage capacity will not be required until later stages of mining. It is therefore proposed to stage construction of the Pit Water Dam.

A preliminary assessment of the Hazard Category of the proposed dams has been undertaken in accordance with the failure to contain criteria in the *Draft Manual for Assessing Hazard Categories and Hydraulic Performance of Regulated Dams constructed as part of environmentally relevant activities pursuant to the Environmental Protection Act 1994* Version 3.1 (DERM, 2012) (the DERM Dams Manual).

Water captured in the Saline Water Management System has the potential to come into contact with runoff from acid-forming material, and may contain elevated metal concentrations. All dams in the saline water management system have therefore been assigned a "Significant Hazard Category" for the purpose of establishing the design storage capacity requirements for this assessment.

Water captured in the Waste Rock Runoff System is likely to be of a higher quality than that captured in the Saline Water System. Based on the results of the geochemical overburden assessment, and experience with similar systems in the Bowen Basin, it is unlikely that when operated as proposed, the contents of these dams will exceed the contaminant concentrations triggering a "Significant Hazard" rating under the DERM Dams Manual, and the sediment dams would not be deemed "regulated dams" under the "failure to contain" hazard assessment.

Notwithstanding this assessment, given the potential uncertainty in Waste Rock Runoff quality, it is proposed to design and operate both the Saline Runoff and Waste Rock Runoff System in accordance with the "Design Storage Allowance" (DSA) provisions of the DERM Dams Manual, unless future water quality assessments demonstrate that this is not required.

Under typical site Environmental Authority water conditions, the DSA must be provided in the dam as at 1 November each year to prevent a discharge to an annual exceedance probability specified in the DERM Dams Manual. The manual makes provision for distributing the DSA across multiple regulated dams in an Integrated Water Management System, so long as the system is operated in accordance with a certified system design plan. If the DSA is to be shared between two or more regulated dams, then each dam must be capable of accommodating at least 20% of that dam's individual DSA volume for its catchment on 1 November.

Table 4.5 Catchment Areas Draining to Site Water Storages and Pits

	Catchment Area (ha)								
	2013	2016	2017	2022	2027	2032	2037	2042	2045
Water Storage	**Y1**	**Y4**	**Y5**	**Y10**	**Y15**	**Y20**	**Y25**	**Y30**	**Y33**
North Pit	0.0	0.0	337.0	472.7	581.6	581.5	981.4	981.7	1,053.2
South Pit	427.1	621.5	621.2	1,323.7	1,324.3	1,408.4	1,530.1	1,532.0	1,531.4
Sed Dam N	0.0	0.0	831.9	693.1	693.1	693.1	517.3	517.5	520.5
ROM Dump N Dam	0.0	0.0	90.4	90.4	90.4	90.4	89.9	89.9	89.9
Product Stockpile Dam	54.3	54.3	54.3	54.3	54.3	54.3	54.3	54.3	54.3
ROM Stockpile Dam	38.3	38.3	38.3	38.3	38.3	38.3	38.3	38.3	38.3
MIA Dam	25.1	25.1	25.1	25.1	25.1	25.1	25.1	25.1	25.1
Sed Dam Central	0.0	361.8	361.8	361.8	359.4	361.8	0.0	0.0	0.0
ROM Dump S Dam	57.5	57.5	58.0	58.0	58.0	58.0	58.0	58.0	58.0
Sediment Dam S	629.5	531.2	531.2	533.2	533.2	533.2	532.5	532.3	533.2
ROM Dump X Dam	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5
Total	**1,238**	**1,696**	**2,956**	**3,657**	**3,764**	**3,851**	**3,833**	**3,836**	**3,910**

Water will be transferred between the storages as indicated in the schematic diagram shown in Figure 4.9. Details of the proposed operating rules are provided in Appendix A.

Table 4.6 Design Storage Capacities of Water Management Dams at Y33

Storage	Capacity to Spillway Level (ML)
Raw Water Dam	406
Dirty Water Dam	100
Sediment Dam South	582
Sediment Dam Central	394
Sediment Dam North	1,148
Pit Water Dam	24,220
Product Stockpile Dam	347
ROM Stockpile Dam	245
MIA Dam	160
ROM Dump South Dam	370
ROM Dump North Dam	577
ROM Dump X Dam	42
Dam A	201
Dam B	201

The conceptual water management system design has been developed on the basis that the Pit Water Dam would be sized to contain the full range of modelled historical inflows without discharge and without long-term in-pit water storage. This approach would minimise disruptions to mining operations, and reduce the potential for water quality to deteriorate in-pit.

The proposed dirty water dam is a relatively small structure proposed as a staging point for transferring water from the Sediment Dams to the infrastructure area for reuse without mixing in the Pit Water Dam. During the detailed feasibility study, storage may be shared differently between the Dirty Water Dam and the Pit Water Dam to allow greater separation of Saline Water and Waste Rock Runoff Water.

The modelled pit water storage capacity adopted for modelling is very large. In practice, if this capacity is required to manage pit inflows, alternative methods of storage and disposal of pit water are likely to be required, as discussed in the following sections.

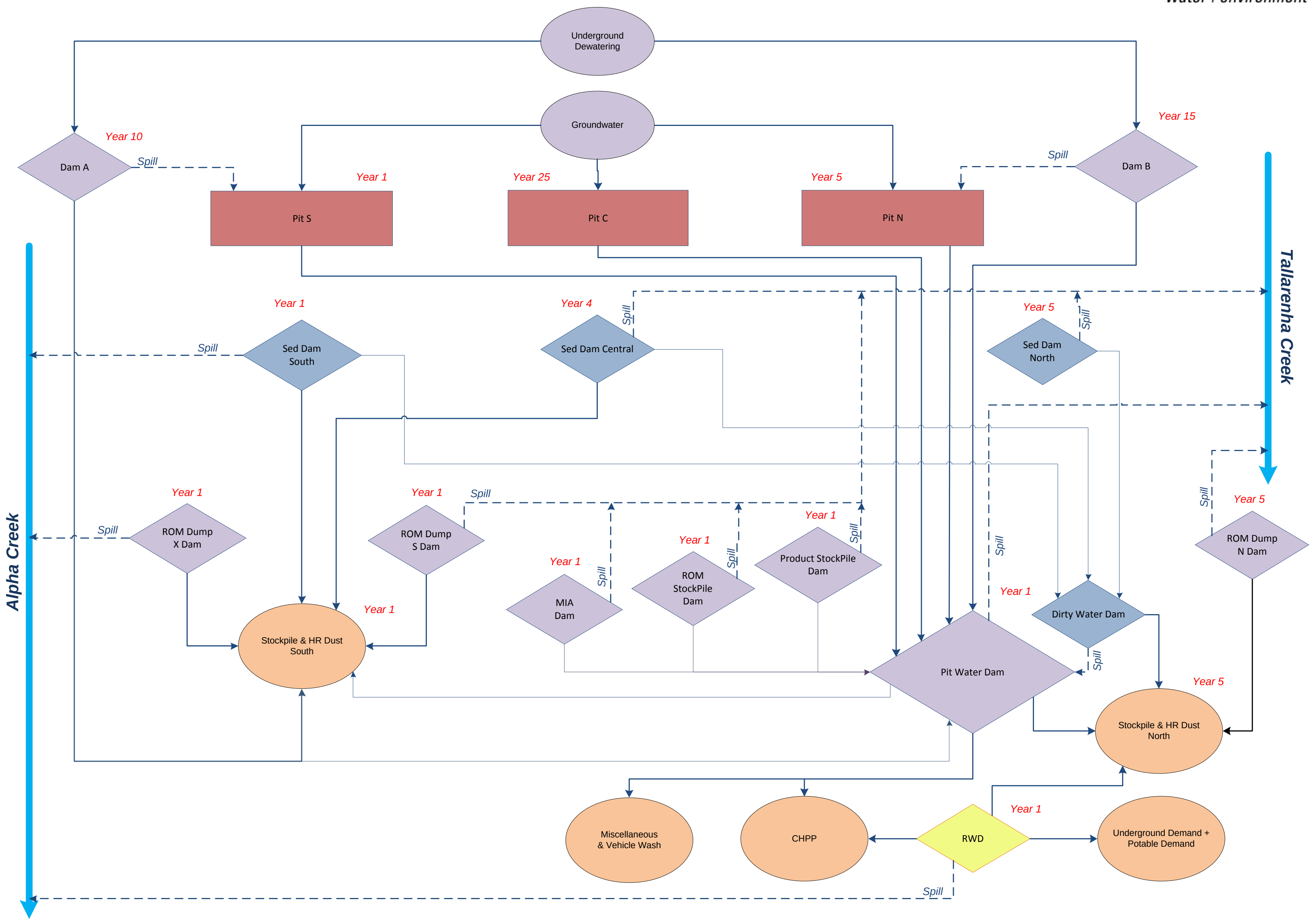


Figure 4.9 Schematic Diagram of Proposed Water Management System

Pit Water Management

Groundwater inflows into the proposed operation are expected to be significant. The Hydrogeological Assessment (RPS Aquaterra, 2012), predicts total groundwater inflow rates to the proposed operations will increase to over 9,125ML/a (25ML/d) for a short period in Year 5. For much of the project life, the inflow rate will exceed 14ML/d.

In the water balance analysis of this investigation, the portion of the groundwater inflows flowing to the open cut were reduced to account for evaporation from the pit faces and the entrained moisture losses due to mining.

- Evaporation from the open cut pits was based on pit face lengths estimated from the mine stage plans and a typical 5m coal seam height. An evaporation rate of 2.35mm/d was adopted based on a Morton's Lake Average rate of 5mm/d, an evaporation factor of 0.94 and a storage factor for deep pits of 0.5.

- The entrained losses due to mining were calculated from the production schedule and an assumption that the raw feed to the CHPP has a moisture content of 8%.

Additional pit water will be generated by the collection of surface water runoff from areas draining to the open cut. Pit water may have elevated salinity and may also contain suspended sediment and dissolved metals. Contaminant concentrations in pit water at the SGCP are likely to be in excess of levels required for protection of downstream receiving water values, and hence, will be transferred to the Pit Water Dam for long-term storage and reuse at the CHPP as required.

The adopted pit water dam capacity requirement is substantial, increasing significantly as the mine develops and the pit catchment increases. Alternative water management approaches (which have not been modelled) will also be considered to reduce the total Pit Water Dam storage requirement, including:

- Off-site discharge of excess water (to Alpha Creek or Tallarenha Creek) in accordance with release conditions to be specified in the Environmental Authority;
- Beneficial reuse (including treatment if necessary) to reduce the net site water surplus;
- In-pit water storage, in areas of the void which will not cause disruptions to operations in the open cut or underground operations.

The Pit Water Dam volume requirements increase significantly as the mine develops and the pit catchment increases. It is therefore proposed to stage construction of the Pit Water Dam. Based in the results of the modelling, the Pit Water Dam should be sized to contain 3,000ML by Year 2, 10,000ML by Year 5, 15,000ML by Year 8, with full capacity likely to be required by Year 12.

The volume stored in the storage will be constantly re-evaluated. In the event that water inventories become so high that the risk of future pit inundation is unacceptable, additional storage compartments may be constructed.

Figure 4.10 and Figure 4.11 show that with the proposed arrangement, pits could be expected to store more than 1,200ML of water for less than 1% of the time during all stages of the mine life.

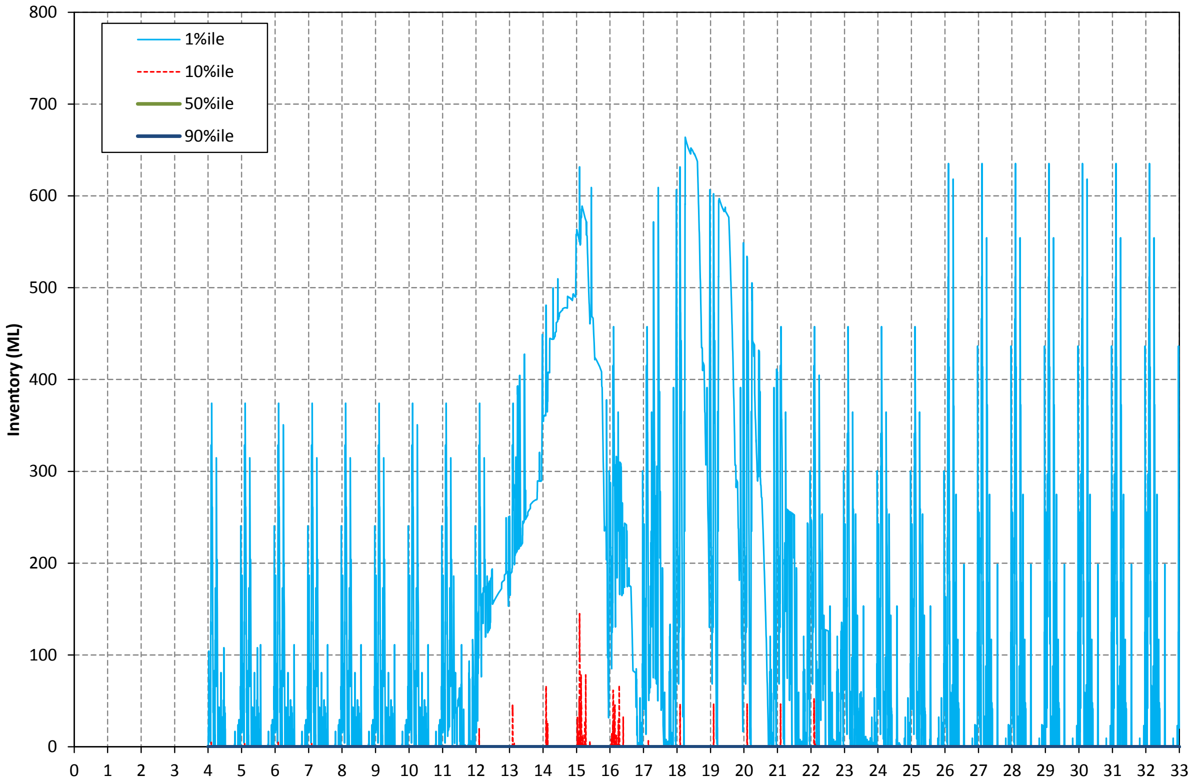


Figure 4.10 Frequency of North Pit Inundation

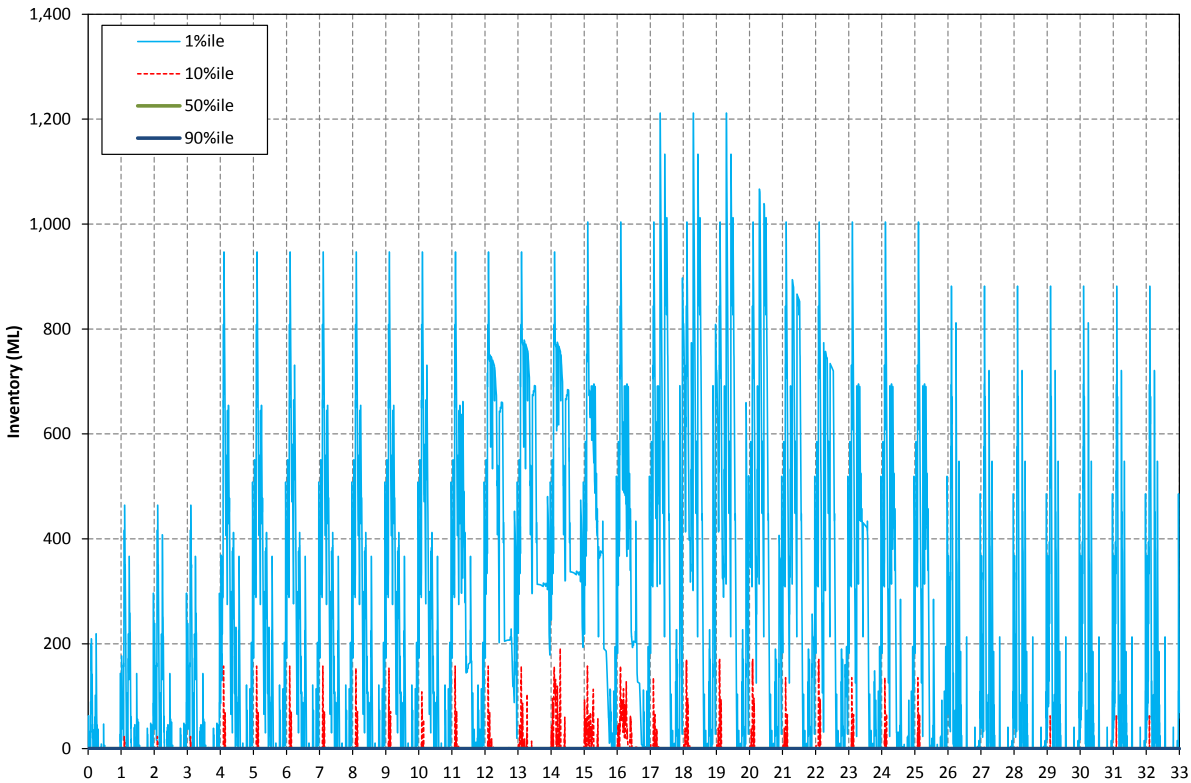


Figure 4.11 Frequency of South Pit Inundation

Waste Rock Runoff Management
Based on the geochemistry assessment and proposed methods for managing PAF material, the waste rock runoff is expected to have elevated suspended solids but only moderate levels of salinity and other pollutants. Waste rock runoff will be captured and temporarily stored in sediment dams. A total of three (3) sediment dams are required over the project life to intercept runoff from waste rock emplacements around the site.

The dams will be operated as part of the integrated water management system for the mine, and will be sized to contain 20% of the runoff from the 1 in 20 AEP 3 month rainfall. The dams will allow coarse sediment to settle and reduce the turbidity of runoff. This design storage capacity will be sufficient to limit the frequency of uncontrolled off-site discharge only to periods of relatively high and prolonged rainfall (when there is a reasonable prospect of natural flow in the receiving waters).

If water quality allows, excess stored water will be released from the sediment dams in accordance with the site EA release conditions. Otherwise water will be re-used on site, and if necessary returned to the Pit Water Dam to ensure that 20% of the Design Storage Allowance (DSA) requirement for each dam is provided by November 1 of each year.

Sediment Dam Central is a temporary dam which will be required after Year 1 (when placement of waste rock extends north of the initial boxcut) until after vegetation is well established on the rehabilitated waste rock emplacement, around Year 25.

Site Water Balance
The results of the site water balance modelling is summarised in Table 4.7, which describes the average annual site water balance. Once the project is established, on average, there is a small water excess. However, as explained in the previous sections, water will accumulate during periods of prolonged high rainfall.

Table 4.7 Average Annual Site Water Balance

		Year 1	Year 4	Year 10	Year 20	Year 33
Parameters						
ROM Coal (Mtpa)						
	Open Cut	*5.61*	*4.79*	*5.04*	*5.17*	*5.9*
	Underground	*0*	*7.09*	*13.59*	*11.99*	*4.53*
	Total	*5.61*	*11.88*	*18.63*	*17.16*	*10.43*
Water Inputs (ML)						
Rainfall/Runoff Yield						
	Dirty Water System	*164*	*242*	*479*	*182*	*130*
	Saline Water System	*1,025*	*1,168*	*1,762*	*1,762*	*1,878*
	Clean Water System	*47*	*47*	*47*	*254*	*147*
	Total	1,235	1,457	2,448	2,199	2,156
Raw Water from Pipeline		0	1,258	659	659	1,106
Groundwater Inflow to Open Cut Pits		0	117	194	0	0
Groundwater Inflow to Underground		0	1,932	5,738	5,044	2,312
Gross Water Inputs		**1,235**	**4,764**	**9,038**	**7,902**	**5,574**
Water Outputs (ML)						
Evaporation from Dams and Ponds						
	Dirty Water System	*27*	*47*	*389*	*107*	*6*
	Saline Water System	*702*	*1,027*	*2,964*	*2,360*	*1,502*
	Clean Water System	*149*	*149*	*149*	*279*	*160*
	Total	878	1,223	3,502	2,746	1,668
Dam Overflows (off-site)						
	Dirty Water System	*2*	*0*	*43*	*3*	*0*
	Saline Water System	*0*	*0*	*69*	*0*	*0*
	Clean Water System	*0*	*0*	*0*	*12*	*0*
	Total	2	0	112	15	0
Net Loss from CHPP		656	2,103	3,298	3,037	1,846
Haul Road Dust Suppression		333	453	883	910	1,043
Vehicle Wash + Misc. Industrial Use		131	131	131	131	131
Stockpile Dust Suppression		300	300	300	300	300
Underground Water Use (Potable Quality)		0	470	470	470	470
Potable Water Demand		62	84	84	84	84
Gross Water Outputs		**2,363**	**4,763**	**8,780**	**7,693**	**5,541**
Water Balance (ML)						
Change in Storage		**14**	**1**	**258**	**209**	**29**
Gross Water Balance (Deficit)		**1,141**	**0**	**0**	**0**	**2**

Impacts on Surface Water Runoff to Receiving Waters

A number of aspects of the proposed SGCP could potentially reduce downstream streamflow and sediment movement:

1. Runoff from disturbed areas around the open cut will be captured in the WMS and reused for mining purposes. Water will only be released from the site dams in compliance with the EA conditions and when water inventories are high. The estimated impact on mean annual flow is summarised in Table 4.8. The estimated reduction in average Tallarenha Creek streamflow (at the Beta Creek confluence, approximately 25km downstream of the SGCP (see Figure 2.4)) is 1% in Year 1, increasing to 9% by Year 25. The impact on flows in Alpha Creek is much less significant. The estimated reduction in average flow to DERM's Violet Grove streamflow gauge on Native Companion Creek is predicted to be less than 0.4% throughout the life of the project. At mine closure, there will be residual impacts of 7% and 3% in Tallarenha and Native Companion Creeks respectively.

2. Figure 4.4 shows that mine-induced subsidence would result in the formation of deep new pools along the Tallarenha Creek main channel. Catchment surface flow and flood flows could also be trapped in depressions formed by mine-induced subsidence in the floodplain areas. A monitoring plan will be established over the subsidence impact zone surrounding Tallarenha Creek. The purpose of the plan will be to identify subsidence-induced changes to the creek profile and floodplain drainage patterns that could prevent flow draining downstream. If these impacts are identified through aerial and ground survey of the area, channels will be constructed to direct flows downstream. Figure 4.12 and Figure 4.13 show the expected requirements for drainage mitigation works based on the predicted subsidence impacts.

3. Subsidence-induced cracking will enhance infiltration in the affected catchment areas. However, it is expected that these areas will be self-sealing within 1 wet-season of subsidence occurring (*RPS Aquaterra* 2012, *pers. comm.*, 5 Oct). As a result, if free drainage is maintained, it is unlikely that additional infiltration losses will significantly impact on downstream streamflow.

Impacts on Sediment Movement

The proposed channel works to ensure that streams remain free-draining following subsidence, should ensure that the movement of sediment is not restricted. Significant impacts on downstream QMAN holders in Lagoon Creek are therefore unlikely.

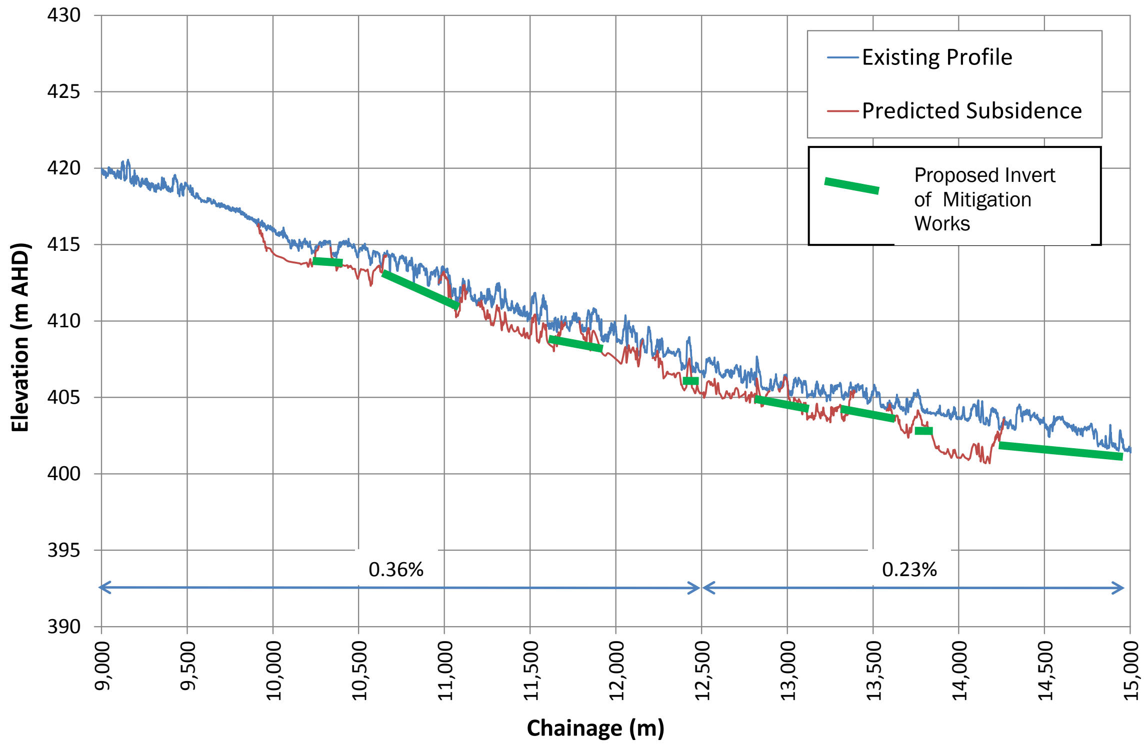


Figure 4.12 Tallarenha Creek Channel– Proposed Mitigation Works

Impacts on Sediment Loads to Great Barrier Reef Lagoon

Land disturbance caused by construction and operational activities within with the MLA areas, and during construction of the infrastructure corridor, has the potential to increase sediment loads in the receiving waters. However, the impact on the Great Barrier Reef Lagoon will be minimal because:

- Erosion and sediment control measures will be implemented to reduce sediment loads in runoff from construction sites. An Erosion and Sediment Control Plan will be developed prior to all construction work in accordance with recommendations of the International Erosion Control Association's Best Practice guidelines;
- Runoff from disturbed areas of the operational site will be captured in the site water management system, so that coarse sediment will settle, and the risk of discharge of sediment-laden runoff will be low
- The project area makes up less than 0.3% of the catchment area to Burdekin Falls Dam and the contribution of other downstream catchments to total sediment loads is far greater than that of Native Companion Creek (the sediment load is less than 5% of the total Burdekin River sediment load and less than 2% of the total sediment load to the Great Barrier Reef Lagoon).
- The Burdekin Falls Dam further reduces sediment loads from its tributary catchments (including Native Companion Creek) by between 60% and 90%.

Table 4.8 Catchment Interception & Reduction in Runoff

Including diverted rehab areas Years 25,30,33 *Note: 2072ha diverted from Sapling Creek to Dead Horse Creek*

Receiving Water Location	Area (ha)										
	Existing	Year 1	Year 4	Year 5	Year 10	Year 15	Year 20	Year 25	Year 30	Year 33	Post Mining
Tallarenha Creek to confluence of Beta Creek	38,140	37,740	37,290	36,030	35,880	35,770	35,690	34,700	34,700	35,610	35,510
Native Companion Creek at Violet Grove	406,500	405,501	405,501	405,501	404,946	404,946	404,935	404,917	404,917	404,917	405,473

Receiving Water Location	Percentage of Existing (%)										
		Year 1	Year 4	Year 5	Year 10	Year 15	Year 20	Year 25	Year 30	Year 33	Post Mining
Tallarenha Creek to confluence of Beta Creek		99%	98%	94%	94%	94%	94%	91%	91%	93%	93%
Native Companion Creek at Violet Grove		99.8%	99.8%	99.8%	99.6%	99.6%	99.6%	99.6%	99.6%	99.6%	99.7%

Receiving Water Location	Mean Annual Flow Volume (ML/a)										
	Existing	Year 1	Year 4	Year 5	Year 10	Year 15	Year 20	Year 25	Year 30	Year 33	Post Mining
Tallarenha Creek to confluence of Beta Creek	7,073	6,999	6,916	6,682	6,654	6,634	6,619	6,435	6,435	6,604	6,586
Native Companion Creek at Violet Grove	75,389	75,204	75,204	75,204	75,101	75,101	75,099	75,096	75,096	75,096	75,199

Receiving Water Location	Percentage Reduction (%)										
		Year 1	Year 4	Year 5	Year 10	Year 15	Year 20	Year 25	Year 30	Year 33	Post Mining
Tallarenha Creek to confluence of Beta Creek		1%	2%	6%	6%	6%	6%	9%	9%	7%	7%
Native Companion Creek at Violet Grove		0.2%	0.2%	0.2%	0.4%	0.4%	0.4%	0.4%	0.4%	0.4%	0.3%

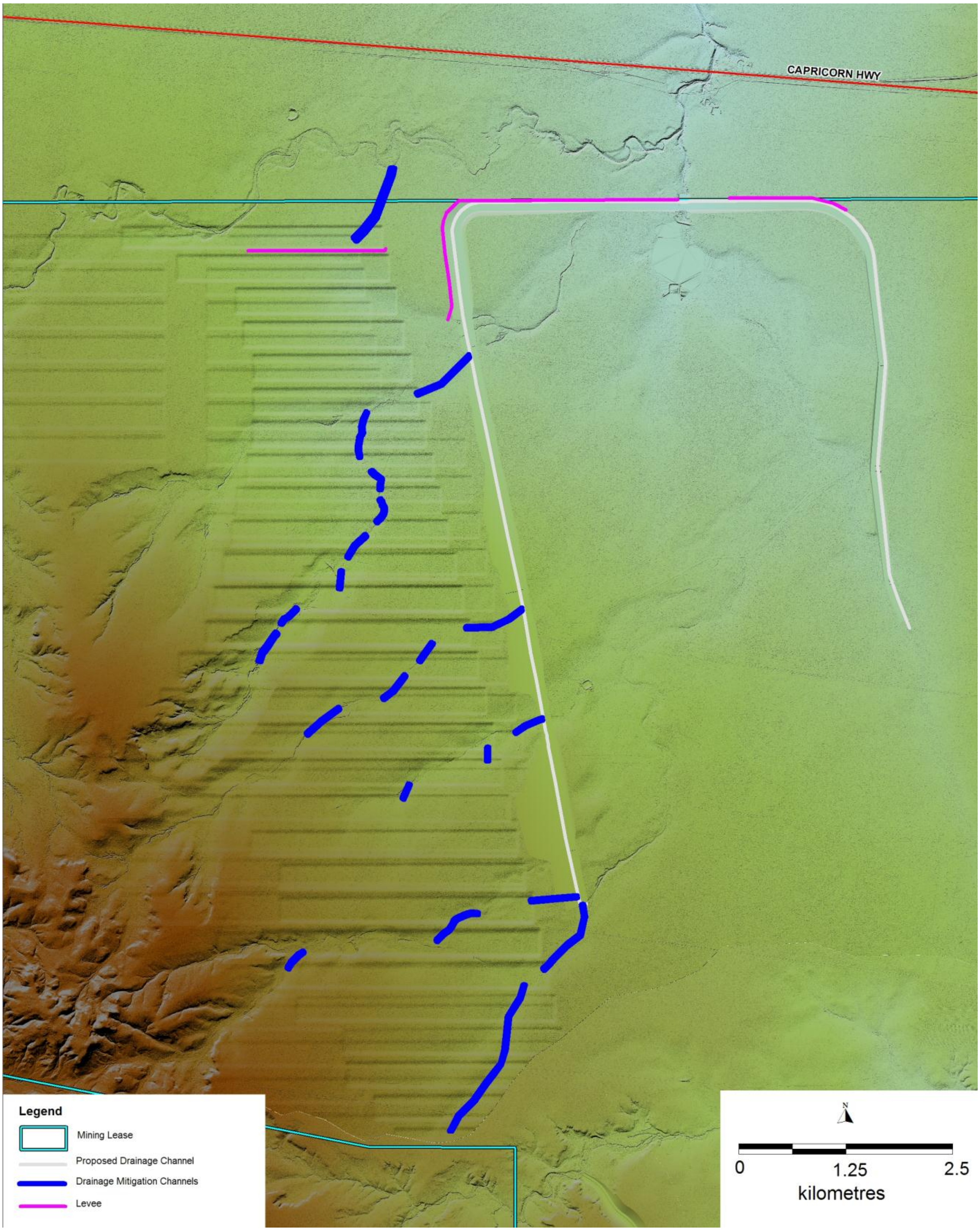


Figure 4.13 Potential Drainage Works for Draining Subsidence Zones

4.5 CUMULATIVE IMPACTS

There are a number of proposed coal mines in the vicinity of the SGCP, including:

- Galilee Coal (Northern Export Facility) (also known as the China First Coal Project), proposed by Waratah Coal Pty Ltd;
- Alpha Coal Mine, proposed by Hancock Prospecting Pty Ltd;
- Kevin's Corner, proposed by Hancock Galilee Pty Ltd; and
- Carmichael Coal Mine and Rail Project, proposed by Adani Mining Pty Ltd.

As shown in Figure 4.14, these projects are all located downstream of the proposed SGCP. As the proposed water management system will aim to maximise onsite water reuse by providing a large on-site water storage, the potential for impacts on downstream receiving water quality is limited.

Water will only be released from the site dams in compliance with the EA conditions, which will be developed in consultation with DERM to manage potential cumulative impacts.

Depending on the arrangement of the downstream projects, there will be some potential for cumulative impacts on downstream streamflow. However, given the contribution to streamflow from large downstream and adjacent catchments not affected by proposed mining projects, the percentage cumulative reduction in downstream flows is likely to be less than the impact in the immediate vicinity of the SGCP.

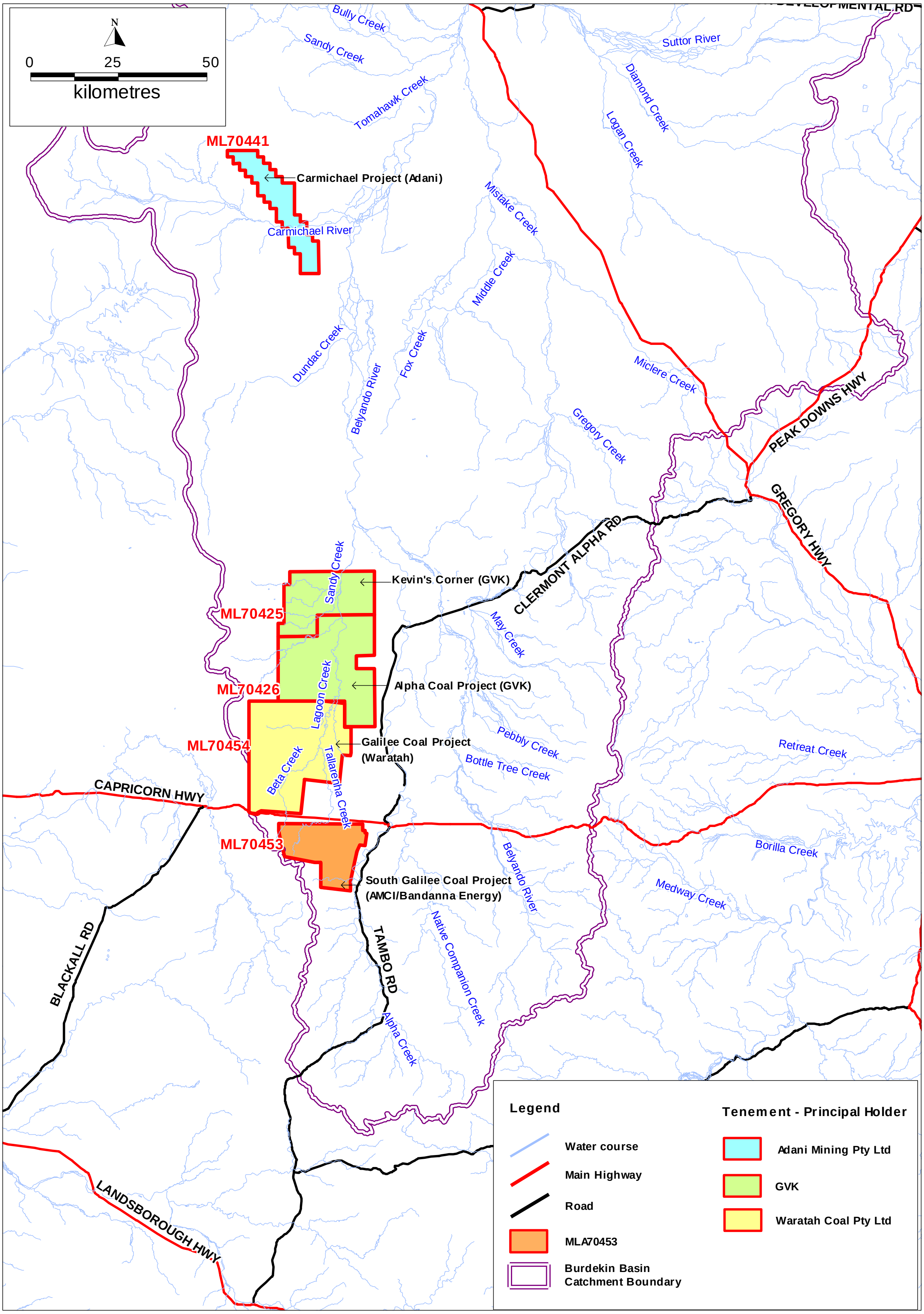


Figure 4.14 Proposed Projects in the Vicinity of the SGCP

4.6 INFRASTRUCTURE CORRIDOR

As discussed in section 2.2.2, the proposed infrastructure corridor does not cross significant floodplain areas. Therefore, the impact of the proposed infrastructure on flooding in major streams is likely to be insignificant.

During the Detailed Feasibility Study, drainage works will be designed for the proposed infrastructure. The infrastructure will need to be sized to ensure that the cross-drainage structures do not significantly increase upstream flood levels in the minor stream and drainage paths crossing the alignment.

Cross-drainage works will also need to be adequately sized to limit localised increases in flow velocity. Where this is unavoidable, the appropriate scour protection works will be implemented to limit localised erosion.

4.7 WATER MANAGEMENT AT DECOMMISSIONING

4.7.1 Local Drainage Patterns

At mine closure, a final void will remain, and the drainage system will be largely as proposed for Year 33. Dams will be decommissioned, and rehabilitated catchments will drain from the project via the proposed high-wall and low-wall channels, which will become part of the post-mine drainage system. The potential long-term impact on downstream streamflow is summarised in Table 4.8.

At watercourse confluences, suitable dumped rock erosion protection will be provided if required to prevent excessive erosion.

4.7.2 Final Void Flood Immunity

The final voids will be protected from flood events by the proposed diversion channels and levee, which will become part of the final landform. The extent of flooding for the probable maximum flood (PMF) is shown in Appendix E.

4.7.3 Final Void Water Levels

Final void water levels have been simulated using a simplified OPSIM water balance model assuming long-term groundwater inflows are approximately 1.6ML/d (based on the results of groundwater modelling and advice from RPS Aquaterra).

Based on model results, water levels eventually stabilise at a level at which the average net contribution to the pit final void from rainfall, runoff and infiltration are balanced by evaporative losses.

Long term expected water levels in the South Galilee Final Void are presented in Figure 4.15. A range of initial water levels were chosen to investigate sensitivity. Figure 4.15 indicates the following:

- Long term water levels in the final void appear to stabilise at around 325m AHD (a depth of approximately 40m compared to the total void depth of 90m); and
- The long-term final void water level is relatively insensitive to the initial water level.

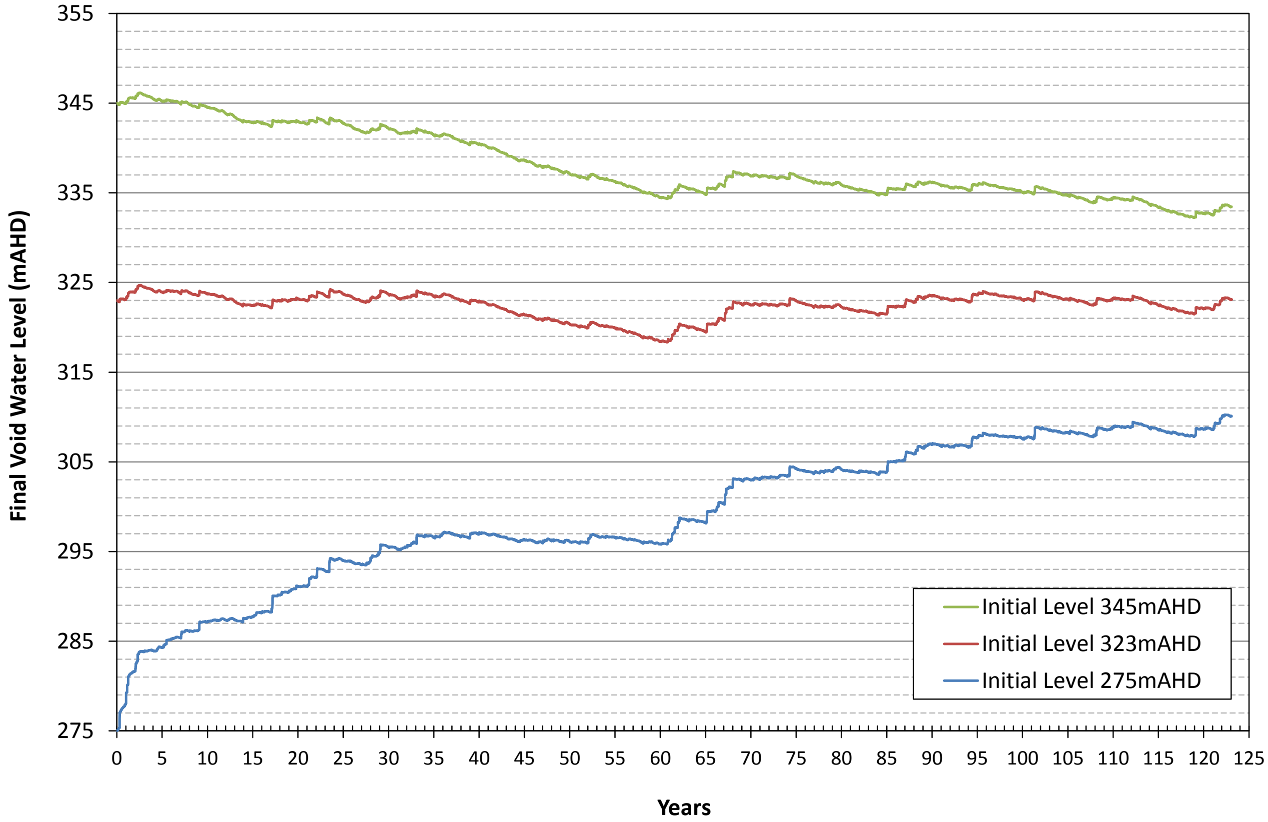


Figure 4.15 South Galilee Final Void Water Level Behaviour

The overall behaviour of the water stored in the final void was found to be relatively insensitive to basic assumptions regarding runoff parameters and evaporation and seepage rates. If the proportion of rainfall that becomes runoff is less than expected, the long-term water level will be reduced. Results of the sensitivity analysis are provided in Appendix C.

4.7.4 Long Term Final Void Salinity

In any void which does not have a mechanism for salts to flow out (e.g. by flushing through flood inflows and discharges, or by fresh groundwater inflows), salinity will tend to increase over time. OPSIM modelling of the final void shows that if initial water levels are low, the salinity will eventually increase beyond safe stock watering levels.

5 SURFACE WATER MONITORING

A surface water monitoring program will be required to measure compliance with the EA conditions. Monitoring points will be provided at points where contaminants could potentially be released from the WMS at concentrations that could cause environmental harm in the receiving waters. Monitoring points will also measure receiving water quality upstream and downstream of the release points.

Table 5.1 lists the contaminant release points from the mine water management system and the associated receiving waters. The locations of the release points are shown in Figure 5.1.

Table 5.1 Contaminant Release Points and Receiving Waters

Release Point	Latitude	Longitude	Contaminant Source and Location	Monitoring Point	Receiving Waters
RP1 (Discharge Point 1)	146.495	-23.667	Dirty Water Dam	Low Level Pipe Outlet	Tallarenha Creek
RP2 (Discharge Point 2)	146.495	-23.663	Pit Water Dam	Low Level Pipe Outlet	Tallarenha Creek
RP3 (Discharge Point 3)	146.491	-23.657	Sediment Dam North	Low Level Pipe Outlet	Tallarenha Creek
RP4 (Discharge Point 4)	146.492	-23.700	Sediment Dam Central	Low Level Pipe Outlet	Tallarenha Creek
RP5 (Discharge Point 5)	146.512	-23.745	Sediment Dam South	Low Level Pipe Outlet	Sapling Creek/ Alpha Creek

Monitoring requirements to establish site-specific trigger values, and assess the potential impact of releases on downstream receiving waters are listed in Table 5.2.

Table 5.2 Proposed Receiving Water Monitoring Points

Monitoring Point	Description	Water Level (Flow)	REMP Water Quality Sampling	Continuous Water Quality Sampling	Full suite as per note below
TCU	Tallarenha Creek (Upstream Monitoring Point)	X	During Flow Event	X	Daily During Release
TCD	Tallarenha Creek (Downstream Monitoring Point)	X	-	X	Daily During Release
SCU	Sapling Creek (Upstream Monitoring Point)	-	During Flow Event	-	Daily During Release
SCD	Sapling Creek (Downstream Monitoring Point)	X	During Flow Event	-	Daily During Release
DCU	Dead Horse Creek (Upstream Monitoring Point)	-	During Flow Event	-	Daily During Release
DCD	Dead Horse Creek (Downstream Monitoring Point)	X	-	-	Daily During Release
ACU	Alpha Creek (Upstream Monitoring Point)	X	-	X	Daily During Release
ACD	Alpha Creek (Downstream Monitoring Point)	X	-	X	Daily During Release
HC	Highwall Channel (Downstream Monitoring Point)	-	-	-	Daily During Release
LC	Lowwall Channel (Downstream Monitoring Point)	X	-	-	Daily During Release

Gauge boards will be provided at all dams to allow storage water levels and volumes to be monitored and enable inflows and outflows to be estimated. Automatic monitoring equipment may be installed at key storages.

The event-based sampling will enable quantification of discharge water quality from the site and any potential corresponding impact on receiving waters. On-site monthly sampling from the water storages will allow for any potential problem areas with respect to pollutant generation to be identified in advance to ensure that appropriate remedial action can be taken in time.

In addition to the above water quality and streamflow monitoring points, a monitoring system will be established in the Sapling Creek Diversion and in Dead Horse Creek downstream of the Sapling Creek Diversion outlet. The monitoring program will include regular assessments of the geomorphic condition following flow events, and will include collection of site photographs, aerial photographs, and aerial survey data. The purpose of the monitoring points will be to establish baseline creek conditions and monitoring ongoing performance during both operations and following mine closure. The monitoring program will be designed taking into account the recommendations in the ACARP program 'Monitoring and Evaluation Program for Bowen Basin Diversions' (ID&A 2000).

A monitoring program will also be established for the underground subsidence zone surrounding Tallarenha Creek. The purpose of the program will be to identify subsidence-induced changes to the creek profile and floodplain drainage patterns that could prevent flow draining downstream. If these impacts are identified through aerial and ground survey of the area, channels will be constructed to direct flows downstream. The highwall and lowwall flood protection channels and levees will also be routinely monitored to ensure they can safely convey flood flows around the active mining area.

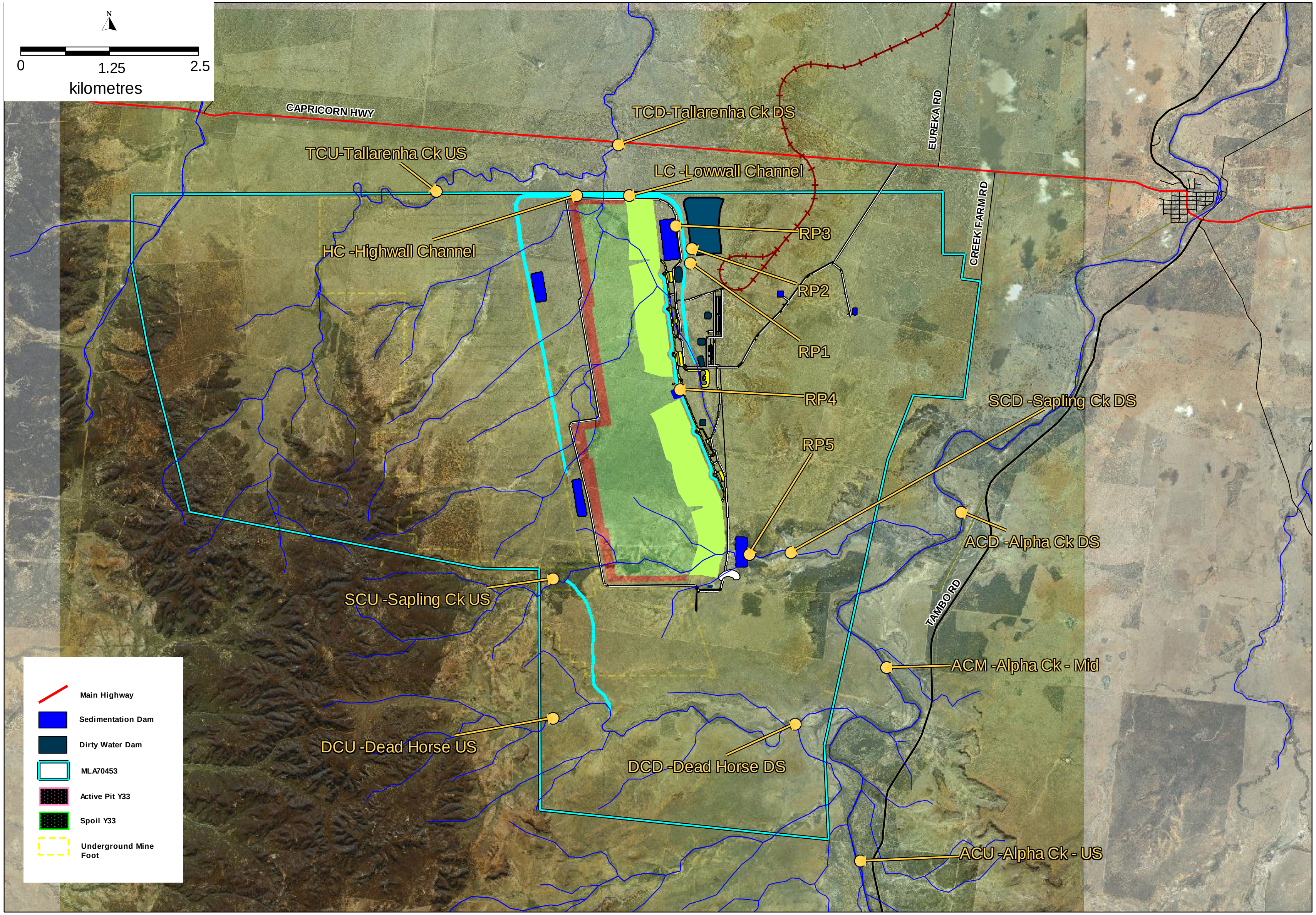


Figure 5.1 Proposed Receiving Water Monitoring Points

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APPENDIX A

SITE WATER BALANCE MODEL

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1 MODELLING APPROACH

An Operational Simulation (OPSIM) model was developed for the SGCP. The OPSIM model dynamically simulates the operation of the site's water management system, and in doing so accounts for the movement of all site water, including representative water quality. The model operates on a daily timestep using long-term historical climate data.

The model has been configured to simulate all major components of the water management system including:

- Climatic variability – rainfall and evaporation;
- Catchment runoff and collection;
- Pit dewatering;
- Pump and gravity transfers;
- Water storage filling, spilling and leaking; and
- Industrial water extraction, usage and return
- Water imports from external sources.

For modelling purposes, the Project life has been broken into 5 stages as outlined in Table 1.1.

Table 1.1 Stages of Evolution of the Site Water Management System

Stage	Project Year	Calendar Years
1	1	2014
2	2-4	2015-2017
3	5-15	2018-2028
4	16-26	2029-2039
5	27-33	2040-2046

2 RUNOFF MODEL

The OPSIM model uses the Australian Water Balance Model (AWBM) (Boughton & Chiew 2003) to estimate daily runoff from daily rainfall. The AWBM is a saturated overland flow model which allows for variable source areas of surface runoff.

The AWBM uses a group of connected conceptual storages (three surface water storages and one ground water storage) to represent a catchment. Water in the conceptual storages is replenished by rainfall and is reduced by evapotranspiration. Simulated surface runoff occurs when the storages fill and overflow. Figure 2.1 shows a conceptual configuration of the AWBM model.

The model uses daily rainfalls and estimates of catchment evapotranspiration to calculate daily values of runoff using a daily balance of soil moisture. The model has a baseflow component which simulates the recharge and discharge of a shallow subsurface store. Runoff depth calculated by the AWBM model is converted into runoff volume by multiplying by the contributing catchment area.

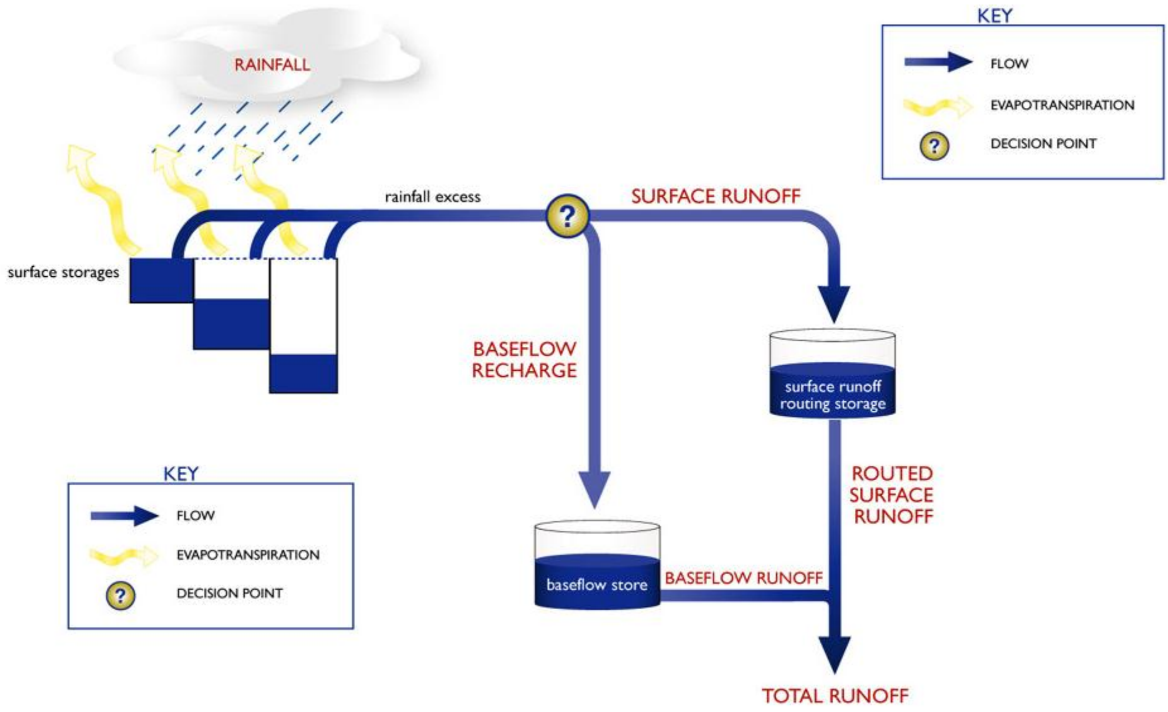

Figure 2.1 AWBM Model Configuration

At each time step, the AWBM computes as follows:

- Rainfall is added to each of the 3 surface moisture stores and evapotranspiration is subtracted from each store. The water balance equation is:
  $$store_n = store_n + rain - evap \quad (n = 1 \text{ to } 3).$$
- If the value of moisture in the store becomes negative, it is reset to zero. If the value of moisture in the store exceeds the capacity of the store, the moisture in excess of capacity becomes runoff and the store is reset to the capacity.
- When runoff occurs from any store, part of the runoff becomes recharge of the baseflow store. The fraction of the runoff used to recharge the baseflow store is BFI*runoff, where BFI is the baseflow index.
- The remainder of the runoff, i.e. (1.0 - BFI)*runoff, is surface runoff.
- The baseflow store is depleted at the rate of (1.0 - K)*BS where BS is the current moisture in the baseflow store and K is the baseflow recession constant of the time step being used (daily or hourly).
- The surface runoff can be routed through a store if required to simulate the delay of surface runoff reaching the outlet of a medium to large catchment. The surface store acts in the same way as the baseflow store, and is depleted at the rate of (1.0 - KS)*SS, where SS is the current moisture in the surface runoff store and KS is the surface runoff recession constant of the time step being used.

The model parameters define the storage depths, the proportion of the catchment draining to each of the storages, and the rate of flux between them (Boughton, 2003).

The AWBM model parameters were selected for consistency with the modelling undertaken for nearby Alpha Coal Project EIS (Parsons Brinckerhoff, 2011). Note that in the absence of better data, the Rehabilitated Waste land type was assumed to have the same runoff characteristics as Natural land type. The adopted parameters and long term runoff coefficients for the various catchment land use types are presented in Table 2.1.

Table 2.1 Adopted AWBM Parameters for Various Catchment Land Use Types

Parameter	Natural	Hardstand	Open Cut	Active Waste	Rehab Waste	Stockpile
A1	0.1	0.134	0.2	0.136	0.1	0.1
A2	0.3	0.433	0.2	0.270	0.3	0.9
C1	70	2.3	5	50	70	5
C2	135	22.9	70	100	135	50
C3	500	45.7	90	500	500	0
BFI	0.03	0	0	0.103	0.03	0.5
Kb	0.98	1	1	1	0.98	0.98
Long Term Runoff Coefficient	3.3%	33.1%	21.1%	4.7%	3.3%	23.9%

3 MODEL INPUTS

3.1 RAINFALL

A Patched Point Dataset for the Bureau of Meteorology's Alpha Post Office station was obtained from the Department of Environment and Resource Management (DERM). The dataset contains 124 years of data.

3.2 EVAPORATION

Daily estimates of Morton's Lake evaporation (obtained from DERM's Data Drill) were used for estimating evaporation from open water surfaces. The following factors, where applicable, were applied to evaporation rates for different surfaces.

Table 3.1 Other Evaporation/Evapotranspiration Factors

Description	Factor	Applied to:
Evapotranspiration Factor	0.94	Convert lake evaporation to actual evapotranspiration
Storage Factor	0.7	Reduction in evaporation in open cut pits due to lower wind effects and shading from pit walls.
Salinity Factor	$1/(1+Sx10^{-6})$	Reduction in evaporation due to salinity - using Morton's relationship– i.e. $E'=E/(1+Sx10^{-6})$ – where S is salinity in parts per million

3.3 GROUNDWATER INFLOWS

Groundwater inflows into the proposed operation are expected to be significant. The Hydrogeological Assessment (RPS Aquaterra, 2012), predicts total groundwater inflow rates to the proposed operations will increase to over 9,125ML/a (25ML/d) for a short period in Year 5. For much of the project life, the inflow rate will exceed 14ML/d.

The groundwater inflow rates were averaged over the years covered by each mine stage. This is illustrated in Figure 3.1. Table 3.2 shows the adopted groundwater inflow rates to the open cut and underground mine during different stages of mining.

The open cut inflows were reduced to account for evaporation from the pit faces and the entrained moisture losses due to mining.

- Evaporation from the open cut pits was based on pit face lengths estimated from the mine stage plans and a typical 5m coal seam height. An evaporation rate of 2.35mm/d was adopted based on a Morton's Lake Average rate of 5mm/d, an evapotranspiration factor of 0.94 and a storage factor for deep pits of 0.5.
- The entrained losses due to mining were calculated from production schedule and an assumption that the raw feed to the CHPP has a moisture content of 8%.

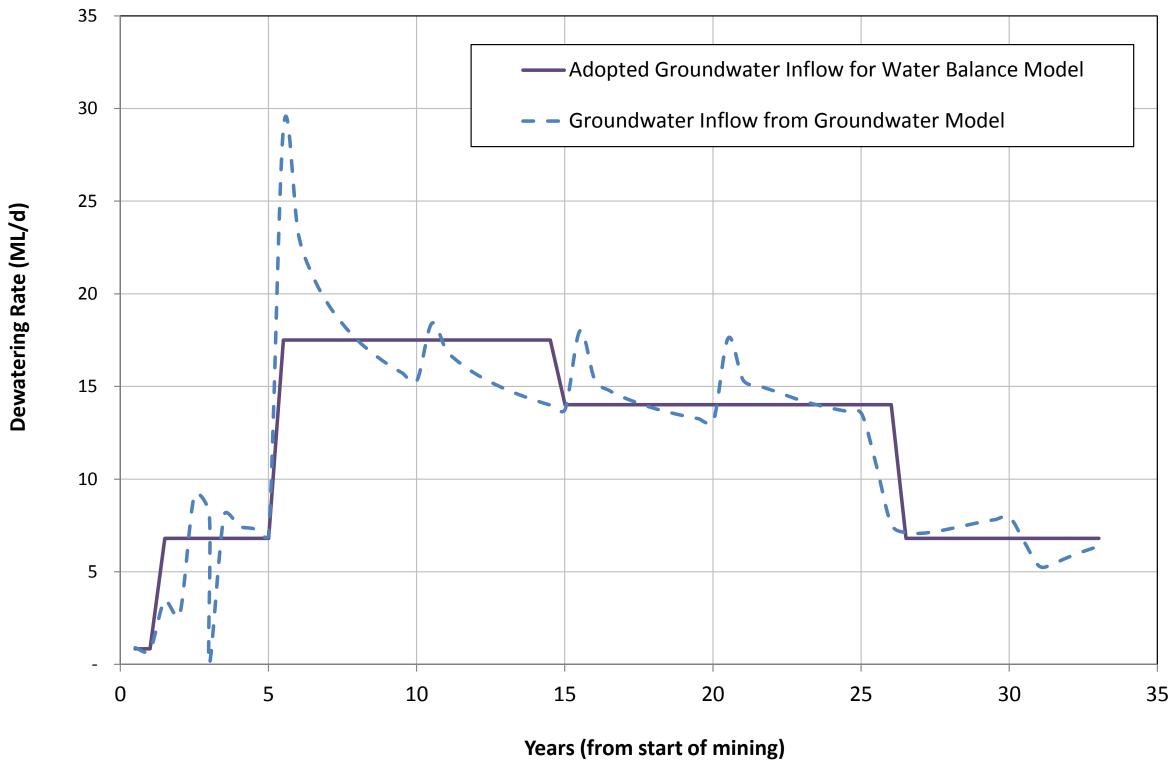

Figure 3.1 Estimated Groundwater Inflow Rates

Table 3.2 Adopted Groundwater Inflows for Surface Water Balance

Stage	Underground GW Inflows (ML/a)	Open Cut GW Inflows (ML/a)	Total Mine GW Inflow (ML/a)	Open Cut Losses (ML/a)	Net Open Cut Inflows (ML/a)
Year 1	0	307	307	493	0
Year 4	3,290	798	4,088	437	361
Year 10	5,432	487	5,918	459	28
Year 20	5,043	74	5,116	470	0
Year 33	2,311	172	2,483	537	0

4 WATER DEMANDS

4.1 OVERVIEW

Table 4.1 summarises the demands included in the water balance model. Details are provided in Sections 4.2 to 4.6.

Table 4.1 Demand Summary

	Average Annual Demand (ML)				
Demand	**Year 1**	**Year 4**	**Year 10**	**Year 20**	**Year 33**
CHPP	656	2,103	3,298	3,037	1,846
Haul Road Dust Suppression	336	456	889	917	1,051
Stockpile Dust Suppression	300	300	300	300	300
Potable Demand	62	84	84	84	84
Underground (Potable)	0	470	470	470	470
Misc. + Vehicle Wash	131	131	131	131	131
Total	**1,485**	**3,544**	**5,172**	**4,939**	**3,882**

4.2 COAL HANDLING AND PREPARATION PLANT (CHPP)

The CHPP will be operated as a dry tailings system until the end of Year 3 and as a wet tailings system from Year 4 onwards. The South Galilee Coal Pre-feasibility Study estimated that the CHPP demand during the dry tailings phase will be 117L/tonne of ROM (dry) and during the wet tailings phase will be 177L/tonne of ROM (dry). The adopted CHPP water usage over each modelled stage based on the production schedule is provided in Table 4.2.

Table 4.2 Estimated CHPP Water Demand

Stage	ROM Throughput (Mtpa)	CHPP Demand (ML/a)
Year 1	5.61	656
Year 4	11.88	2,103
Year 10	18.63	3,298
Year 20	17.16	3,037
Year 33	10.43	1,846

4.3 DUST SUPPRESSION

Dust suppression requirements at South Galilee Coal comprise haul road dust suppression and stockpile dust suppression.

For the purpose of the water balance assessment, haul road dust suppression watering rates have been applied to haul road areas that vary with the stage of mine development. Haul road lengths have been extracted from the mine stage plans provided by AMCI and are assumed to be 22m wide. The modelled dust suppression requirements are dependent on the daily rainfall. The following rules have been applied to determine the applied dust suppression rate on any given day of the historical rainfall record:

- for a dry day, the haul road watering rate is $4L/m^2/d$;
- for a rain day when rainfall is less than5 mm/d, the haul road watering rate is reduced and is only required to make up the remaining demand to $4L/m^2/d$;
- for a rain day when rainfall exceeds 5mm/d, no haul road watering is required.

The resultant average dust suppression demand is presented in Table 4.3.

Table 4.3 Estimated Haul Road Dust Suppression Requirements

Stage	Haul Road Area (ha)	Average Demand (ML/a)*1
Year 1	25.5	336
Year 4	34.5	456
Year 10	67.4	889
Year 20	69.5	917
Year 33	79.6	1,051

*1 Based on long-term average including rainfall days.

The adopted stockpile dust suppression is 300ML/a over each modelled stage. This is based on an assumed demand of 100ML/a per coal stockpile, of which there are three in each stage.

4.4 POTABLE WATER

The potable water demands for each modelled stage, presented in Table 4.4, have been based on the findings of the South Galilee Coal Pre-feasibility Study.

Table 4.4 Estimated Potable Water Demand

Potable Water Demand Type	Demand Rate (L/p/d)	Year 1		Year 4, 10, 20 & 33	
		No. of Staff	Potable Water Demand (ML/a)	No. of Staff	Potable Water Demand (ML/a)
Office and Maintenance Personnel	80	288	8	288	8
Mine Operations Personnel	200	222	16	372	27
Serviced Accommodation Berths	200	510	37	660	48
Total			**62**		**84**

4.5 UNDERGROUND DEMAND

Underground water requirements are assumed to be constant at 470ML/a (1,286m^3/day). Note that underground mining commences in Year 3. The water supplied for underground use is to be of potable quality.

4.6 VEHICLE WASH AND MISCELLANEOUS DEMAND

Light and heavy vehicle washdown and miscellaneous demands have been adopted from findings of the South Galilee Coal Pre-feasibility Study. The adopted rate is 131ML/a.

5 WATER MANAGEMENT SYSTEM

5.1 CONCEPTUAL ARRANGEMENT

A conceptual design of the SGCP water management system (WMS) has been prepared. Details of the system will be finalised in later stages of design, but the system will comprise the elements described in the following sections and meet the design criteria set out in the Environmental Authority and the performance targets outlined in this report.

The proposed system comprises three separate sub-systems, according to the water quality characteristics of the site catchments:

5.1.1 Saline Water System

This system manages catchment runoff which is potentially coal-affected. Water captured in this system is expected to have high salinity, and will potentially have elevated concentrations of dissolved metals. Water in this system will be pumped to the Pit Water Dam with a goal of containing all water on site for later reuse. Dams forming part of this system are:

- Pit Water Dam;
- ROM Dump X Dam;
- ROM Dump S Dam;
- MIA Dam;
- ROM Stockpile Dam;
- Product Stockpile Dam;
- ROM Dump N Dam;
- Dam A;
- Dam B.

The locations of the above dams are shown in Figure 5.1 and Appendix B.

5.1.2 Waste Rock Runoff Water System

This system manages runoff from waste rock areas, which are expected to have high turbidity, and a risk of moderately elevated salinity, and a lower risk of elevated metal concentrations. Dams forming part of this system are:

- Sediment Dam South;
- Sediment Dam Central;
- Sediment Dam North;
- Dirty Water Dam.

The locations of the above dams are shown in Figure 5.1 and Appendix B.

5.1.3 Raw Water System

Water supplied from the external raw water supply source is expected to have low salinity levels, will be managed separately to all other waters, and stored in the Raw Water Dam.

5.1.4 WMS Staging

The WMS layout will evolve over the 33 year mine life. As additional catchments are disturbed, new dams will be brought on line to treat waste rock runoff. For the purposes of concept design, and impact assessment, a number of stages have been assumed to be in place as follows:

Table 5.1 Mine Stages

Stage Label	Period	Duration
Stage Y1	2013-2015	3 years
Stage Y4	2016-2016	1 years
Stage Y5	2017-2021	5 years
Stage Y10	2022-2026	5 years
Stage Y15	2027-2031	5 years
Stage Y20	2032-2036	5 years
Stage Y25	2037-2041	5 years
Stage Y30	2042-2024	5 years
Stage Y33	2045-2047	2 years

Figure 5.1 shows the layout of the WMS at Year 33 of the project. Plans showing the conceptual water management layout at each of the above stages are provided in Appendix B.

For the purposes of the impact assessment, the operational catchment areas have been classified into different types based on their hydrological and geochemical characteristics. The adopted areas are summarised in Table 5.2. Further details are provided in Appendix B. The area intercepted by the planned SGCP water management system increases from 1,238ha to 3,910ha over the mine life.

With the exception of the South Sediment Dam, and ROM Dump X Dam, all dams are to be constructed in the Tallarenha Creek catchment.

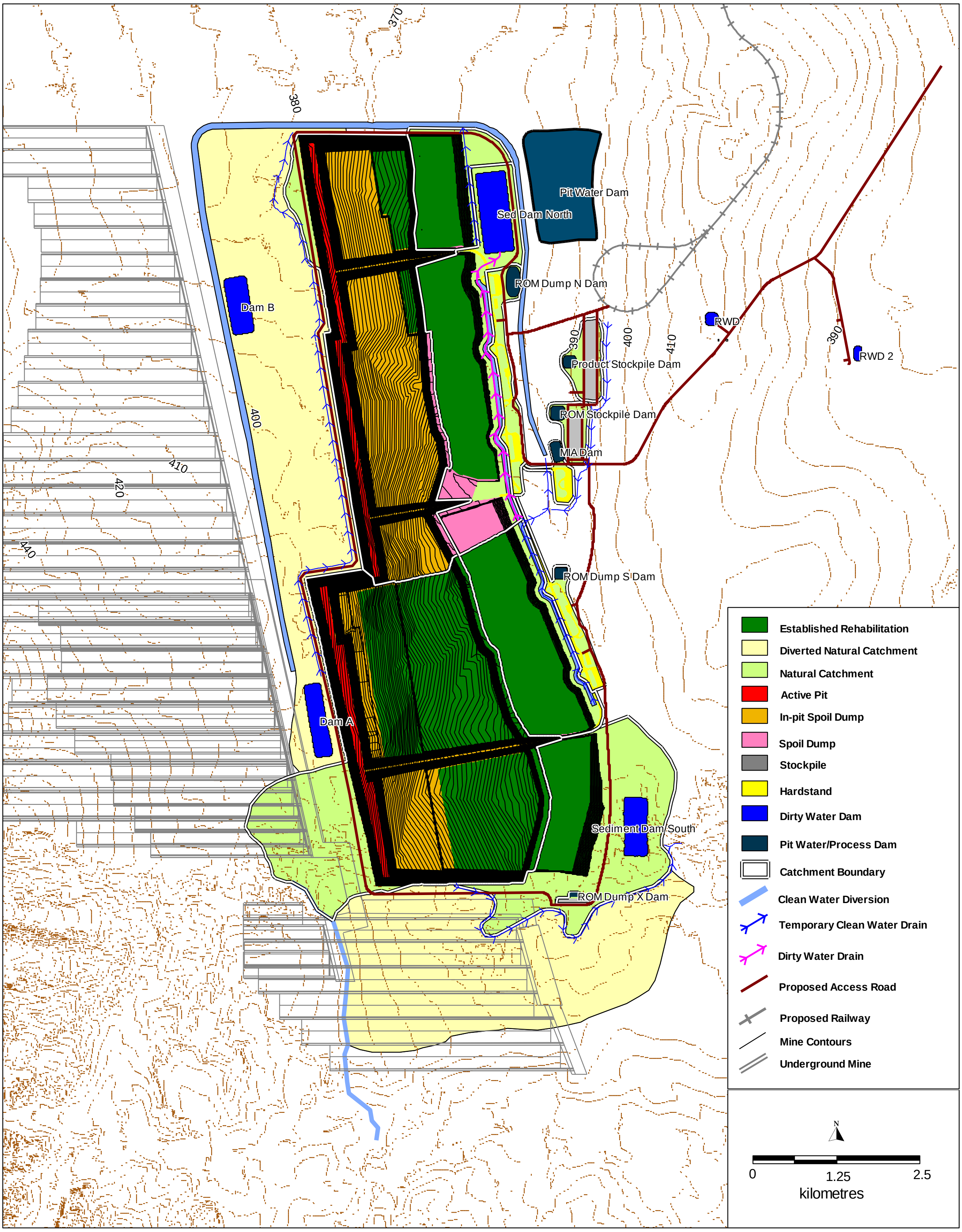

Figure 5.1 Mine Arrangement and WMS Layout at Year 33

Table 5.2 Changes in Catchment Areas and Land Use Types Intercepted by WMS

Stage	Land Use Classification (ha)							Total Area (ha)
	Natural (grassed)	Stockpile	Hardstand	Active Pit	In Pit Dump	Out Pit Dump	Established Rehabilitation	
Year 1	811	43	34	87	0	263	0	1,238
Year 4	685	43	34	153	273	508	0	1,696
Year 5	1,502	43	65	222	355	768	0	2,956
Year 10	1,690	43	65	312	620	561	365	3,657
Year 15	1,584	43	65	289	714	386	683	3,764
Year 20	1,409	43	65	312	808	144	1,068	3,851
Year 25	1410	43	63	411	758	162	986	3833
Year 30	1122	43	63	351	880	110	1266	3836
Year 33	1048	43	63	328	1,048	111	1,268	3,910

5.2 DAM SIZING

5.2.1 Failure to Contain Hazard Assessment

A preliminary assessment of the Hazard Category of the proposed dams has been undertaken in accordance with the failure to contain criteria in the *Draft Manual for Assessing Hazard Categories and Hydraulic Performance of Regulated Dams constructed as part of environmentally relevant activities pursuant to the Environmental Protection Act 1994* Version 3.1 (DERM, 2012) (the DERM Dams Manual).

All dams with a 'Significant" Hazard Category or greater are deemed "regulated dams".

Saline Water Management System Dams

Water captured in the saline water management system has the potential to come into contact with runoff from acid-forming material, and may contain elevated metal concentrations.

However, with the exception of the southernmost saline water dams (which are relatively small) the Saline Water Management System Dams would overflow to Tallarenha Creek in a failure to contain scenario. Given this location, the high potential dilution, and experience with water quality in similar structures in Central Queensland, serious impacts on human health or stock are unlikely. Significant environmental values have not been identified in the immediate vicinity. A "High Hazard" rating is therefore not justified, and a "Significant Hazard" rating would be appropriate under the "Failure to Contain" scenario for dams in the Saline Water Management System.

It should be noted, that given its large size, proximity to Capricorn Highway and Railway, the Pit Water Dam would require a High Hazard Rating under the Dam Break Scenario.

Waste Rock Runoff System Dams

Water captured in the Waste Rock Runoff System is likely to be of a higher quality than that captured in the Saline Water System, The proposed handling and capping measures for dealing with potentially acid-forming (PAF) material should ensure that stored contaminant concentrations in these dams would be well below potentially harmful levels at the point of overflowing. The most likely issue of concern for potential environmental harm will be suspended solids.

Given the potential uncertainty in Waste Rock Runoff quality, it is proposed to design and operate the Waste Rock Runoff System in accordance with the “Design Storage Allowance” (DSA) provisions of the DERM Dams Manual, unless future water quality assessments demonstrate that this is not required.

5.2.2 Hydraulic Performance Criteria

All regulated dams must provide for the “Design Storage Allowance” (DSA). Under typical site Environmental Authority water conditions, the DSA must be provided in the dam as at 1 November each year to prevent a discharge to an annual exceedance probability specified in the DERM Dams Manual.

The manual makes provision for distributing the DSA across multiple regulated dams in an Integrated Water Management System, so long as the system is operated in accordance with a certified system design plan. If the DSA is to be shared between two or more regulated dams, then each dam must be capable of accommodating at least 20% of that dam’s individual DSA volume for its catchment on 1 November.

The DSA has been estimated based on the Method of Deciles for Volumetric Containment specified in the DERM Manual for Dams. The results of this analysis are shown in Figure 5.2 and Table 5.3. The adopted 1 in 20 AEP design rainfall depth is 639mm (or 6.4ML/ha). 20% of this allowance is 128mm (or 1.3ML/ha).

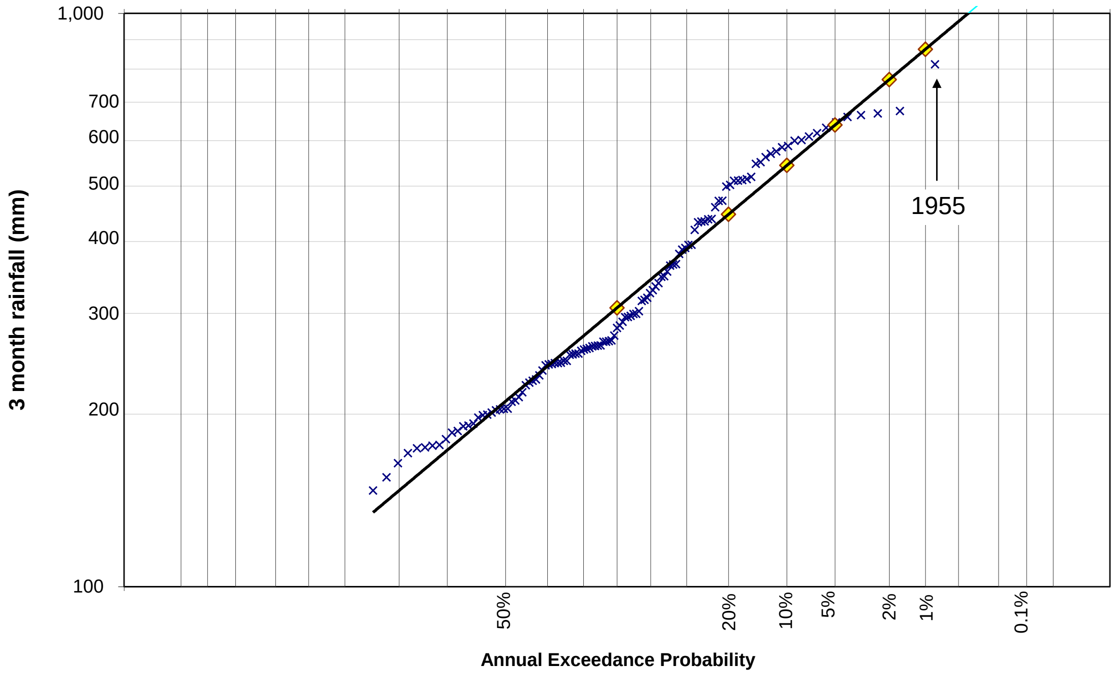

Figure 5.2 Deciles Analysis for Deriving Design Rainfall Depths

Table 5.3 Design 3-month Rainfall Depths

AEP	ARI	Design Rainfall
	y	mm
0.1%	1000	1,217
1%	100	866
2%	50	766
5%	20	639
10%	10	543
20%	5	446
50%	2	306

5.2.3 Waste Rock Runoff Dam Sizes

Sediment Dams collecting runoff from Waste Rock Dumps have been sized to provide 20% of the DSA for a “Significant Hazard Dam” (1.3ML/ha) (though the stored water quality is not expected to meet the manual requirements for dams of this type). This is equivalent to providing enough volume to contain the runoff from a 1 in 10 AEP 72 hour storm with a volumetric runoff coefficient of 0.85. The remainder of the DSA will be provided in the Pit Water Dam, with water to be transferred from the Sediment Dams, if significant quantities remain as November 1 approaches. The adopted design storage capacities for dams in the Waste Rock Runoff system are listed in Table 5.4.

Table 5.4 Design Storage Capacity of Waste Rock Runoff Dams

Storage	Capacity (ML)
Dirty Water Dam	100
Sediment Dam South	582
Sediment Dam Central	394
Sediment Dam North	1,148

The dirty water dam is a relatively small structure proposed as a staging point for transferring water from the Sediment Dams to the infrastructure area for reuse without mixing in the Pit Water Dam. During the detailed feasibility study, storage may be shared differently between the Dirty Water Dam and the Pit Water Dam to allow greater separation of Saline Water and Waste Rock Runoff Water.

Sediment Dam Central is a temporary dam which will be required after Year 1 (when placement of waste rock extends north of the initial boxcut) until after vegetation is well established on the rehabilitated waste rock emplacement, around Year 25.

5.2.4 Saline Water Dams

Saline Water System dams have been sized to provide 100% of the 1 in 20 AEP DSA (6.4ML/ha). Additional storage has been provided in the Pit Water Dam to accommodate additional surface water to be dewatered from the open cut following rainfall. The volume required has been based on the results of the site water balance model.

The conceptual water management system design has been developed on the basis that the Pit Water Dam would be sized to contain the full range of modelled historical inflows without discharge and without long-term in-pit water storage. This approach would minimise disruptions to mining operations, and reduce the potential for water quality to deteriorate in-pit.

Table 5.5 Design Storage Capacity of Saline Water Management Dams

Storage	Capacity (ML)
Pit Water Dam	24,220
Product Stockpile Dam	347
ROM Stockpile Dam	245
MIA Dam	160
ROM Dump South Dam	370
ROM Dump North Dam	577
ROM Dump X Dam	42
Dam A	201
Dam B	201

As the total Pit Water Dam capacity requirements are substantial, alternative water management approaches (which have not been modelled) will need to be considered to reduce the total Pit Water Dam storage requirement, including:

- Off-site discharge of excess water (to Alpha Creek or Tallarenha Creek) in accordance with release conditions to be specified in the Environmental Authority;
- Beneficial reuse (including treatment if necessary) to reduce the net site water surplus;
- In-pit water storage, in areas of the void which will not cause disruptions to operations in the open cut or underground operations.

5.3 PUMP TRANSFER RATES

The schematic diagram in Figure 5.3 shows how the site storages are interconnected. The following pump capacities have been adopted for the interlinking pipelines:

- Pit dewatering – 300L/s;
- Raw Water Dam pipeline supply – 200L/s;
- Sediment Dams South, Central, North to Pit Water Dam – 50L/s;
- MIA, ROM Stockpile, Product Stockpile Dams to Pit Water Dam – 50L/s;
- Dams A, B to Pit Water Dam – 100L/s.

5.4 PUMP OPERATING RULES

Supply to the CHPP, dust suppression and vehicle wash demands is based on the following order of priority:

1. Saline Water System;
2. Waste Rock Runoff System;
3. Raw Water System;

The potable water and underground demand are supplied solely from the Raw Water system. Details of the adopted operating rules in the model are outlined in the Table 5.6:

Table 5.6 WMS Operating Rules for OPSIM model

Item	Node Name	Operating Rules
1.0 1.1	*Water Supply* External Raw Water Supply Scheme	▪ Supplies to the Raw Water Dam as required at a maximum allocation of 3,000ML/a. ▪ Not available in Year 1.
2.0 2.1	*Water Demands* CHPP	▪ Supplied from the following locations in order of priority: – Pit Water Dam – Raw Water Dam ▪ Demands at the following rates: – Year 1: 2,719kL/d – Year 4: 5,757kL/d – Year 10: 9,028kL/d – Year 20: 8,316kL/d – Year 33: 5,054kL/d
2.2	North Haul Road Dust Suppression	▪ Supplied from the following locations in order of priority: – ROM Dump North Dam – Pit Water Dam

		– Dirty Water Dam – Raw Water Dam ▪ 100% loss assumed.
2.3	South Haul Road Dust Suppression	▪ Supplied from the following locations in order of priority: – ROM Dump X and South Dams – Dam A & Pit Water Dam – Sediment Dams South and Central – Raw Water Dam ▪ 100% loss assumed.
2.4	Vehicle Wash + Miscellaneous Use	▪ Supplied at a rate of 359kL/d from the following locations in order of priority: – Pit Water Dam – Raw Water Dam ▪ 100% loss assumed.
2.5	North Stockpile Dust Suppression	▪ Supplied at a rate of 548kL/d from the following locations in order of priority: – Pit Water Dam – Raw Water Dam ▪ 100% loss assumed.
2.6	South Stockpile Dust Suppression	▪ Supplied at a rate of 274kL/d from the following locations in order of priority: – ROM Dump X and South Dams – Dam A & Pit Water Dam – Sediment Dams South and Central – Raw Water Dam ▪ 100% loss assumed.
2.7	Underground Demand	▪ Supplied from the Raw Water Dam at a rate of 1,286kL/d. ▪ 100% loss assumed.
2.8	Potable Demand	▪ Supplied from the Raw Water Dam at the following rates: – Year 1: 169kL/d – Year 4, 10, 20 & 33: 229kL/d ▪ 100% loss assumed.
<u>3.0</u> 3.1	*<u>Operational Pits</u>* North Pit	▪ Continuous dewatering as required to Pit Water Dam at a nominal rate of 300L/s ▪ Receives groundwater inflows at the following rates: – Year 1: 1,025kL/d – Year 4: 1,901kL/d – Year 10: 1,872kL/d – Year 20: 1,856kL/d – Year 33: 1,765kL/d
3.2	South Pit	▪ Continuous dewatering as required to Pit Water Dam at a nominal rate of 300L/s ▪ Receives groundwater inflows at the following rates: – Year 1: 1,025 kL/d – Year 4: 1,901 kL/d – Year 10: 1,872 kL/d – Year 20: 1,856 kL/d – Year 33: 1,765 kL/d
<u>4.0</u> 4.1	*<u>Water Storages</u>* Pit Water Dam	▪ Primary saline water storage dam ▪ Maximum Operating Level allows for a nominal 620ML capacity to spillway ▪ Supplies to the following locations as required: – CHPP – Vehicle Wash & Miscellaneous Use – North & South Haul Road Dust Suppression – North & South Stockpile Dust Suppression ▪ Receives pumped transfers from the following locations: – North & South Pits – Dirty Water Dam – MIA Dam

		– ROM Stockpile Dam – Product Stockpile Dam – Dam A & Dam B ▪ Storages overflows to Tallarenha Creek
4.2	Raw Water Dam	▪ Primary raw water storage dam ▪ Demands from External Water Supply Scheme to maintain at 80% capacity at a nominal rate of 200L/s. ▪ Supplies to the following locations as required: – Potable Water Demand – Underground Demand – CHPP – North & South Haul Road Dust Suppression – North & South Stockpile Dust Suppression – Vehicle Wash & Miscellaneous Use ▪ Storage overflows to Alpha Creek
4.3	Dirty Water Dam	▪ Primary waste rock water storage dam ▪ Maximum Operating Level at set at a nominal 80% of capacity ▪ Receives pumped transfers from the following locations: – Sediment Dam South – Sediment Dam Central – Sediment Dam North ▪ Supplies to the following locations as required: – North Haul Road Dust Suppression – North Stockpile Dust Suppression – Pit Water Dam ▪ Storage overflows to Pit Water Dam
4.4	Sediment Dam South	▪ Supplies to the following locations as required: – South Haul Road Dust Suppression – South Stockpile Dust Suppression ▪ Pump transfers to Dirty Water Dam at >25ML inventory at a nominal rate of 50L/s. ▪ Storage overflows to Alpha Creek
4.5	Sediment Dam Central	▪ Supplies to the following locations as required: – South Haul Road Dust Suppression – South Stockpile Dust Suppression ▪ Pump transfers to Dirty Water Dam at >40ML inventory at a nominal rate of 50L/s. ▪ Storage overflows to Tallarenha Creek
4.6	Sediment Dam North	▪ Pump transfers to Dirty Water Dam at a nominal rate of 50L/s to maintain empty ▪ Storage overflows to Tallarenha Creek
4.7	MIA Dam, ROM Stockpile Dam, Product Stockpile Dam	▪ Pump transfers to Pit Water Dam to maintain empty at a nominal rate of 50L/s ▪ Storages overflow to Tallarenha Creek
4.8	ROM Dump X Dam	▪ Pump transfers to Pit Water Dam at a nominal rate of 100L/s. ▪ Supplies to the following locations as required: – South Haul Road Dust Suppression – South Stockpile Dust Suppression ▪ Storage overflows to Alpha Creek
4.9	ROM Dump S Dam	▪ Pump transfers to Pit Water Dam at a nominal rate of 100L/s. ▪ Supplies to the following locations as required: – South Haul Road Dust Suppression – South Stockpile Dust Suppression ▪ Storage overflows to Tallarenha Creek
4.10	ROM Dump N Dam	▪ Pump transfers to Pit Water Dam at a nominal rate of 100L/s. ▪ Supplies to the following locations as required:

		– North Haul Road Dust Suppression – North Stockpile Dust Suppression ▪ Storage overflows to Tallarenha Creek
4.11	Dam A & Dam B	▪ Storages overflow to operational pits
<u>*5.0*</u> 5.1	<u>*Receiving Waters*</u> Alpha Creek	▪ Receives storage overflows from the following locations: – Sediment Dam South – ROM Dump X Dam – Raw Water Dam
5.2	Tallarenha Creek	▪ Receives storage overflows from the following locations: – Sediment Dam Central & North – ROM Dump South & North Dams – Pit Water Dam – MIA Dam – ROM Stockpile Dam – Product Stockpile Dam

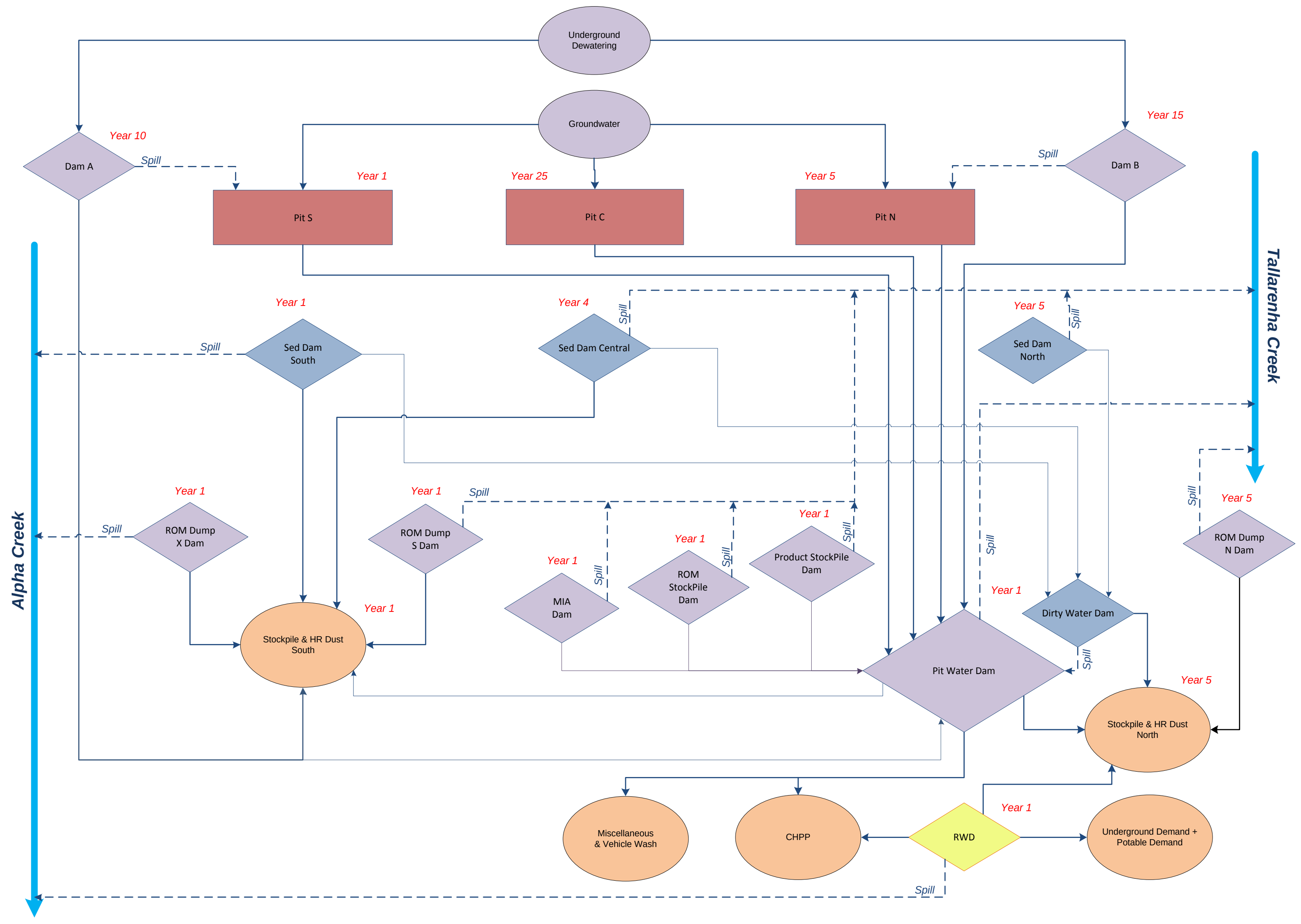

Figure 5.3 SGCP Water Management System – Schematic Diagram

6 MODEL RESULTS

6.1 EXAMPLE BEHAVIOUR

Figure 6.1 shows examples of the behaviour of the Pit Water Dam over a dry period of the climate record in 1940-41. Figure 6.1 shows demands initially being supplied from the raw water pipeline, until runoff to the site catchments results in inflows to the Pit Water Dam. A portion of demand for underground operations continues to be drawn from the raw water pipeline, but other supplies are drawn from the Pit Water Dam until the available supply is exhausted.

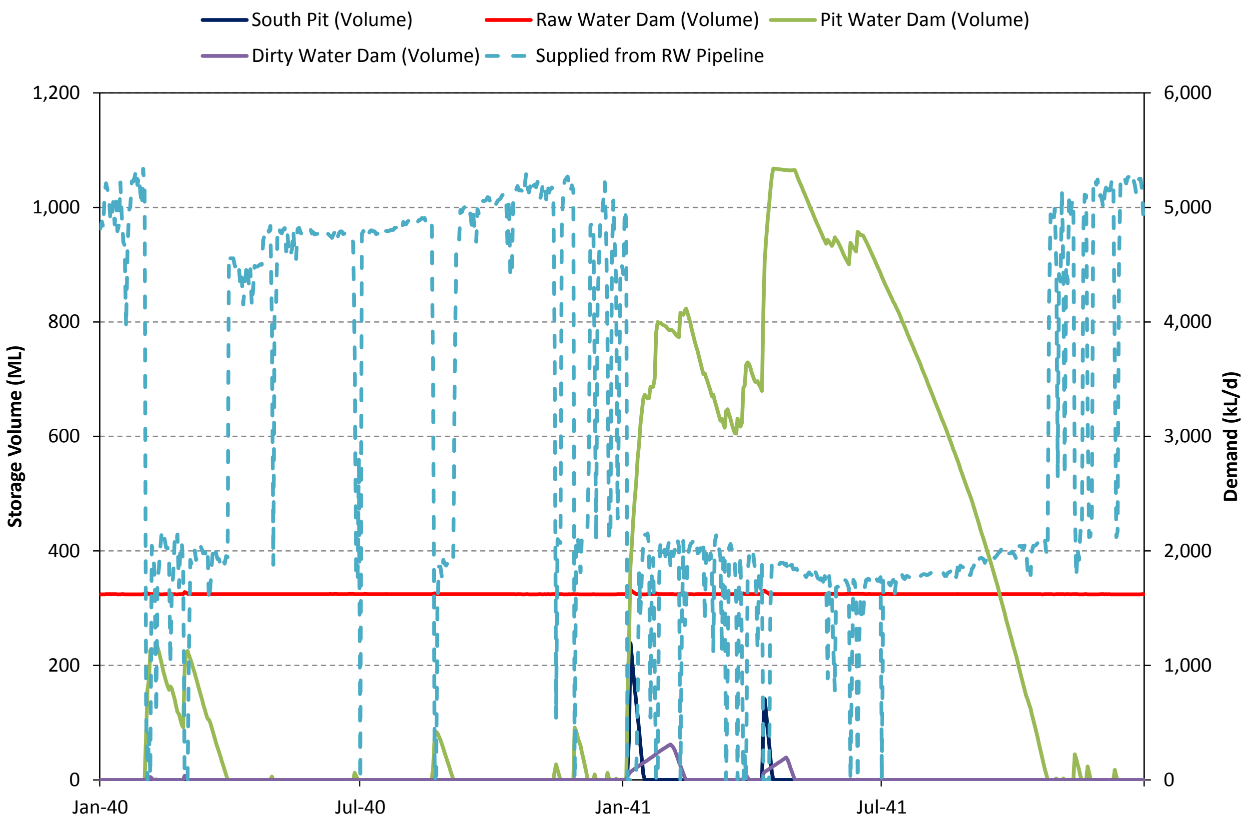

Figure 6.1 Pit Water Dam Behaviour

6.2 OVERALL SITE WATER BALANCE

The predicted overall average annual site water balance is summarised in Table 6.1 based on simulations of each mine stage over 123 years of historical climate data. In summary, the following observations can be made on the average annual water balance over the project life:

Outflows

- Total water demand ranges between approximately 3,010ML/a and 7,325ML/a;
- Total evaporation loss from dams ranges between approximately 878ML/a and 3,502ML/a;

Inflows

- Rainfall and runoff yield contributes between approximately 1,235ML/a and 2,448ML/a;
- Net groundwater inflows (to underground and open cut pits) contribute between approximately 0ML/a and 5,932ML/a;
- After Year 1, external, raw water requirements vary from approximately 659ML/a to 1,258ML/a.

Table 6.1 South Galilee Project Average Annual Water Balance

		Year 1	Year 4	Year 10	Year 20	Year 33
Parameters						
ROM Coal (Mtpa)						
	Open Cut	*5.61*	*4.79*	*5.04*	*5.17*	*5.9*
	Underground	*0*	*7.09*	*13.59*	*11.99*	*4.53*
	Total	*5.61*	*11.88*	*18.63*	*17.16*	*10.43*
Water Inputs (ML)						
Rainfall/Runoff Yield						
	Dirty Water System	*164*	*242*	*479*	*182*	*130*
	Saline Water System	*1,025*	*1,168*	*1,762*	*1,762*	*1,878*
	Clean Water System	*47*	*47*	*47*	*254*	*147*
	Total	1,235	1,457	2,448	2,199	2,156
Raw Water from Pipeline		0	1,258	659	659	1,106
Groundwater Inflow to Open Cut Pits		0	117	194	0	0
Groundwater Inflow to Underground		0	1,932	5,738	5,044	2,312
Gross Water Inputs		**1,235**	**4,764**	**9,038**	**7,902**	**5,574**
Water Outputs (ML)						
Evaporation from Dams and Ponds						
	Dirty Water System	*27*	*47*	*389*	*107*	*6*
	Saline Water System	*702*	*1,027*	*2,964*	*2,360*	*1,502*
	Clean Water System	*149*	*149*	*149*	*279*	*160*
	Total	878	1,223	3,502	2,746	1,668
Dam Overflows (off-site)						
	Dirty Water System	*2*	*0*	*43*	*3*	*0*
	Saline Water System	*0*	*0*	*69*	*0*	*0*
	Clean Water System	*0*	*0*	*0*	*12*	*0*
	Total	2	0	112	15	0
Net Loss from CHPP		656	2,103	3,298	3,037	1,846
Haul Road Dust Suppression		333	453	883	910	1,043
Vehicle Wash + Misc. Industrial Use		131	131	131	131	131
Stockpile Dust Suppression		300	300	300	300	300
Underground Water Use (Potable Quality)		0	470	470	470	470
Potable Water Demand		62	84	84	84	84
Gross Water Outputs		**2,363**	**4,763**	**8,780**	**7,693**	**5,541**
Water Balance (ML)						
Change in Storage		**14**	**1**	**258**	**209**	**29**
Gross Water Balance (Deficit)		**1,141**	**0**	**0**	**0**	**2**

6.3 EXTERNAL WATER SUPPLY PIPELINE DEMAND

The primary water source for site water demands is the Pit Water Dam. The Raw Water Dam will supply potable water and underground water demands and will source water from an external raw water supply source. When the Pit Water Dam is unable to meet the site demands, the shortfall will also be met from this source.

Initial plans are for off-lease water to be connected to the Project by Q1 2015, which means a raw water supply will not be available during Year 1 of the Project.

Table 6.2 shows the modelled demand for water from the external water source over the period of historical climate data.

Table 6.2 Annual Raw Water Demand from the Raw Water Pipeline (Financial Year)

	Annual Raw Water Demand from the Raw Water Pipeline (ML/a)			
Financial Year	90%ile	50%ile	10%ile	1%ile
0/1	0	0	0	0
1/2	320	677	810	866
2/3	791	1,363	1,606	1,696
3/4	740	1,354	1,601	1,691
4/5	726	1,144	1,391	1,517
5/6	627	668	889	1,016
6/7	629	661	827	1,021
7/8	627	659	735	1,016
8/9	627	658	684	1,016
9/10	627	658	682	978
10/11	629	660	684	703
11/12	624	657	682	701
12/13	624	657	682	692
13/14	624	657	682	692
14/15	626	659	683	692
15/16	624	656	681	690
16/17	624	657	681	690
17/18	627	658	682	699
18/19	626	660	685	762
19/20	624	659	686	838
20/21	627	661	691	880
21/22	627	661	690	931
22/23	629	663	690	933
23/24	624	661	686	930
24/25	624	663	686	931
25/26	624	663	686	931
26/27	626	665	690	1,234
27/28	625	665	691	1,662

28/29	629	665	691	1,662
29/30	629	665	1,297	1,662
30/31	632	667	1,384	1,668
31/32	630	666	1,381	1,666
32/33	629	666	1,517	1,666
33/34	325	343	788	936

**Note: Raw Water Pipeline not available until Q1 2015.*

Figure 6.3 and Figure 6.4 below shows how the site water demand would have been supplied for each of the modelled stages for the period of the historical climate record.

Review of the results indicates that in most years, the water management system will make a significant contribution to the site water supply. However, every stage requires a proportion of annual supply to be drawn from the Raw Water Pipeline.

During the modelled Year 1 stage, the initially planned external raw water supply source would not yet be complete. As a result, the expected demand may need to be met from another (as yet unidentified) temporary raw water source.

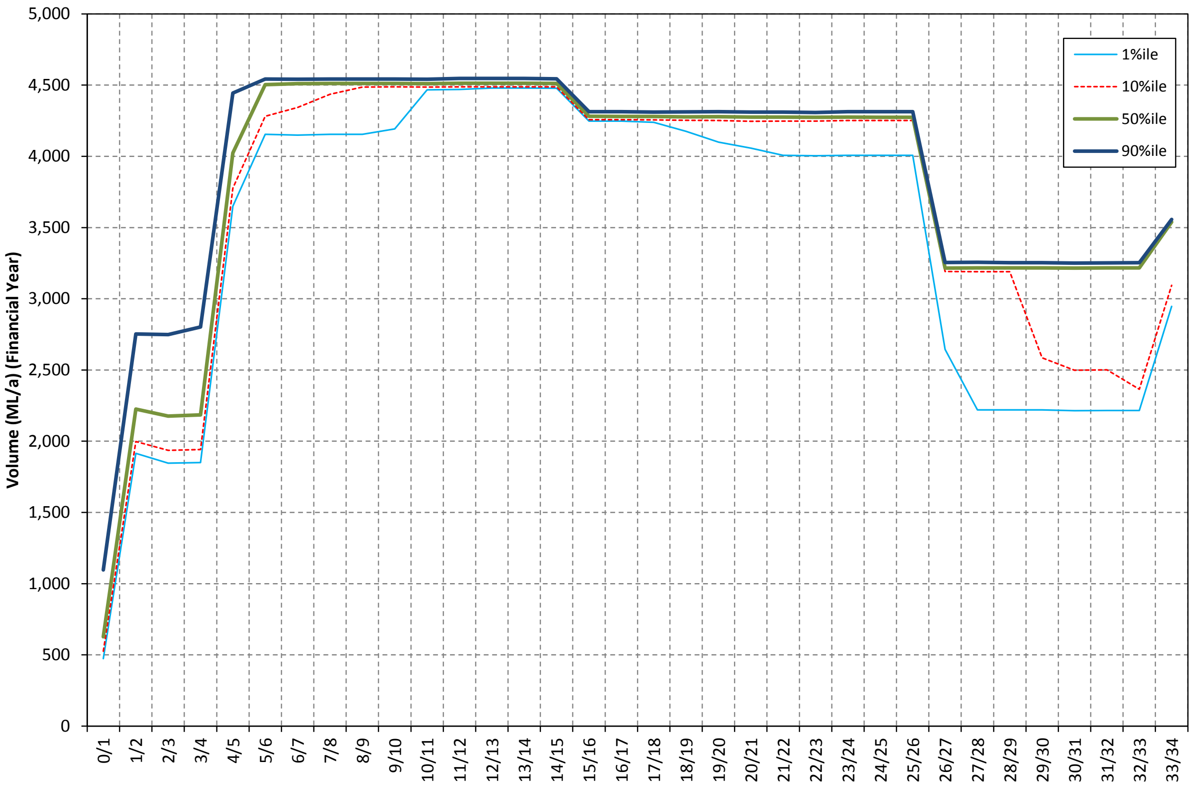

Figure 6.2 Simulated Water Supplies – Mine Water Management System (Financial Year)

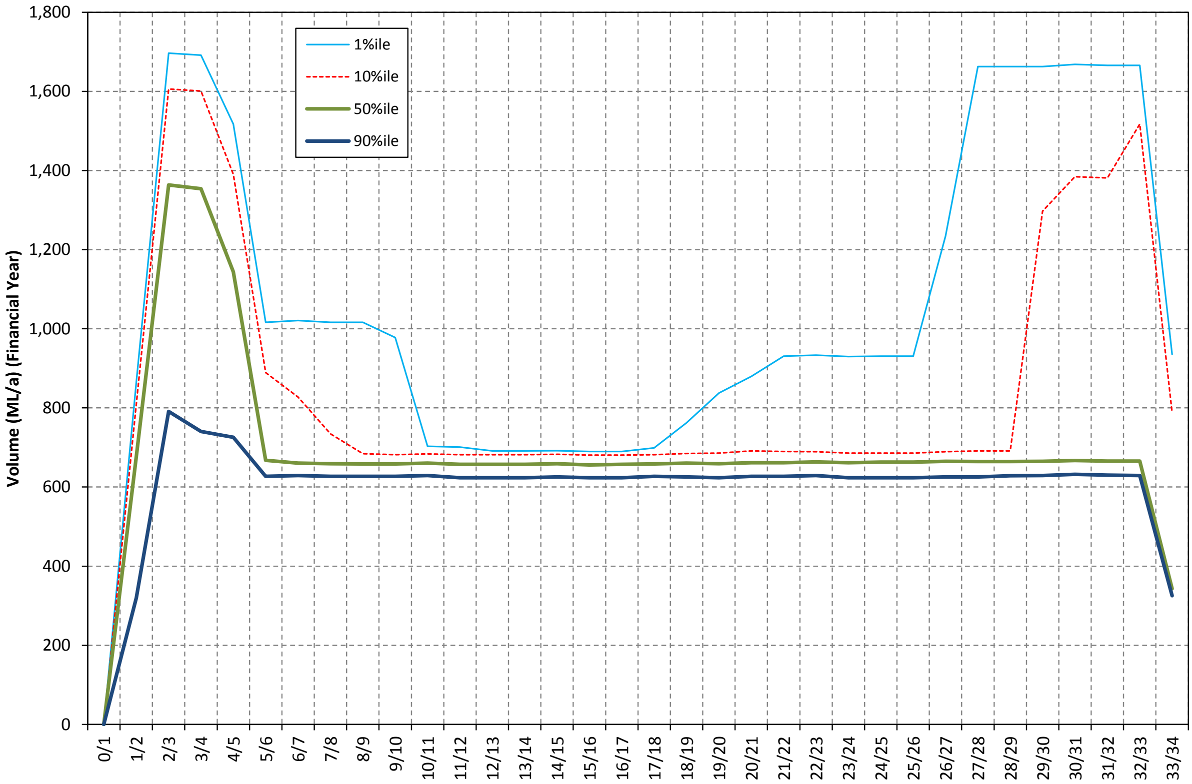

Figure 6.3 Simulated Water Supplies – Raw Water Pipeline (Financial Year)

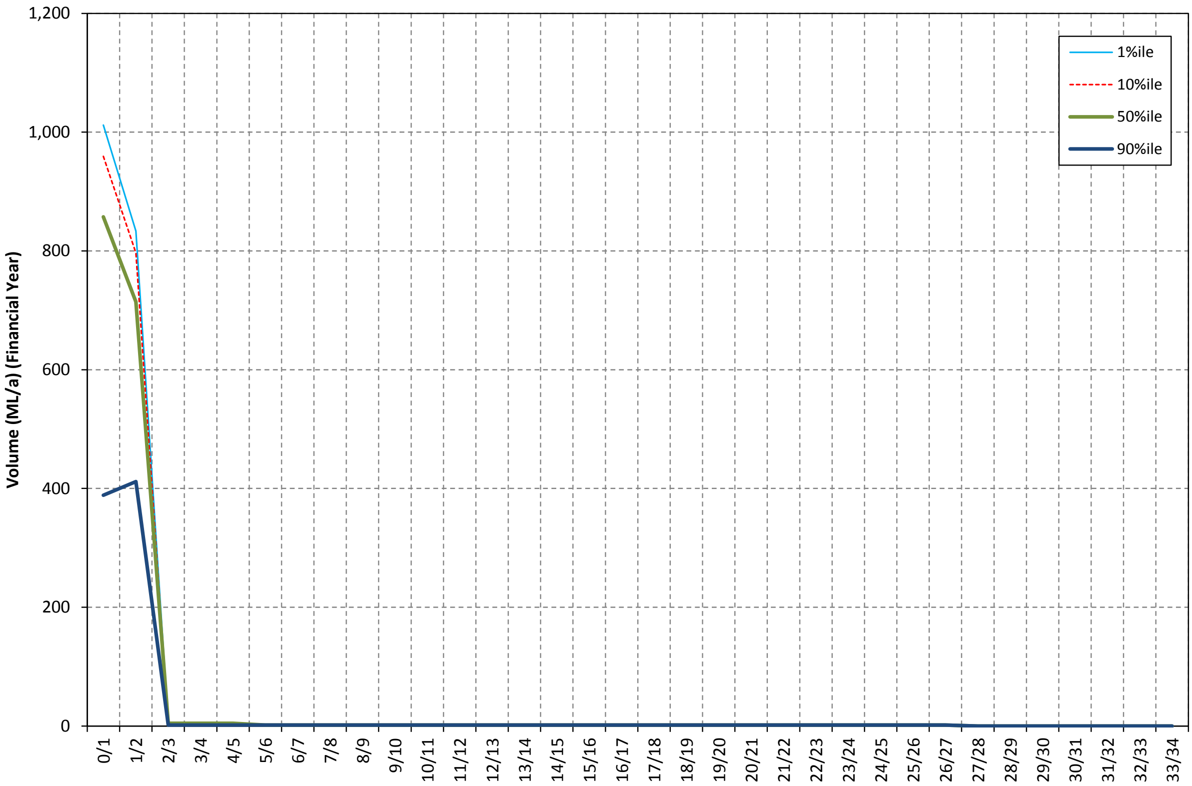

Figure 6.4 Simulated Water Supplies – Unidentified External Source (Financial Year)

6.4 UNCONTROLLED OFFSITE DISCHARGES

The water balance model results show that there are no simulated uncontrolled discharges from the saline or dirty water systems for the 1 percentile confidence trace in any year of the Project life.

6.5 PIT AVAILABILITY

Figure 6.5 and Figure 6.6 show the simulated inventory stored in the pits during the Project life.

Inundation risks are generally very low due to the very large modelled capacity of the Pit Water Dam. Inundation risks are highest in Years 26 to 33 in the North Pit, and Years 15 to 26 in the South Pit. For all years, the 10th percentile confidence trace shows minor pit inundation (up to 300ML in the North Pit and 1,000ML in the South Pit). The median (50th percentile) confidence trace shows no pit inundation for both pits. There is a 1% probability of accumulating 1,600ML in North Pit and 2,500ML in the South Pit.

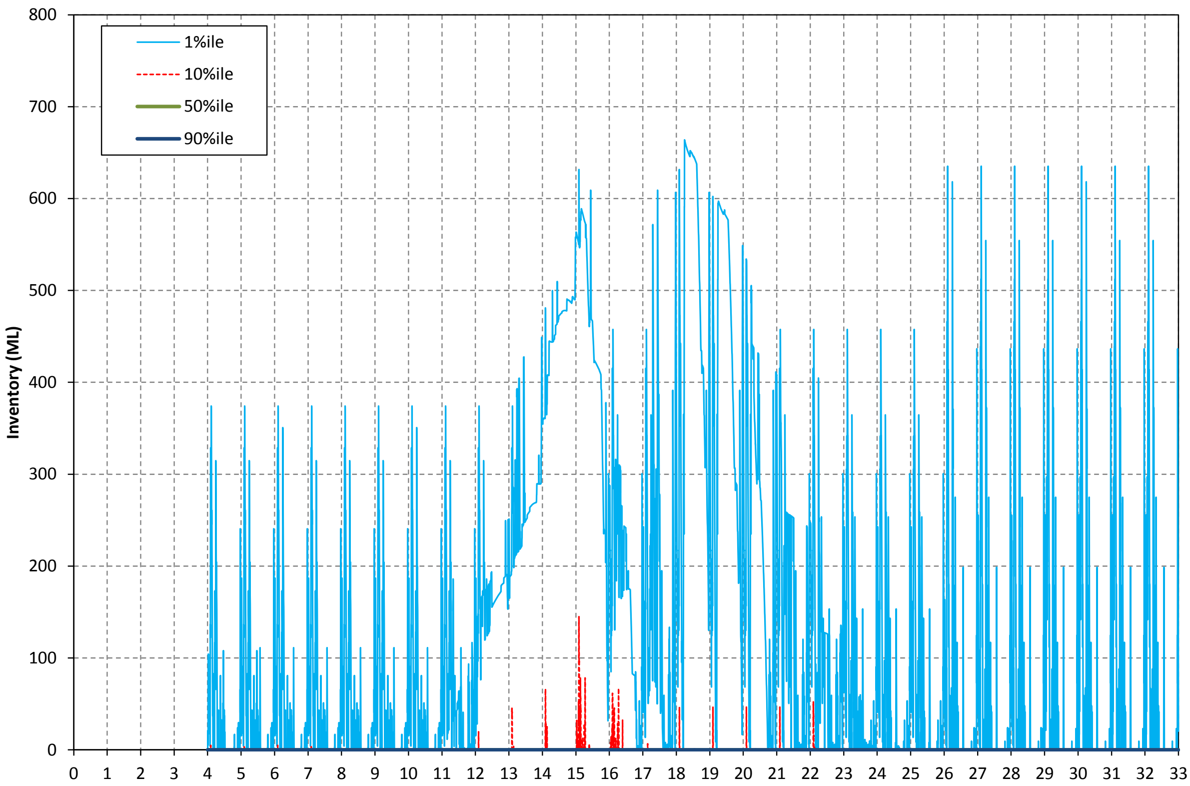

Figure 6.5 North Pit Stored Inventory

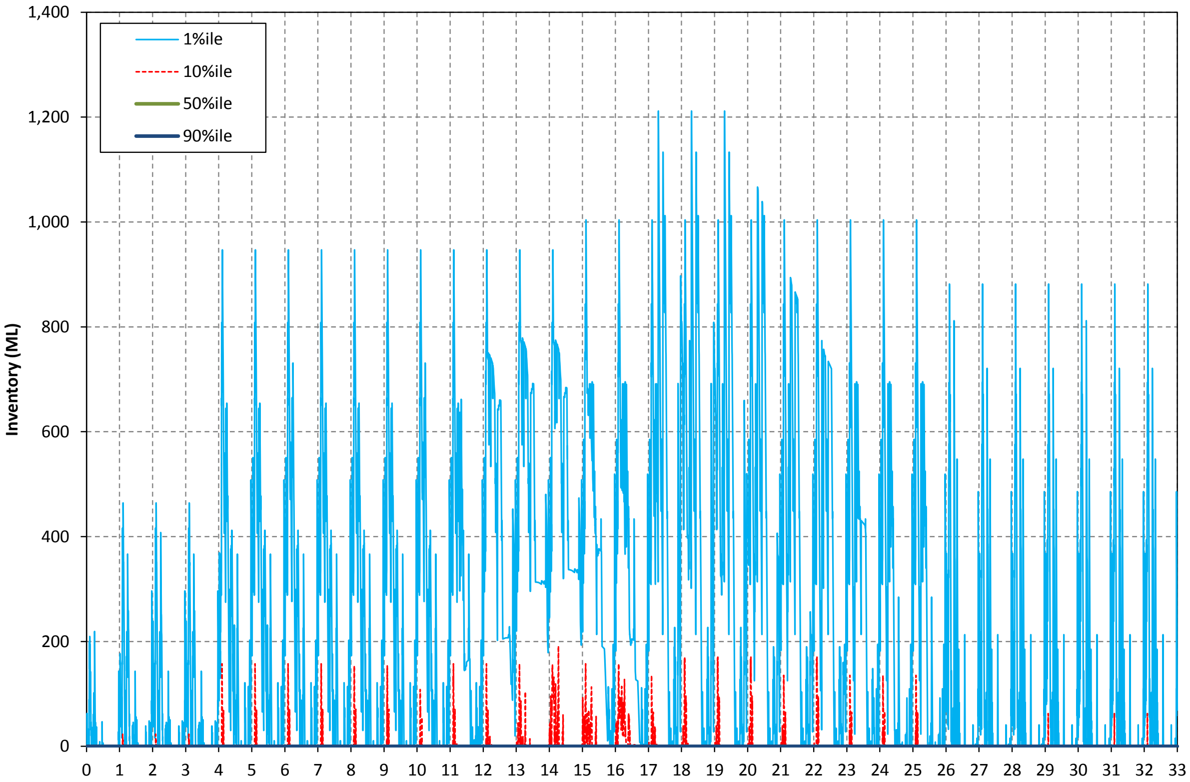

Figure 6.6 South Pit Stored Inventory

6.6 PIT WATER DAM INVENTORY

Figure 6.7 shows the simulated inventory stored in the Pit Water Dam over the Project life.

The results indicate that for the median (50 percentile) confidence trace, the Pit Water Dam accumulates water for the first 24 years of the Project, and thereafter decreases in inventory.

During the early stages (first four years), the Pit Water Dam is not likely to require greater than 5,000ML capacity for a 1 percentile confidence trace. During Years 12 to 24, the Pit Water Dam is operating at or near its maximum operating level of 23,600ML for the 1 percentile confidence trace. This would result in a risk of a greater volume of water being retained in the active mining pits and potentially impacting on production.

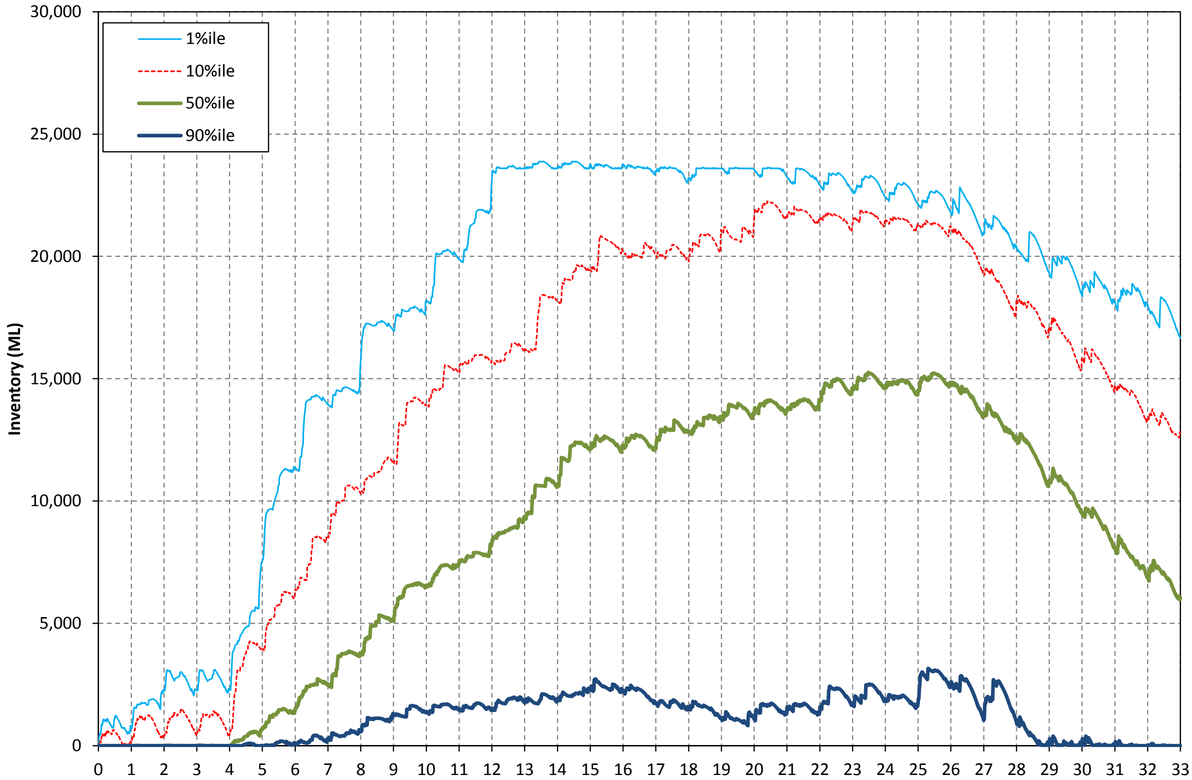

Figure 6.7 Pit Water Dam Stored Inventory

7 SENSITIVITY ANALYSIS – HIGH RUNOFF CASE

7.1 OVERVIEW

Further analysis was undertaken to assess the sensitivity of the system behaviour to an increase in the runoff to rainfall ratio. Under this scenario, the AWBM runoff parameters were modified as summarised in Table 7.1.

Table 7.1 AWBM Parameters for High Runoff Scenario (ML/a)

Parameter	Natural		Active Waste	
	Normal Case	'High runoff' case	Normal Case	'High runoff' case
A1	0.1	0.2	0.136	0.2
A2	0.3	0.4	0.270	0.4
C1	70	60	50	60
C2	135	90	100	90
C3	500	180	500	180
BFI	0.03	0.03	0.103	.03
Kb	0.98	0.98	1	0.98
Long Term Runoff Coefficient	3.3%	9.1%	4.7%	9.1%

7.2 OVERALL SITE WATER BALANCE

The revised site water balance over the life of the project is presented in Table 7.2. On average:

Outflows

- Total water demand ranges between approximately 3,250ML/a and 7,970ML/a;
- Evaporation ranges between approximately 1,050ML/a and 3,974ML/a;

Inflows

- Runoff yield is increased to contribute between approximately 1,577ML/a and 3,528ML/a;
- Net groundwater inflows (to underground and open cut pits) are unchanged and contribute between approximately ,0ML/a and 5,932ML/a;
- After Year 1, raw water requirements vary from approximately 658ML/a to 1,138ML/a. These are essentially unchanged from the low runoff case.

Table 7.2 South Galilee Project Annual Water Balance (High Runoff) (ML/a)

		Year 1	Year 4	Year 10	Year 20	Year 33
Parameters						
ROM Coal (Mtpa)						
	Open Cut	*5.61*	*4.79*	*5.04*	*5.17*	*5.9*
	Underground	*0*	*7.09*	*13.59*	*11.99*	*4.53*
	Total	*5.61*	*11.88*	*18.63*	*17.16*	*10.43*
Water Inputs						
Rainfall Yield						
	Dirty Water System	*350*	*488*	*972*	*427*	*307*
	Saline Water System	*1,180*	*1,293*	*2,510*	*2,443*	*2,558*
	Clean Water System	*47*	*47*	*47*	*581*	*330*
	Total	1,577	1,827	3,528	3,451	3,195
Raw Water from Pipeline		0	1,138	658	659	793
Groundwater Inflow to Open Cut Pits		0	117	194	0	0
Groundwater Inflow to Underground		0	1,932	5,738	5,044	2,312
Gross Water Inputs		**1,577**	**5,014**	**10,118**	**9,154**	**6,301**
Water Outputs						
Evaporation from Dams and Ponds						
	Dirty Water System	*58*	*70*	*518*	*227*	*23*
	Saline Water System	*842*	*1,170*	*3,306*	*2,820*	*2,001*
	Clean Water System	*149*	*149*	*149*	*405*	*184*
	Total	1,050	1,389	3,974	3,452	2,208
Dam Overflows (off-site)						
	Dirty Water System	*82*	*71*	*337*	*109*	*4*
	Saline Water System	*0*	*0*	*92*	*34*	*0*
	Clean Water System	*0*	*0*	*0*	*184*	*47*
	Total	82	71	428	327	51
Net Loss from CHPP		656	2,103	3,298	3,037	1,846
Haul Road Dust Suppression		336	53	883	917	1,051
Vehicle Wash + Misc. Industrial Use		131	131	131	131	131
Stockpile Dust Suppression		300	300	300	300	300
Underground Water Use (Potable Quality)		0	470	470	470	470
Potable Water Demand		62	84	84	84	84
Gross Water Outputs		**2,615**	**5,000**	**9,568**	**8,711**	**6,133**
Water Balance						
Change in Storage Volume		**14**	**15**	**550**	**443**	**166**
Gross Water Balance (Deficit)		**1,051**	**0**	**0**	**0**	**0**

7.3 EXTERNAL WATER SUPPLY PIPELINE DEMAND

Further details of the modelled demand from the external raw water source under the high runoff sensitivity analysis scenario are presented in Table 7.3 and Figure 7.1 to Figure 7.3 below.

Table 7.3 Annual Raw Water Demand from the Raw Water Pipeline (High Runoff) (Financial Year)

	Annual Raw Water Demand from the Raw Water Pipeline (ML/a)			
Financial Year	90%ile	50%ile	10%ile	1%ile
0/1	0	0	0	0
1/2	320	677	810	866
2/3	791	1,363	1,606	1,696
3/4	740	1,354	1,601	1,691
4/5	726	1,144	1,391	1,517
5/6	627	668	889	1,016
6/7	629	661	827	1,021
7/8	627	659	735	1,016
8/9	627	658	684	1,016
9/10	627	658	682	978
10/11	629	660	684	703
11/12	624	657	682	701
12/13	624	657	682	692
13/14	624	657	682	692
14/15	626	659	683	692
15/16	624	656	681	690
16/17	624	657	681	690
17/18	627	658	682	699
18/19	626	660	685	762
19/20	624	659	686	838
20/21	627	661	691	880
21/22	627	661	690	931
22/23	629	663	690	933
23/24	624	661	686	930
24/25	624	663	686	931
25/26	624	663	686	931
26/27	626	665	690	1,234
27/28	625	665	691	1,662
28/29	629	665	691	1,662
29/30	629	665	1,297	1,662
30/31	632	667	1,384	1,668
31/32	630	666	1,381	1,666
32/33	629	666	1,517	1,666
33/34	325	343	788	936

**Note: Raw Water Pipeline not available until Q1 2015.*

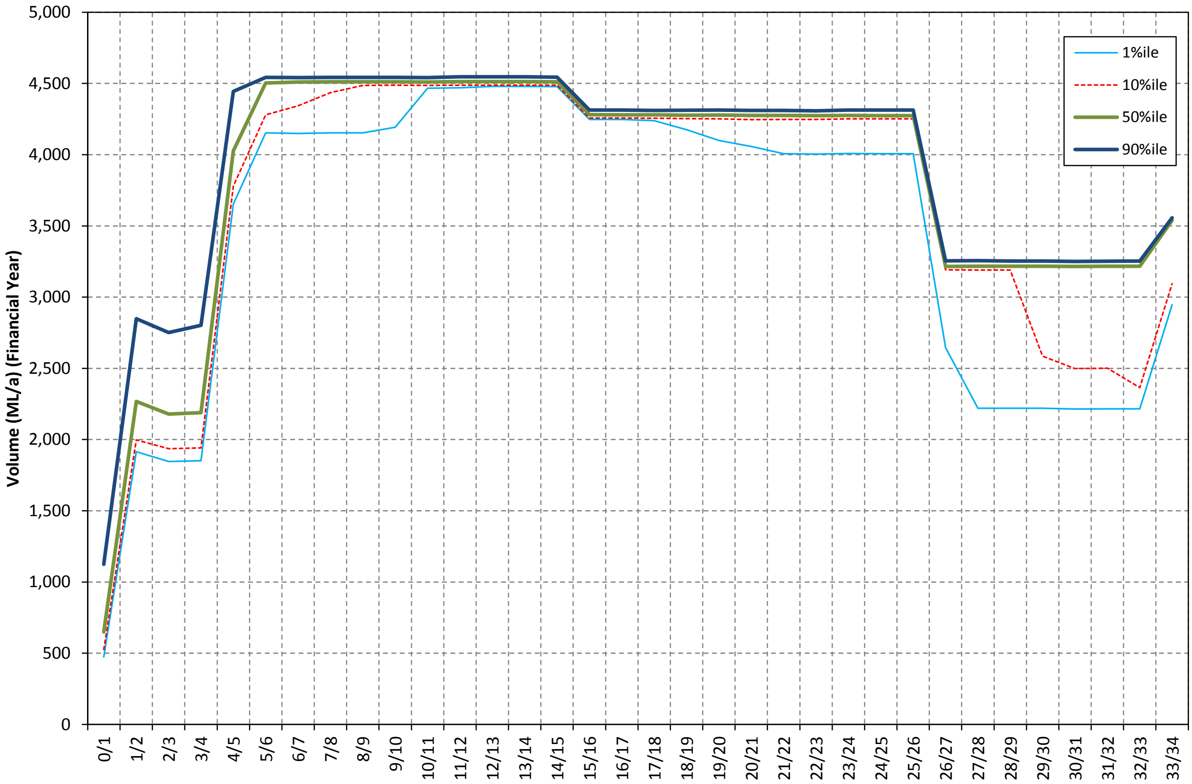

Figure 7.1 Simulated Water Supplies from Mine Water Management System (Financial Year) – High Runoff

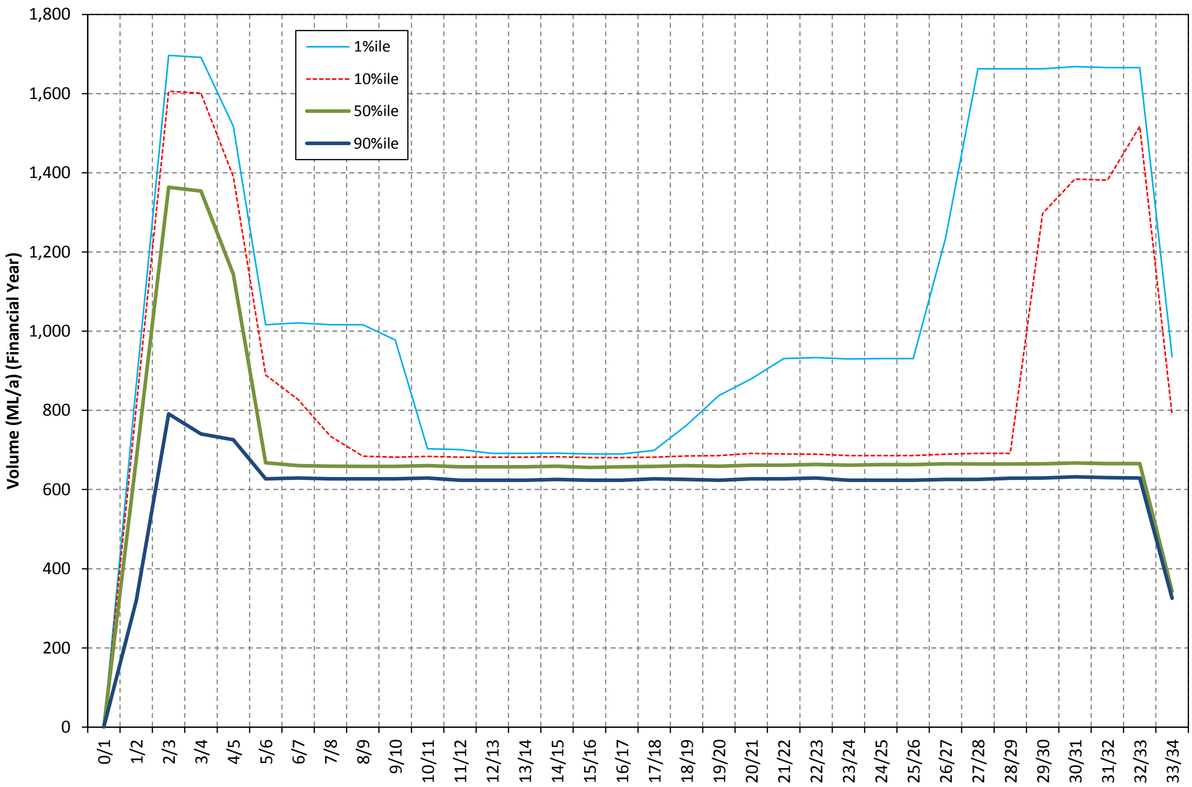

Figure 7.2 Simulated Water Supplies Raw Water Pipeline (Financial Year) – High Runoff

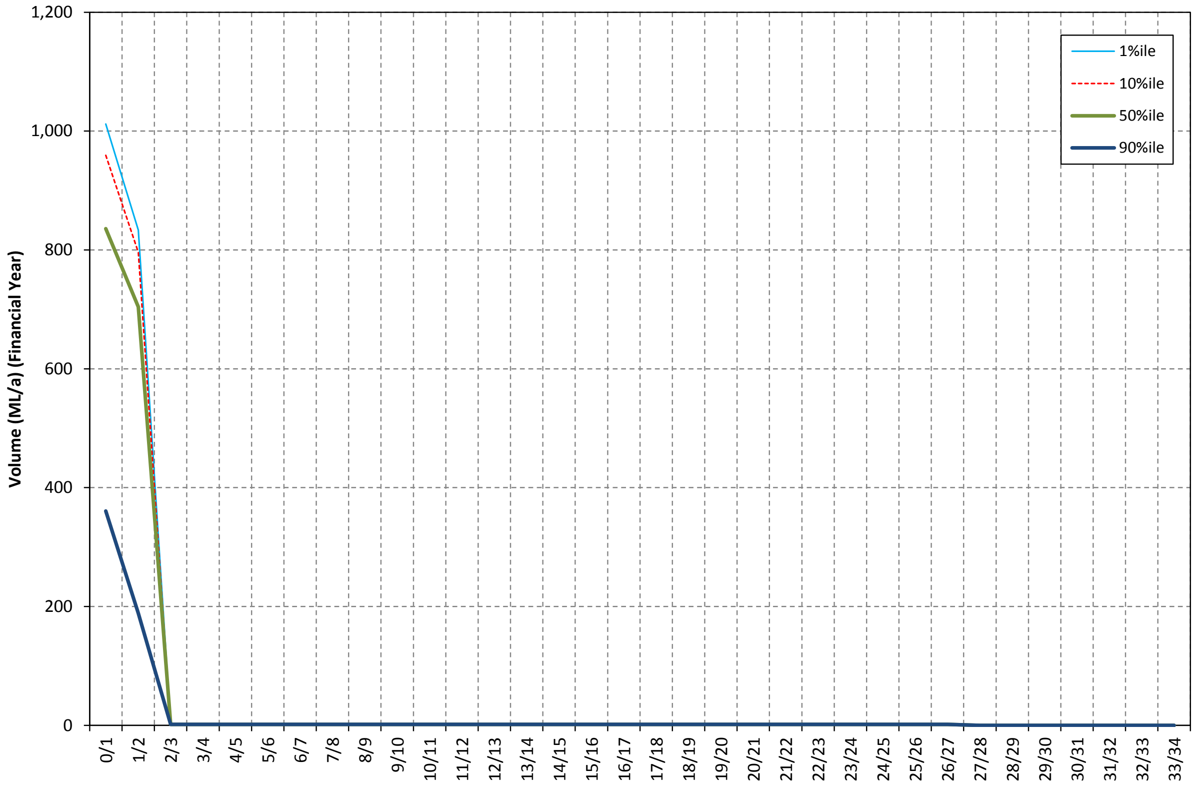

Figure 7.3 Simulated Water Supplies Unidentified External Source (Financial Year) – High Runoff

7.4 UNCONTROLLED OFFSITE DISCHARGES

The water balance model results show that there are no simulated uncontrolled discharges from the saline water system for the 1 percentile confidence trace in any year of the Project life.

Total simulated uncontrolled discharges from the dirty and clean water systems are shown in Figure 7.4 and Figure 7.5 respectively.

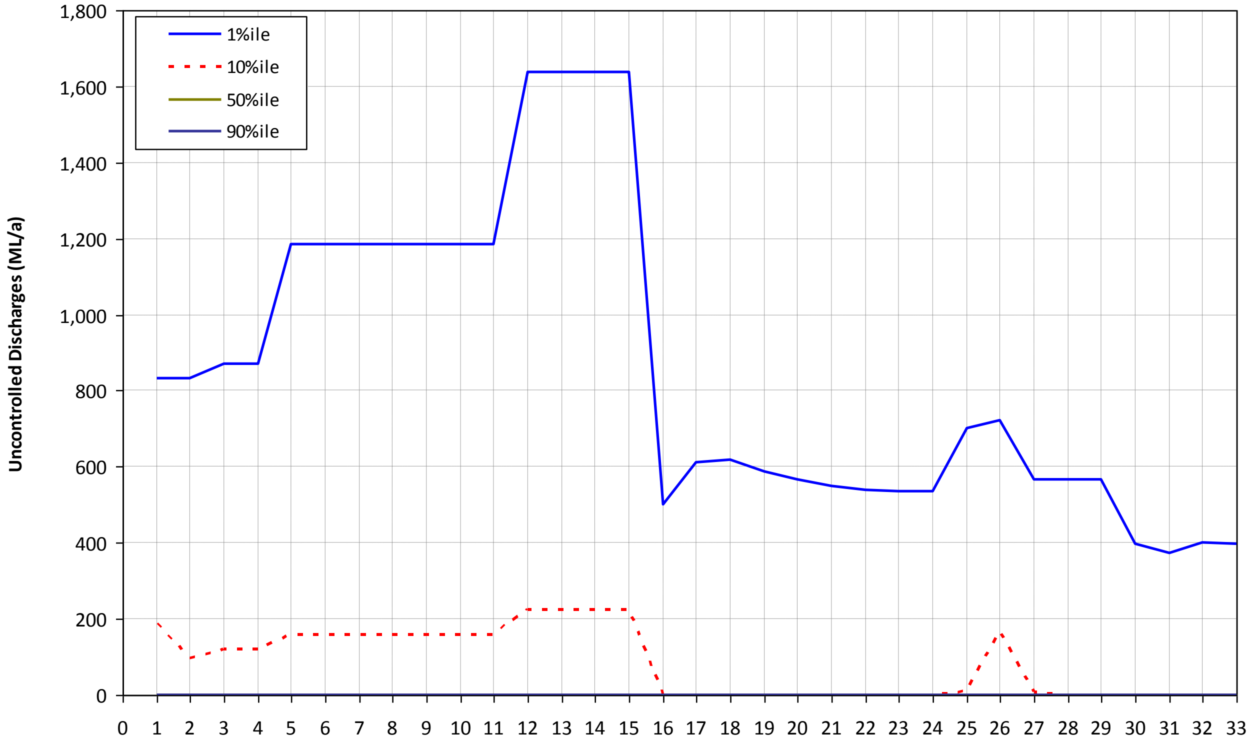

Figure 7.4 Uncontrolled Offsite Discharge Volumes – Dirty Water System

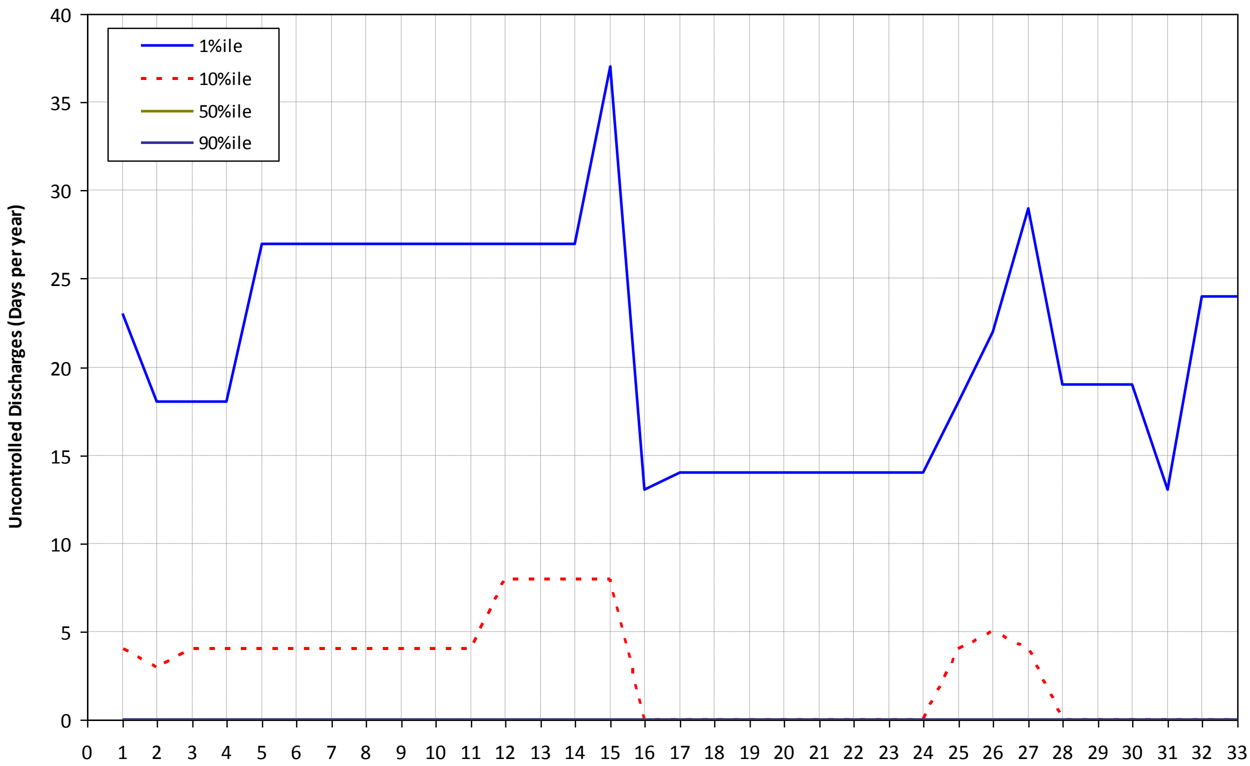

Figure 7.5 Uncontrolled Offsite Discharge Days – Dirty Water System

7.5 PIT AVAILABILITY

Figure 7.6 and Figure 7.7 show the simulated inventory stored in the pits during the Project life for the high runoff sensitivity analysis scenario.

Inundation risks are highest in Year 33 for the North Pit and Years 15 to 25 for the South Pit. For Year 33, there is a 1 percentile confidence trace of 1,600ML accumulating in the North Pit. For Years 15 to 25, there is a 1 percentile confidence trace of accumulating 2,500ML in the South Pit.

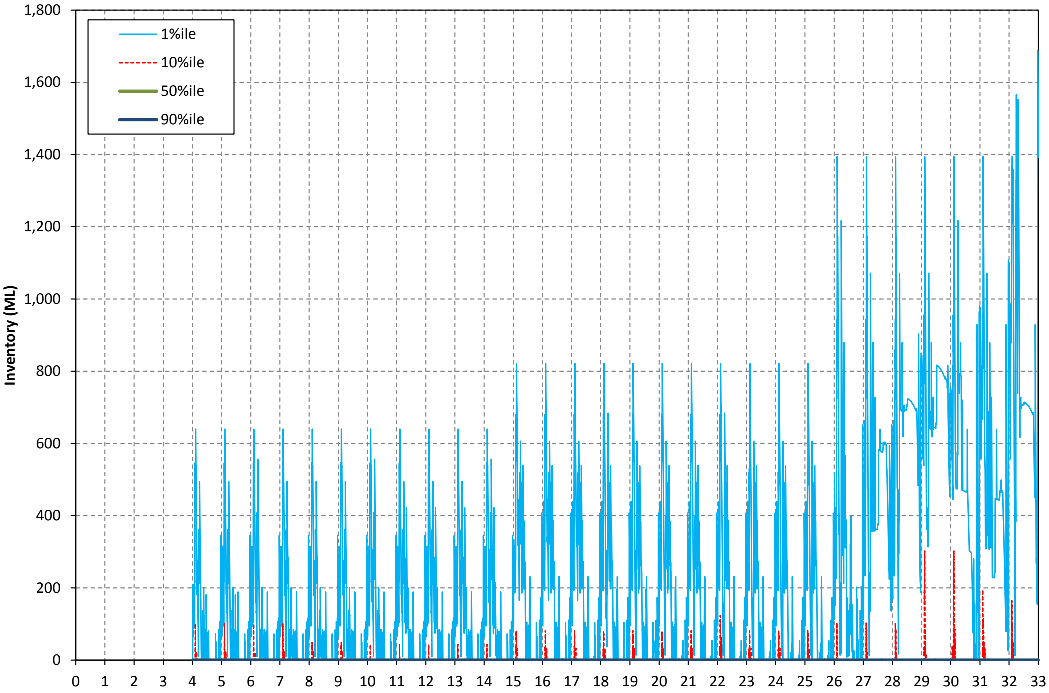

Figure 7.6 North Pit Stored Inventory – High Runoff

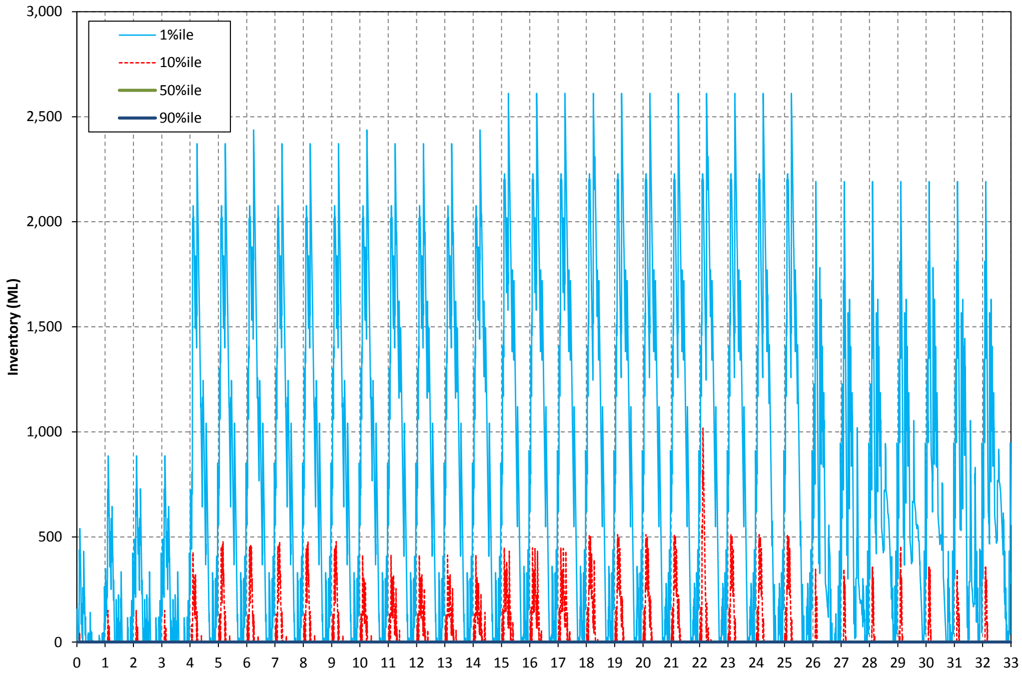

Figure 7.7 South Pit Stored Inventory – High Runoff

7.6 PIT WATER DAM INVENTORY

Figure 7.8 shows the simulated inventory stored in the Pit Water Dam over the Project life for the high runoff sensitivity analysis scenario. The maximum pit water dam capacity requirement for the high runoff sensitivity analysis scenario is 46,700ML (when modelled with the same footprint as for the low runoff scenario).

The results indicate that for the median (50 percentile) confidence trace, the Pit Water Dam accumulates water for the first 24 years of the Project, and thereafter decreases slightly in inventory.

During the early stages (first four years), the Pit Water Dam is not likely to require greater than 10,000ML capacity for a 1 percentile confidence trace. During Years 26 to 33, the Pit Water Dam is operating at or near its maximum operating level of 46,100ML for the 1 percentile confidence trace. This would result in a greater volume of water being retained in the active mining pits and potentially impacting on production.

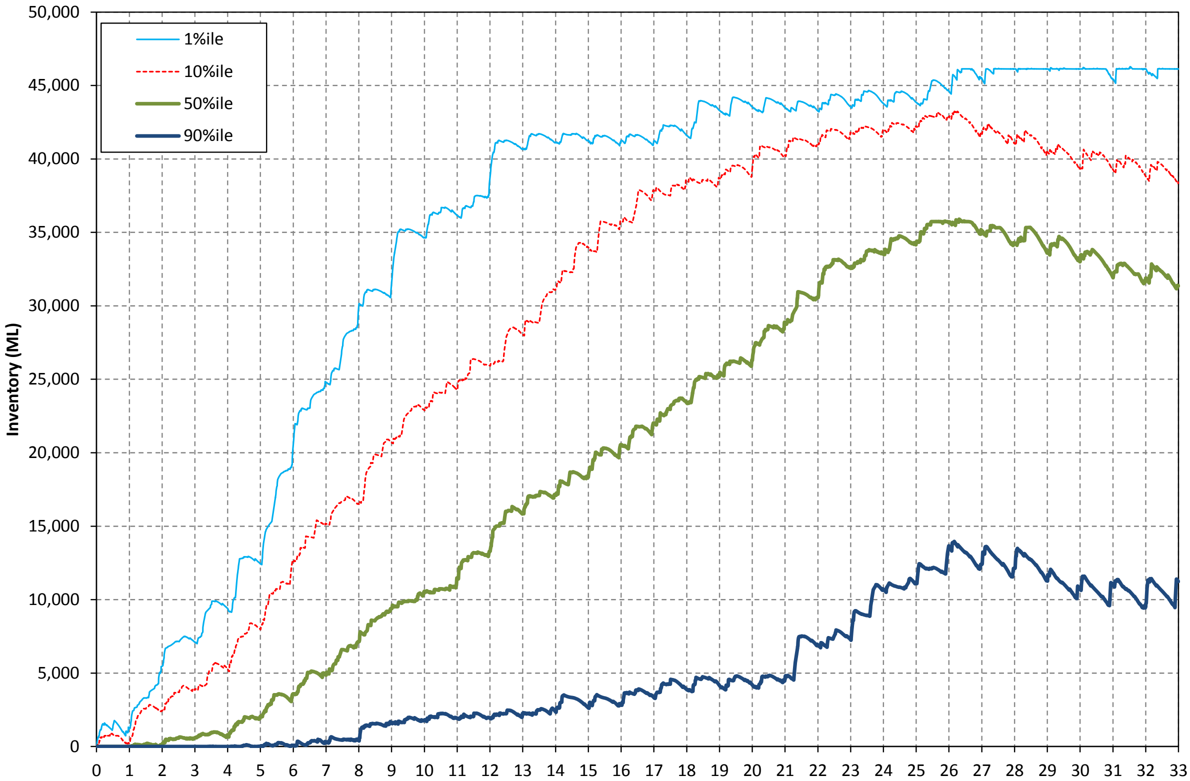

Figure 7.8 Pit Water Dam Stored Inventory – High Runoff

8 MODEL LIMITATIONS

The water balance analysis results should be interpreted with a number of potential uncertainties in mind:

- Catchment response to rainfall – mine site catchment behaviour can vary significantly from site to site (and from pit to pit). In the absence of sufficient site-specific data, AWBM model parameters have been adopted from experience with models at Bowen Basin mine sites.

- System operation – the model assumes the water management system is operated in a systematic, and predictable way. In reality, day-to-day water management decisions can be driven by other operational imperatives, and this will affect the system performance.

- Groundwater inflows – the predicted groundwater inflows make up a significant proportion of the water balance. If the actual groundwater inflows are significantly different from the modelled inflows, the implications could be potential water shortages or increased risk of discharge.

9 REFERENCES

Boughton, 2003	Boughton, *Calibrations of the AWBM for use on Ungauged Catchments,* Technical Report 03/15*,* Co-operative Research Centre for Catchment Hydrology, December 2003.
Pilgrim (1999)	'Australian Rainfall and Runoff. A Guide to Flood Estimation'. Book VI, Estimation of Large and Extreme Floods, Nathan, R.J. and Weinmann, P.E. (Ed.s), Revised Edition, Institution of Engineers Australia, 1999.
RPS Aquaterra (2012)	'*South Galilee Coal Project (SGCP) Groundwater Assessment and Modelling*', RPS Aquaterra, October 2012.
Water Solutions (2010)	'OPSIM Reference Manual - A General Purpose Operational Simulation Modelling System for the Management, Design and Assessment of Water Resources Systems', Water Solutions Pty Ltd 2010.

B APPENDIX

CATCHMENT LAND USE TYPES

LIST OF TABLES

LIST OF FIGURES

1 WATER MANAGEMENT SYSTEM LAYOUT

The WMS layout will evolve over the 33 year mine life. As additional catchments are disturbed, new sediment dams will be brought on line to treat waste rock runoff. For the purposes of concept design, and impact assessment, a number of stages have been assumed to be in place as follows:

Table 1.1 Mine Stages

Stage Label	Period	Duration
Stage Y1	2013-2015	3 years
Stage Y4	2016-2016	1 years
Stage Y5	2017-2021	5 years
Stage Y10	2022-2026	5 years
Stage Y15	2027-2031	5 years
Stage Y20	2032-2036	5 years
Stage Y25	2037-2041	5 years
Stage Y30	2042-2024	5 years
Stage Y33	2045-2047	2 years

Figure 1.1 to Figure 1.9 show plans of the conceptual water management layout at each of the stages listed in Table 1.1. Table 1.2 to Table 1.10 show the different types of land use areas draining to each of the proposed water management dams.

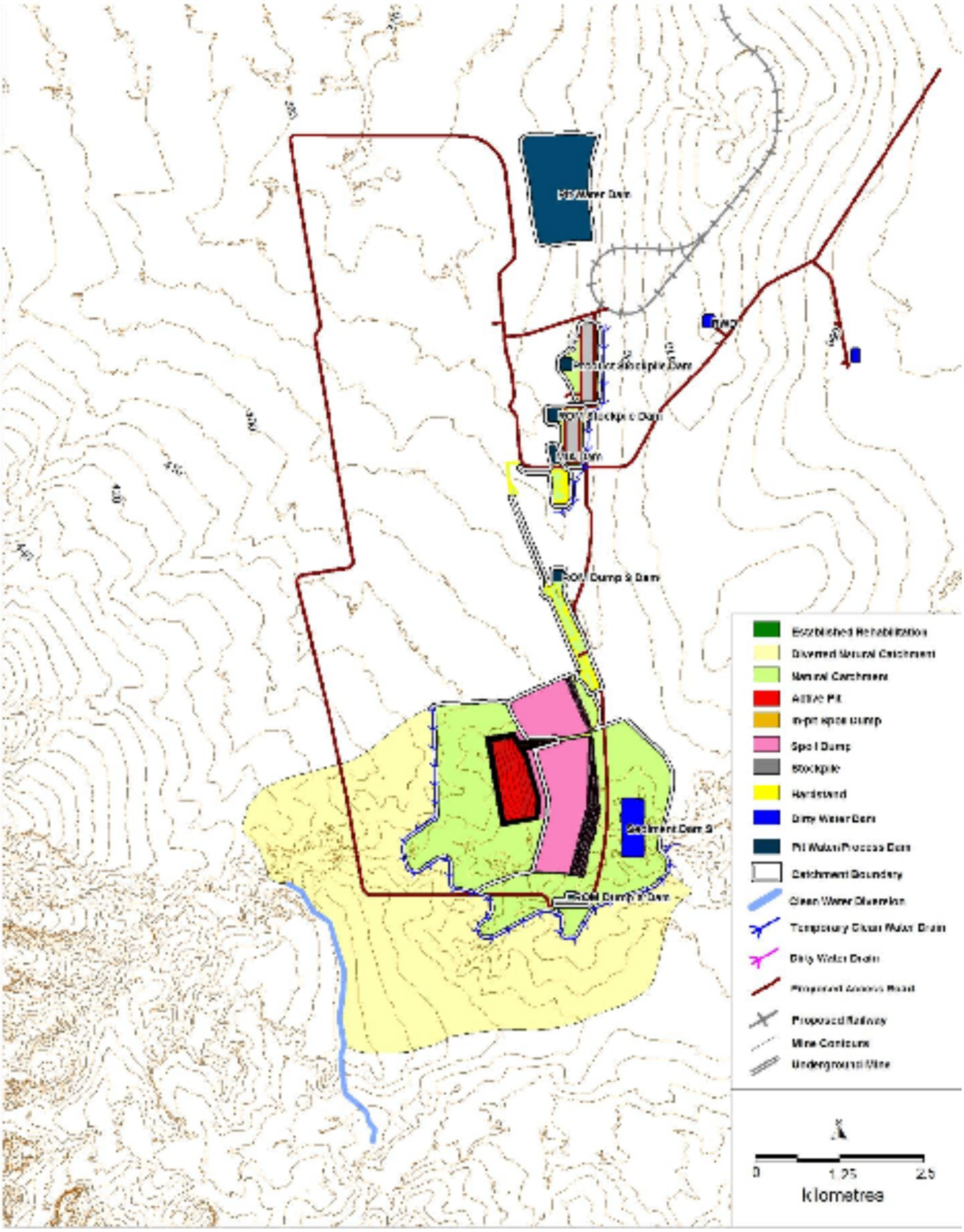

Figure 1.1 Y1 Water Management System Layout

Table 1.2 Catchment Areas Draining to Site Water Storages in Y1

Storage Name	Land Use Classification (ha)							Total Area (ha)
	Natural (grassed)	Stockpile	Hardstand	Active Pit	In Pit Dump	Out Pit Dump	Established Rehabilitation	
North Pit								0.0
South Pit	320.9			87.0		19.2		427.1
Sed Dam N								0.0
ROM Dump N Dam								0.0
Product Stockpile Dam	28.0	26.3						54.3
ROM Stockpile Dam	24.6	13.6						38.3
MIA Dam	15.2		9.9					25.1
Sed Dam Central								0.0
ROM Dump S Dam	36.2		21.3					57.5
Sediment Dam S	383.1		3.0			243.4		629.5
ROM Dump X Dam	3.0	3.4						6.5
Total	811.1	43.4	34.2	87.0	0.0	262.6	0.0	1,238.3

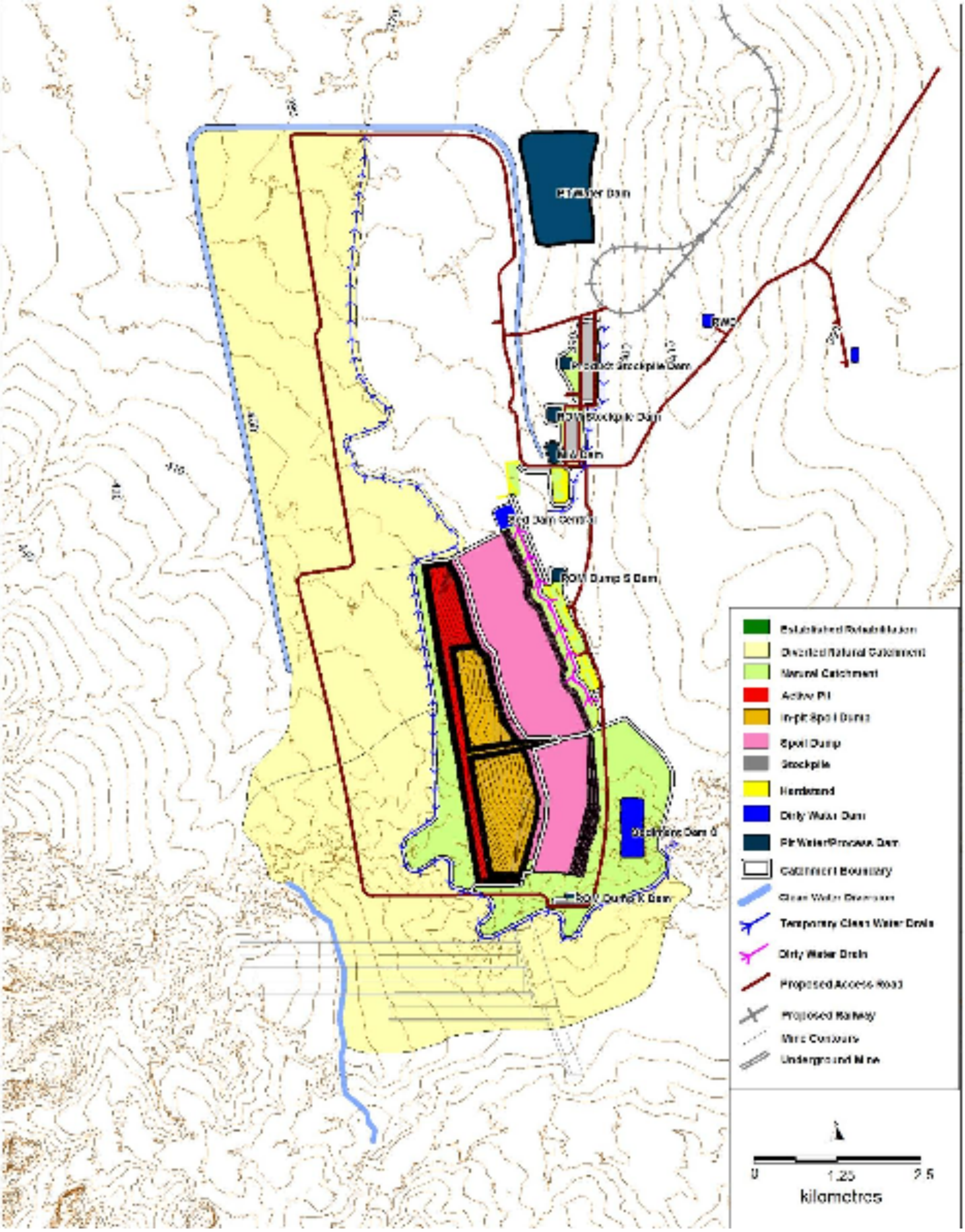

Figure 1.2 Y4 Water Management System Layout

Table 1.3 Catchment Areas Draining to Site Water Storages in Y4

Storage Name	Land Use Classification (ha)							Total Area (ha)
	Natural (grassed)	Stockpile	Hardstand	Active Pit	In Pit Dump	Out Pit Dump	Established Rehabilitation	
North Pit								0.0
South Pit	150.3			152.7	272.7	45.8		621.5
Sed Dam N								0.0
ROM Dump N Dam								0.0
Product Stockpile Dam	28.0	26.3						54.3
ROM Stockpile Dam	24.6	13.6						38.3
MIA Dam	15.2		9.9					25.1
Sed Dam Central	59.5		2.6			299.7		361.8
ROM Dump S Dam	36.2		21.3					57.5
Sediment Dam S	367.9		0.4			162.9		531.2
ROM Dump X Dam	3.0	3.4						6.5
Total	684.8	43.4	34.2	152.7	272.7	508.4	0.0	1,696.2

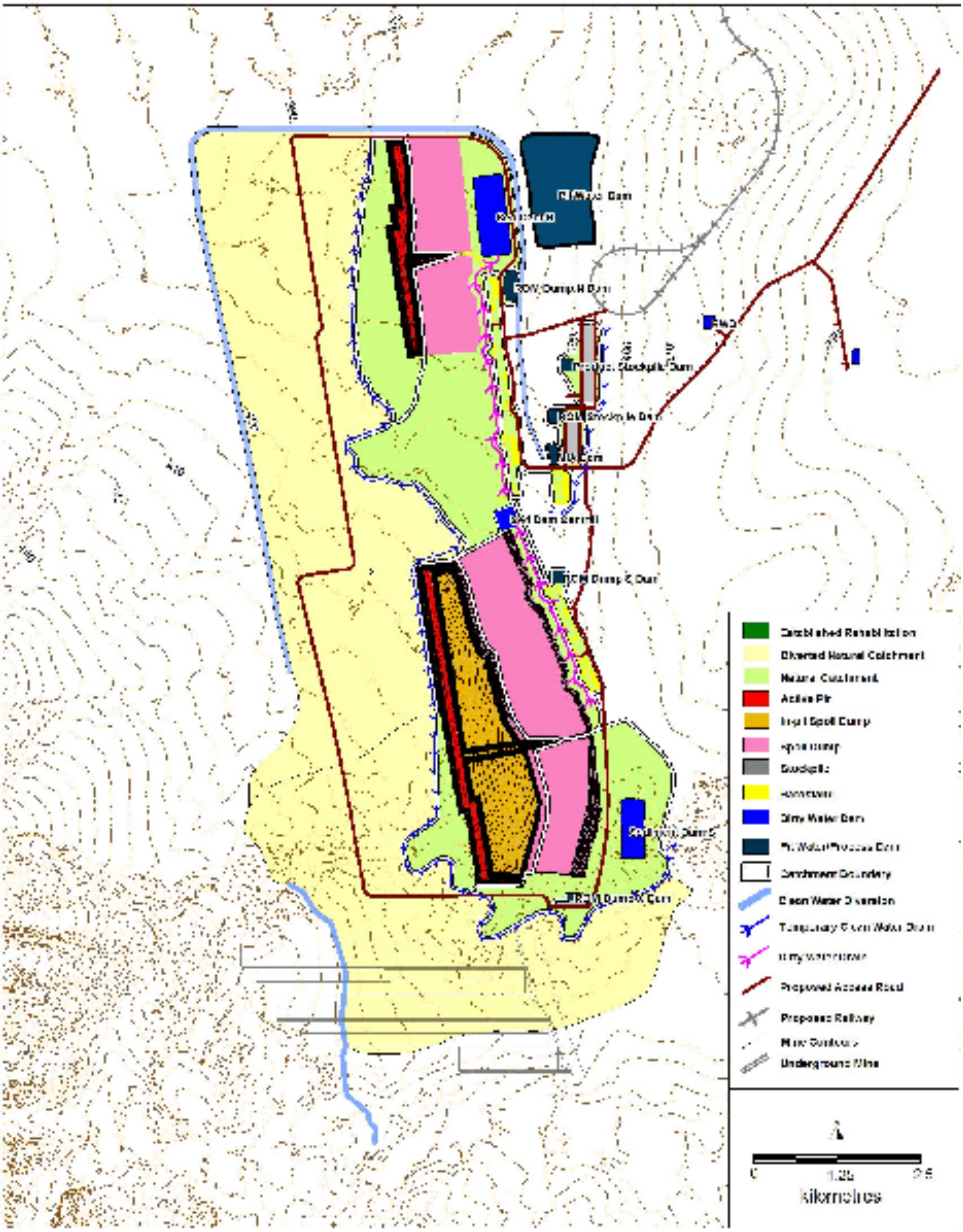

Figure 1.3 Y5 Water Management System Layout

Table 1.4 Catchment Areas Draining to Site Water Storages in Y5

Storage Name	Land Use Classification (ha)							Total Area (ha)
	Natural (grassed)	Stockpile	Hardstand	Active Pit	In Pit Dump	Out Pit Dump	Established Rehabilitation	
North Pit	222.4			113.2		1.4		337.0
South Pit	110.4			109.1	354.5	47.2		621.2
Sed Dam N	571.3		3.5			257.1		831.9
ROM Dump N Dam	62.7		27.7					90.4
Product Stockpile Dam	28.0	26.3						54.3
ROM Stockpile Dam	24.6	13.6						38.3
MIA Dam	15.2		9.9					25.1
Sed Dam Central	59.5		2.6			299.7		361.8
ROM Dump S Dam	36.7		21.3					58.0
Sediment Dam S	367.9		0.4			162.9		531.2
ROM Dump X Dam	3.0	3.4						6.5
Total	1,501.6	43.4	65.5	222.3	354.5	768.4	0.0	2,955.7

wrm water + environment

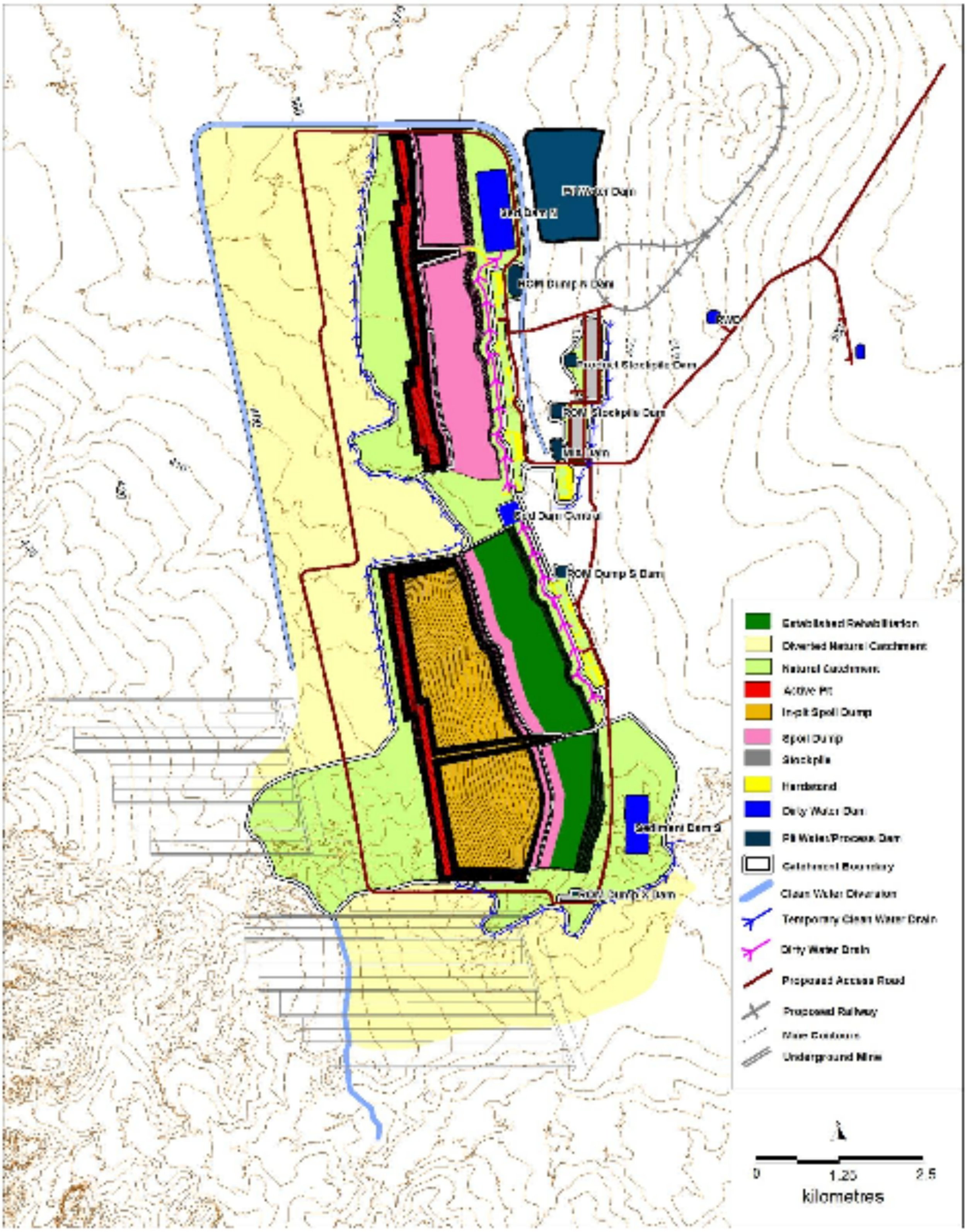

Figure 1.4 Y10 Water Management System Layout

Table 1.5 Catchment Areas Draining to Site Water Storages in Y10

Storage Name	Land Use Classification (ha)							Total Area (ha)
	Natural (grassed)	Stockpile	Hardstand	Active Pit	In Pit Dump	Out Pit Dump	Established Rehabilitation	
North Pit	268.3			185.0		19.4		472.7
South Pit	529.7			127.0	619.8	47.2		1,323.7
Sed Dam N	292.7		3.5			396.9		693.1
ROM Dump N Dam	62.7		27.7					90.4
Product Stockpile Dam	28.0	26.3						54.3
ROM Stockpile Dam	24.6	13.6						38.3
MIA Dam	15.2		9.9					25.1
Sed Dam Central	59.5		2.6			61.8	237.9	361.8
ROM Dump S Dam	36.7		21.3					58.0
Sediment Dam S	369.8		0.4			35.3	127.6	533.2
ROM Dump X Dam	3.0	3.4						6.5
Total	1,690.3	43.4	65.5	311.9	619.8	560.7	365.5	3,657.0

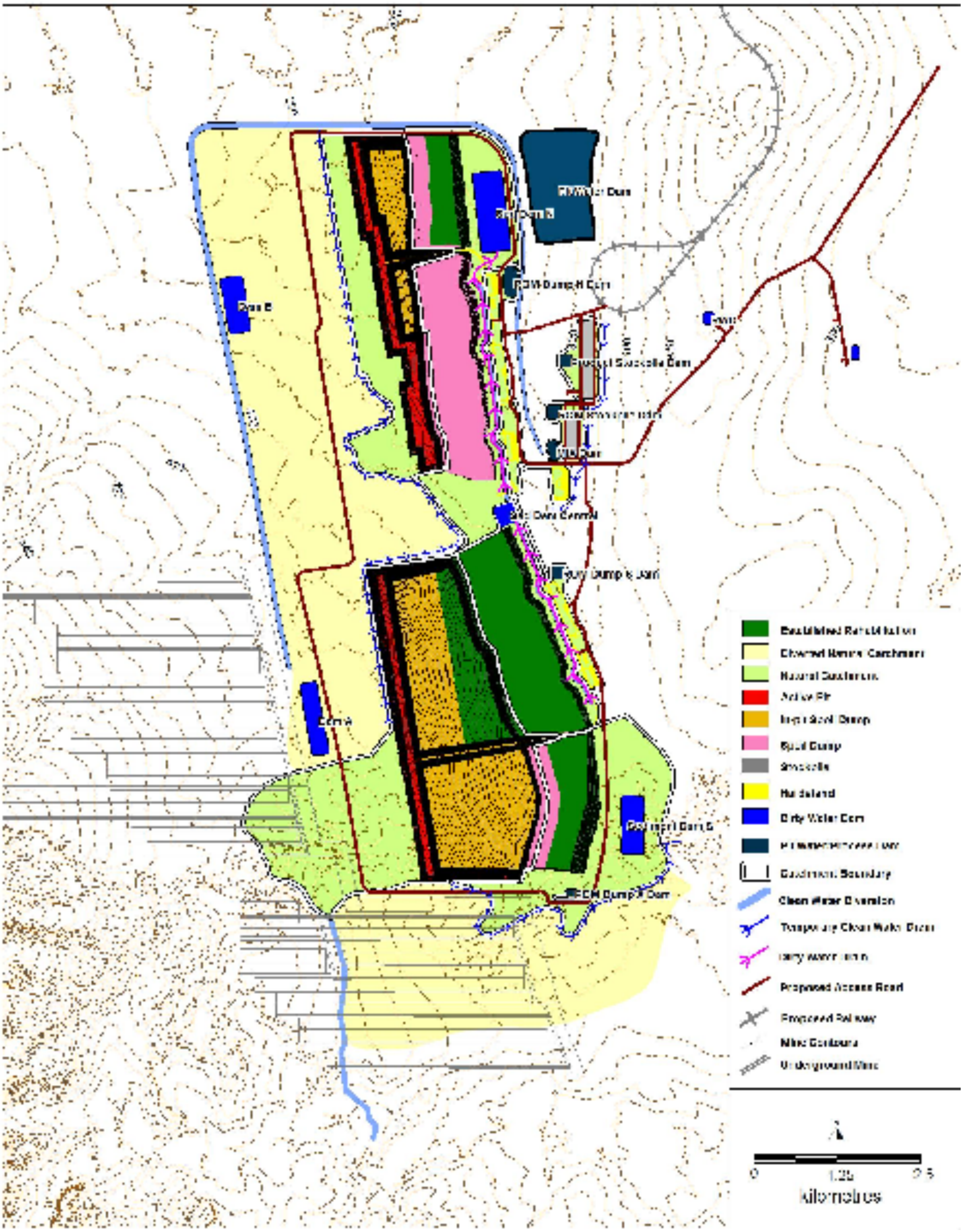

Figure 1.5 Y15 Water Management System Layout

Table 1.6 Catchment Areas Draining to Site Water storages in Y15

Storage Name	Land Use Classification (ha)							Total Area (ha)
	Natural (grassed)	Stockpile	Hardstand	Active Pit	In Pit Dump	Out Pit Dump	Established Rehabilitation	
North Pit	234.0			161.1	167.1	19.4		581.6
South Pit	457.4			128.2	546.8	22.0	170.0	1,324.3
Sed Dam N	292.7		3.5			308.3	88.6	693.1
ROM Dump N Dam	62.7		27.7					90.4
Product Stockpile Dam	28.0	26.3						54.3
ROM Stockpile Dam	24.6	13.6						38.3
MIA Dam	15.2		9.9					25.1
Sed Dam Central	59.5		2.6			0.0	297.3	359.4
ROM Dump S Dam	36.7		21.3					58.0
Sediment Dam S	369.9		0.4			36.1	126.8	533.3
ROM Dump X Dam	3.0	3.4						6.5
Total	1,583.7	43.4	65.5	289.3	713.9	385.8	682.7	3,764.2

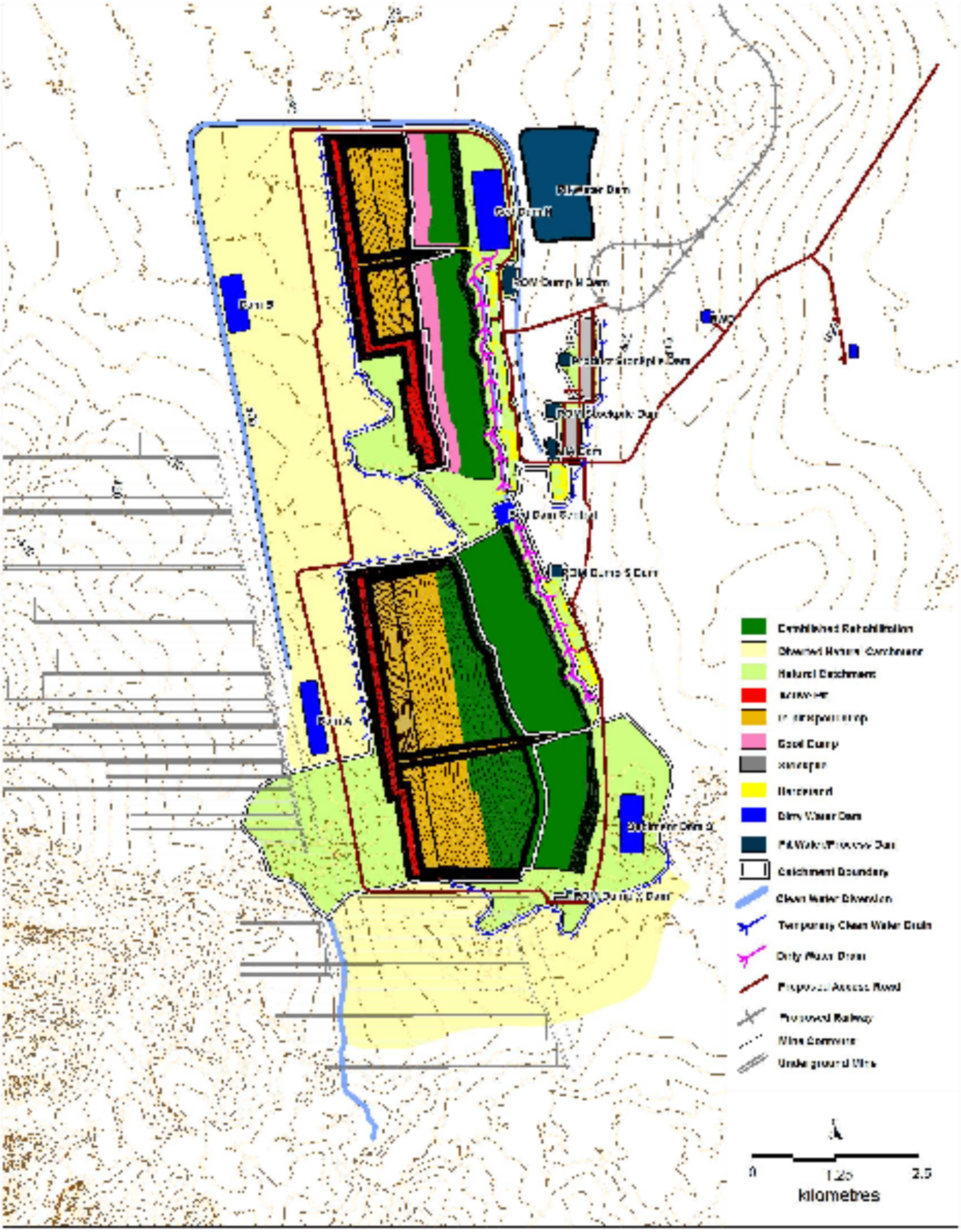

Figure 1.6 Y20 Water Management System Layout

Table 1.7 Catchment Areas Draining to Site Water storages in Y20

Storage Name	Land Use Classification (ha)							Total Area (ha)
	Natural (grassed)	Stockpile	Hardstand	Active Pit	In Pit Dump	Out Pit Dump	Established Rehabilitation	
North Pit	110.6			172.8	278.7	19.4		581.5
South Pit	406.5			139.3	529.8	0.0	332.8	1,408.4
Sed Dam N	292.7		3.5			124.5	272.3	693.1
ROM Dump N Dam	62.7		27.7					90.4
Product Stockpile Dam	28.0	26.3						54.3
ROM Stockpile Dam	24.6	13.6						38.3
MIA Dam	15.2		9.9					25.1
Sed Dam Central	59.5		2.6			0.0	299.7	361.8
ROM Dump S Dam	36.7		21.3					58.0
Sediment Dam S	369.8		0.4			0.0	162.9	533.2
ROM Dump X Dam	3.0	3.4						6.5
Total	1,409.4	43.4	65.5	312.1	808.5	143.9	1,067.8	3,850.5

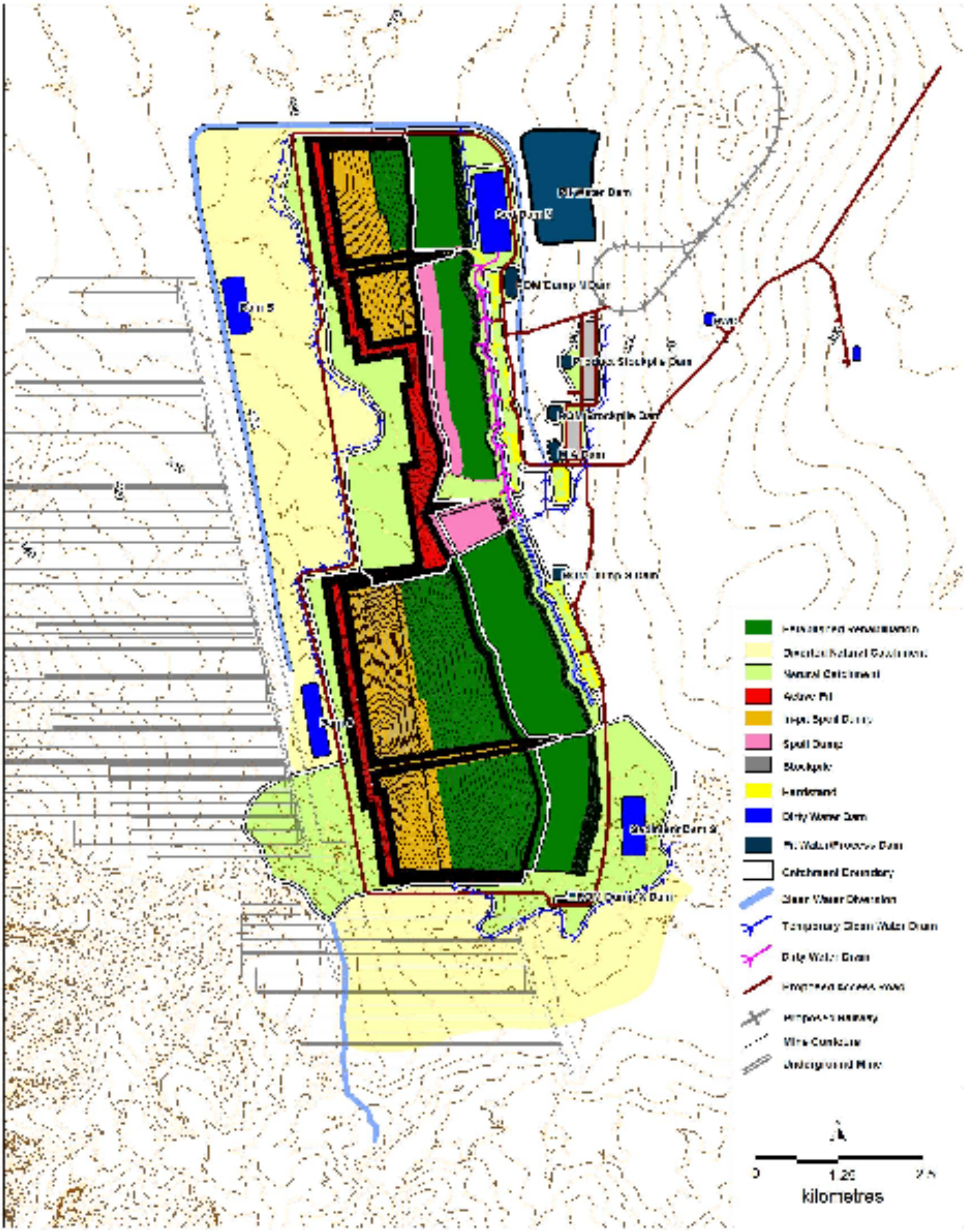

Figure 1.7 Y25 Water Management System Layout

Table 1.8 Catchment Areas Draining to Site Water storages in Y25

Storage Name	Land Use Classification (ha)							Total Area (ha)
	Natural (grassed)	Stockpile	Hardstand	Active Pit	In Pit Dump	Out Pit Dump	Established Rehabilitation	
North Pit	311.8			272.2	292.6	26.1	78.6	981.4
South Pit	363.0			139.0	465.2	0.0	563.0	1,530.1
Sed Dam N	195.5		3.5			136.1	182.2	517.3
ROM Dump N Dam	62.2		27.7					89.9
Product Stockpile Dam	28.0	26.3						54.3
ROM Stockpile Dam	24.6	13.6						38.3
MIA Dam	15.2		9.9					25.1
Sed Dam Central								
ROM Dump S Dam	36.7		21.3					58.0
Sediment Dam S	369.8		0.4			0.0	162.3	532.5
ROM Dump X Dam	3.0	3.4						6.5
Total	1,409.9	43.4	62.9	411.2	757.8	162.2	986.1	3,833.3

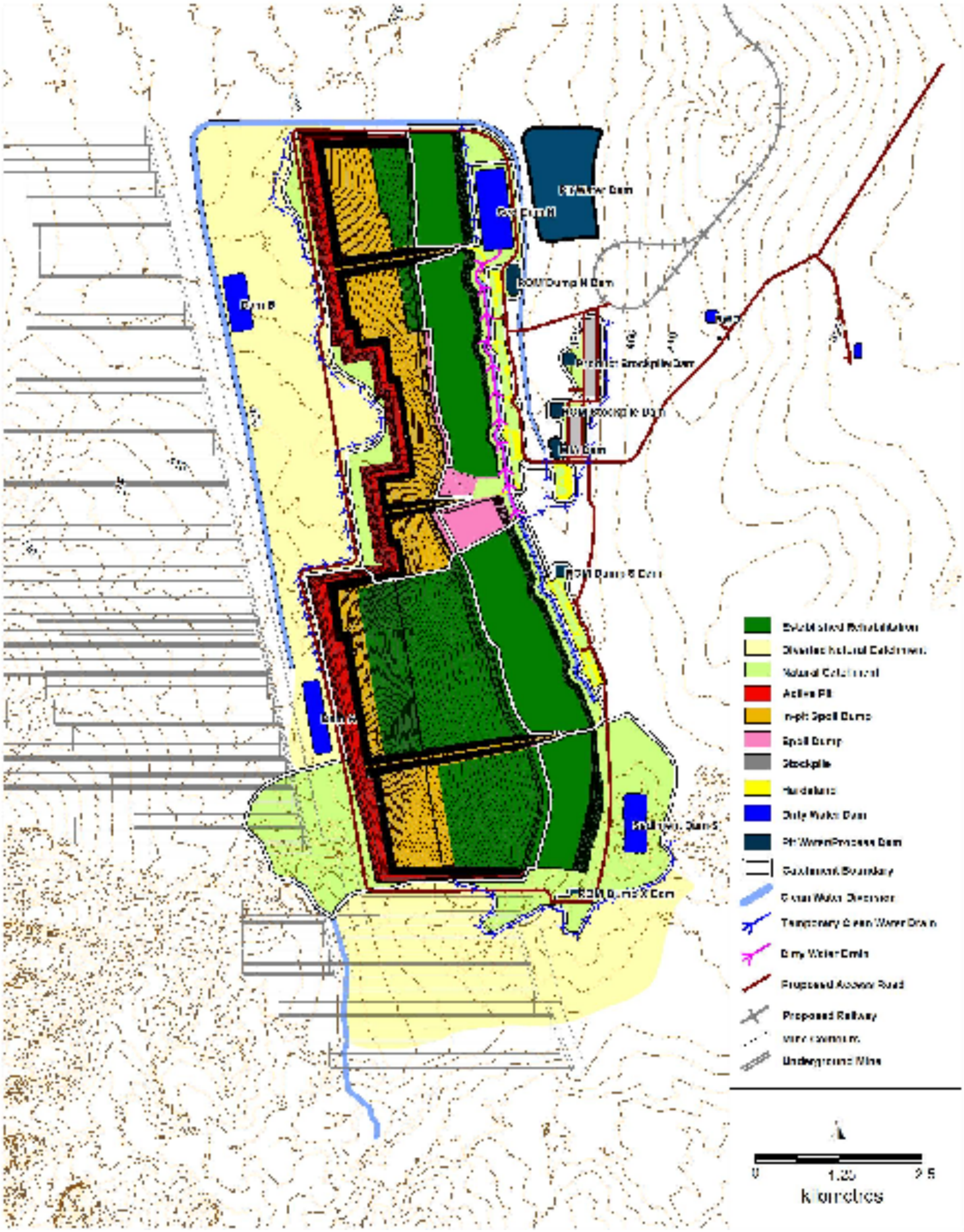

Figure 1.8 Y30 Water Management System Layout

Table 1.9 Catchment Areas Draining to Site Water storages in Y30

Storage Name	Land Use Classification (ha)							Total Area (ha)
	Natural (grassed)	Stockpile	Hardstand	Active Pit	In Pit Dump	Out Pit Dump	Established Rehabilitation	
North Pit	112.6			191.8	547.1	25.2	104.9	981.7
South Pit	291.9			159.1	332.6	1.9	746.5	1,532.0
Sed Dam N	178.0		3.5			83.0	253.0	517.5
ROM Dump N Dam	62.2		27.7					89.9
Product Stockpile Dam	28.0	26.3						54.3
ROM Stockpile Dam	24.6	13.6						38.3
MIA Dam	15.2		9.9					25.1
Sed Dam Central								0.0
ROM Dump S Dam	36.7		21.3					58.0
Sediment Dam S	369.8		0.4			0.0	162.1	532.3
ROM Dump X Dam	3.0	3.4						6.5
Total	1,122.1	43.4	62.9	350.9	879.8	110.1	1,266.4	3,835.5

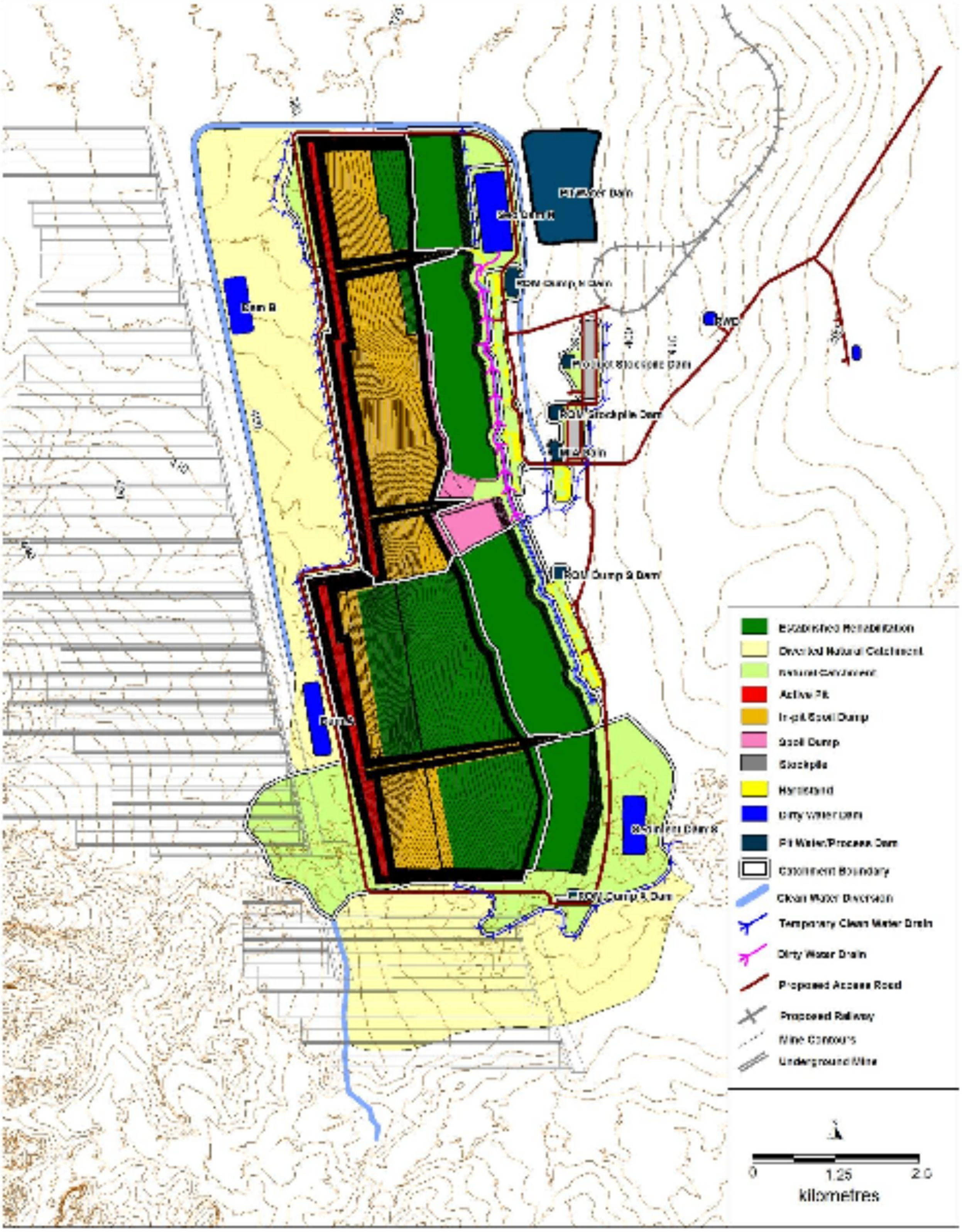

Figure 1.9 Y33 Water Management System Layout

Table 1.10 Catchment Areas Draining to Site Water storages in Y33

Storage Name	Land Use Classification (ha)							Total Area (ha)
	Natural (grassed)	Stockpile	Hardstand	Active Pit	In Pit Dump	Out Pit Dump	Rehab	
North Pit	38.3			169.9	714.9	25.2	104.9	1,053.2
South Pit	291.9			158.4	333.5	0.0	747.7	1,531.4
Sed Dam N	178.0		3.5			86.1	253.0	520.5
ROM Dump N Dam	62.2		27.7					89.9
Product Stockpile Dam	28.0	26.3						54.3
ROM Stockpile Dam	24.6	13.6						38.3
MIA Dam	15.2		9.9					25.1
Sed Dam Central								
ROM Dump S Dam	36.7		21.3					58.0
Sediment Dam S	369.8		0.4			0.0	162.9	533.2
ROM Dump X Dam	3.0	3.4						6.5
Total	1,047.8	43.4	62.8	328.2	1,048.4	111.3	1,268.5	3,910.4

C APPENDIX

FINAL VOID ANALYSIS

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1 INTRODUCTION

1.1 OVERVIEW

Final void water levels in each pit have been simulated using a simplified OPSIM water balance model.

1.2 MODEL DETAILS

1.2.1 Runoff Model

It was assumed that at mine closure, the major diversions of the western catchments (incl. Sapling Creek) will remain in place, but the minor clean water drains around the pit highwall will have been decommissioned. The catchments flowing to the void will include:

- The pit floor itself;
- The natural catchment upslope (west) of the highwall and east of the diversion;
- The in-pit overburden (which will have been shaped to its final profile, topsoiled and revegetated).

Table 1.1 Final Void Land Use Areas (ha)

Storage	Land Use Classification			Total Catchment Area
	Pit Void	Established Rehabilitation	Natural/ Undisturbed	
Final Void	328	1,082	11	1,421

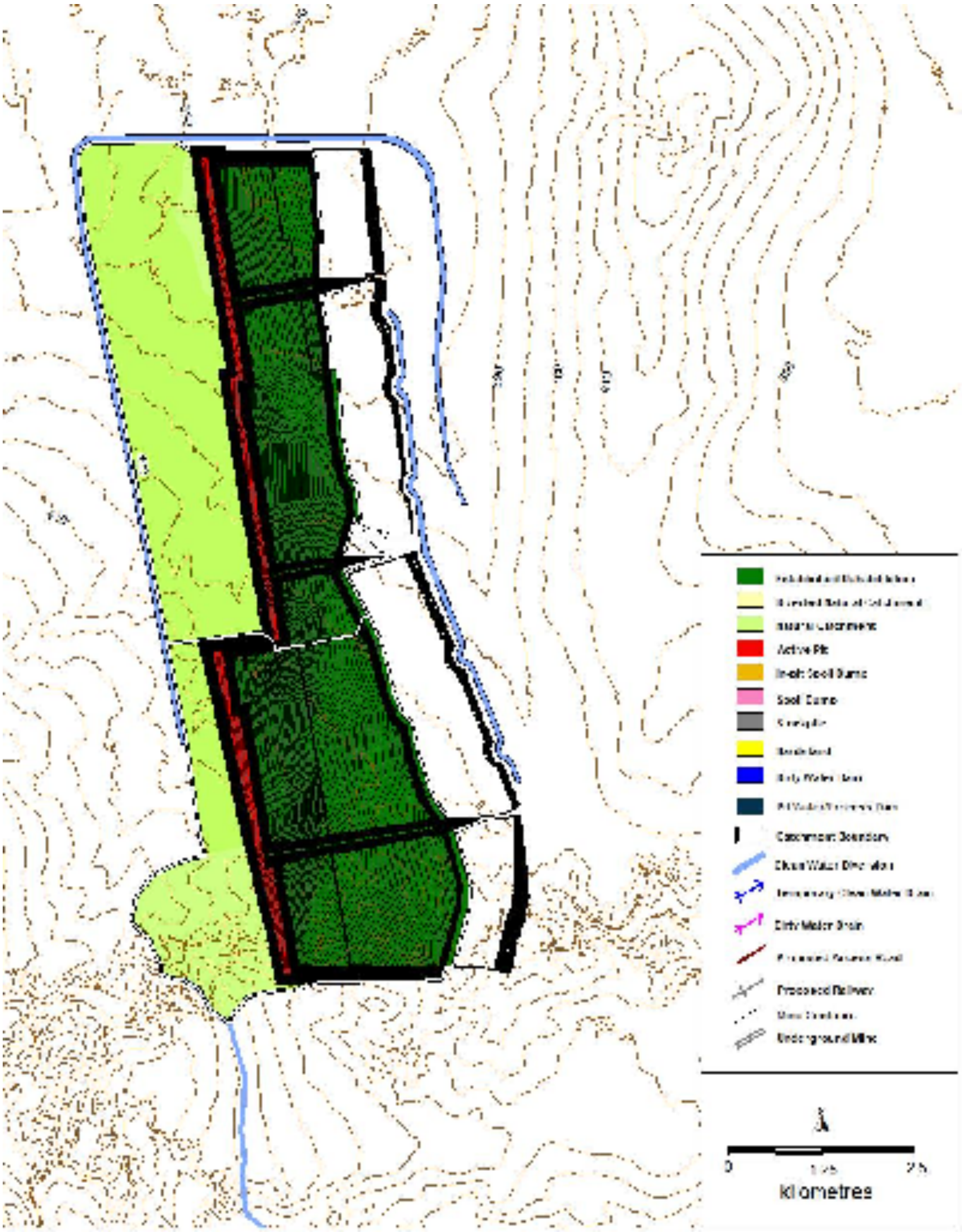

Figure 1.1 Final Void Catchment and Land Use Classifications

1.2.2 Storage Curves

Stage storage characteristics for the final void were derived using pit design at Year 33 of the mine schedule. The adopted final void stage storage characteristics are presented Figure 1.2.

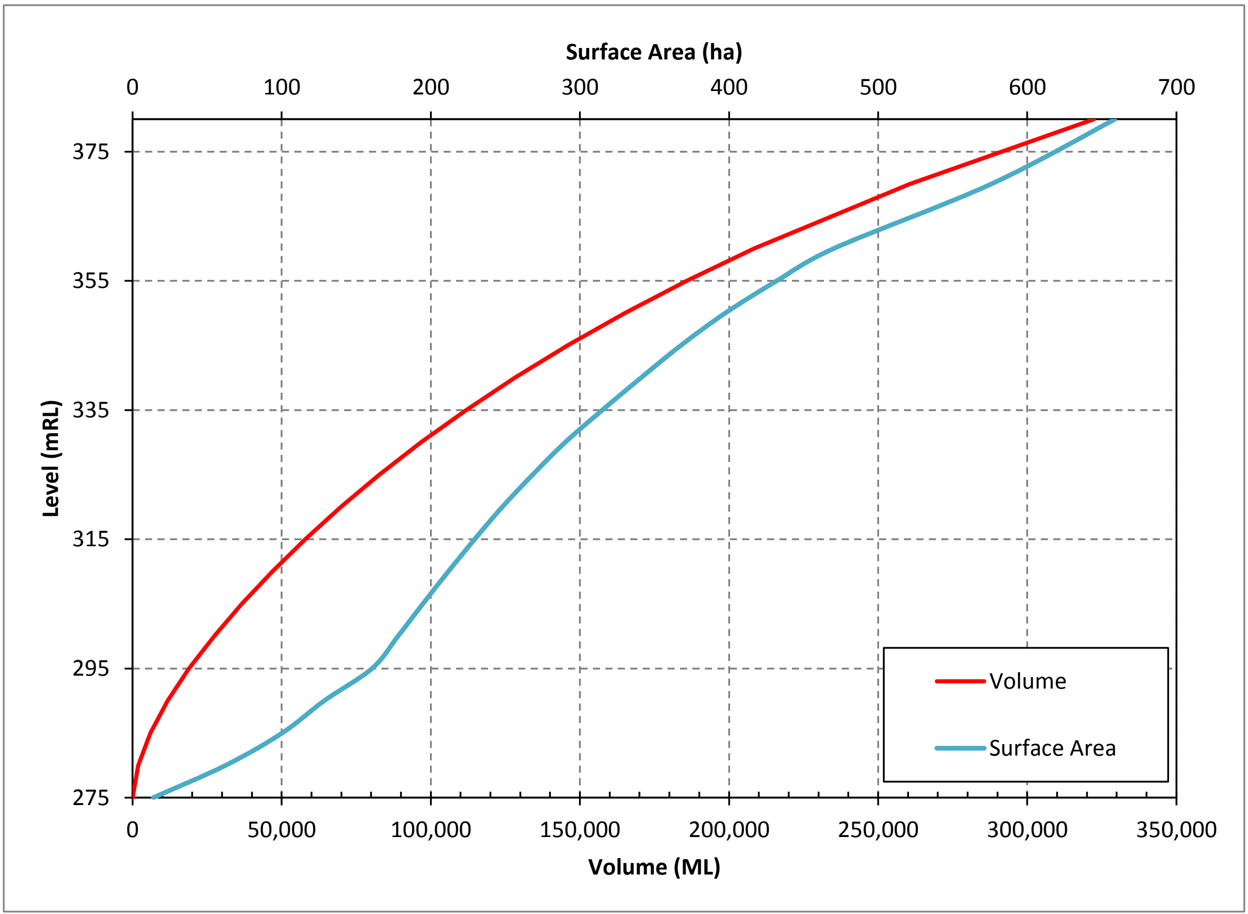

Figure 1.2 Final Void Stage Storage Characteristics

1.2.3 Runoff Salinity

Runoff water quality salinities measured as electrical conductivity (EC) have been considered so as to allow an estimate of final void water levels and quality. Each catchment type has been assigned a runoff salinity as presented in Table 1.2 below. The adopted salinities were based on experience at similar mining operations.

Table 1.2 Final Void Land Use Classifications

Catchment Type	Electrical Conductivity (µS/cm)
Natural/Undisturbed	250
Pit Void	1,000
Rehabilitated Spoil	700

1.2.4 Groundwater Inflows

RPS Aquaterra advised that based on results of groundwater modelling undertaken for the SGCP Hydrogeological assessment, an assumption of a long-term groundwater inflow of 1.6ML/d to the final void would be reasonable for the purposes of this analysis. A constant groundwater salinity of 2,000µS/cm was also adopted.

1.3 RESULTS

1.3.1 Long Term Water Level Behaviour

Based on a simulation period of 125 years, long term expected water level behaviour in the South Galilee Final void is presented in Figure 1.3. A range of initial water levels were chosen to investigate sensitivity. Figure 1.3 indicates the following:

- Long term water levels in the final void would appear to stabilise at around 325m AHD; and
- The long-term final void water level appears relatively insensitive to the initial water level.

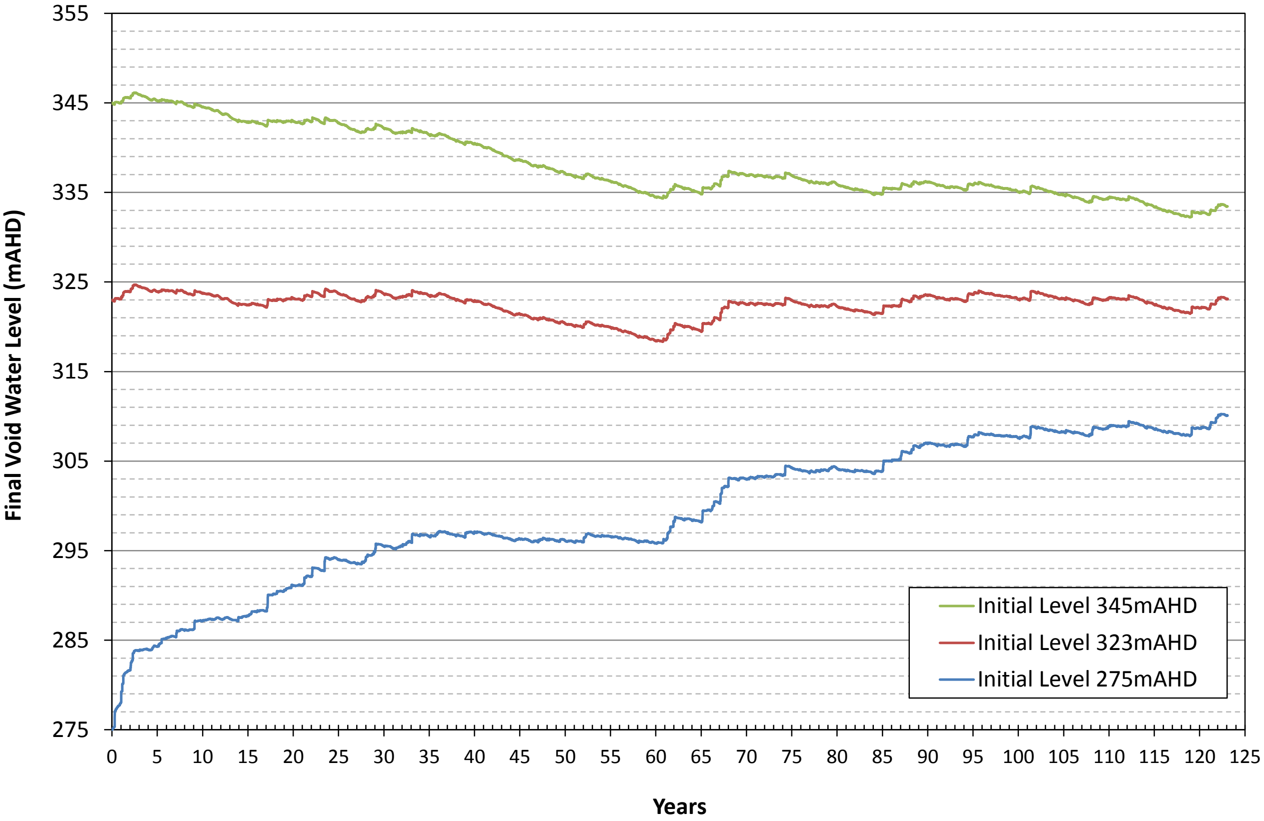

Figure 1.3 South Galilee Final Void Water Level Behaviour

1.3.2 Long Term Salinity

In any void which does not have a mechanism for salts to flow out (e.g. by flushing through flood inflows and discharges, or by fresh groundwater inflows), salinity will tend to increase over time. OPSIM modelling of the voids show that if initial water levels are low, the salinity will eventually increase beyond safe stock watering levels.

1.4 SENSITIVITY ANALYSIS

1.4.1 AWBM Runoff Characteristics

A sensitivity analysis was undertaken on the final void water level behaviour to the adopted runoff characteristics of the natural/undisturbed land use type. AWBM parameters yielding a higher long term runoff coefficient of 8% were used to investigate the effects on the final void water level behaviour.

Results of the sensitivity analysis are presented in Figure 1.4. Review of Figure 1.4 indicates the following:

- The final void water level behaviour is sensitive to the natural/undisturbed land use type runoff characteristics;
- In this case, long term water levels in the final void appear to stabilise at around 340m AHD; and
- The long-term final void water level appears relatively insensitive to the initial water level.

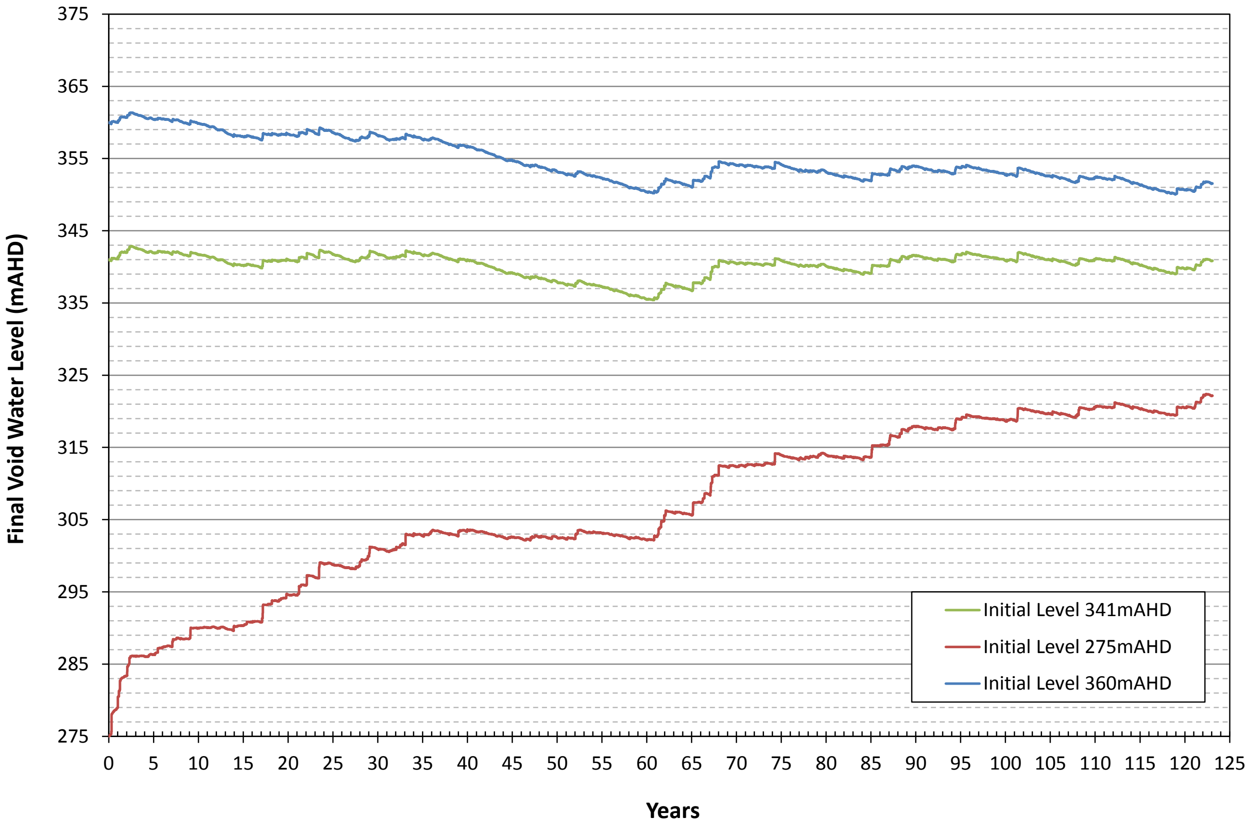

Figure 1.4 South Galilee Final Void Water Level – Sensitivity to Runoff Characteristics

1.4.2 Runoff Salinity

A sensitivity analysis was performed on the effect of initial water quality to the long term final void water level and water quality; results are presented in Figure 1.5 . Simulated water quality results are presented in Figure 1.6. Review of the results indicates that the final void water level behaviour is mostly unaffected by the initial water quality, with a difference of 50,000µS/cm resulting in an increase in water level of approximately 3.5m after 125 years. The simulated water quality results are largely dependent on the initial water quality. However, salinity tends to slowly increase over time due to the continuing addition of salt from the source catchments and groundwater.

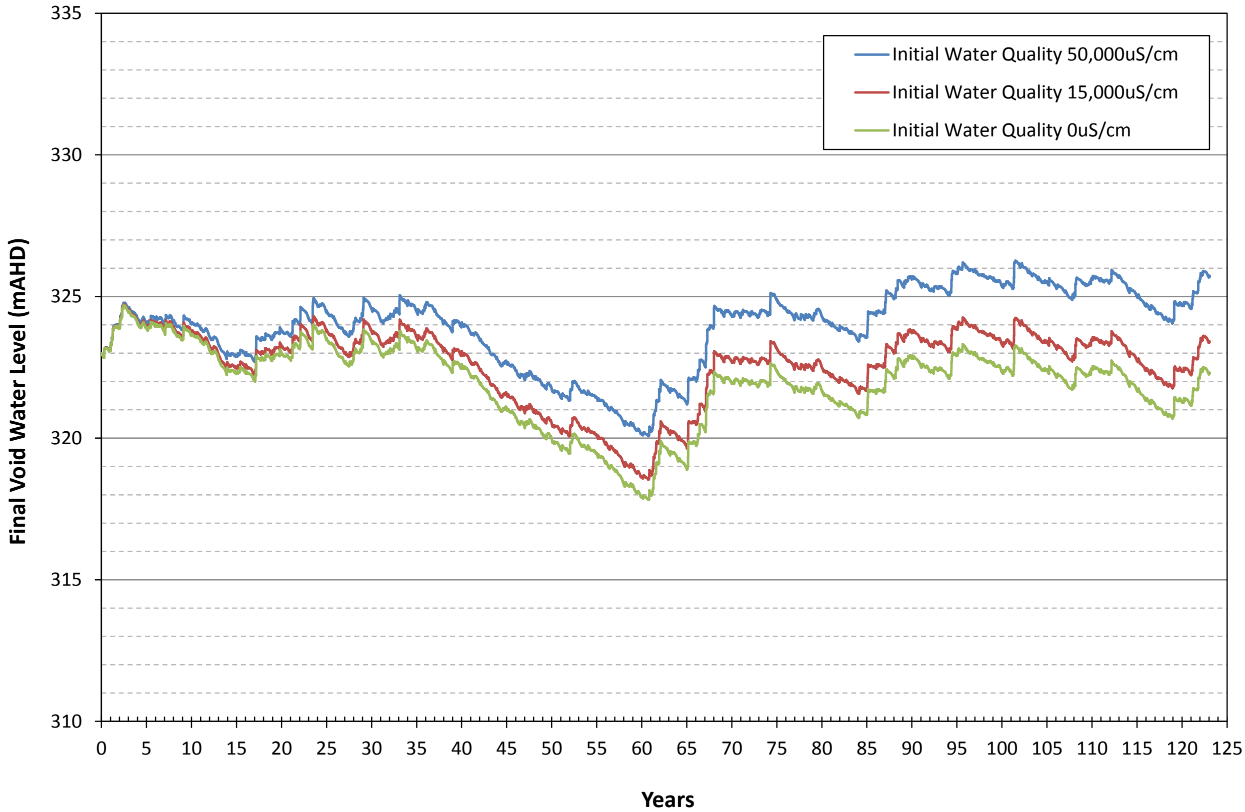

Figure 1.5 Final Void Water Level Sensitivity Analysis to Initial Water Quality

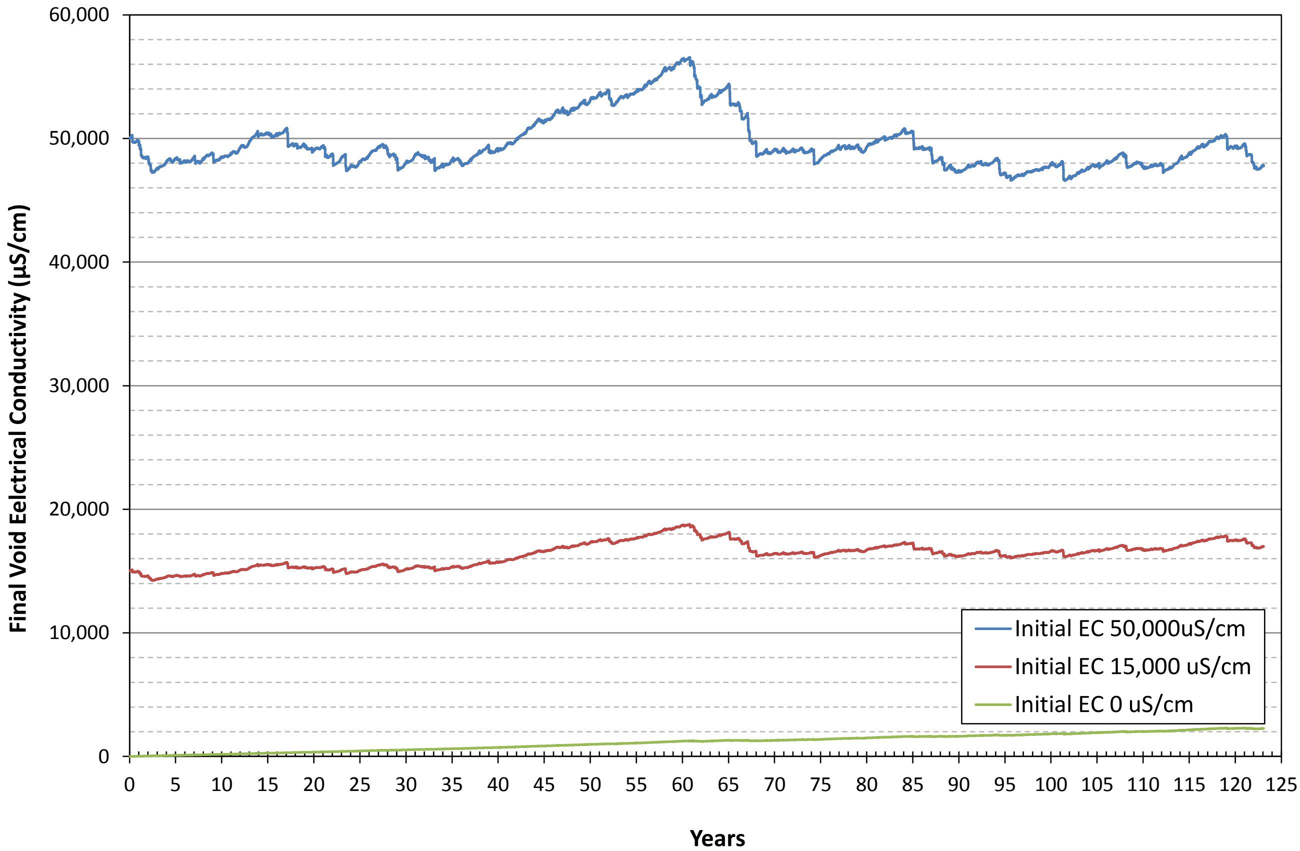

Figure 1.6 Final Void Simulated Water Quality Results

D APPENDIX

FLOOD MODELLING AND CREEK DIVERSION ASSESSMENT

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1 INTRODUCTION

1.1 INTRODUCTION

Detailed flood modelling was carried out for the major drainage paths crossing the SGCP disturbance area including the proposed Infrastructure Corridor to the north. The modelling results define existing flood conditions in the above drainage paths as well as conditions following development of the SGCP, including details of the design flows in the proposed Sapling Creek Diversion.

This appendix details the study methodology, and the diversion design details. The modelling results are presented graphically in Appendix E.

The flood investigations detailed in this Appendix have been undertaken for the purpose of determining existing flow behaviour and impact assessment to address the terms of reference for the SGCP EIS. The results presented herein should not be used for any other purpose without seeking advice from WRM Water & Environment regarding its applicability.

2 FLOOD HYDROLOGY

2.1 METHODOLOGY

The URBS runoff-routing model (Carroll, 2004) was used to estimate flood discharges in the Tallarenha Creek, Sapling Creek and Dead Horse Creek catchments. A separate model was also used for the areas draining through the proposed Infrastructure Corridor catchment area. URBS is a runoff-routing computer model that uses a network of conceptual storages to represent the routing of rainfall excess through a catchment. URBS is used extensively throughout Australia by the Bureau of Meteorology for flood forecasting on major river systems.

For this study, the URBS model was used in the “split mode” which enables the simulation of separate catchment and channel routing. Adopted rainfall losses are subtracted from the total rainfall hyetograph to obtain rainfall excess. Rainfall excess is routed through a conceptual storage representing each sub-catchment of the model before being added to the creek or river channel. Routing through the creek or river system uses the Muskingum method.

The model parameters were chosen to provide peak discharge estimates consistent with estimates obtained using the Rational Method at the upstream and downstream extents of the mine workings. Design flood discharge hydrographs were output for a range of Average Recurrence Intervals (ARIs) up to the Probable Maximum Flood (PMF), and a range of storm durations up to 72 hours.

Design rainfall intensities were derived in accordance with Australian Rainfall and Runoff (Pilgrim, 1998), and probable maximum precipitation (PMP) design rainfall depths were estimated using the Generalised Short-Duration Method (GSDM) for durations up to 6 hours (BoM, 2003).

The proposed project will result in the diversion of watercourses within three (3) catchments – Tallarenha Creek, Sapling Creek and Dead Horse Creek. Separate hydrologic models were developed for these catchments for the existing and post-developed scenarios, including levee, drainage and creek diversion works. No diversion works are proposed in the Infrastructure Corridor and, as such, only the existing case scenario has been analysed using the hydrologic model.

2.1.1 Model Parameters

Rainfall and streamflow data are not available for historical flood events at the project site and as such, the hydrologic models are not calibrated. In the absence of calibration data, the URBS model parameters were selected so that the peak discharges matched the modelled peak discharges estimated using the Rational Method at various locations.

Table 2-1 shows the adopted model parameters for the three (3) URBS models.

Table 2-1 Adopted URBS Model Parameters

System	α (channel lag parameter)	β (catchment lag parameter)	m (catchment non-linearity parameter)
Tallarenha Creek	0.2	1.0	0.8
Sapling Creek	0.15	1.0	0.8
Dead Horse Creek	0.2	1.0	0.8
Infrastructure Corridor	0.2	1.0	0.8

Table 2-2 shows the adopted uniform initial loss and continuing loss rates for the Tallarenha Creek, Sapling Creek, Dead Horse Creek and Infrastructure Corridor catchments.

Table 2-2 Adopted Initial and Continuing Losses

ARI	Initial Loss (mm)	Continuing Loss (mm/hr)
2	30.0	2.5
50	25.0	2.5
100	15.0	2.5
1000	0.0	1.4
PMF	0.0	1.4

Table 2-3 shows the adopted design rainfall depths for the Tallarenha Creek, Sapling Creek and Dead Horse Creek catchments for the 2, 50 and 100 year ARI design events. Table 2-4 shows the adopted design rainfall depths for the Infrastructure Corridor catchment for the 10 and 50 year ARI design events. The 10 and 50 year ARI design events were assessed in order to comply with design requirements for the proposed Infrastructure Corridor.

Table 2-3 Adopted Design Rainfall Depths (mm)

Storm Duration (hours)	2 Year ARI	50 Year ARI	100 Year ARI
1	36.8	77.1	86.6
1.5	41.7	87.2	98.0
2	45.6	95.2	107.0
3	50.1	105.0	117.9
4.5	54.9	114.5	128.6
6	58.5	121.8	136.8
9	64.5	134.9	151.9
12	69.1	145.2	163.2
18	77.6	165.9	187.1
24	84.2	182.4	206.2
30	89.7	196.9	222.9
36	94.3	209.6	237.5
48	102.2	231.6	262.7
72	108.7	252.0	287.3

Table 2-4 Adopted Design Rainfall Depths (mm) Infrastructure Corridor

Storm Duration (hours)	10 Year ARI	50 Year ARI
1	56.2	78.5
1.5	63.6	88.7
2	69.4	96.8
3	76.5	106.5
4.5	83.5	115.9
6	88.8	123.0
9	98.3	136.2
12	105.6	146.4
18	119.7	167.3
24	130.8	183.8
30	140.3	198.4
36	148.6	211.1
48	162.7	232.8
72	175.7	253.4

2.1.2 Existing Conditions URBS Model Configurations

The configuration of the Tallarenha Creek Existing Conditions URBS model is shown in Figure 2-1. The model extends to approximately 3.4km downstream (north) of the mining lease and consists of 83 sub-catchments. Summary details of sub-catchment areas are given in Table 2-5.

The configuration of the Sapling Creek Existing Conditions URBS model is shown in Figure 2-2. The model covers the entire catchment and consists of 46 sub-catchments. Summary details of sub-catchment areas are given in Table 2-6.

The configuration of the Dead Horse Creek Existing Conditions URBS model is shown in Figure 2-3. The model covers the entire catchment and consists of 37 sub-catchments. Summary details of sub-catchment areas are given in Table 2-7.

The configuration of the Infrastructure Corridor URBS model is shown in Figure 2-4. The model covers the entire catchment and consists of 47 sub-catchments. Summary details of sub-catchment areas are given in Table 2-8.

Table 2-5 Adopted Tallarenha Creek (Existing Conditions) URBS Model Sub-Catchment Areas

Sub-Catchment Number	Area (km²)	Sub-Catchment Number	Area (km²)	Sub-Catchment Number	Area (km²)
1	8.53	36	0.73	71	0.48
2	4.36	37	0.77	72	0.45
3	3.66	38	0.55	73	0.45
4	3.47	39	1.07	74	0.45
5	6.89	40	1.54	75	0.44
6	5.90	41	1.50	76	1.18
7	4.82	42	1.97	77	0.97
8	3.90	43	5.70	78	6.44
9	4.33	44	4.49	79	2.74
10	5.96	45	5.71	80	5.58
11	5.77	46	6.19	81	7.41
12	5.33	47	3.34	82	7.44
13	6.73	48	0.68	83	6.04
14	5.32	49	0.81		
15	0.35	50	0.64		
16	0.50	51	0.41		
17	0.39	52	0.24		
18	0.43	53	0.30		
19	0.45	54	0.39		
20	0.80	55	0.13		
21	0.59	56	0.11		
22	0.50	57	0.17		
23	0.56	58	0.76		
24	0.45	59	6.36		
25	6.44	60	5.68		
26	1.42	61	0.91		
27	1.41	62	0.96		
28	0.83	63	1.42		
29	0.98	64	0.94		
30	0.60	65	1.04		
31	6.60	66	0.35		
32	8.18	67	0.53		
33	1.85	68	0.73		
34	1.08	69	0.67		
35	0.91	70	0.73		

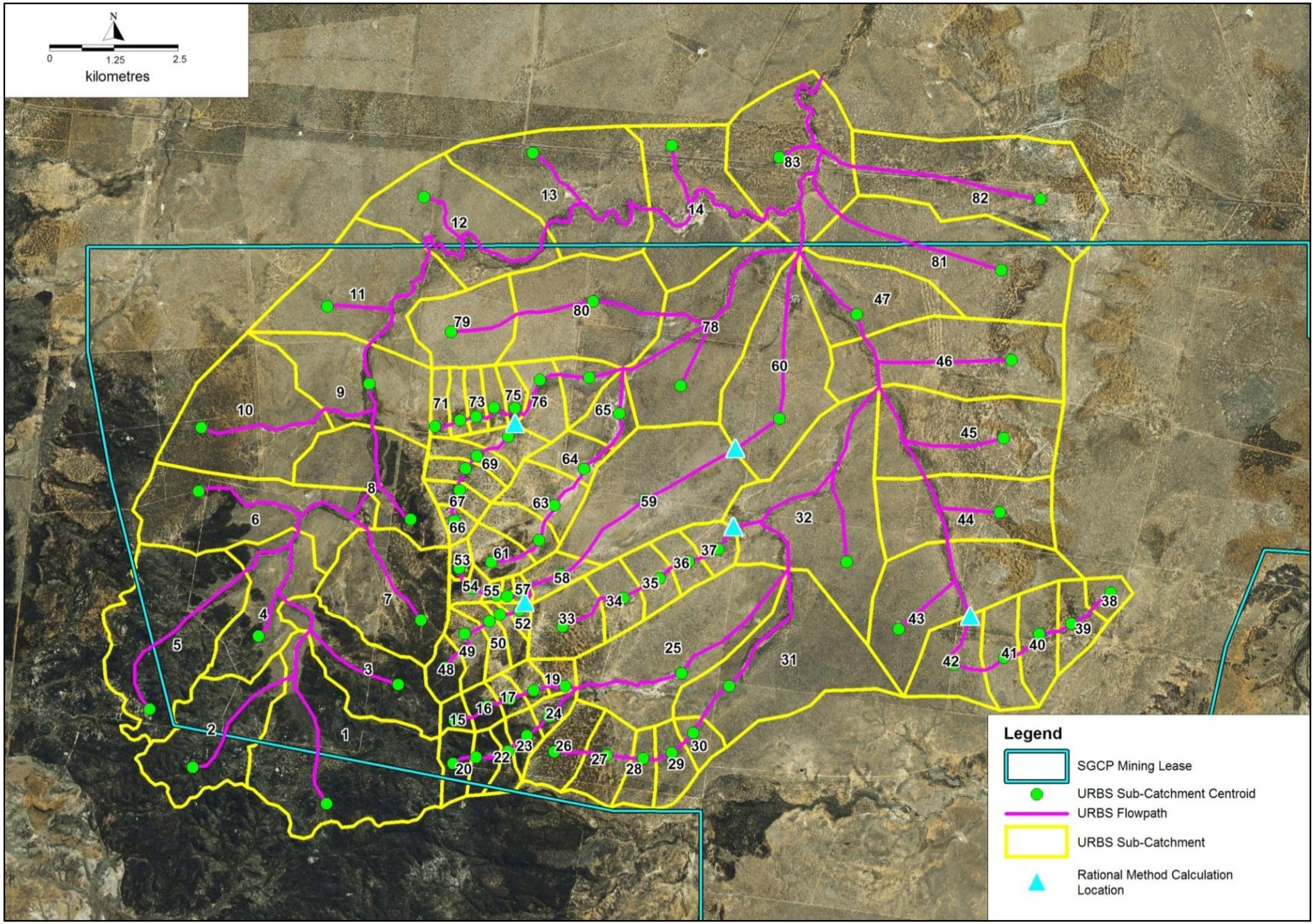

Figure 2-1 Tallarenha Creek (Existing Conditions) URBS Model Configuration

Table 2-6 Adopted Sapling Creek (Existing Conditions) URBS Model Sub-Catchment Areas

Sub-Catchment Number	Area (km^2)	Sub-Catchment Number	Area (km^2)	Sub-Catchment Number	Area (km^2)
1	1.14	17	0.69	33	1.60
2	1.30	18	0.89	34	1.11
3	0.80	19	0.45	35	0.83
4	0.92	20	0.37	36	1.52
5	0.73	21	0.40	37	2.75
6	2.38	22	0.55	38	2.09
7	0.16	23	0.27	39	2.04
8	0.48	24	3.50	40	2.18
9	0.49	25	3.42	41	1.45
10	0.53	26	2.01	42	3.02
11	0.70	27	0.72	43	3.09
12	0.74	28	1.40	44	3.48
13	0.59	29	1.60	45	1.52
14	0.84	30	1.35	46	2.22
15	1.06	31	2.19		
16	0.89	32	0.92		

Table 2-7 Adopted Dead Horse Creek (Existing Conditions) URBS Model Sub-Catchment Areas

Sub-Catchment Number	Area (km^2)	Sub-Catchment Number	Area (km^2)	Sub-Catchment Number	Area (km^2)
1	1.66	17	0.84	33	1.73
2	1.84	18	2.38	34	0.89
3	1.43	19	2.98	35	4.89
4	1.61	20	3.42	36	3.00
5	1.39	21	1.11	37	3.83
6	1.05	22	1.80		
7	1.32	23	1.61		
8	1.24	24	1.17		
9	1.26	25	0.87		
10	0.94	26	2.51		
11	1.32	27	2.00		
12	1.97	28	2.55		
13	0.91	29	3.50		
14	0.34	30	1.47		
15	0.65	31	1.35		
16	0.86	32	1.65		

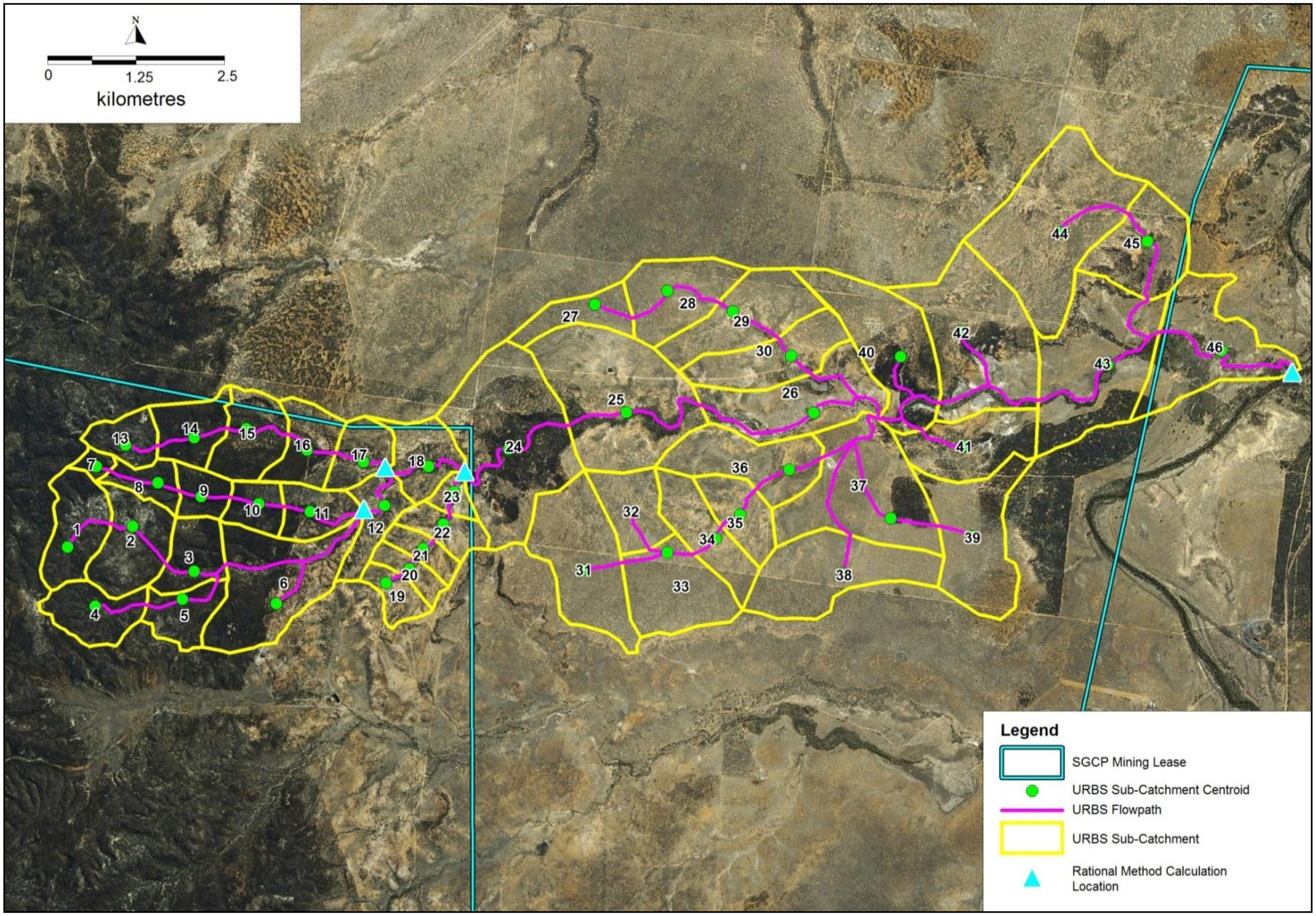

Figure 2-2 Sapling Creek (Existing Conditions) URBS Model Configuration

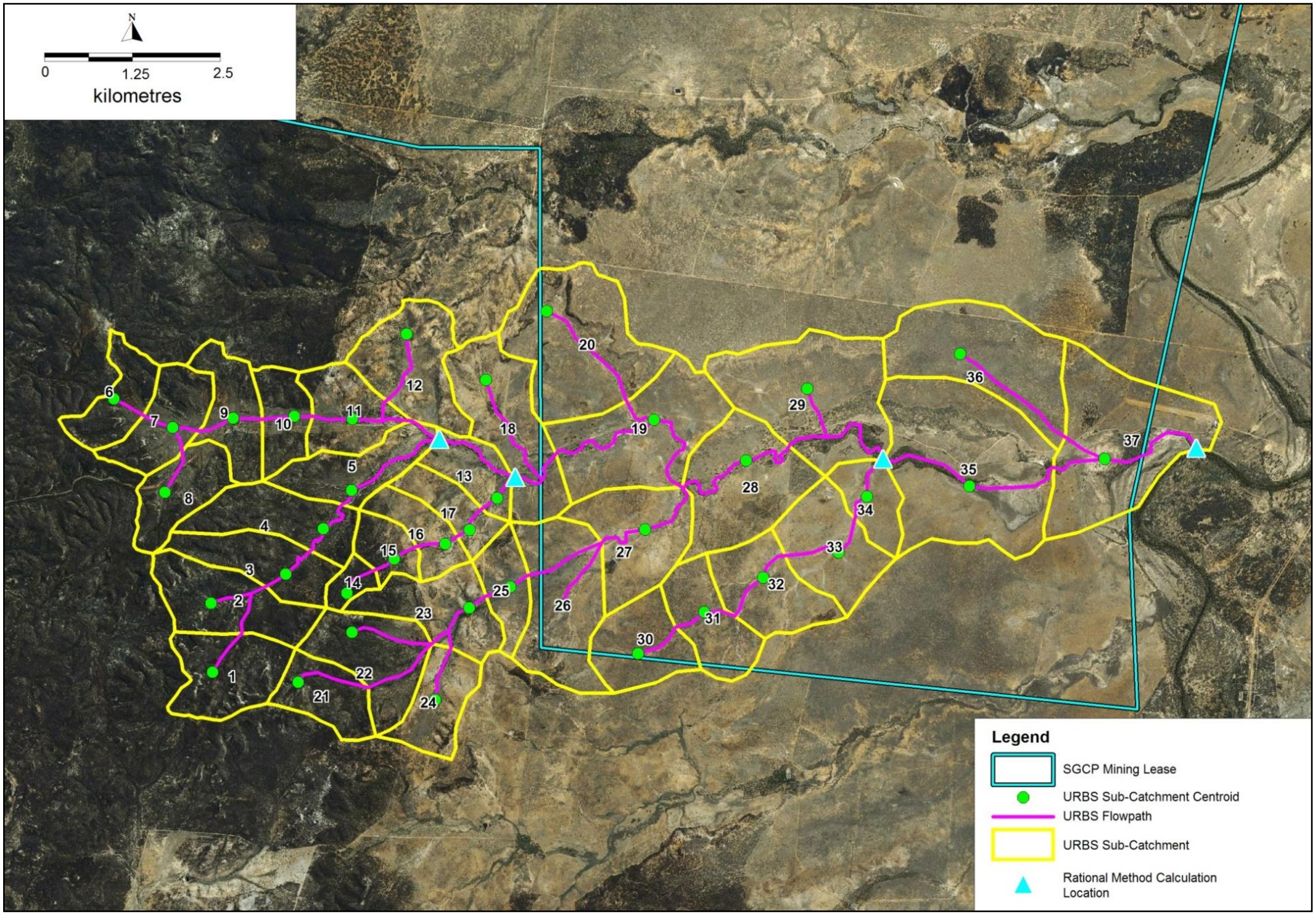

Figure 2-3 Dead Horse Creek (Existing Conditions) URBS Model Configuration

Table 2-8 Adopted Infrastructure Corridor URBS Model Sub-Catchment Areas

Sub-catchment Number	Area (km^2)	Sub-catchment Number	Area (km^2)
1	14.6	31	4.77
2	7.27	32	5.94
3	9.14	33	3.38
4	10.5	34	5.47
5	4.16	35	3.31
6	2.31	36	9.44
7	5.01	37	6.76
8	6.87	38	2.10
9	5.83	39	3.61
10	7.57	40	3.00
11	5.14	41	5.03
12	3.50	42	3.86
13	6.75	43	3.02
14	3.01	44	3.56
15	6.75	45	3.34
16	5.45	46	4.15
17	9.06	47	7.96
18	6.29		
19	13.7		
20	13.2		
21	7.73		
22	3.74		
23	6.11		
24	1.69		
25	1.54		
26	5.38		
27	3.72		
28	5.75		
29	5.11		
30	5.70		

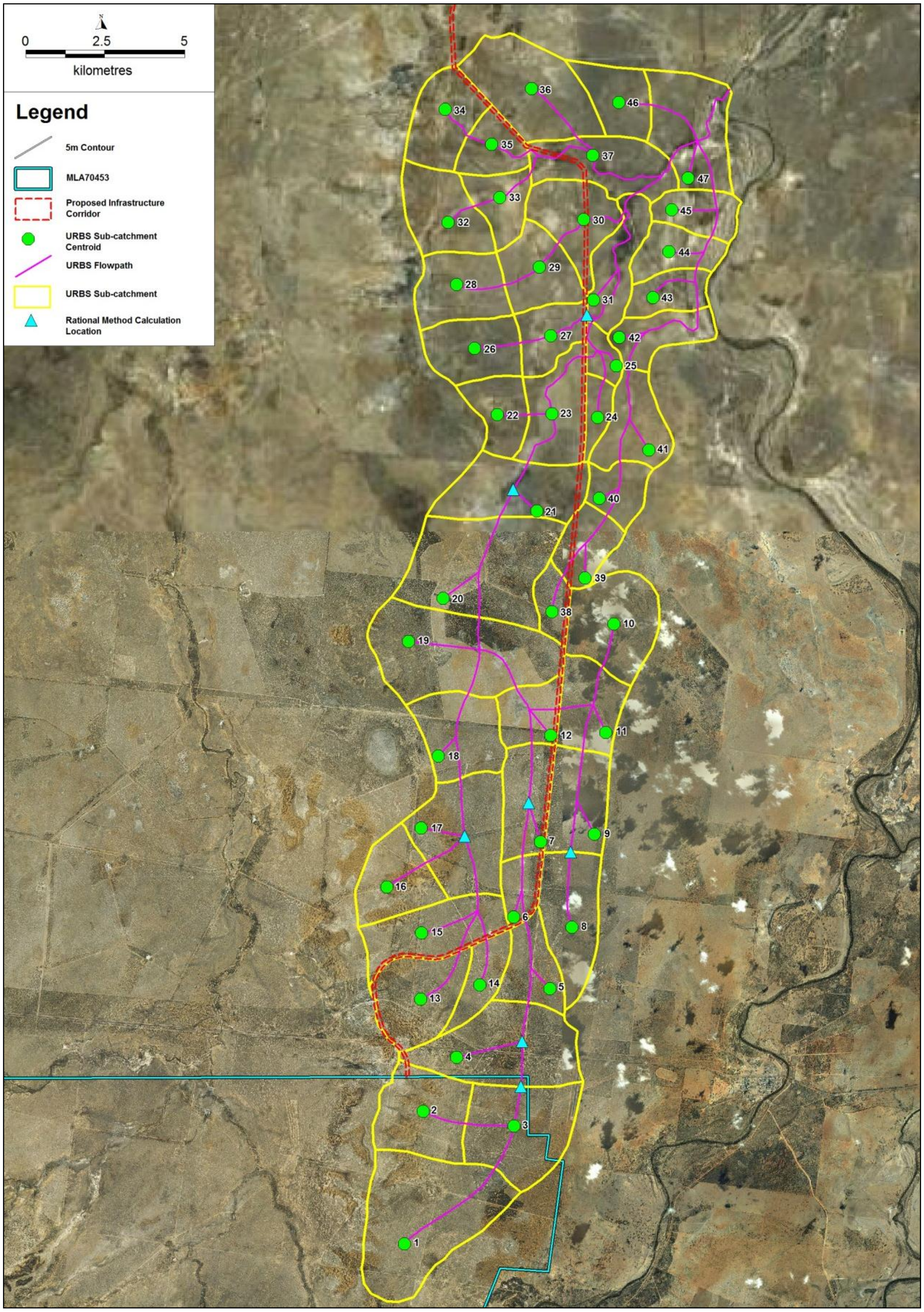

Figure 2-4 Infrastructure Corridor URBS Model Configuration

2.1.3 Post-Developed Conditions URBS Model Configurations

The SGCP mining will impacts the existing hydrologic and hydraulic behaviour of Tallarenha Creek, Sapling Creek and Dead Horse Creek. The proposed impacts are summarised below and shown in Figure 2-5:

- Construction of an open clean water drainage channel which traverses the open-cut operations. Tributaries of Tallarenha Creek will be directed into this channel which will then discharge into the main channel of Tallarenha Creek downstream of the SGCP lease.
- Connecting the upper catchment of Sapling Creek with the Dead Horse Creek catchment with a 4.4km diversion channel.

The configuration of the Tallarenha Creek Post-Developed Conditions URBS model is shown in Figure 2-6. The model covers the entire catchment and consists of 80 sub-catchments. Summary details of sub-catchment areas are given in Table 2-9. The configuration of the Sapling Creek Post Developed Conditions URBS model is shown in Figure 2-7. The model covers the entire catchment and consists of 21 sub-catchments. Summary details of sub-catchment areas are given in Table 2-10. The configuration of the Dead Horse Creek Post Developed Conditions URBS model is shown in Figure 2-8. The model covers the entire catchment and consists of 62 sub-catchments. Summary details of sub-catchment areas are given in Table 2-11.

The Infrastructure Corridor Existing Conditions URBS model was configured to allow extraction of flows at key locations for both existing and post development conditions. Therefore, a Post-Developed URBS model is not required for the Infrastructure Corridor.

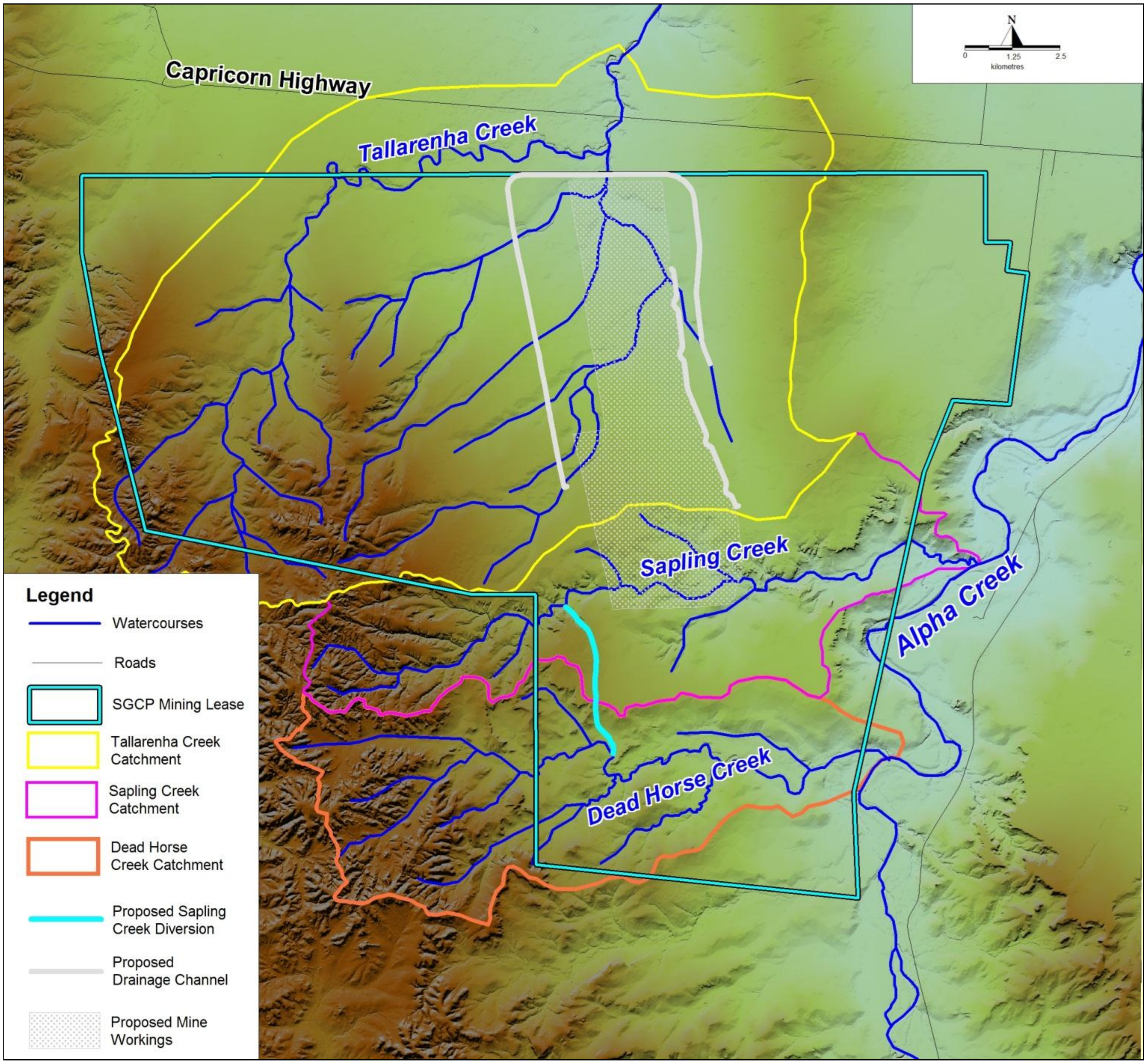

Figure 2-5 Proposed Watercourse Diversion Works

Table 2-9 Adopted Tallarenha Creek (Post-Developed Conditions) URBS Model Sub-Catchment Areas

Sub-Catchment Number	Area (km²)	Sub-Catchment Number	Area (km²)	Sub-Catchment Number	Area (km²)
1	8.53	28	0.83	55	0.35
2	4.36	29	0.98	56	0.53
3	3.66	30	0.60	57	0.73
4	3.47	31	1.67	58	0.67
5	6.89	32	1.85	59	0.73
6	5.90	33	1.08	60	0.48
7	4.82	34	0.91	61	0.45
8	3.90	35	0.73	62	0.45
9	4.33	36	0.47	63	0.45
10	5.96	37	0.68	64	0.44
11	5.77	38	0.81	65	1.18
12	5.33	39	0.64	66	0.97
13	6.90	40	0.41	67	5.96
14	4.38	41	0.24	68	2.74
15	0.35	42	0.30	69	0.55
16	0.50	43	0.39	70	1.07
17	0.39	44	0.13	71	1.54
18	0.43	45	0.11	72	1.50
19	0.45	46	0.17	73	0.69
20	0.80	47	0.76	74	2.95
21	0.59	48	5.14	75	3.19
22	0.50	49	1.61	76	4.24
23	0.56	50	0.91	77	4.58
24	0.45	51	0.96	78	6.93
25	5.20	52	1.42	79	7.44
26	1.42	53	0.94	80	6.32
27	1.41	54	1.04		

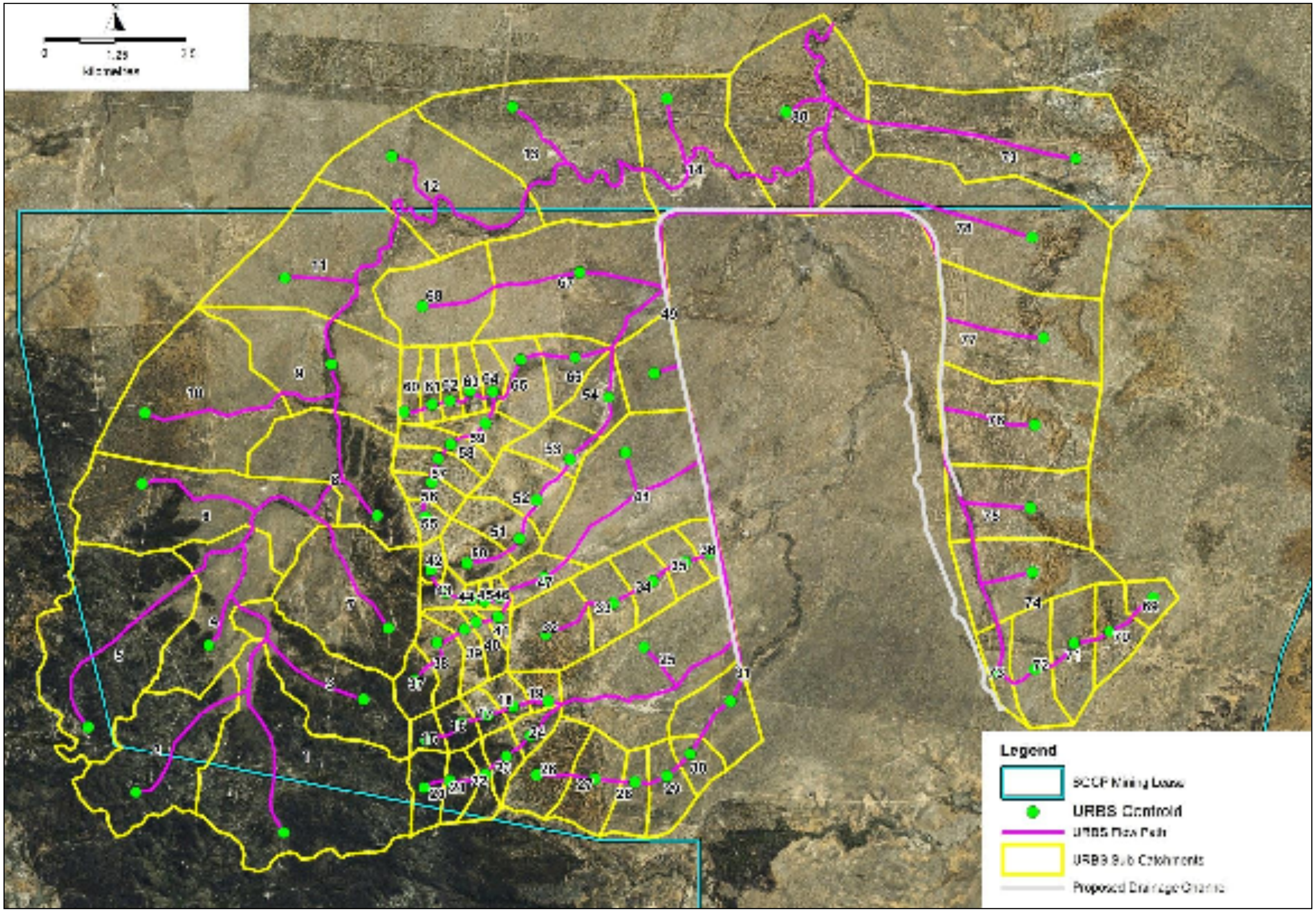

Figure 2-6 Tallarenha Creek (Post-Developed Conditions) URBS Model Configuration

Table 2-10 Adopted Sapling Creek (Post-Developed Conditions) URBS Model Sub-Catchment Areas

Sub-Catchment Number	Area (km^2)	Sub-Catchment Number	Area (km^2)
1	3.42	12	2.75
2	2.01	13	2.09
3	0.72	14	2.04
4	1.40	15	2.19
5	1.63	16	1.45
6	1.33	17	3.02
7	0.92	18	3.09
8	1.60	19	3.48
9	1.11	20	1.52
10	0.83	21	2.22
11	1.52		

Table 2-11 Adopted Dead Horse Creek (Existing Conditions) URBS Model Sub-Catchment Areas

Sub-Catchment Number	Area (km^2)	Sub-Catchment Number	Area (km^2)	Sub-Catchment Number	Area (km^2)
1	1.66	22	1.30	43	0.27
2	1.84	23	0.80	44	3.50
3	1.43	24	0.92	45	2.19
4	1.61	25	0.73	46	1.11
5	1.39	26	2.38	47	1.80
6	1.05	27	0.16	48	1.61
7	1.32	28	0.48	49	1.17
8	1.24	29	0.49	50	0.87
9	1.26	30	0.53	51	2.51
10	0.94	31	0.70	52	2.00
11	1.32	32	0.74	53	2.55
12	1.97	33	0.59	54	3.50
13	0.91	34	0.84	55	1.47
14	0.34	35	1.06	56	1.37
15	0.65	36	0.89	57	1.63
16	0.86	37	0.69	58	1.73
17	0.84	38	0.89	59	0.89
18	2.38	39	0.45	60	4.89
19	2.98	40	0.37	61	3.00
20	3.42	41	0.40	62	3.83
21	1.14	42	0.55		

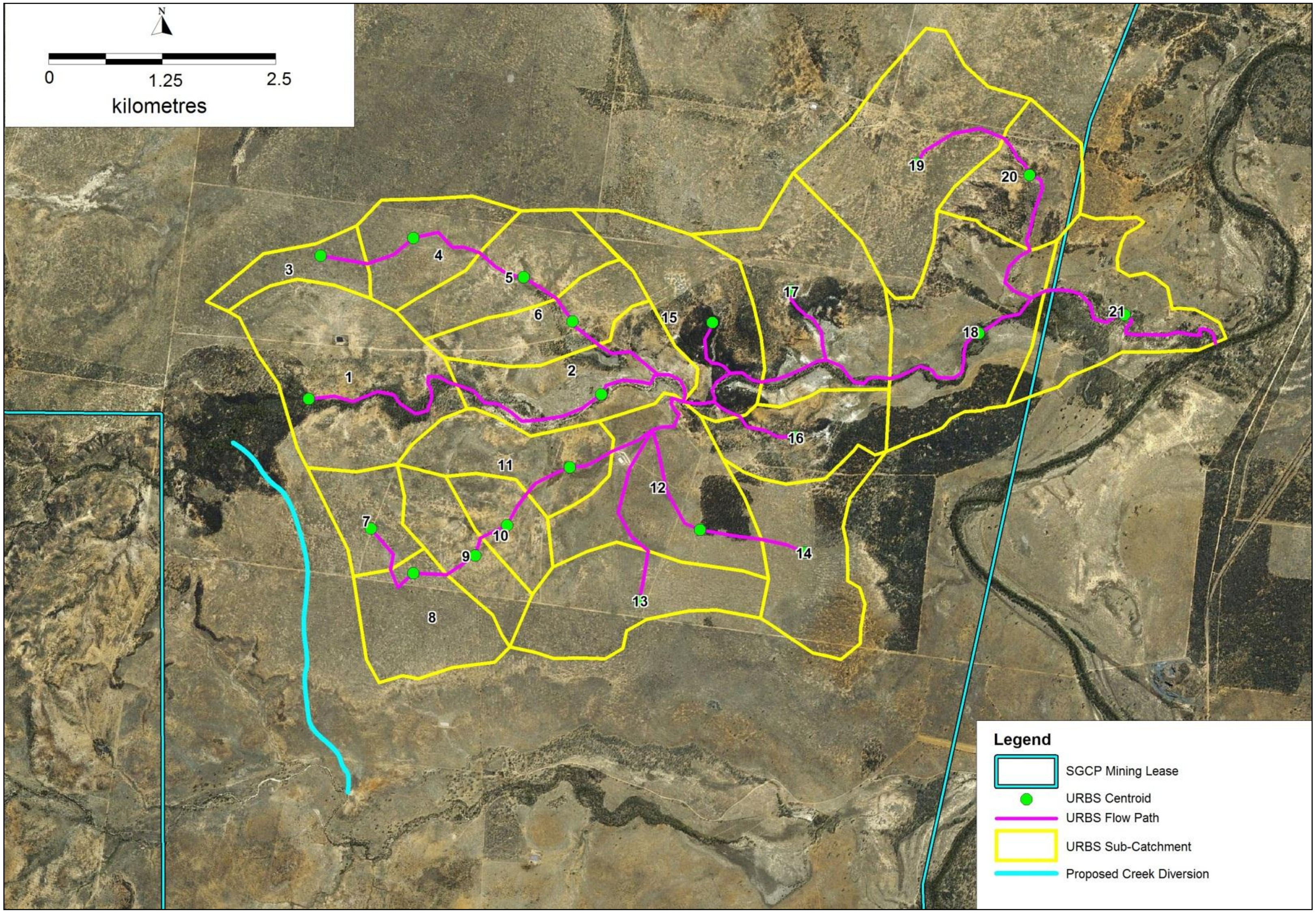

Figure 2-7 Sapling Creek (Post-Developed Conditions) URBS Model Configuration

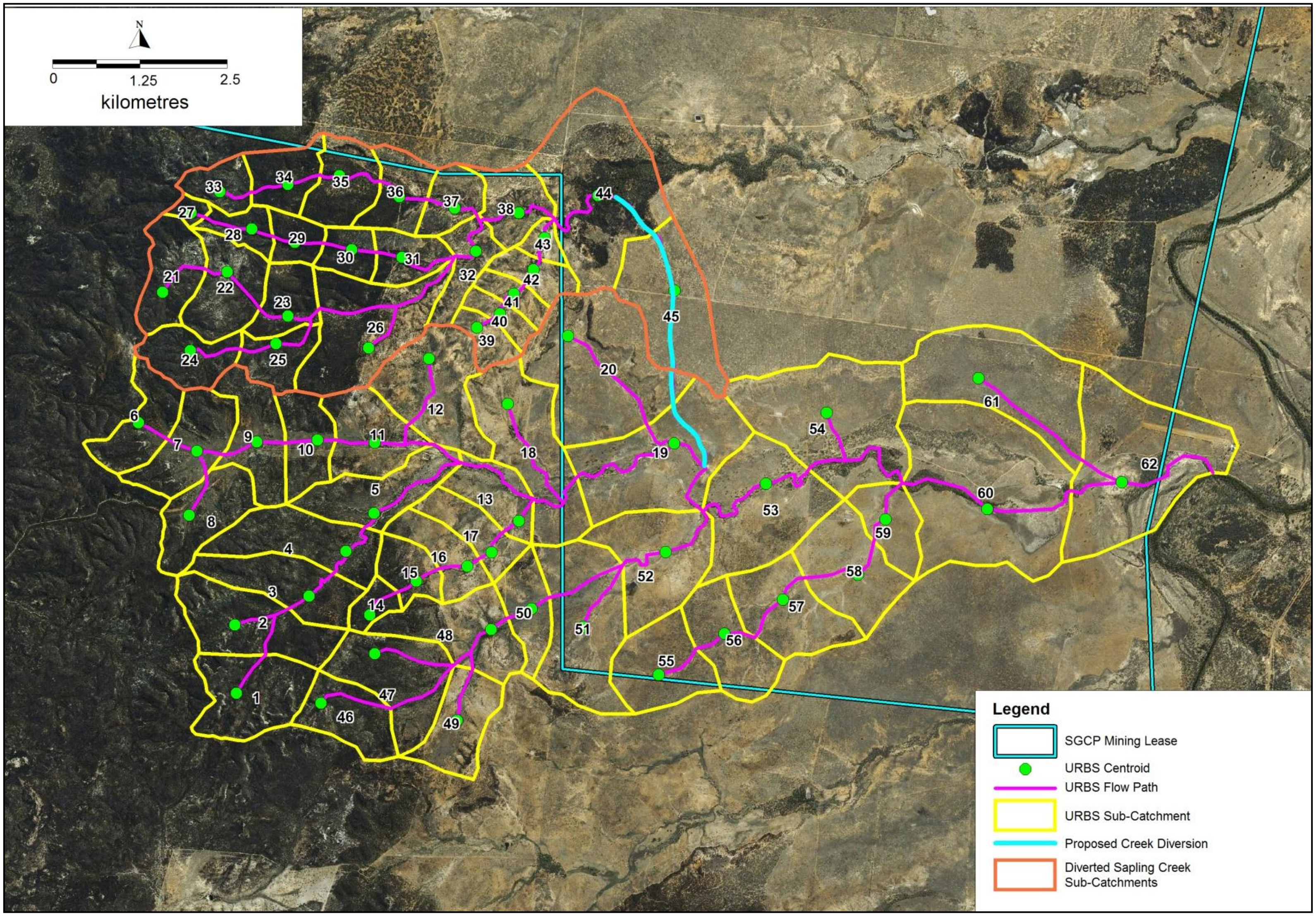

Figure 2-8 Dead Horse Creek (Post-Developed Conditions) URBS Model Configuration

2.1.4 Design rainfall depths up to 100 year ARI

Design rainfall depths for the 2, 50 and 100 year ARI events were estimated for a range of storm durations using the methods outlined in Australian Rainfall and Runoff (Pilgrim, 1998). An Aerial Reduction Factor (ARF) of 1.0 for design rainfalls was adopted for all modelled design events.

Due to the close proximity of the catchments to each other the design rainfall depths do not vary significantly. As such, the rainfall depths given in Table 2-3 were adopted for the 2, 50 and 100 year ARI design rainfall depths for all three (3) modelled catchments.

2.1.5 Design Probable Maximum Precipitation

Design rainfall depths for the Probable Maximum Precipitation (PMP) event for post-developed conditions for the catchment area located within the mining lease were estimated for storm durations ranging from 15 minutes to 72 hours using the Generalised Short-Duration Method (GSDM) outlined in BoM (2003). An interpolation between the design rainfall depths for 50 and 100 year ARI events and the PMP event was then undertaken to estimate the design rainfall depths for the 1,000 and 3,000 year ARI design events using the methodology outlined in Australian Rainfall and Runoff (Pilgrim, 1998).

Due to the close proximity of the catchments to each other the design rainfall depths do not vary significantly. As such, the rainfall depths outlined Table 2-12 were adopted for the 3 hydrologic models. Table 2-12 shows the estimated PMP rainfall depths for durations from 0.25 hours to 6 hours. The PMP rainfall depths were also adopted for the existing case Infrastructure Corridor hydrologic model.

Table 2-12 Extreme Event Design Rainfall Depths (mm)

Storm Duration (hours)	1,000 Year ARI	3,000 Year ARI	PMP
0.25	64.8	76.5	120.0
0.5	95.0	114.4	350.0
1	131.9	158.1	410.0
3	174.0	207.8	470.0
6	205.6	245.0	510.0

2.2 DESIGN DISCHARGES – EXISTING CONDITIONS

Table 2-13 shows the URBS model predicted peak flood discharges and critical storm durations estimated for the Tallarenha Creek, Sapling Creek and Dead Horse Creek catchments for the existing conditions for 2 year, 50 year and 100 year ARI design events.

Table 2-14 shows the URBS model predicted peak flood discharges and critical storm durations estimated for the Infrastructure Corridor. While the longer duration storms (6 hours) generate slightly larger discharges at the downstream outlet, local catchment discharges are larger at the upstream sections of the model in the shorter duration storms (2 hours).

Table 2-13 Existing Conditions URBS Model Predicted Design Discharges and Critical Storm Durations

Catchment	100 Year ARI		50 Year ARI		2 Year ARI	
	Peak Discharge (m^3/s)	Critical Storm Duration (hours)	Peak Discharge (m^3/s)	Critical Storm Duration (hours)	Peak Discharge (m^3/s)	Critical Storm Duration (hours)
Tallarenha Creek	1054.6	3.0	807.2	3.0	178.2	6.0
Sapling Creek	331.7	4.5	257.7	4.5	58.4	6.0
Dead Horse Creek	278.5	4.5	216.0	4.5	48.6	6.0

Table 2-14 Existing Conditions URBS Model Predicted Design Discharges and Critical Storm Durations (Infrastructure Corridor)

Catchment	10 Year ARI		50 Year ARI	
	Peak Discharge (m^3/s)	Critical Storm Duration (hours)	Peak Discharge (m^3/s)	Critical Storm Duration (hours)
Infrastructure Corridor	420.5	6	709.9	6

2.3 COMPARISON WITH RATIONAL METHOD DESIGN DISCHARGES

Table 2-15 to Table 2-18 show a comparison of the estimated Rational Method peak design discharges at key locations within each catchment with the design discharges estimated using the URBS model for existing conditions. The sub-catchment locations used to calculate the Rational Method estimates were selected based on the topographic variations in the catchments and their proximity to URBS output nodes.

The URBS model and Rational Method discharges are in satisfactory agreement at the various locations within the catchment. The Rational Method discharges vary slightly than those estimated by the URBS model, but are considered good for the 10 year, 50 year and 100 year ARI design events. The Rational Method estimates are significantly higher than the equivalent URBS estimates for the 2 year ARI design event. This is due to the significance of the adopted rainfall loss rates in the URBS model. The URBS model discharges have been adopted for this study.

Table 2-15 Comparison of Rational Method and URBS Model Design Discharge Estimates – Tallarenha Creek

Location	Catchment Area (ha)	Channel Length (m)	Slope (m/m)	100 Year ARI		50 Year ARI		2 Year ARI	
				Rational Method Design Discharge (m^3/s)	URBS Peak Discharge (m^3/s)	Rational Method Design Discharge (m^3/s)	URBS Peak Discharge (m^3/s)	Rational Method Design Discharge (m^3/s)	URBS Peak Discharge (m^3/s)
Sub-Catchment 52	279	2,250	0.014	54.6	51.4	45.9	36.6	13.9	6.2
Sub-Catchment 37	535	4,000	0.011	72.5	74.8	60.9	55.5	18.2	10.1
Sub-Catchment 70	302	2,270	0.022	62.8	56.2	53.0	40.1	16.7	6.8
Sub-Catchment 42	664	3,900	0.005	93.4	92.4	78.5	68.7	23.7	12.4
Sub-Catchment 59	832	6,700	0.011	106.0	125.5	88.3	92.9	24.0	17.5

Table 2-16 Comparison of Rational Method and URBS Model Design Discharge Estimates – Sapling Creek

Location	Catchment Area (ha)	Channel Length (m)	Slope (m/m)	100 Year ARI		50 Year ARI		2 Year ARI	
				Rational Method Design Discharge (m^3/s)	URBS Peak Discharge (m^3/s)	Rational Method Design Discharge (m^3/s)	URBS Peak Discharge (m^3/s)	Rational Method Design Discharge (m^3/s)	URBS Peak Discharge (m^3/s)
Sub- Catchment 17	408	4,400	0.022	65.3	68.5	53.9	49.2	16.2	8.9
Sub-Catchment 11	236	4,100	0.024	35.2	44.1	28.8	31.5	8.6	5.7
Sub-Catchment 23	203	2,300	0.018	40.1	45.9	33.7	32.6	10.2	5.7
Total Catchment Area	6,359	17,600	0.008	332.5	331.7	279.2	257.7	84.3	58.4

Table 2-17 Comparison of Rational Method and URBS Model Design Discharge Estimates – Dead Horse Creek

Location	Catchment Area (ha)	Channel Length (m)	Slope (m/m)	100 Year ARI		50 Year ARI		2 Year ARI	
				Rational Method Design Discharge (m^3/s)	URBS Peak Discharge (m^3/s)	Rational Method Design Discharge (m^3/s)	URBS Peak Discharge (m^3/s)	Rational Method Design Discharge (m^3/s)	URBS Peak Discharge (m^3/s)
Sub- Catchment 13	361	3,000	0.021	67.2	62.0	56.4	43.9	16.9	7.7
Sub- Catchment 5	361	3,000	0.021	88.5	99.2	73.9	73.1	21.4	13.9
Sub-Catchment 34	710	2,100	0.011	88.0	93.0	73.4	68.7	20.51	12.9
Total Catchment Area	6,554	16,250	0.008	284.9	278.5	238.9	216.0	73.7	48.6

Table 2-18 Comparison of Rational Method and URBS Model Design Discharge Estimates – Infrastructure Corridor

Location	Catchment Area (ha)	Channel Length (m)	Slope (m/m)	10 Year ARI		50 Year ARI	
				Rational Method Design Discharge (m3/s)	URBS Peak Discharge (m3/s)	Rational Method Design Discharge (m3/s)	URBS Peak Discharge (m3/s)
Sub-Catchment 3	3,097	8,000	0.002	104.6	103.3	192.3	180.5
Sub-Catchment 4	1,000	4,650	0.009	47.3	41.9	91.5	73.2
Sub-Catchment 7	501	3,500	0.002	20.8	24.7	37.6	42.5
Sub-Catchment 8	687	4,100	0.001	26.6	28.9	48.9	50.2
Sub-Catchment 17	1,451	5,500	0.002	58.6	63.2	106.8	109.8
Sub-Catchment 21	773	3,700	0.002	34.7	35.8	63.7	61.9
Sub-Catchment 27	910	5,000	0.003	38.5	40.7	69.2	70.2

2.4 DESIGN DISCHARGES – POST-DEVELOPED CONDITIONS

2.4.1 2 to 100 year ARI Design Event Discharges

Table 2-19 shows the peak flood discharges and critical storm durations for the Tallarenha Creek, Sapling Creek and Dead Horse Creek catchments for 2, 50 and 100 year ARI design events downstream of the SGCP for the post-developed conditions. The design rainfalls, model parameters and rainfall loss rates adopted for the post developed conditions are the same as for the existing conditions.

The same hydrological model for the Infrastructure Corridor was used for both the existing and post-developed conditions, therefore peak flood discharges and critical durations for the post-developed conditions are the same as for the existing conditions shown in Table 2-14.

Table 2-19 Post-Developed Conditions Design Discharges and Critical Storm Durations

Catchment	100 Year ARI		50 Year ARI		2 Year ARI	
	Peak Discharge (m^3/s)	Critical Storm Duration (hours)	Peak Discharge (m^3/s)	Critical Storm Duration (hours)	Peak Discharge (m^3/s)	Critical Storm Duration (hours)
Tallarenha Creek	748.5	4.5	580.0	4.5	129.4	6.0
Sapling Creek	298.1	2.0	229.3	3.0	59.6	3.0
Dead Horse Creek	407.5	4.5	316.5	4.5	71.7	6.0

Table 2-19 indicates that the proposed creek diversion works result in a reduction of peak discharges for both Tallarenha Creek and Sapling Creek. The peak discharges for Dead Horse Creek are increased due to the diversion of flows from the upper catchment of Sapling Creek into Dead Horse Creek.

2.4.2 Extreme Event Design Discharges

Table 2-20 shows the peak flood discharges and critical storm durations for the Tallarenha Creek, Sapling Creek and Dead Horse Creek catchments downstream of the SGCP for the post-developed condition for 100 year ARI, 3000 year ARI, and PMF events. While the longer duration storms (3 and 6 hours) generate slightly larger discharges at the outlet, the peak flows were larger in the upstream sections of the model in the shorter duration storms (1 hour). The post-developed conditions model was used to determine the impacts of the mining operations on existing levels as well as sizing proposed drainage channels. As such, the 1-hour storm duration hydraulic model results are presented for these events.

Table 2-20 also shows the peak flood discharges for the PMF event for the Infrastructure Corridor catchment.

Table 2-20 Post-Developed Conditions URBS Model Design Discharges and Critical Storm Durations for Extreme Design Events

URBS	1,000 Year ARI		3,000 Year ARI		PMF	
Catchment	Peak Discharge (m^3/s)	Critical Storm Duration (hours)	Peak Discharge (m^3/s)	Critical Storm Duration (hours)	Peak Discharge (m^3/s)	Critical Storm Duration (hours)
Tallarenha Creek	1,047	1.0	1,264	1.0	3,369	1.0
Sapling Creek	483	1.0	585	1.0	1,582	1.0
Dead Horse Creek	584	1.0	703	1.0	1,854	1.0
Infrastructure Corridor	-	-	-	-	4,239	1.0

3 FLOOD HYDRAULICS

3.1 AVAILABLE DATA

Topographic aerial survey data for the study area was provided by Met Serve Pty Ltd. The aerial laser scanning (ALS) data, which was obtained from a fixed wing aircraft in May 2010 and covering an area of approximately 1,395 km^2, was supplied in a thinned ground ASCII space delimited format which has a derived point accuracy of +/- 0.1m. This data was converted into a digital terrain model (DTM) for use in the hydraulic modelling and mapping tasks.

3.2 MODELLING OVERVIEW

3.2.1 Sapling Creek and Dead Horse Creek

The one-dimensional HEC-RAS hydraulic model (USACE, 2009), was used to estimate the 2, 50, and 100 year ARI design flood levels along Sapling Creek and Dead Horse Creek for existing and post-development conditions. Both Sapling Creek and Dead Horse Creek are well defined waterways and are well suited for analysis using HEC-RAS.

The discharges estimated using the URBS runoff-routing model were adopted as inflows to the HEC-RAS model.

3.2.2 Tallarenha Creek

Owing to the complex nature of its various watercourses and their interactions, the TUFLOW hydrodynamic model (WBM, 2008) was used to simulate the flow behaviour of Tallarenha Creek and its numerous tributaries for the 2, 50, 100 year ARI design events for existing conditions. The TUFLOW model was used to investigate post-development conditions flood behaviour for the 2, 50, 100, 1,000, 3,000 year and PMF design events. TUFLOW represents hydraulic conditions on a fixed grid 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.

With the exception of the main Tallarenha Creek channel, flow paths within the Tallarenha Creek catchment are not well defined and difficult to analyse using simplistic one-dimensional modelling techniques. The TUFLOW modelling package is suited to simulation of dynamic hydraulic behaviour of complex overland flow in rural areas and was considered the most appropriate investigative tool to determine the flood characteristics of the Tallarenha Creek catchment.

The discharges estimated using the URBS runoff-routing model were adopted as inflows to the TUFLOW model.

3.2.3 Infrastructure Corridor

The TUFLOW modelling package was used to simulate the flow behaviour in areas draining to unknown tributaries of Native Companion Creek for the 10 year, 50 year and PMF design events for existing and post-developed conditions. With the exception of unknown tributaries to native Companion Creek crossing the northern sections of the Infrastructure Corridor, flow paths in the upstream areas are not well defined.

The discharges estimated using the URBS runoff-routing model were adopted as inflows to the TUFLOW model.

3.2.4 Adopted Manning's 'n' Values

Both the TUFLOW and HEC-RAS models use Manning's 'n' values to represent hydraulic resistance (notionally channel or floodplain roughness). In the absence of suitable calibration data for the hydraulic models, Manning's 'n' values were selected based on typical published values (for example, those of Chow, 1959). The adopted Manning's n values for the Sapling Creek and Dead Horse Creek HEC-RAS models were:

- Overbank Areas: 'n' = 0.05
- Creek Channel: 'n' = 0.035

The adopted Manning's n values for the more complex Tallarenha Creek and Infrastructure Corridor TUFLOW models were:

- Channel - Vegetated: 'n' = 0.06
- Channel - Dry: 'n' = 0.035
- Light Scrub Overbanks: 'n' = 0.05
- Roads: 'n' = 0.015
- Vegetated Diversion Channel (Post-Developed Conditions): 'n' = 0.04

The adopted overbank 'n' value is somewhat higher than would be indicated by typical vegetation on the floodplain. This slightly higher value was adopted because flow across the floodplain is expected to be relatively shallow, resulting in a higher hydraulic resistance.

3.2.5 Tailwater Conditions

The downstream boundaries of the models were set well downstream of the SGCP lease area to minimise its influence on flood behaviour predicted for lease areas. The downstream boundary conditions used for the three (3) hydraulic models were:

- Tallarenha Creek: Normal Depth 0.006 m/m;
- Sapling Creek: Normal Depth 0.004 m/m; and
- Dead Horse Creek: Normal Depth 0.004 m/m.
- Infrastructure Corridor; Constant Water level 318 m AHD.

These normal depth slopes are typical of the bed slopes found in each of the creek systems. The model results in the area of interest are insensitive to the adopted downstream boundary condition; changing the flood slope at the boundary by 0.001 resulted in only a minor change (less than 0.1m) to water surface levels throughout the models.

Due to the length of the downstream boundary and the limitations of the TUFLOW software, a normal depth boundary condition for this model resulted in major instabilities. As such, a water level simulating the level in nearby Native Companion Creek was adopted as the tailwater condition for the Infrastructure Corridor TUFLOW model. A sensitivity analysis of the tailwater conditions for the Infrastructure Corridor TUFLOW model was undertaken to determine what impact the constant water level has on water levels throughout the catchment. The results of this sensitivity analysis are outlined in Section 4.3.1.

3.3 MODEL CONFIGURATION –EXISTING CONDITIONS

3.3.1 Sapling Creek and Dead Horse Creek

Figure 3-1 shows the locations of the existing Conditions HEC-RAS model cross-sections for Sapling Creek and Dead Horse Creek.

3.3.2 Tallarenha Creek

Due to the large size of the Tallarenha Creek catchment, three (3) separate TUFLOW models were developed to reduce model simulation times. These separate models analysed the existing flow regime for the Tallarenha Creek main channel and two (2) tributaries of Tallarenha Creek. The location of each of the TUFLOW models is outlined in Figure 3-2. Similar inflow and downstream boundary conditions were adopted for all models in order to establish an existing conditions flood extent for the entire floodplain located within the mining lease.

3.3.3 Infrastructure Corridor

A single TUFLOW model was developed for the infrastructure corridor to analyse the existing flow regime in areas along the Infrastructure Corridor catchment area. Figure 3-3 shows the Existing Conditions TUFLOW model configuration. The model boundary does not include the north-western section of the Infrastructure Corridor as no topographical data was available for this area.

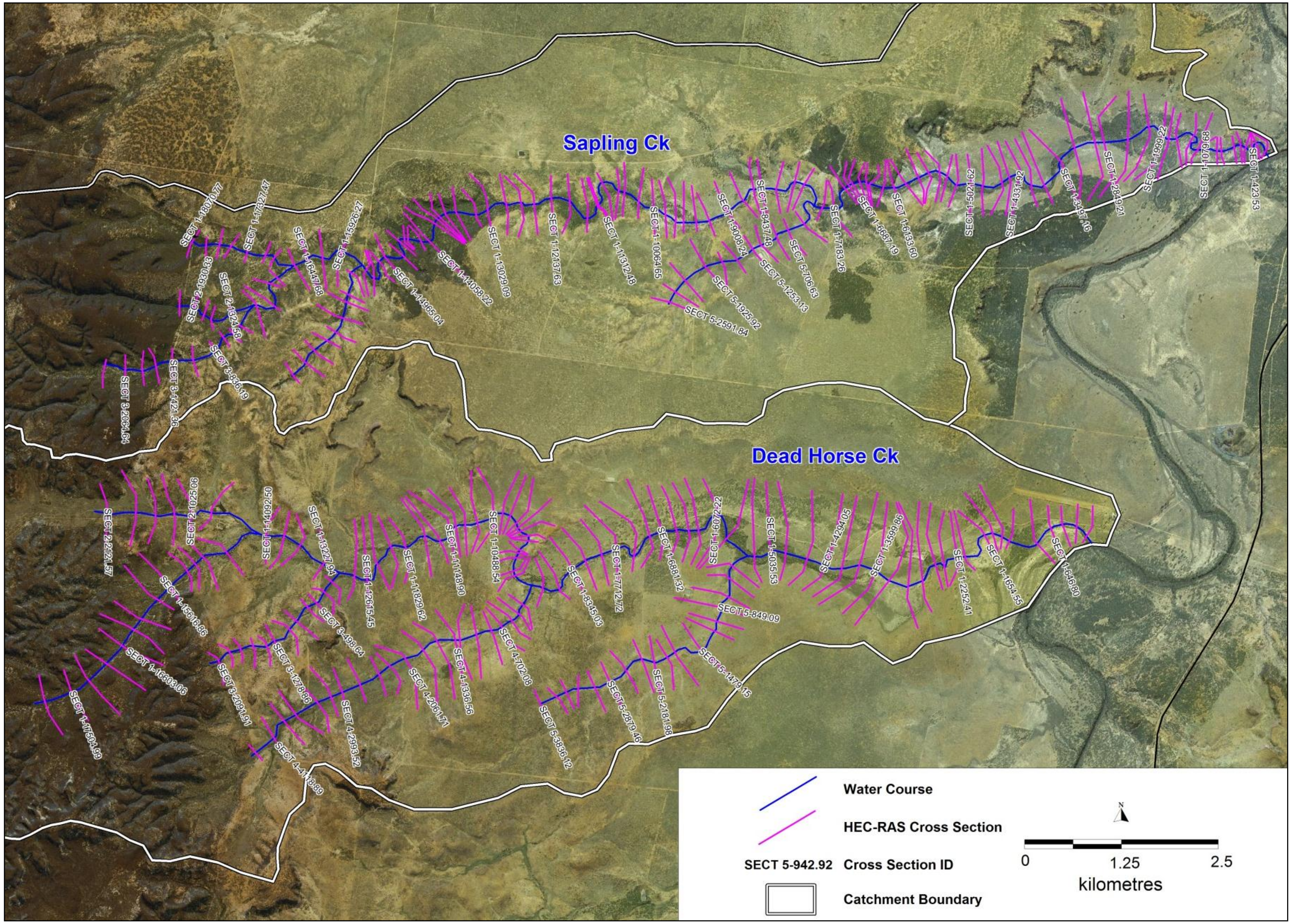

Figure 3-1 Existing Conditions HEC-RAS Layout – Sapling and Dead Horse Creeks

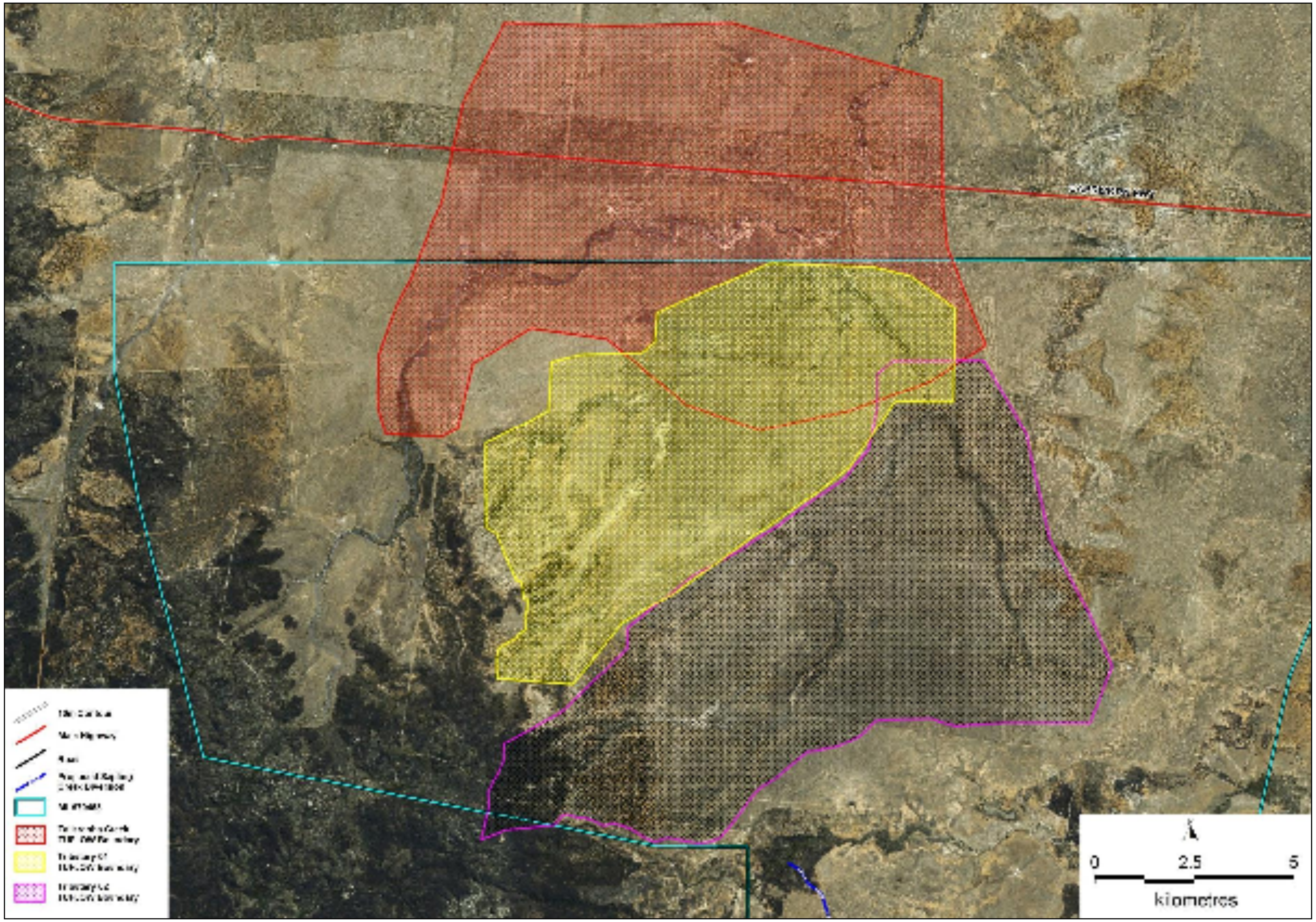

Figure 3-2 Tallarenha Creek Existing Conditions TUFLOW Model Configuration

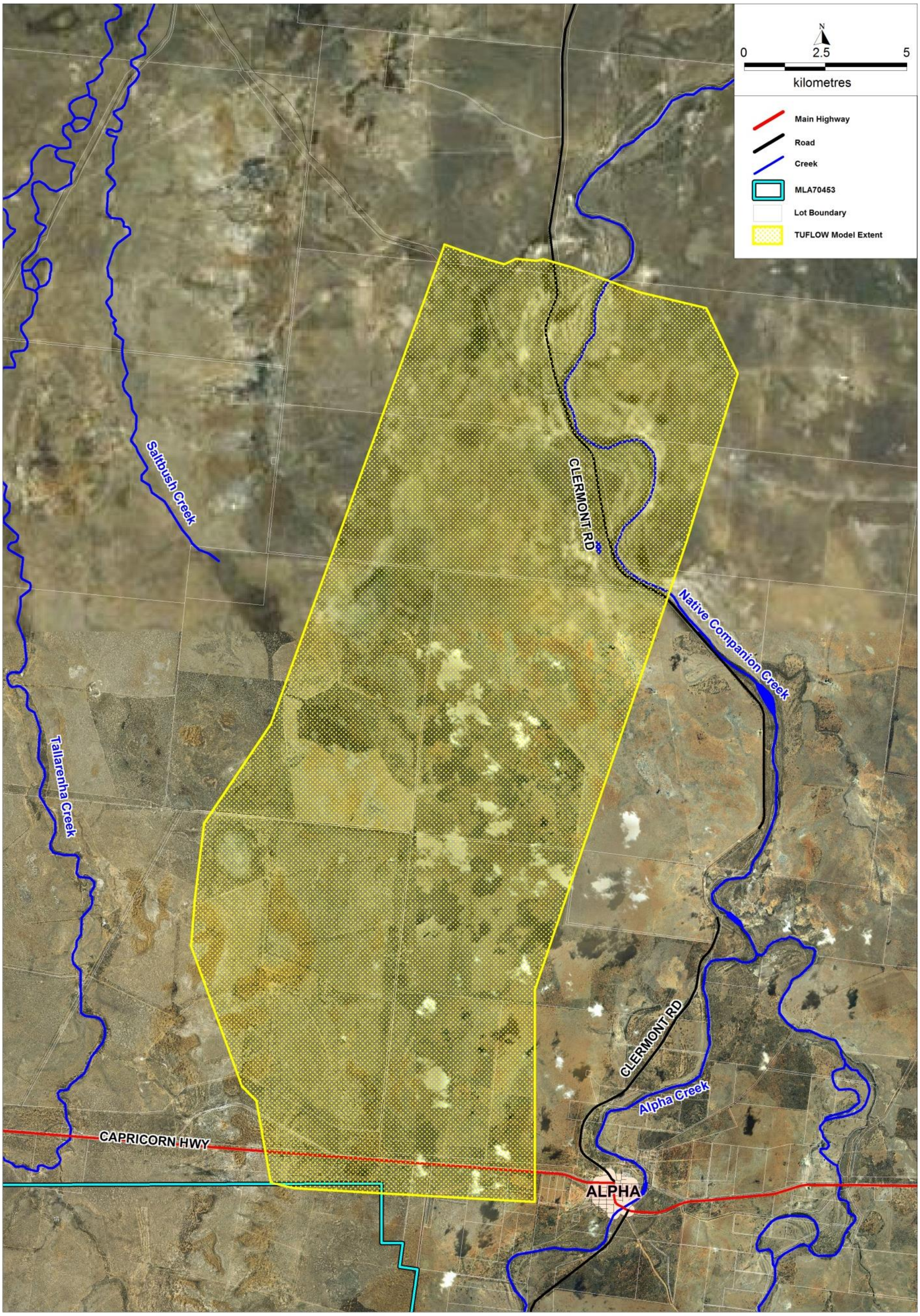

Figure 3-3 Infrastructure Corridor Existing TUFLOW Model Configuration

3.4 MODEL CONFIGURATION – POST-DEVELOPED CONDITIONS

3.4.1 Sapling Creek and Dead Horse Creek

Figure 3-4 shows the locations of the Post-Developed Conditions HEC-RAS model cross-sections for Sapling Creek and Dead Horse Creek. This model contains the upper-catchment of Sapling Creek and the entire Dead Horse Creek system which is linked by the Sapling Creek Diversion Channel.

3.4.2 Tallarenha Creek

The Post-Developed Conditions TUFLOW model for Tallarenha Creek was constructed using the existing DTM supplemented with subsidence contours. The DTM was then amended to reflect the proposed drainage channel. Due to the changes to the DTM from the subsidence contours, adjustment of the TUFLOW model was undertaken to ensure the subsidence contours were free draining. This adjustment was represented in the model by the inclusion of small drainage channels linking each subsidence contour. These small drainage channels were developed using the Z_Shape tool in TUFLOW. A nominal width of 30 m was adopted for all of the small drainage channels with the exception of the drainage channel to the north-west which contains flows of larger magnitude than the rest of the catchment. This drainage channel was modelled with a width of 60 m.

In addition to the small drainage channels, several levees were incorporated into the TUFLOW model to ensure containment of water on the mining lease and to prevent overflowing of water into the final void. Figure 3-5 shows the configuration of the post-developed Tallarenha Creek TUFLOW model.

3.4.3 Infrastructure Corridor

Figure 3-6 shows the post-Developed conditions TUFLOW model for the Infrastructure Corridor. The Infrastructure Corridor was modelled as an embankment. Openings were included at key locations such as creek crossings and high-flow areas to maintain existing flow patterns to the downstream areas of the model. The size of these embankment openings is critical in reducing afflux upstream of the embankment and have been sized by trial and error to ensure existing flow paths are maintained. During detailed design, the sizing of these openings may be further optimised without significantly impacting on the flood management outcomes.

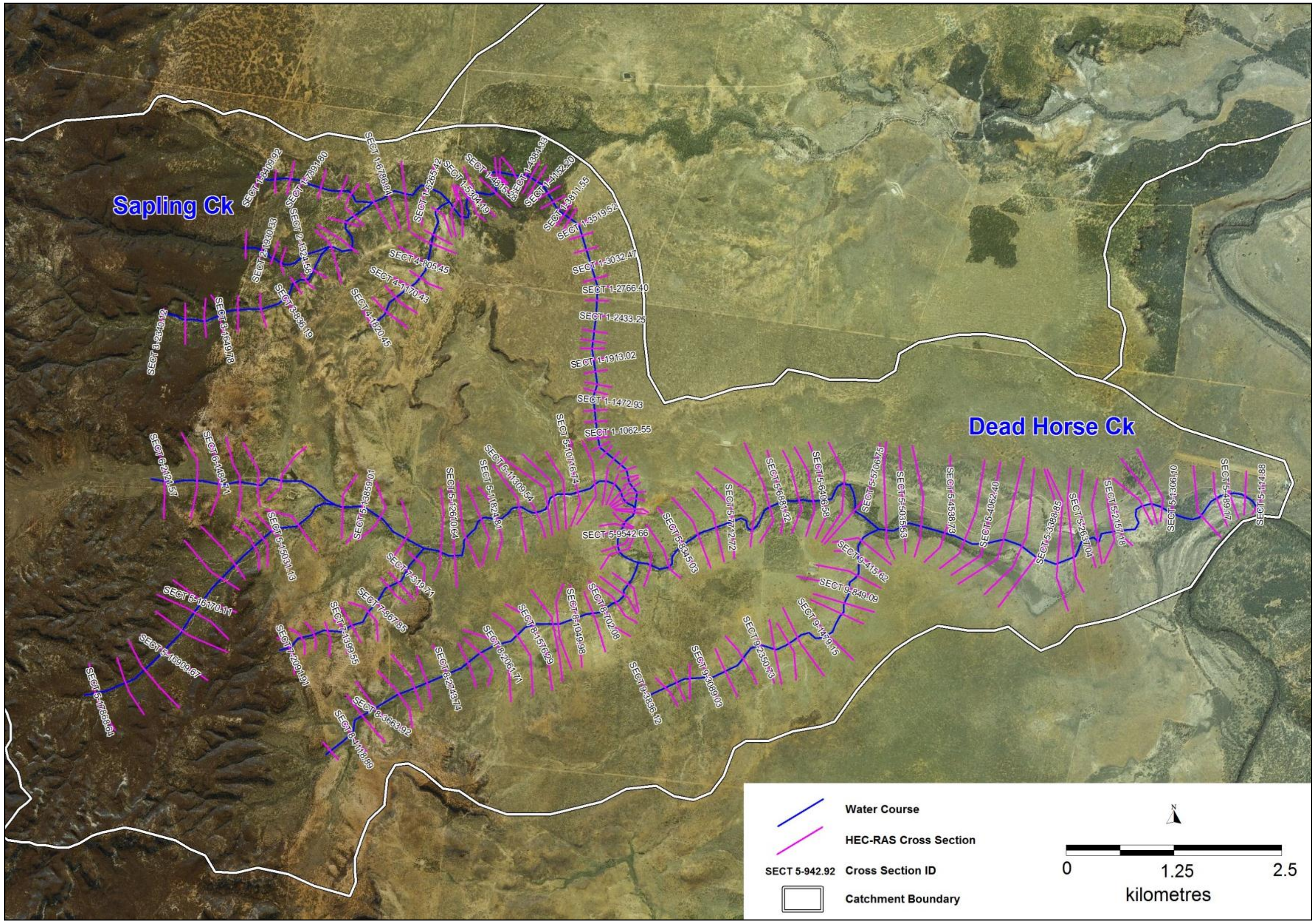

Figure 3-4 Existing Conditions HEC-RAS Layout – Sapling and Dead Horse Creeks

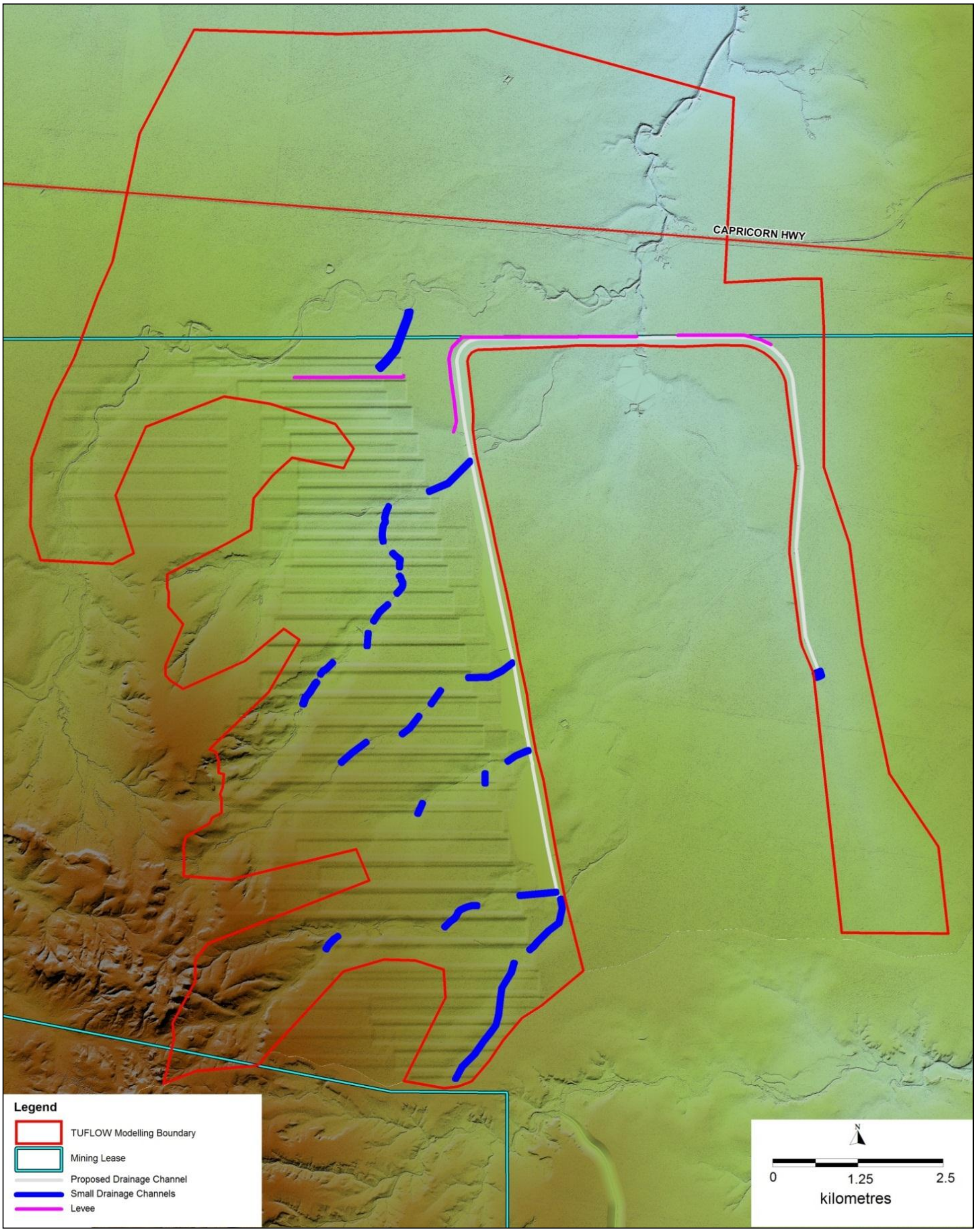

Figure 3-5 Tallarenha Post-Developed TUFLOW Model Configuration

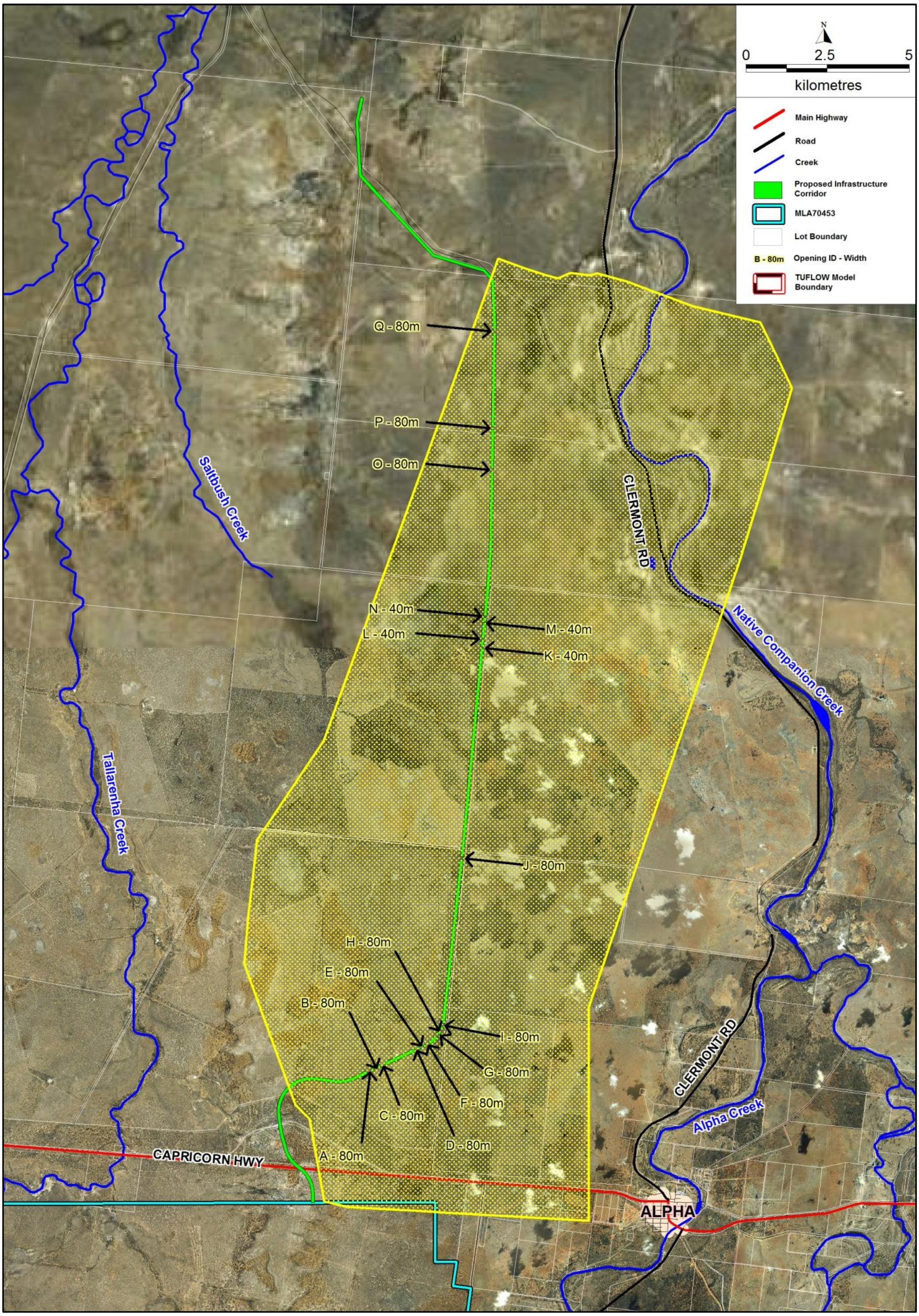

Figure 3-6 Infrastructure Corridor Post-Developed TUFLOW Model Configuration

4 HYDRAULIC MODEL RESULTS

The results of the existing and post-developed models for all four (4) catchments are outlined in the following sections.

4.1 TALLARENHA CREEK

Plans showing the flood depth, extent and velocity for the existing and post-developed Tallarenha Creek models are presented in Appendix E. An analysis of the model results for the existing and post-developed scenario indicates that the presence of the proposed post-development works results in little to no increase in the duration of inundation downstream of the mine lease.

4.2 SAPLING CREEK AND DEAD HORSE CREEK

Plans showing the flood depth, extent and velocity for the existing and post-developed Sapling Creek and Dead Horse Creek models are located in Appendix E.

The following series of figures show the results of the modelling of the existing Sapling Creek and Dead Horse Creek channels. The figures show each of the key hydraulic parameters listed in the DERM Central Queensland Watercourse Diversion Guidelines.

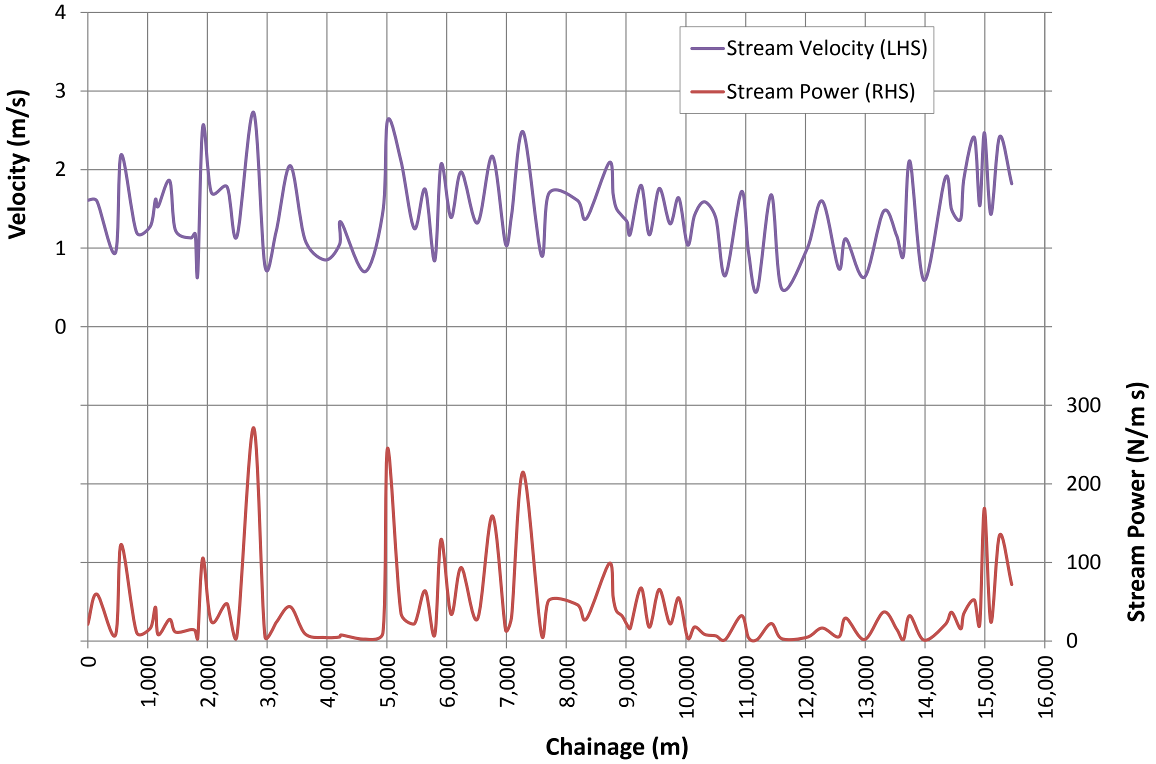

Figure 4-1 Sapling Creek Existing Conditions, 2 Year ARI Velocity and Stream Power Variations

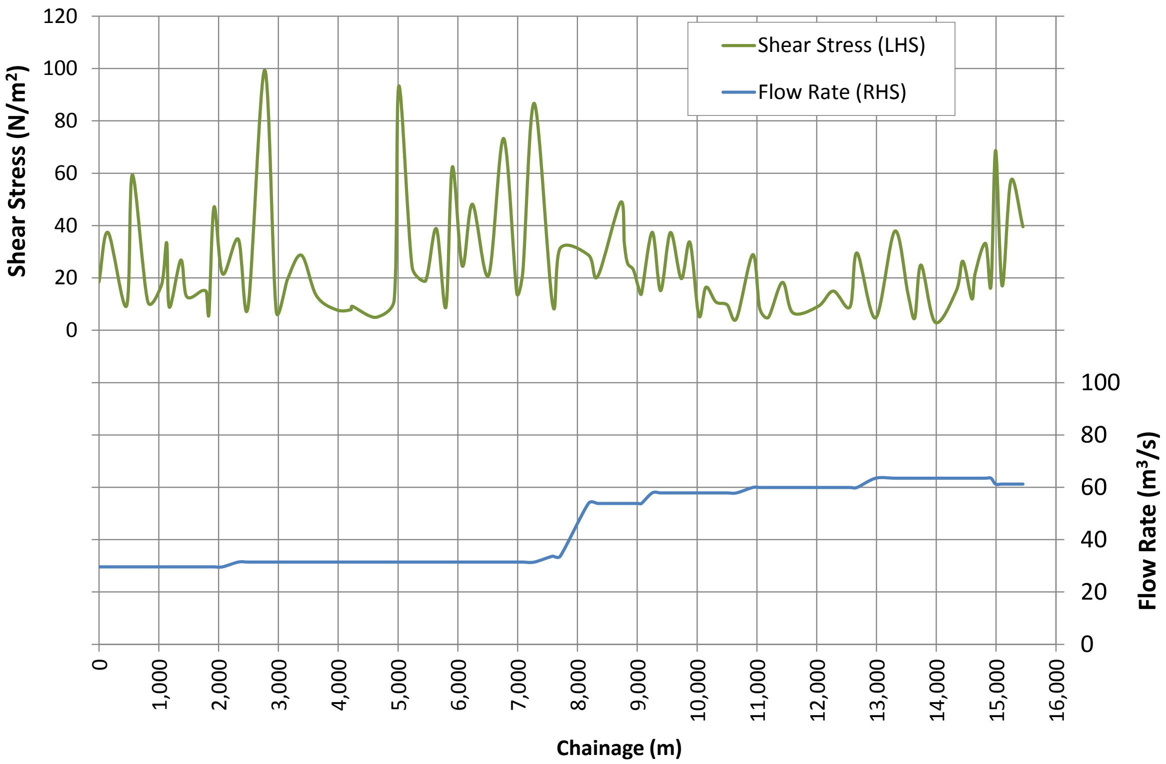

Figure 4-2 Sapling Creek Existing Conditions , 2 Year ARI Flow Rate and Shear Stress Variations

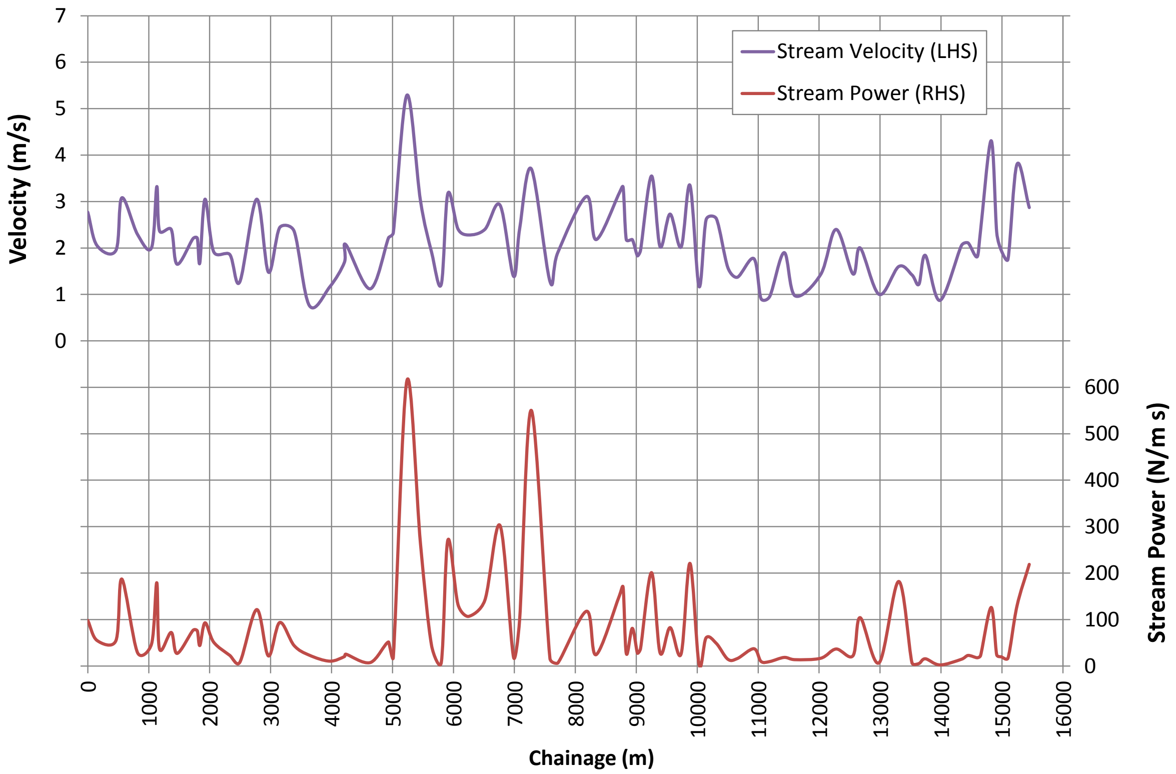

Figure 4-3 Sapling Creek Existing Conditions, 50 Year ARI Velocity and Stream Power Variations

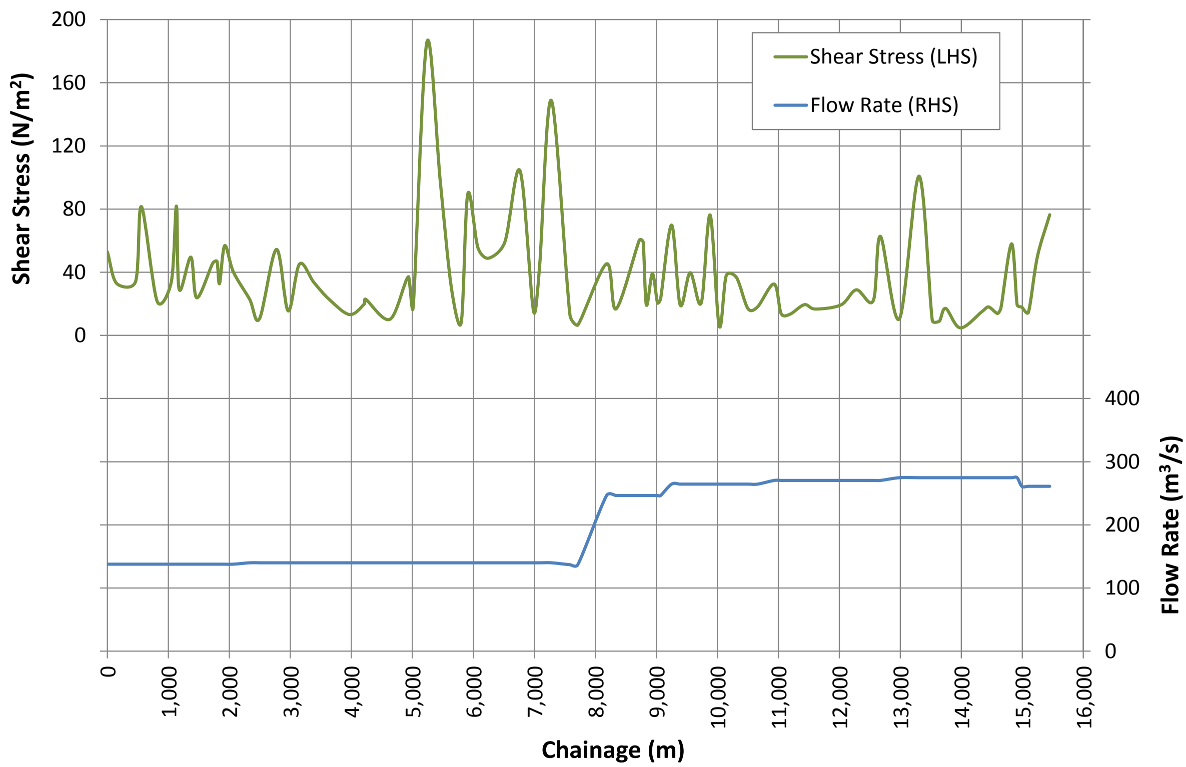

Figure 4-4 Sapling Creek Existing Conditions, 50 Year ARI Flow Rate and Shear Stress Variations

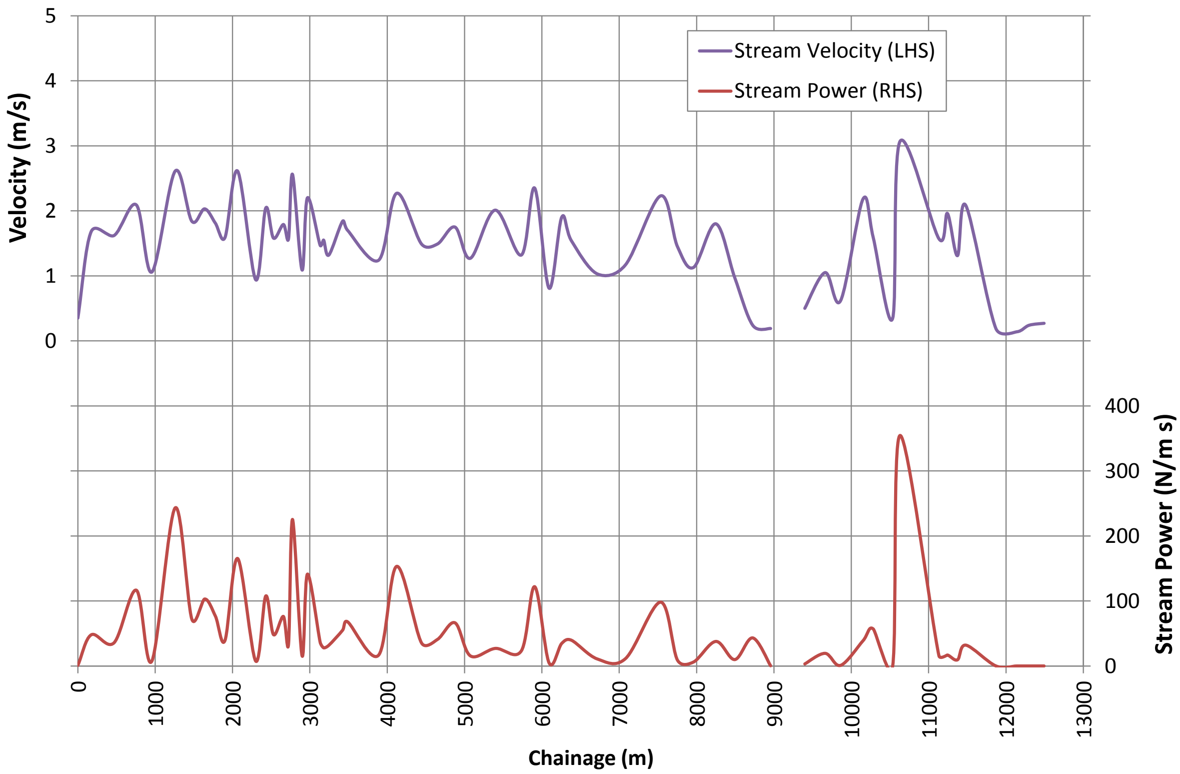

Figure 4-5 Dead Horse Creek Existing Conditions, 2 Year ARI Velocity and Stream Power Variations

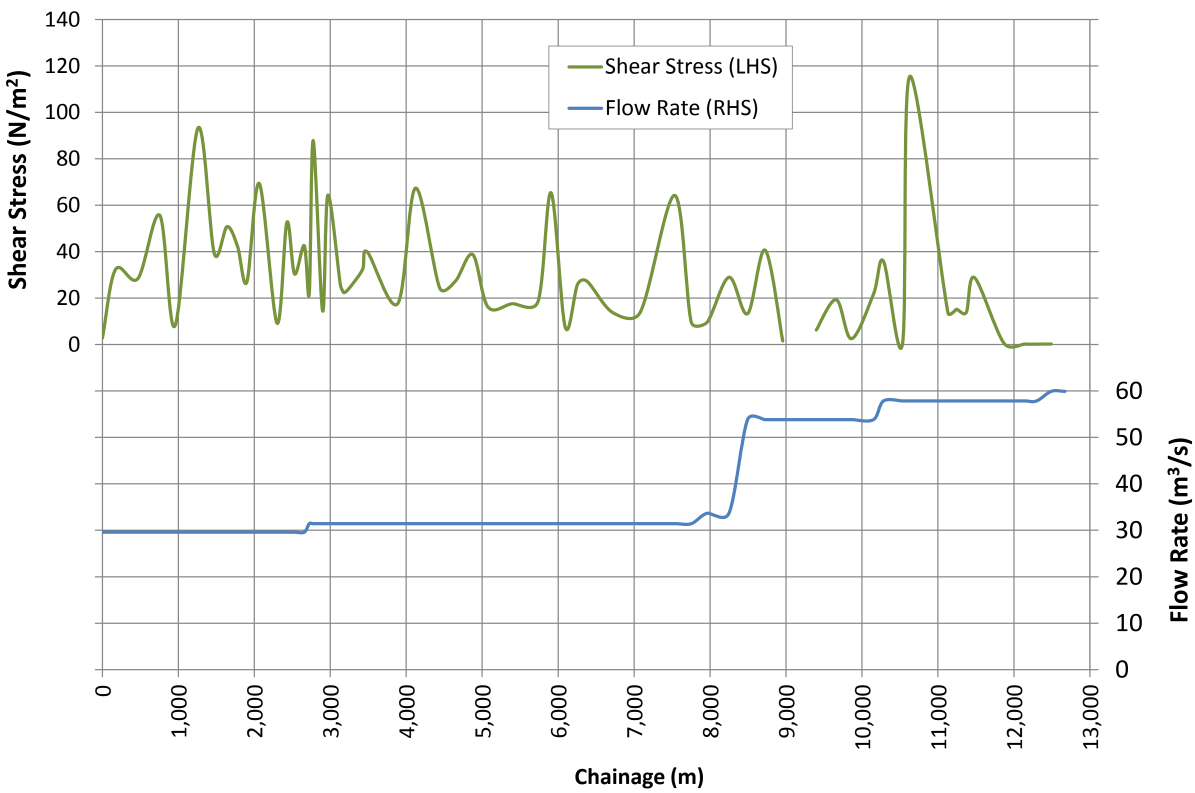

Figure 4-6 Dead Horse Creek Existing Conditions, 2 Year ARI Flow Rate and Shear Stress Variations

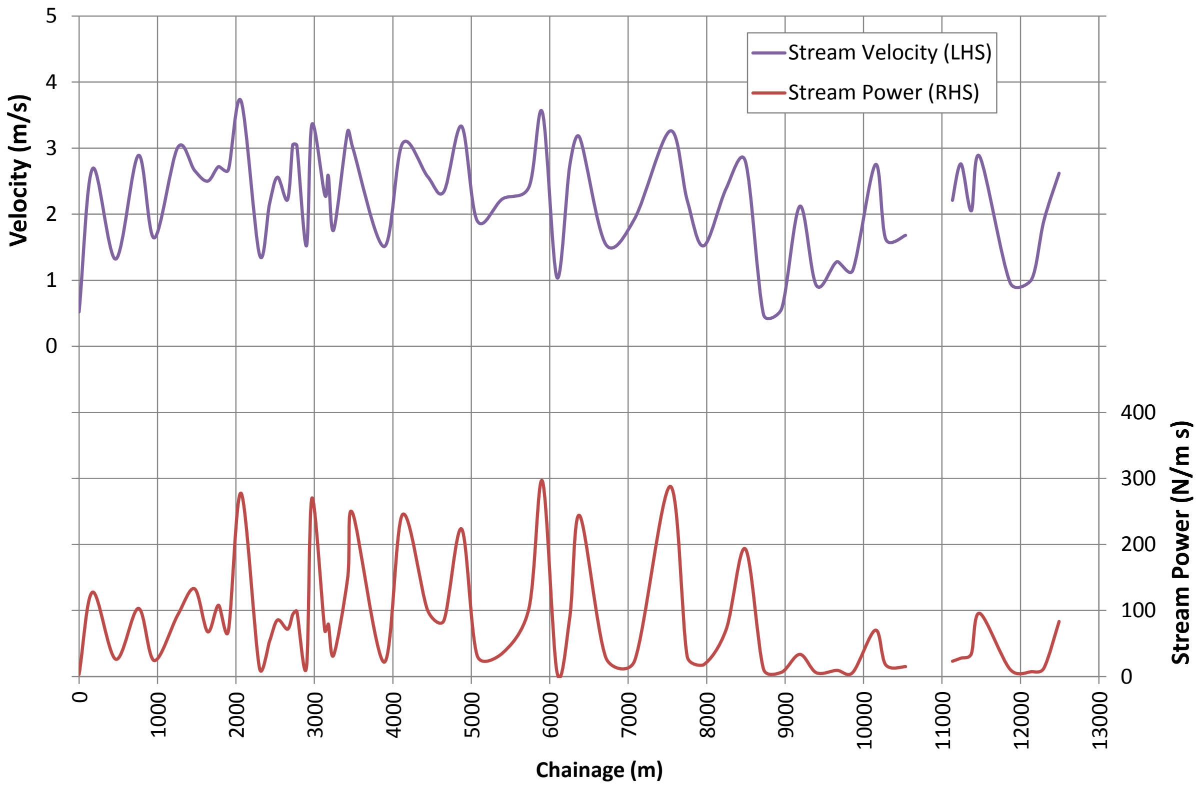

Figure 4-7 Dead Horse Creek Existing Conditions, 50 Year ARI Velocity and Stream Power Variations

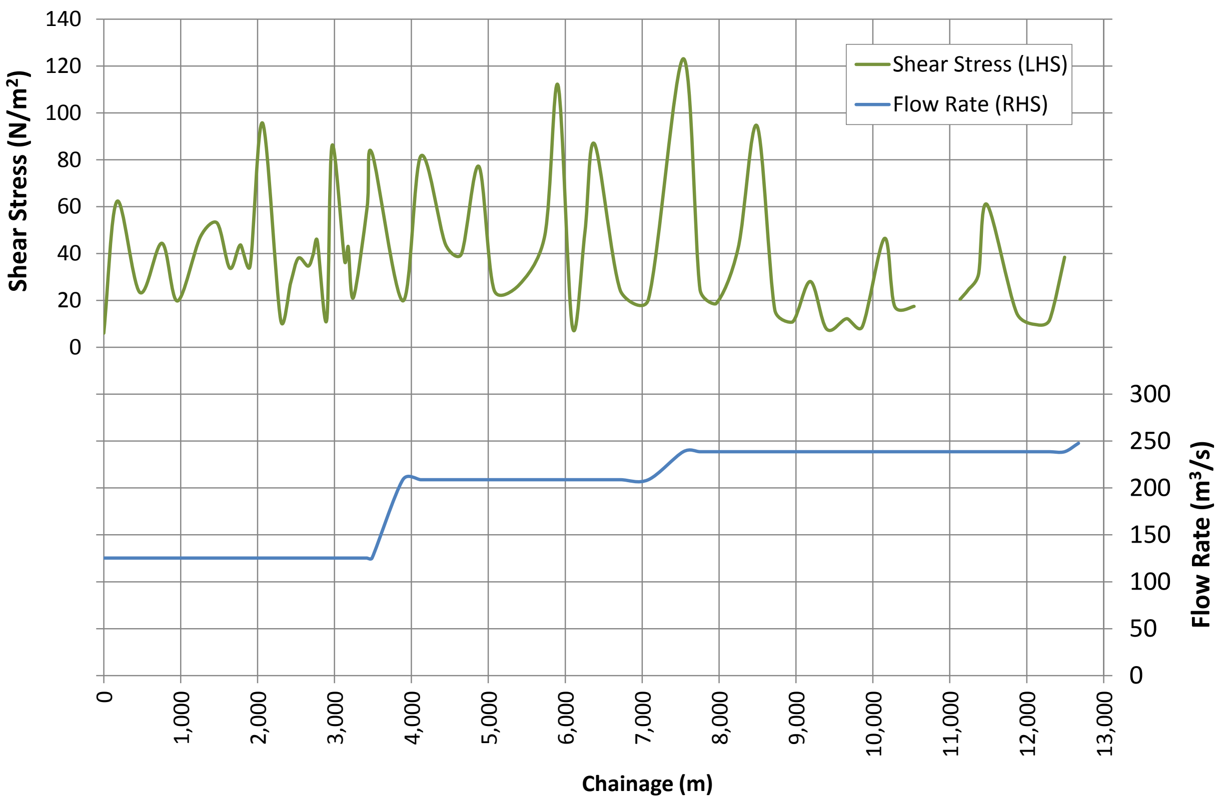

Figure 4-8 Dead Horse Creek Existing Conditions, 50 Year ARI Flow Rate and Shear Stress Variations

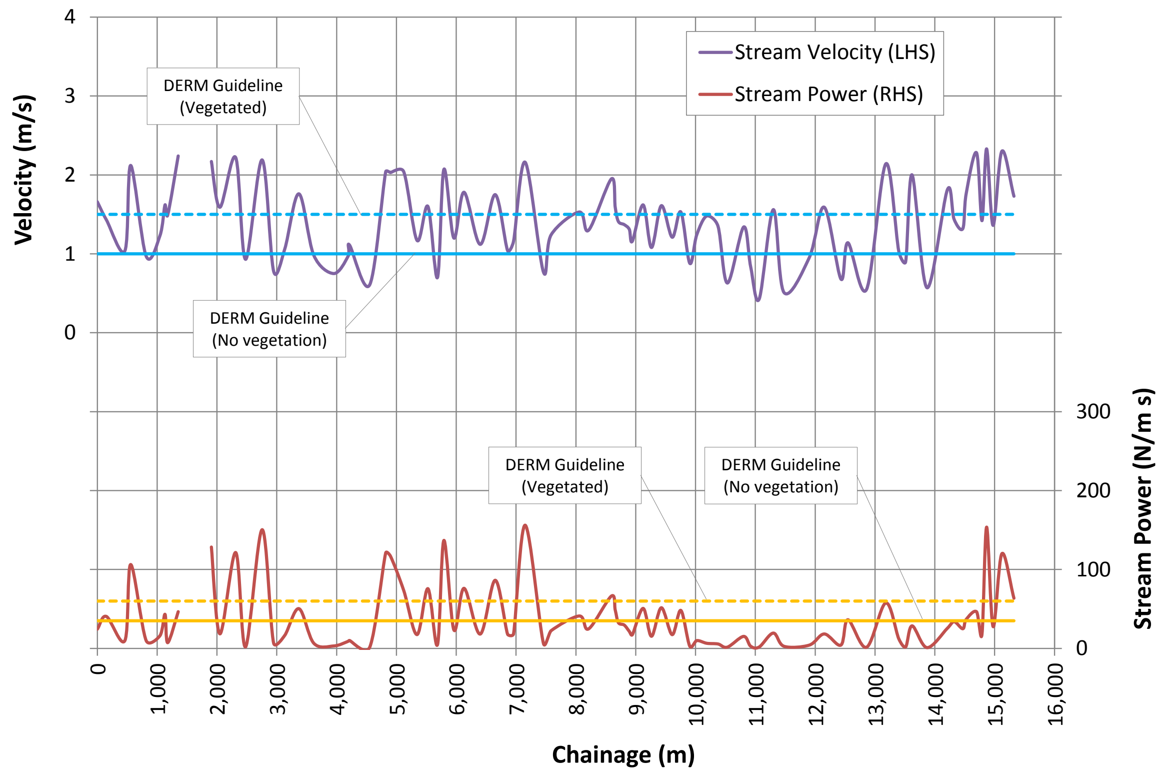

Figure 4-9 Sapling Creek Post Development, 2 Year ARI Velocity and Stream Power Variations

Figure 4-10 Sapling Creek Post Development, 2 Year ARI Flow Rate and Shear Stress Variations

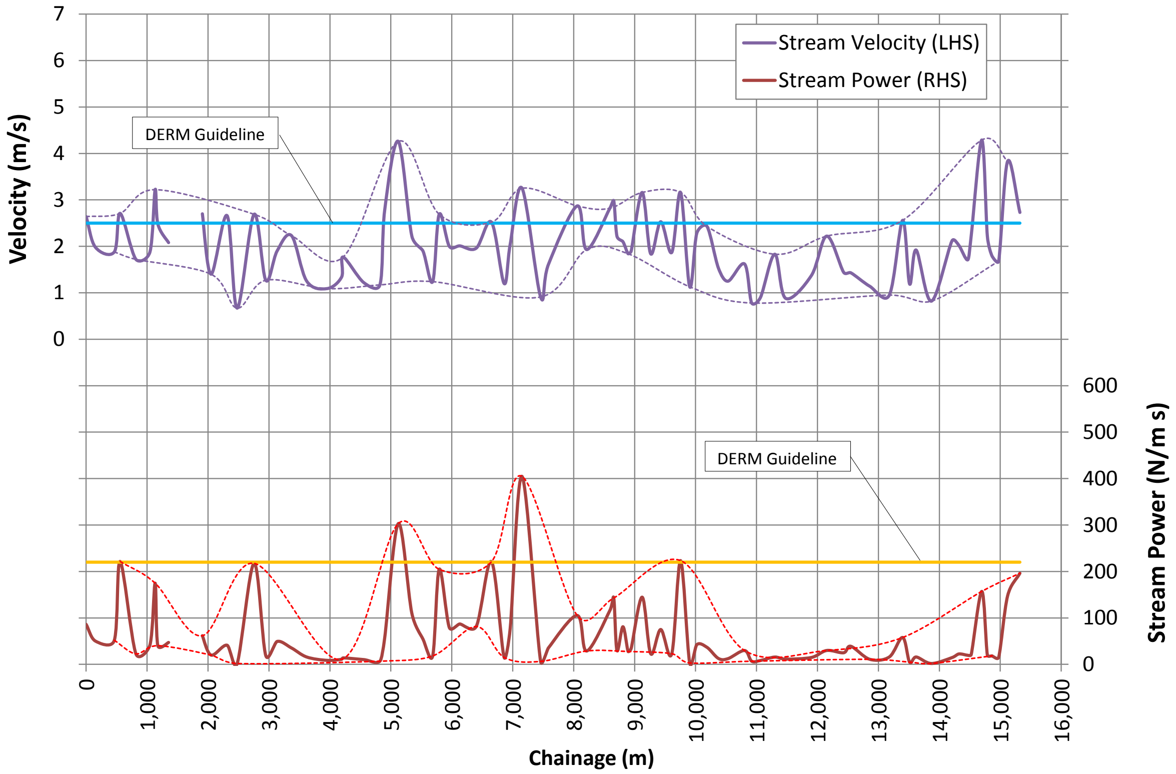

Figure 4-11 Sapling Creek Post Development, 50 Year ARI Velocity and Stream Power Variations

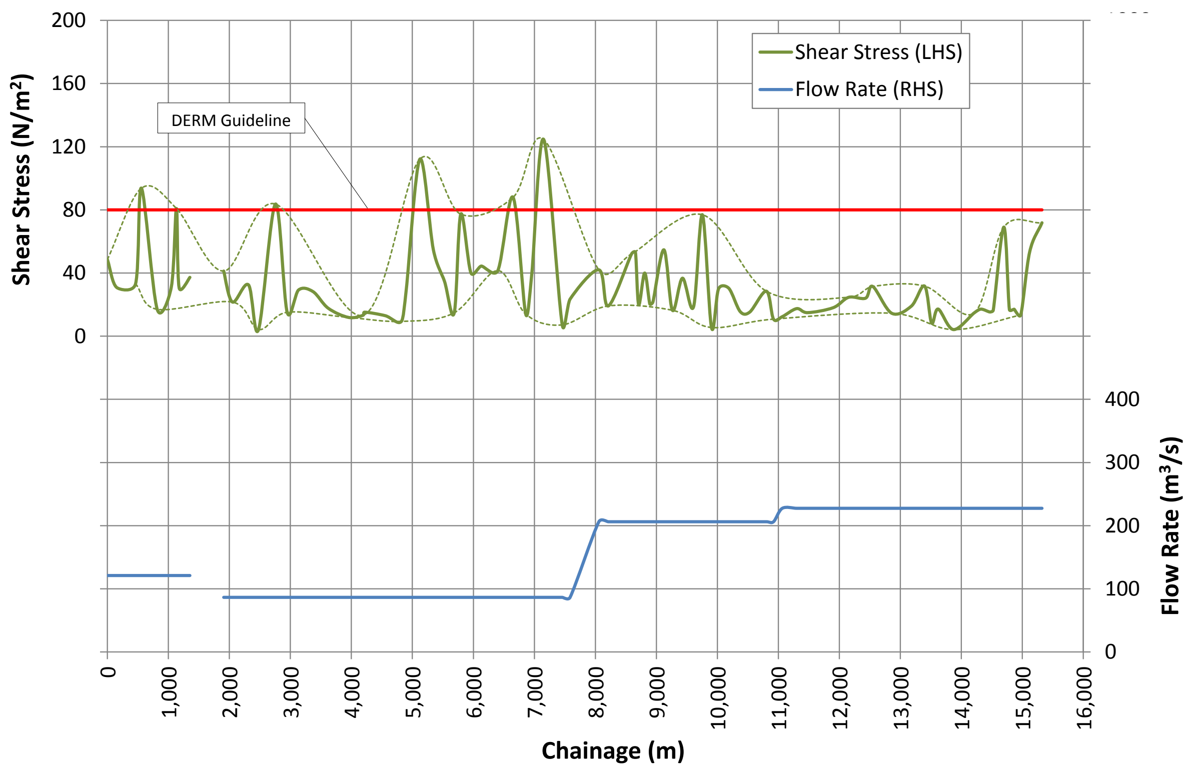

Figure 4-12 Sapling Creek Post Development, 50 Year ARI Flow Rate and Shear Stress Variations

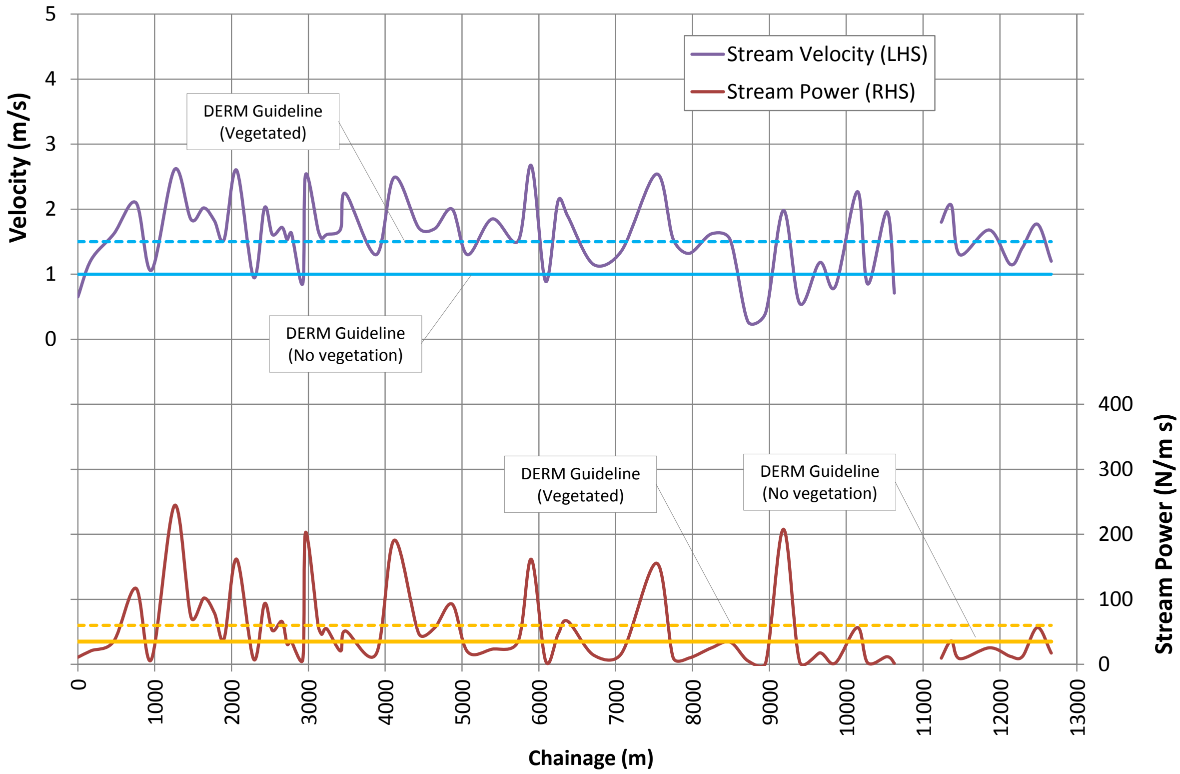

Figure 4-13 Dead Horse Creek Post Development, 2 Year ARI Velocity and Stream Power Variations

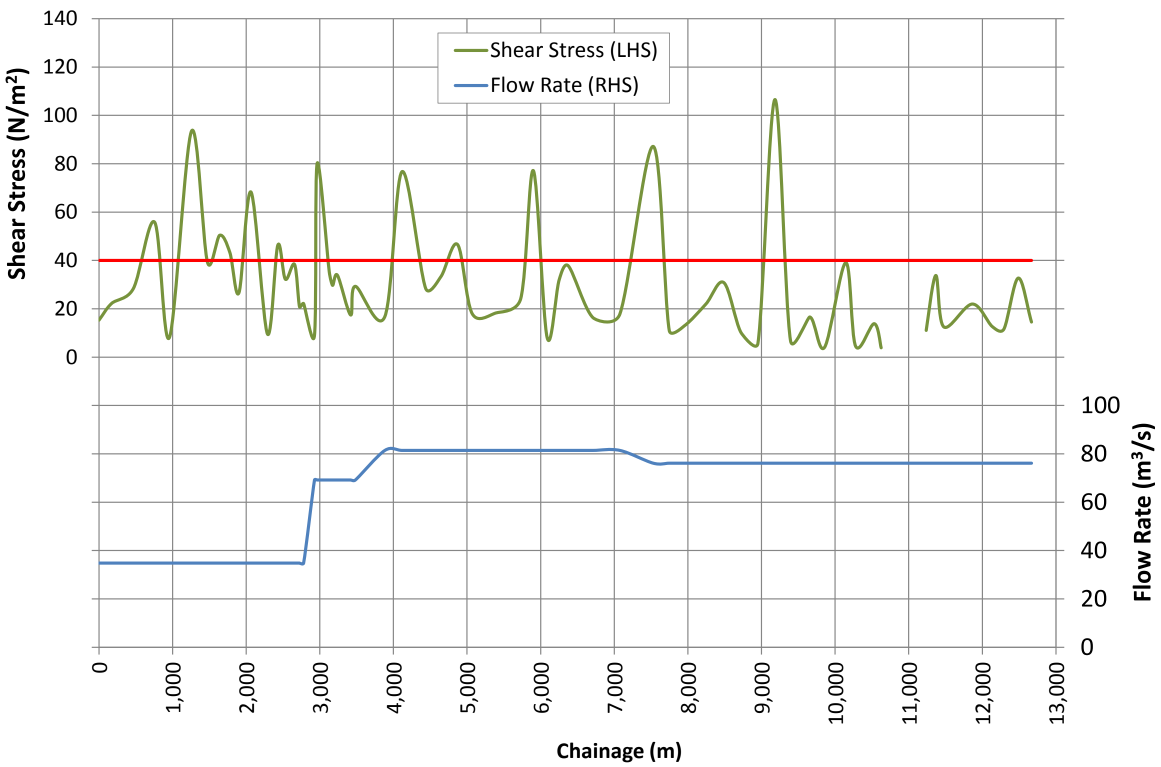

Figure 4-14 Dead Horse Creek Post Development, 2 Year ARI Flow Rate and Shear Stress Variations

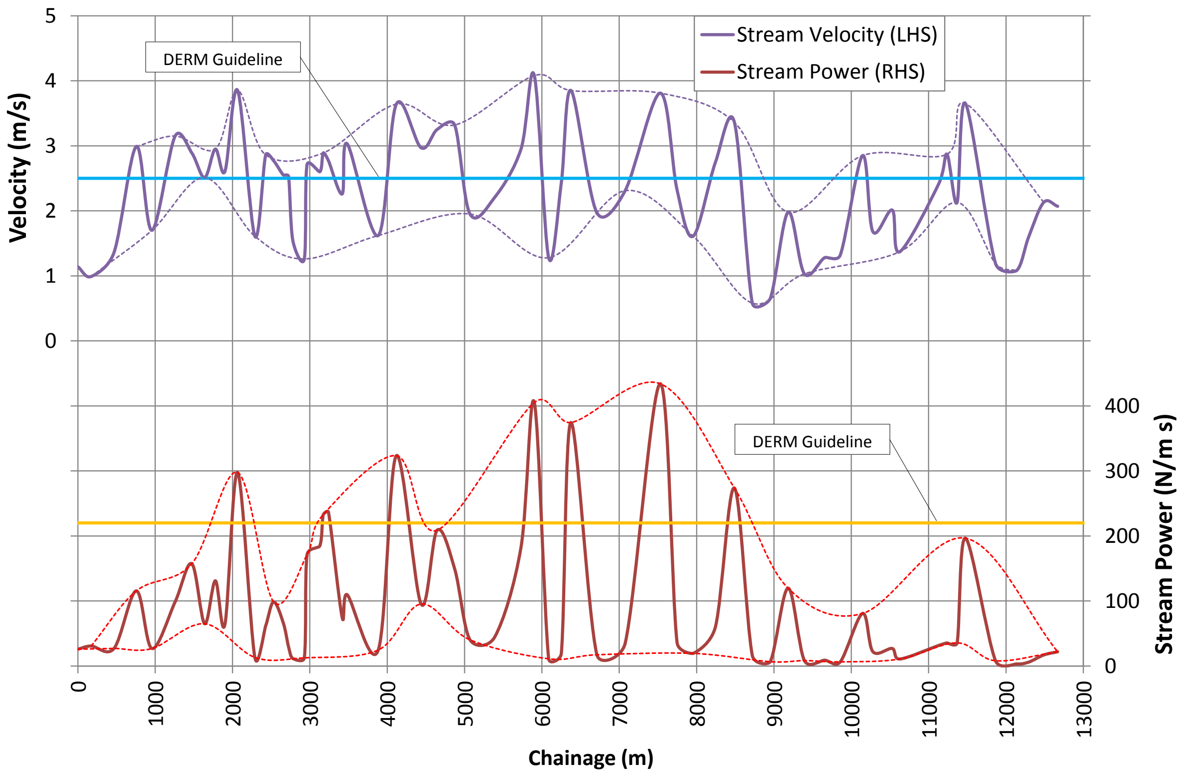

Figure 4-15 Dead Horse Creek Post Development, 50 Year ARI Velocity and Stream Power Variations

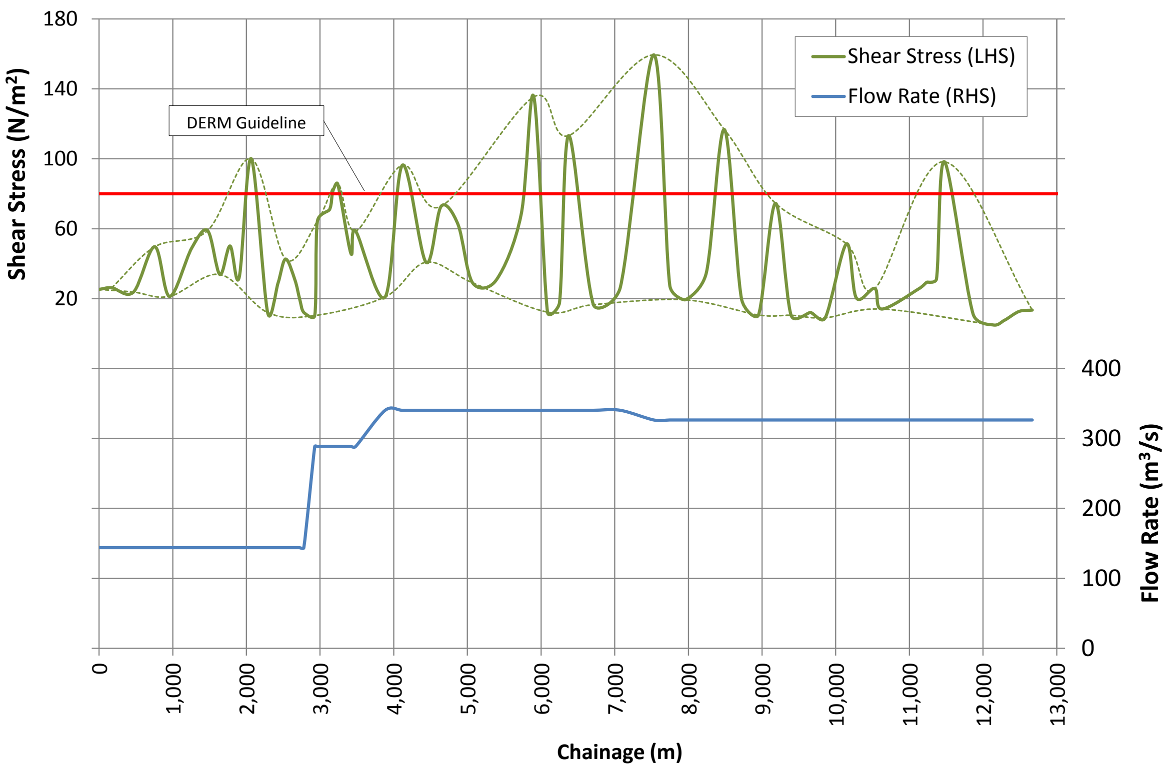

Figure 4-16 Dead Horse Creek Post Development, 50 Year ARI Flow Rate and Shear Stress Variations

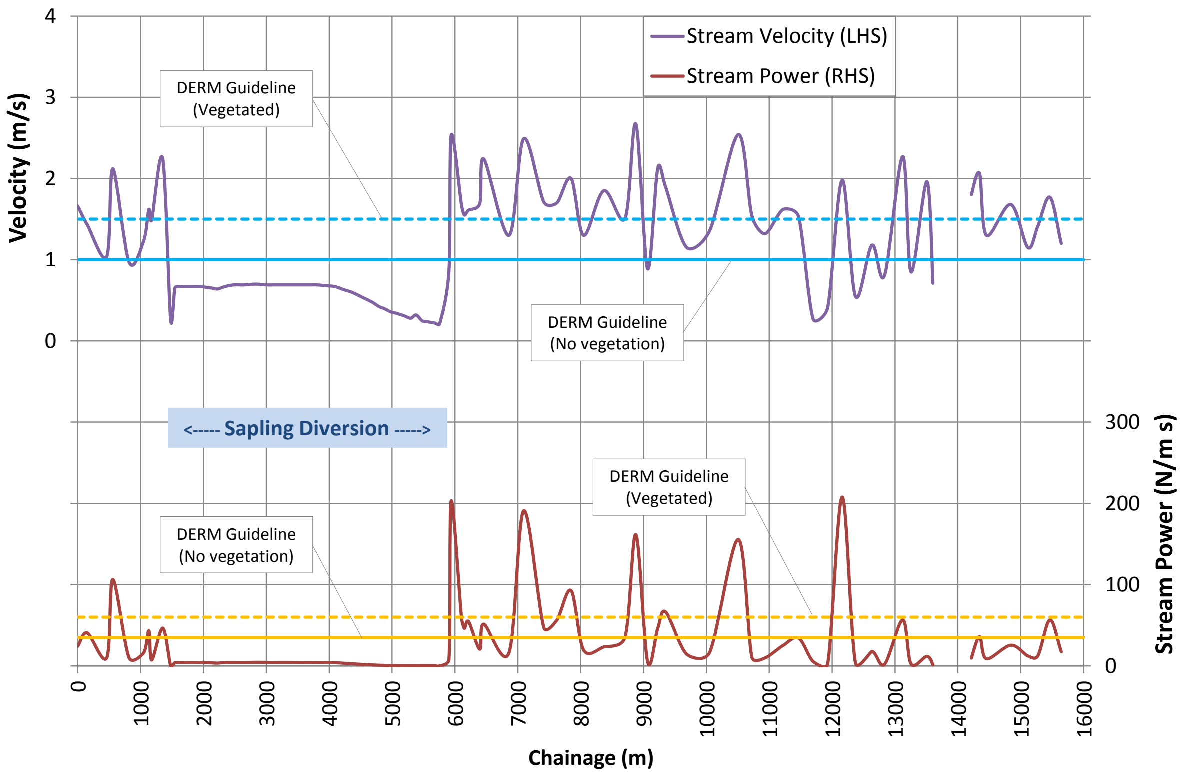

Figure 4-17 Sapling Creek-Dead Horse Creek Post Development, 2 Year ARI Velocity and Stream Power Variations

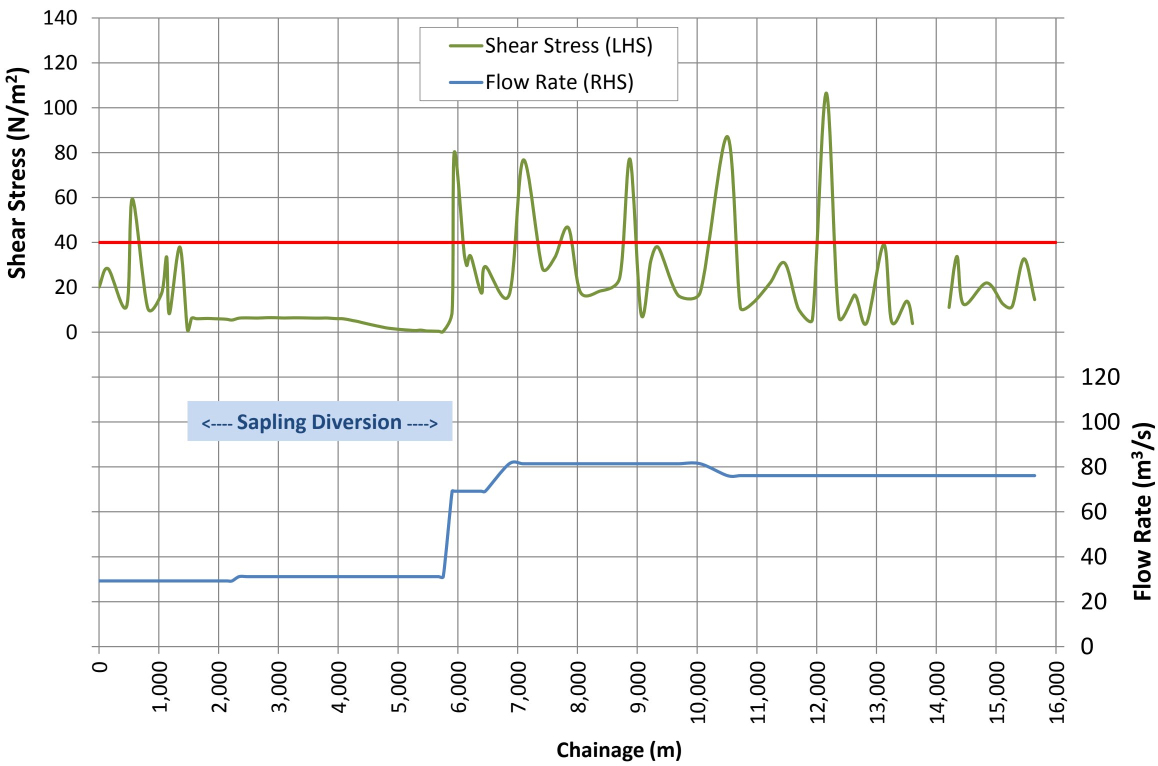

Figure 4-18 Sapling Creek-Dead Horse Creek Post Development, 2 Year ARI Flow Rate and Shear Stress Variations

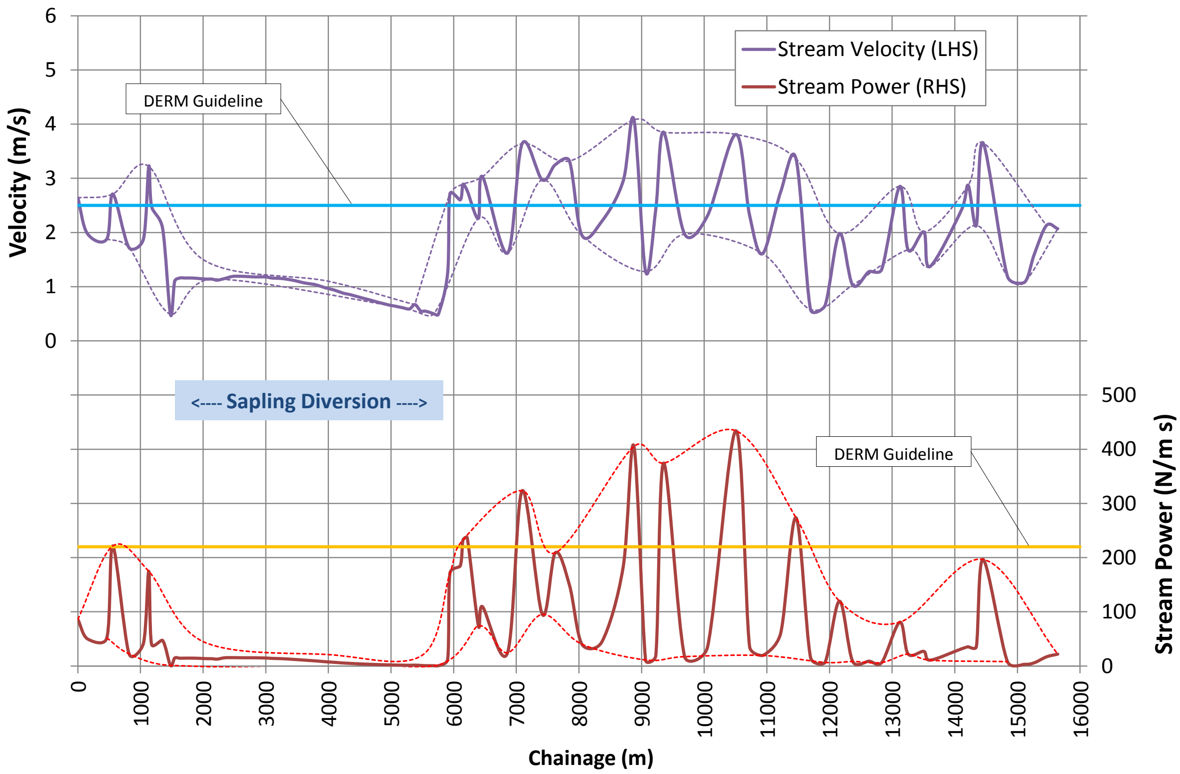

Figure 4-19 Sapling Creek-Dead Horse Creek Post Development, 50 Year ARI Velocity and Stream Power Variations

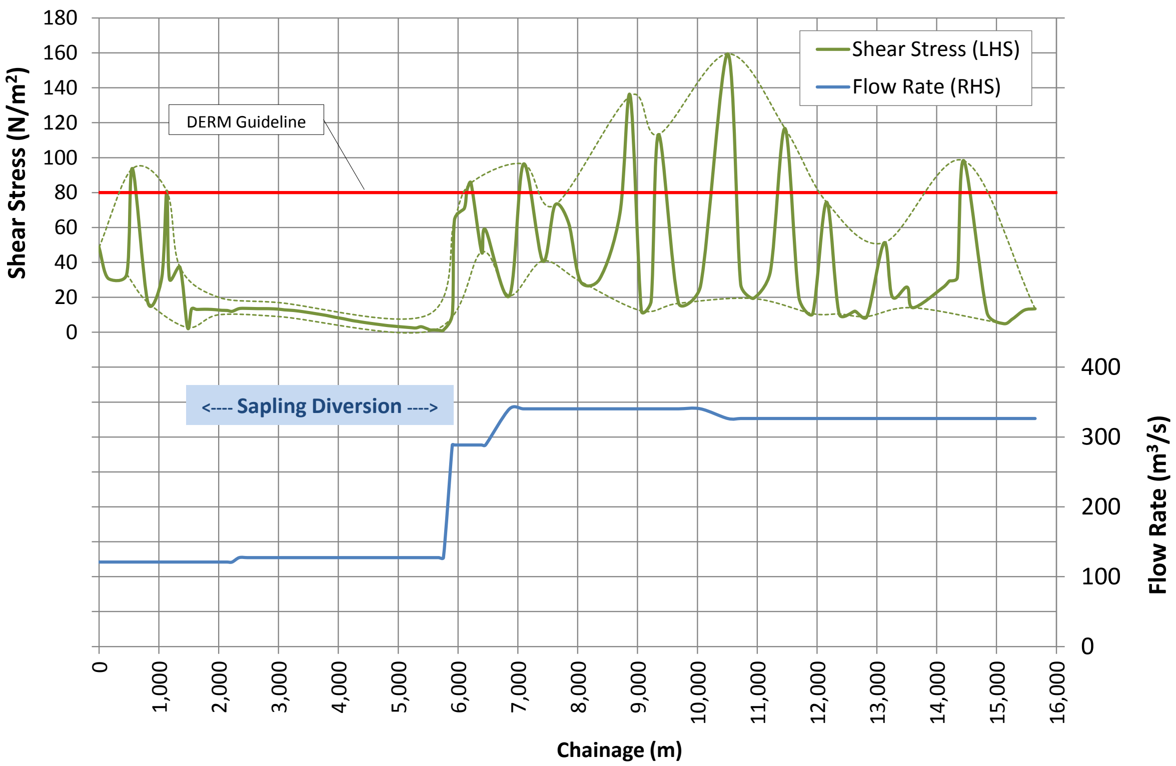

Figure 4-20 Sapling Creek-Dead Horse Creek Post Development, 50 Year ARI Flow Rate and Shear Stress Variations

4.3 INFRASTRUCTURE CORRIDOR

Plans showing the flood depth, extent, velocity and affluxes for the existing and post-developed infrastructure Corridor models are presented in Appendix E. Changes in water level and velocity do not propagate a significant distance upstream of the Infrastructure Corridor. Inflow and downstream boundary locations remain the same as in the existing conditions model outlined in Section 3.2.3.

An analysis of the model results for the existing and post-developed scenario indicates that the presence of the proposed embankment in the Infrastructure Corridor catchment results in little to no increase in the duration of inundation in the 2 year ARI, 50 year ARI and 100 year ARI flood events. Under the currently proposed arrangement, in the 10 year ARI and 20 year ARI events, some areas will become inundated at flows less than would have previously been the case, however, during detailed design it is likely than refinements to the cross-drainage arrangement could further mitigate these impacts.

4.3.1 Sensitivity Analysis

As outlined in Section 3.2.5, a sensitivity analysis of the tailwater conditions was undertaken to determine what impact the adopted constant water level would have on water levels upstream of the downstream model boundary. Four (4) additional 50 Year ARI post-developed model simulations were undertaken with varying downstream boundary conditions ranging from a level of 320 mAHD up to a level of 330 mAHD which represents the flood mapping extent generated by the Queensland Reconstruction Authority for Native Companion Creek. The results of this sensitivity analysis are shown in Figure 4-21 which indicates that a change in water level at the downstream boundary does not propagate a significant distance upstream, and therefore does not affect the impact assessment. However, during detailed design, consideration of the potential localised effect of Native Companion Creek flooding on design rail levels will need to be made.

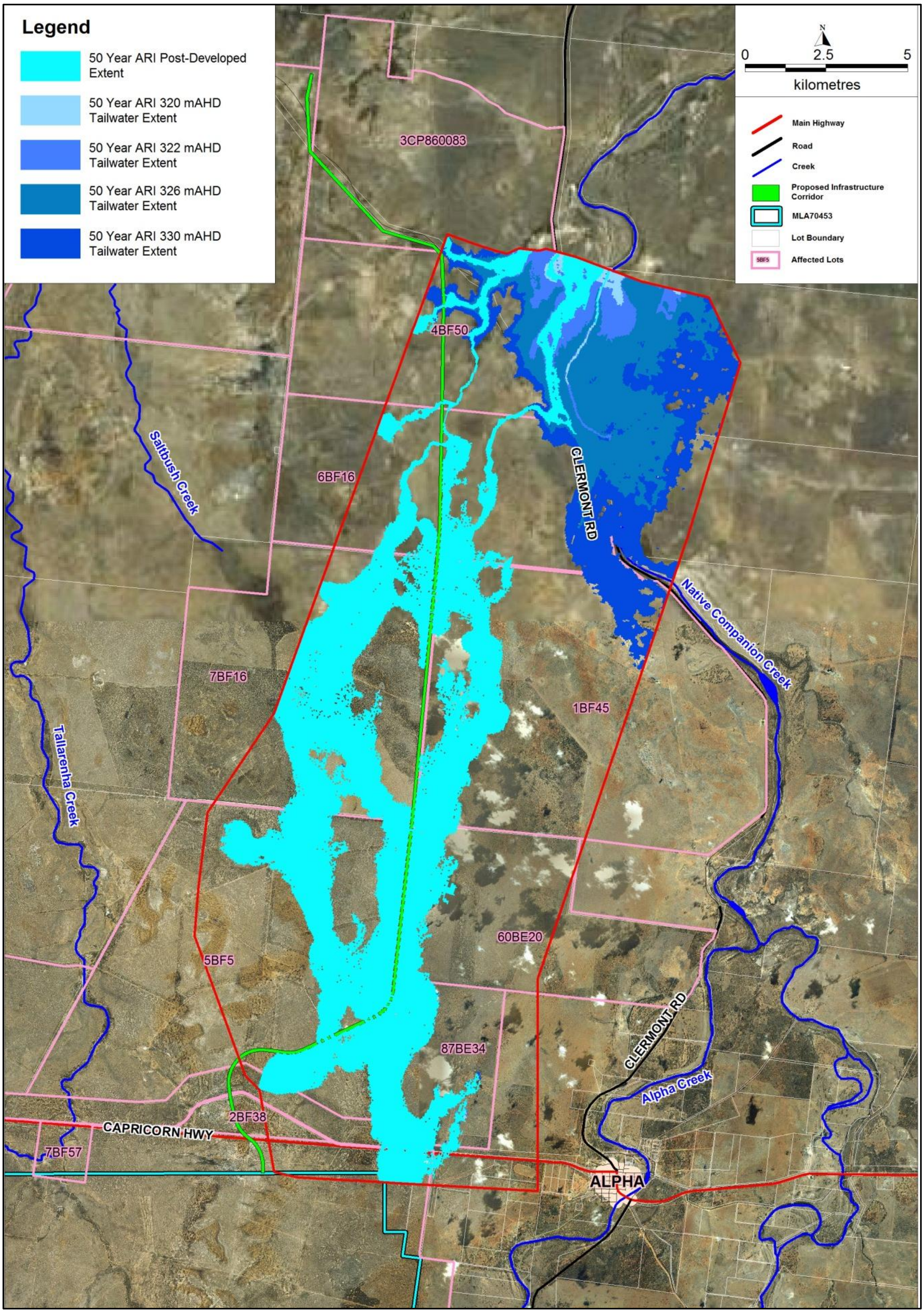

Figure 4-21 Downstream Boundary Sensitivity Analysis

4.4 PROPERTY-SPECIFIC IMPACT ASSESSMENT

The changes in flood conditions induced by the proposed project are summarised in the following tables. The tables show the maximum changes at the various landholdings which are affected, as well as the changes at the identified homesteads. These locations are shown in Figure 4-22.

Table 4-1 Maximum Change in Depth of Flooding (m)

Lot ID	2 Year ARI	10 Year ARI	50 Year ARI	100 Year ARI
2SP136836 [a]		0.00	0.00	
3CP860083 [a]		0.00	0.00	
4BF50 [a]		0.05	0.11	
6BF16 [a]		0.42	0.48	
7BF16 [a]		0.29	0.32	
1BF45 [a]		0.24	0.24	
60BE20 [a]		0.24	0.25	
87BE34 [a]		0.12	0.17	
5BF5 [ab]	0.72	0.27	1.46	1.64
3BF6 [b]	0.00	-	-0.34	-0.87
31BF11 [b]	1.52	-	1.62	1.56
4315PH720 [b]	2.49	-	3.03	3.23
7BF57 [b]	2.20	-	3.76	3.96
301SP108315 [b]	-	-	0.00	-
2BF38 [b]	-	-	0.00	-
1160PH286 [cd]	0.68	-	0.98	1.07
3BF53 [c]	0.98	-	1.46	1.65
1DM3 [d]	2.04	-	3.33	3.52

[a] - Infrastructure Corridor Model [b] - Tallarenha Ck Model [c] - Sapling Ck Model [d] - Dead Horse Creek Model

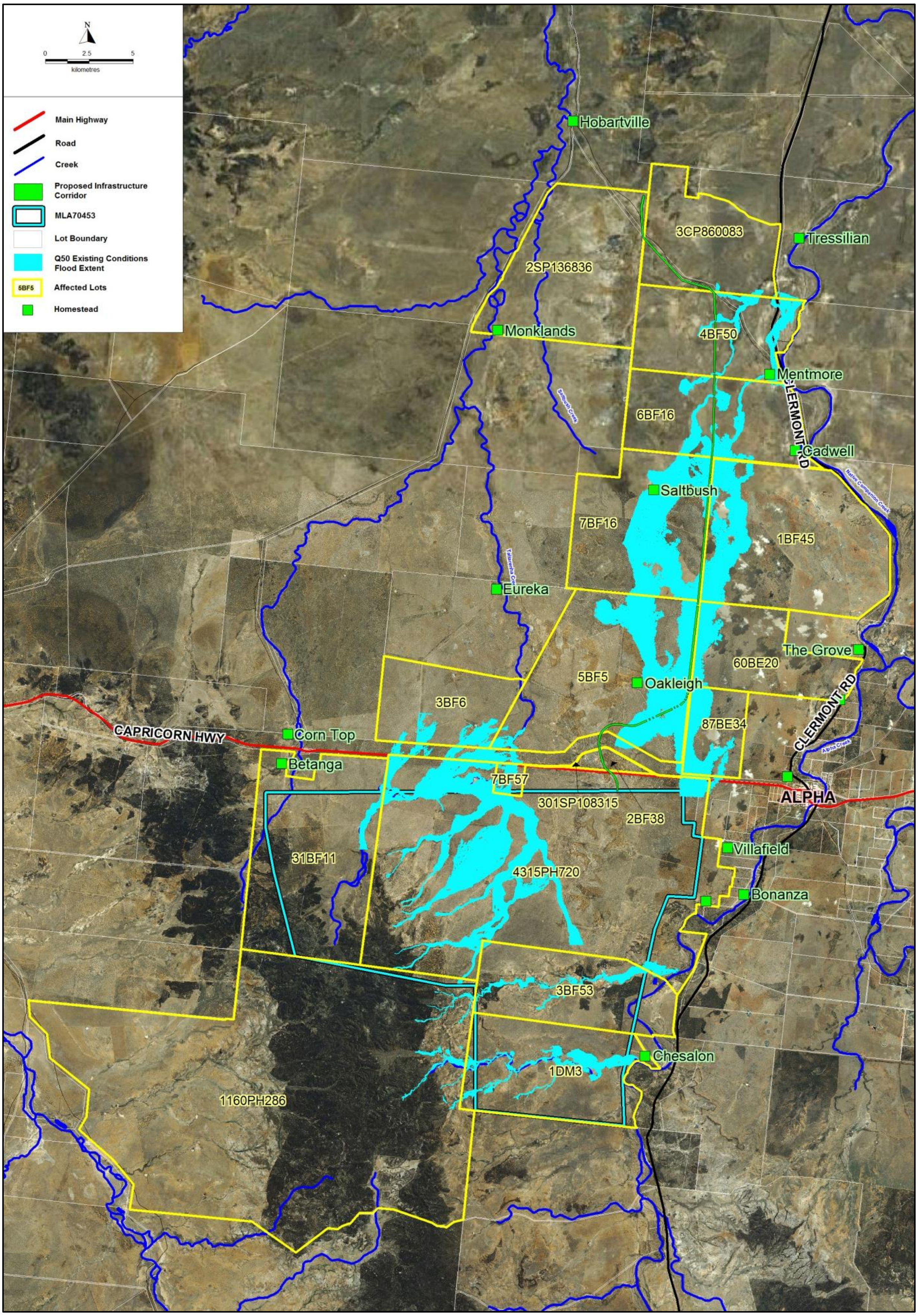

Figure 4-22 Affected Property Lots and Homesteads

Table 4-2 Maximum Increase in Depth of Flooding (Homesteads) (m)

Homestead	2 Year ARI	10 Year ARI	50 Year ARI	100 Year ARI
Hobartville		0	0	
Tressilian		0	0	
Monklands		0	0	
Mentmore		0	0	
Cadwell		0	0	
Saltbush		-0.003	0.002	
Eureka		0	0	
The Grove		0	0	
Oakleigh		0	0	
Corn Top	0	-	-	0
Betanga	0	-	-	0
Villafield	0	-	-	0
Bonanza	0	-	-	0
Creek Farm	0	-	-	0
Chesalon	0	-	-	0
Bedford	0	-	-	0

Table 4-3 Increase in Area of Inundation (km^2) – 2 year ARI

Lot ID	Existing	Post Development	Change
5BF5 [ab]	0.11a	0.10a	-0.01a
3BF6 [b]	-	-	-
31BF11 [b]	0.86	0.85	-0.01
4315PH720 [b]	21.80	7.51	-14.29
7BF57 [b]	0.41	0.41	0.00
301SP108315 [b]	-	-	-
2BF38 [b]	-	-	-
1160PH286 [cd]	0.66	0.70	0.04
3BF53 [c]	1.48	1.47	0.00
1DM3 [d]	2.64	2.92	0.28

[a b] - Tallarenha Ck Model [c] - Sapling Ck Model [d] - Dead Horse Creek Model

Table 4-4 Increase in Area of Inundation (km^2) – 10 year ARI Event

Lot ID	Existing	Post Development	Change
2SP136836 [a]	-	-	-
3CP860083 [a]	0.23	0.23	0.00
4BF50 [a]	4.33	4.33	-0.01
6BF16 [a]	5.11	5.40	0.30
7BF16 [a]	17.03	17.23	0.20
1BF45 [a]	7.34	6.25	-1.09
60BE20 [a]	4.30	4.28	-0.02
87BE34 [a]	5.33	5.29	-0.03
5BF5 ab	20.00	19.11	-0.89
3BF6 [b]	-	-	-
31BF11 [b]	-	-	-
4315PH720 [b]	1.49[b]	1.49[b]	0.00[b]
7BF57 [b]	-	-	-
301SP108315 [b]	-	-	-
2BF38 [b]	-	-	-
1160PH286 [cd]	-	-	-
3BF53 [c]	-	-	-
1DM3 [d]	-	-	-

[a] - Infrastructure Corridor Model [b] - Tallarenha Ck Model [c] - Sapling Ck Model [d] - Dead Horse Creek Model

Table 4-5 Increase in Area of Inundation (km^2) – 50 year ARI Event

Lot ID	Existing	Post Development	Change
2SP136836 [a]	-	-	-
3CP860083 [a]	0.29	0.29	0.00
4BF50 [a]	5.12	5.10	-0.01
6BF16 [a]	6.53	6.83	0.30
7BF16 [a]	20.29	20.10	-0.19
1BF45 [a]	12.75	12.05	-0.70
60BE20 [a]	4.87	4.87	0.01
87BE34 [a]	6.76	6.72	-0.04
5BF5 ab	23.58c	21.79 c	-1.78 c
3BF6 [b]	1.95	0.00	-1.95
31BF11 [b]	1.27	1.22	-0.04
4315PH720 [b]	33.70	12.63	-21.07
7BF57 [b]	1.13	1.25	0.12
301SP108315 [b]	-	-	-
2BF38 [b]	-	-	-
1160PH286 [cd]	1.32	1.30	-0.02
3BF53 [c]	2.67	2.43	-0.24
1DM3 [d]	4.65	5.18	0.53

[a] - Infrastructure Corridor Model [b] - Tallarenha Ck Model [c] - Sapling Ck Model [d] - Dead Horse Creek Model

Table 4-6 Increase in Area of Inundation (km²) – 100 year ARI Event

Lot ID	Existing	Post Development	Change
5BF5 ab	1.29d	0.71d	-0.57d
3BF6 [b]	3.20	0.00	-3.20
31BF11 [b]	1.33	1.30	-0.03
4315PH720 [b]	36.56	14.17	-22.39
7BF57 [b]	1.34	1.41	0.08
301SP108315 [b]	-	-	-
2BF38 [b]	-	-	-
1160PH286 [cd]	1.40	1.49	0.09
3BF53 [c]	2.85	2.61	-0.24
1DM3 [d]	5.33	5.82	0.49

[a] - Tallarenha Ck Model [c] - Sapling Ck Model [d] - Dead Horse Creek Model

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E APPENDIX

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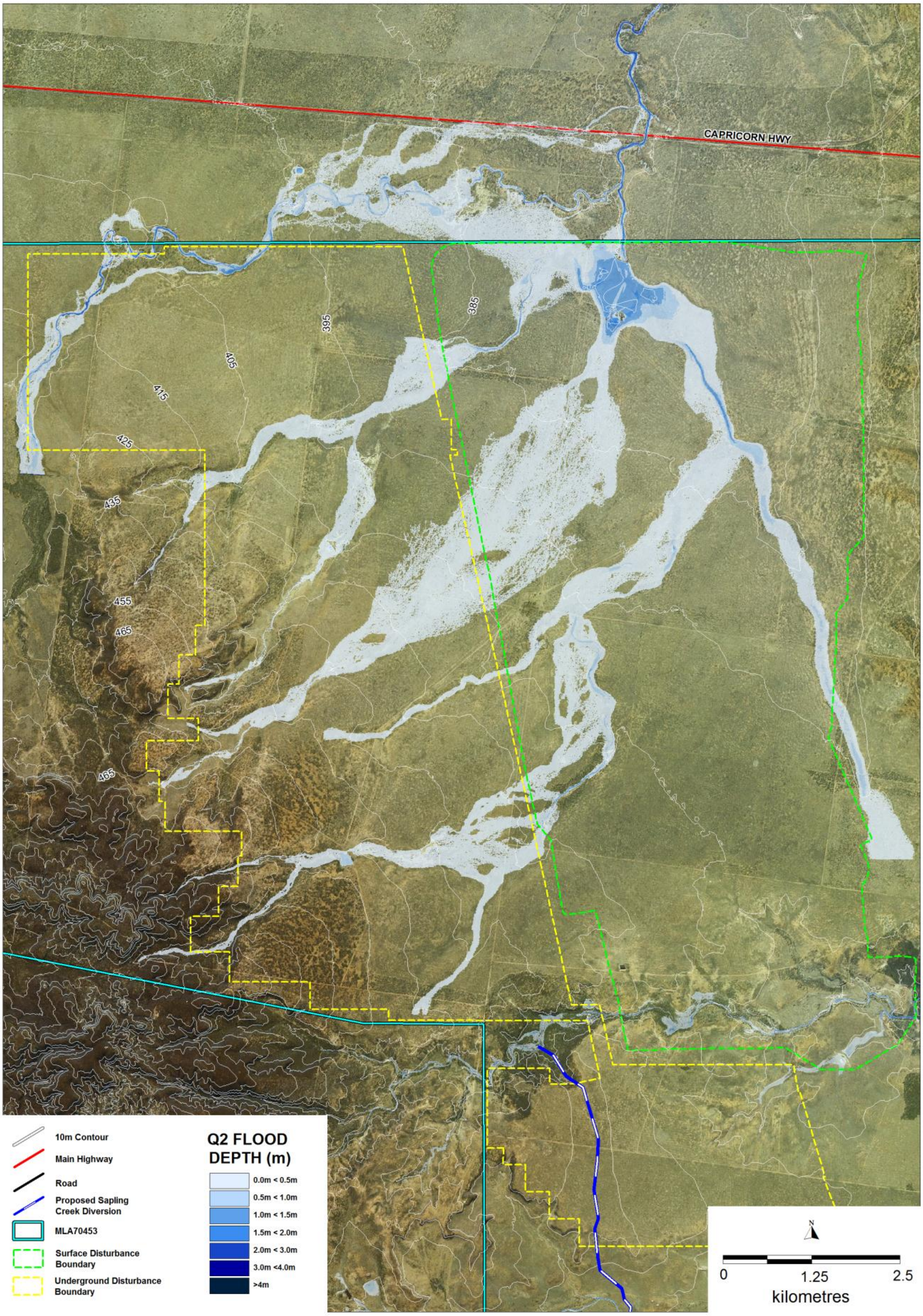

Figure 1.1 Tallarenha Creek – Existing Conditions - 2 Year ARI Flood Depth and Extents

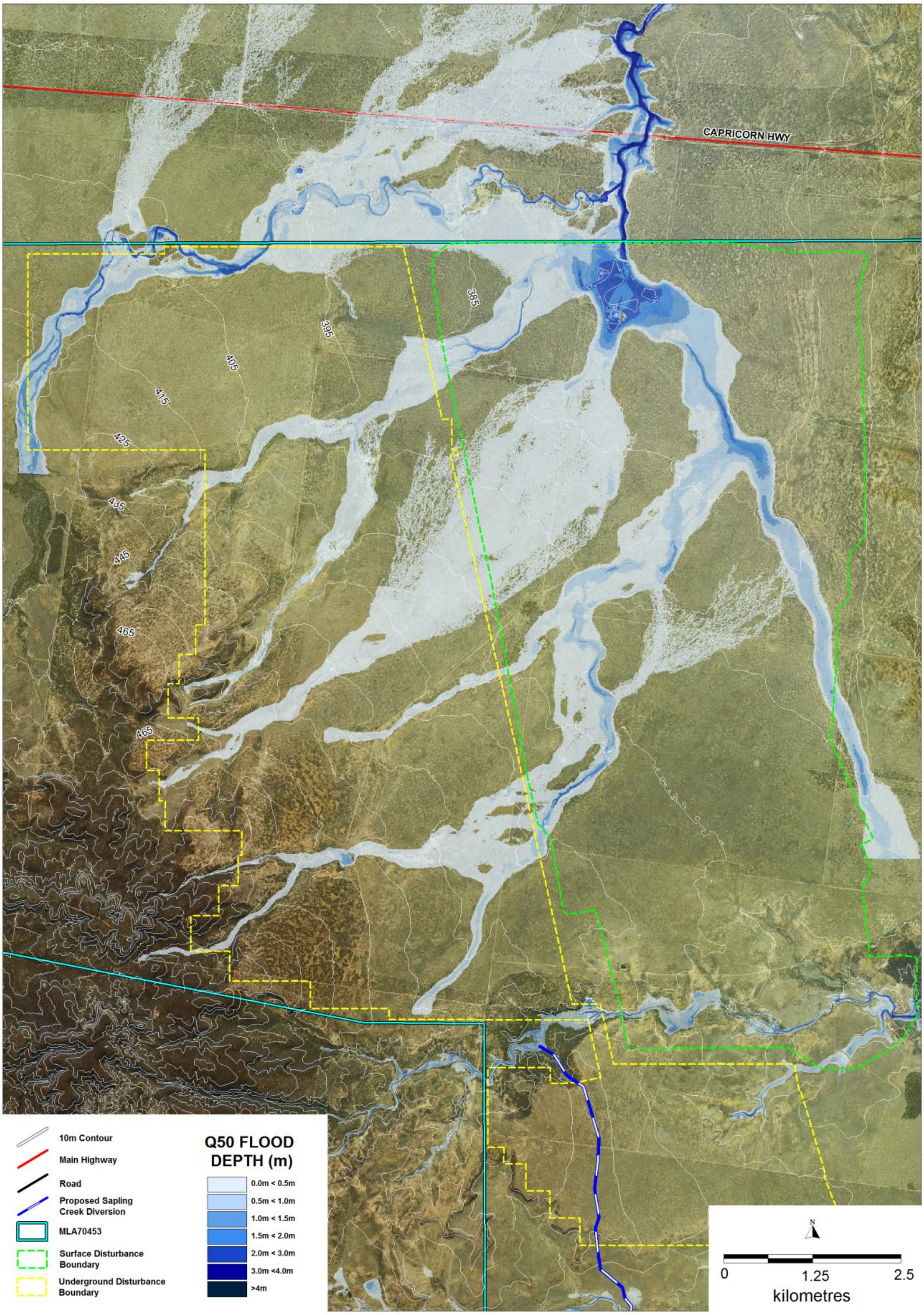

Figure 1.2 Tallarenha Creek – Existing Conditions - 50 Year ARI Flood Depth and Extents

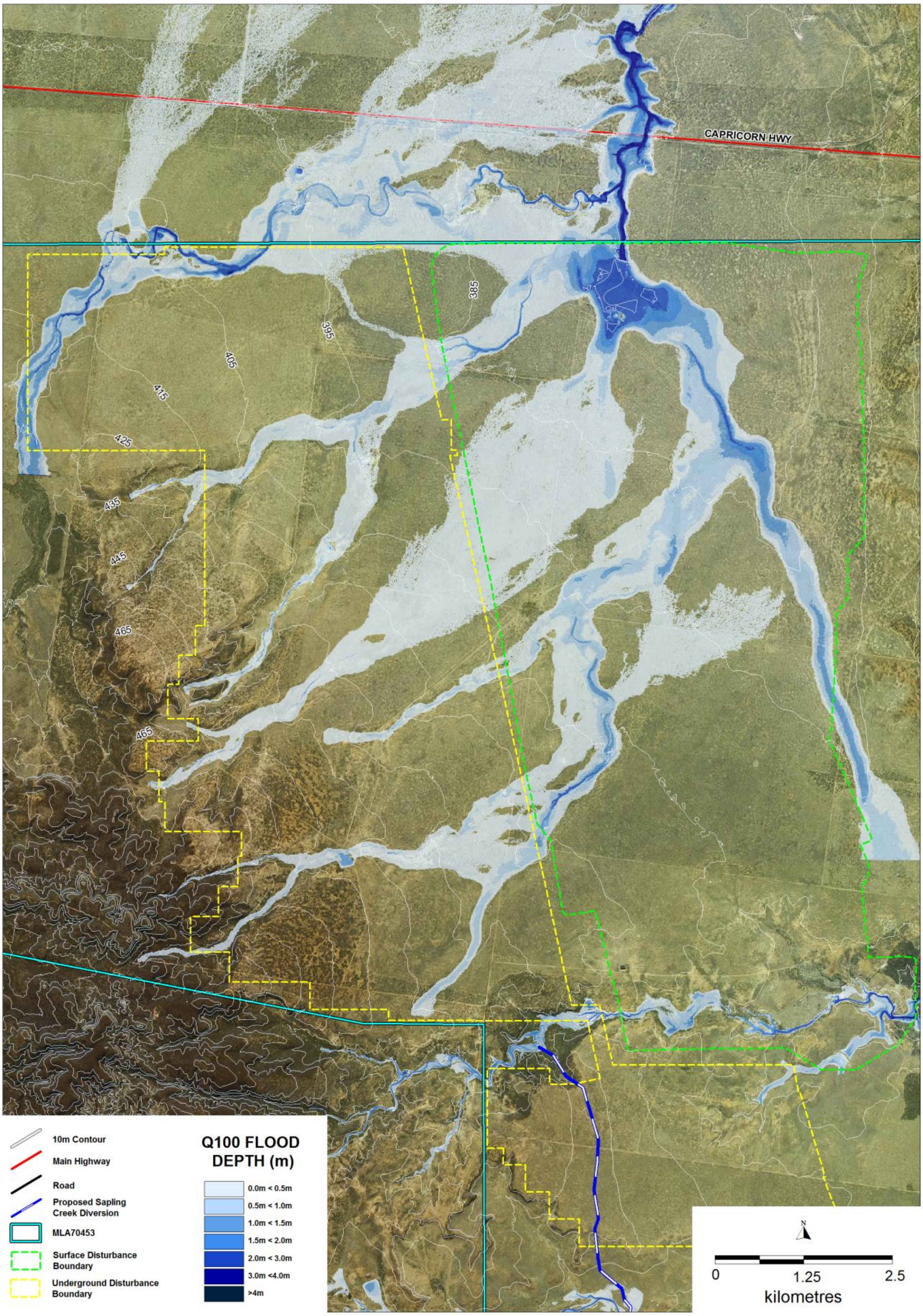

Figure 1.3 Tallarenha Creek – Existing Conditions - 100 Year ARI Flood Depth and Extents

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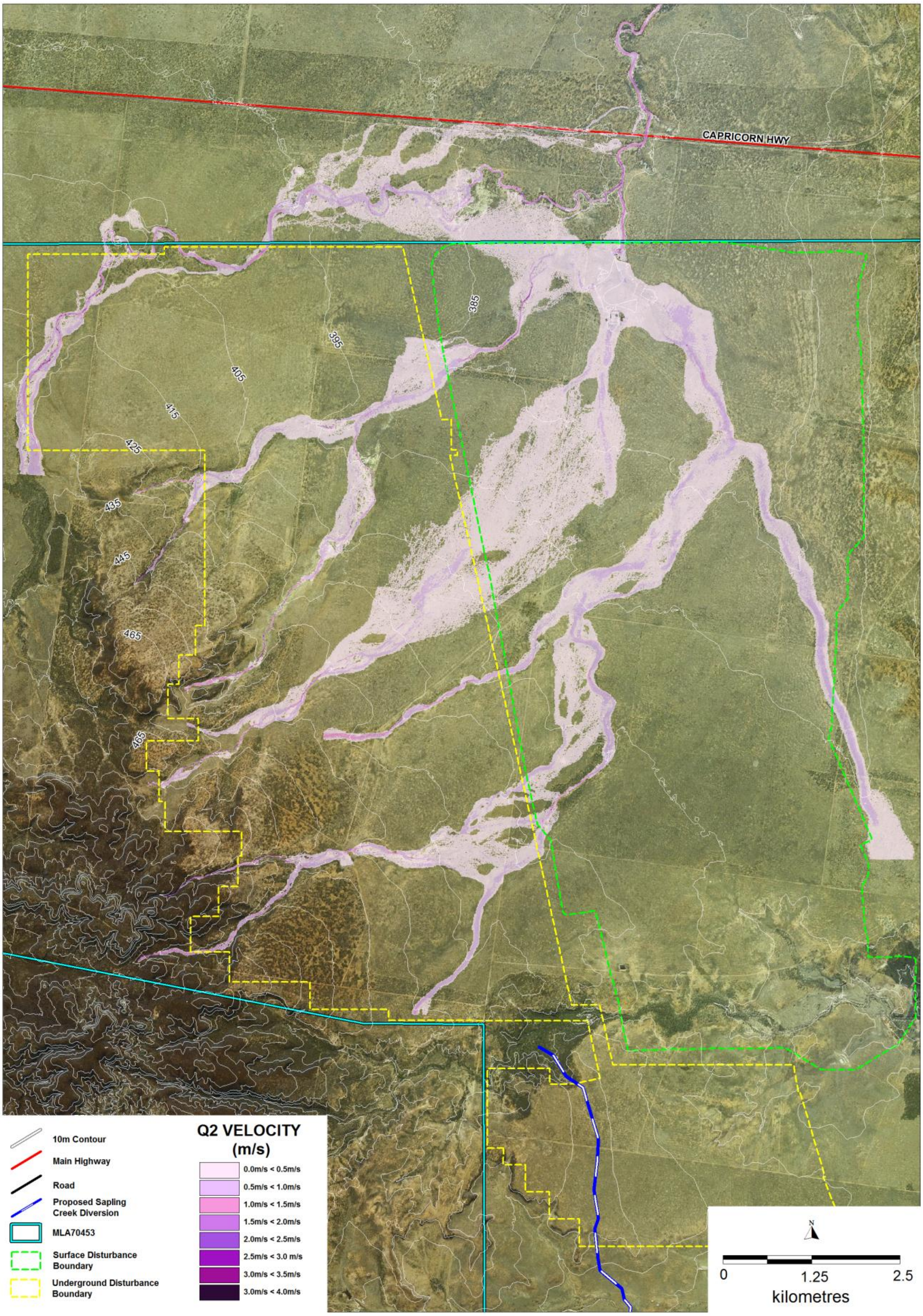
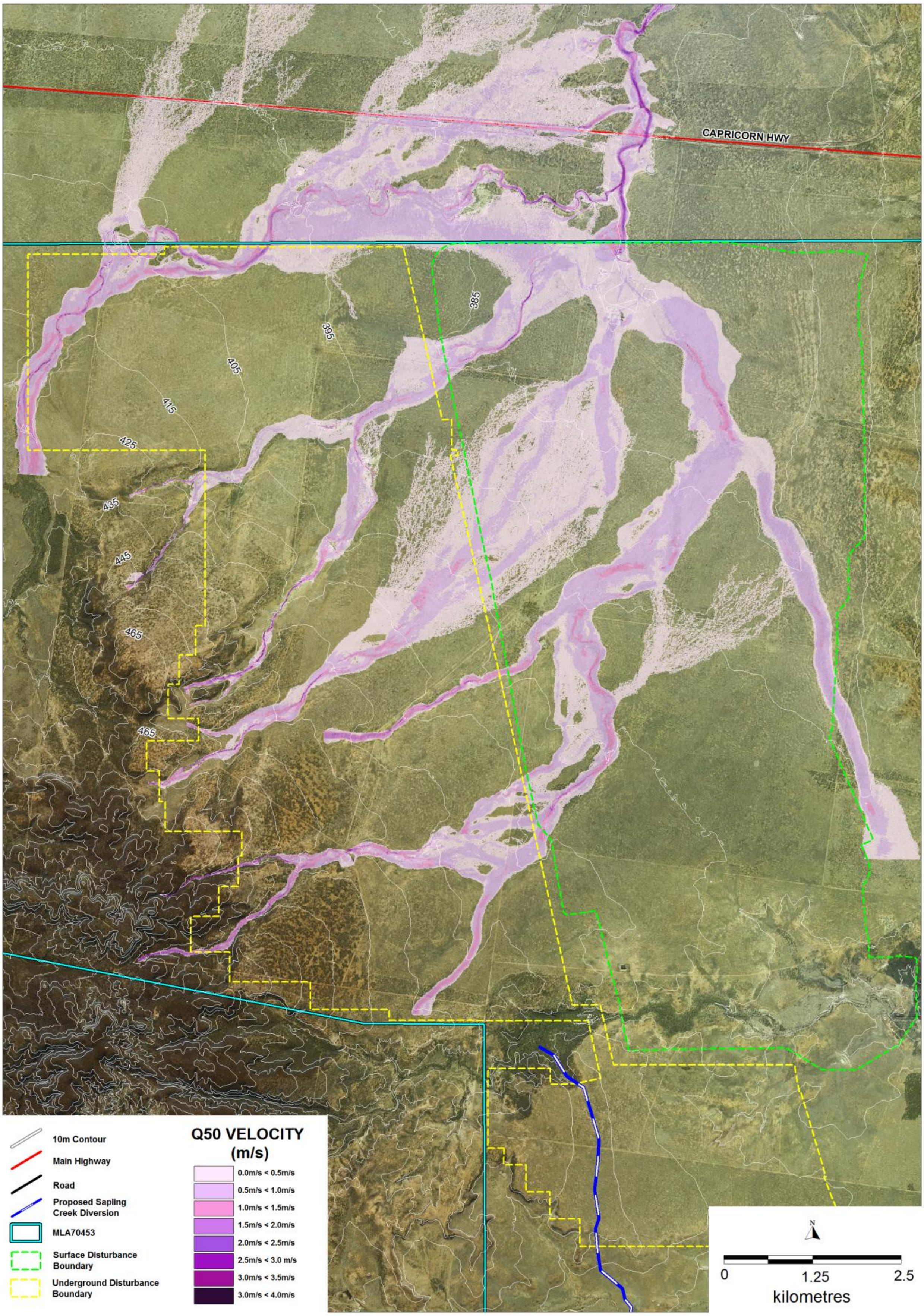


Figure 1.4 Tallarenha Creek – Existing Conditions - 2 Year ARI Flood Velocities



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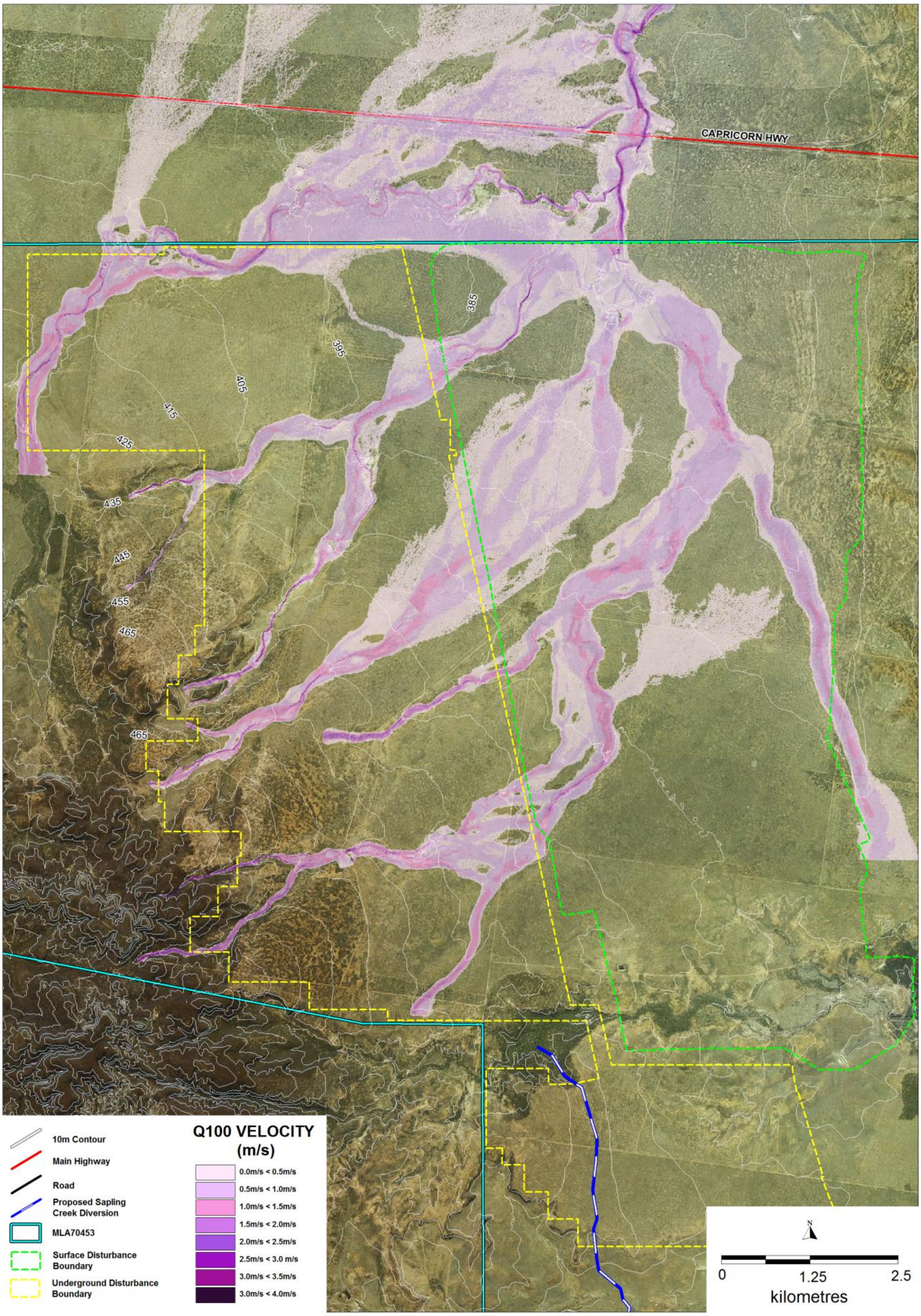

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1.3 POST DEVELOPMENT FLOOD DEPTHS AND EXTENTS

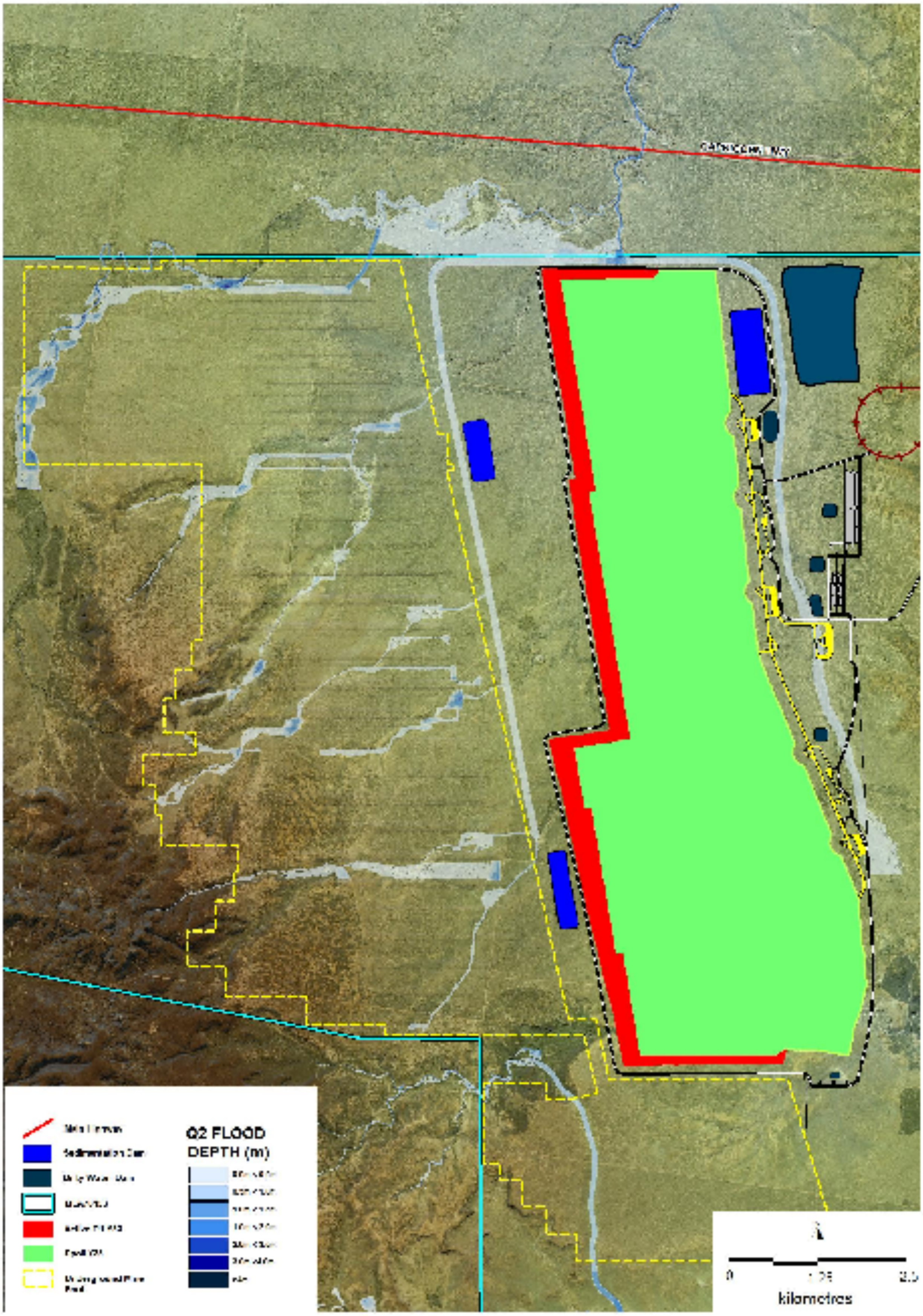


Figure 1.7 Tallarenha Creek – Post Development Conditions - 2 Year ARI Flood Depth and Extents

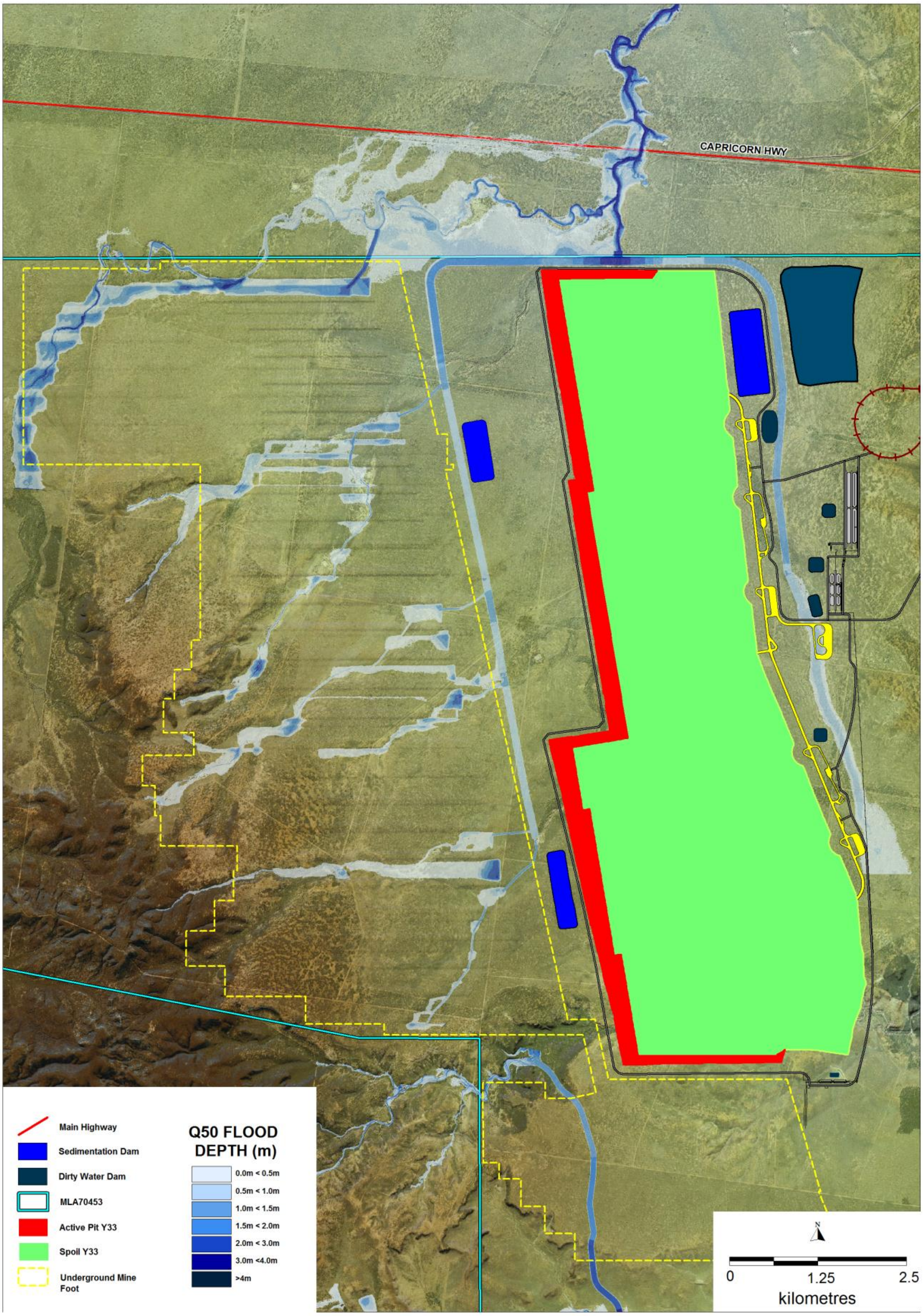
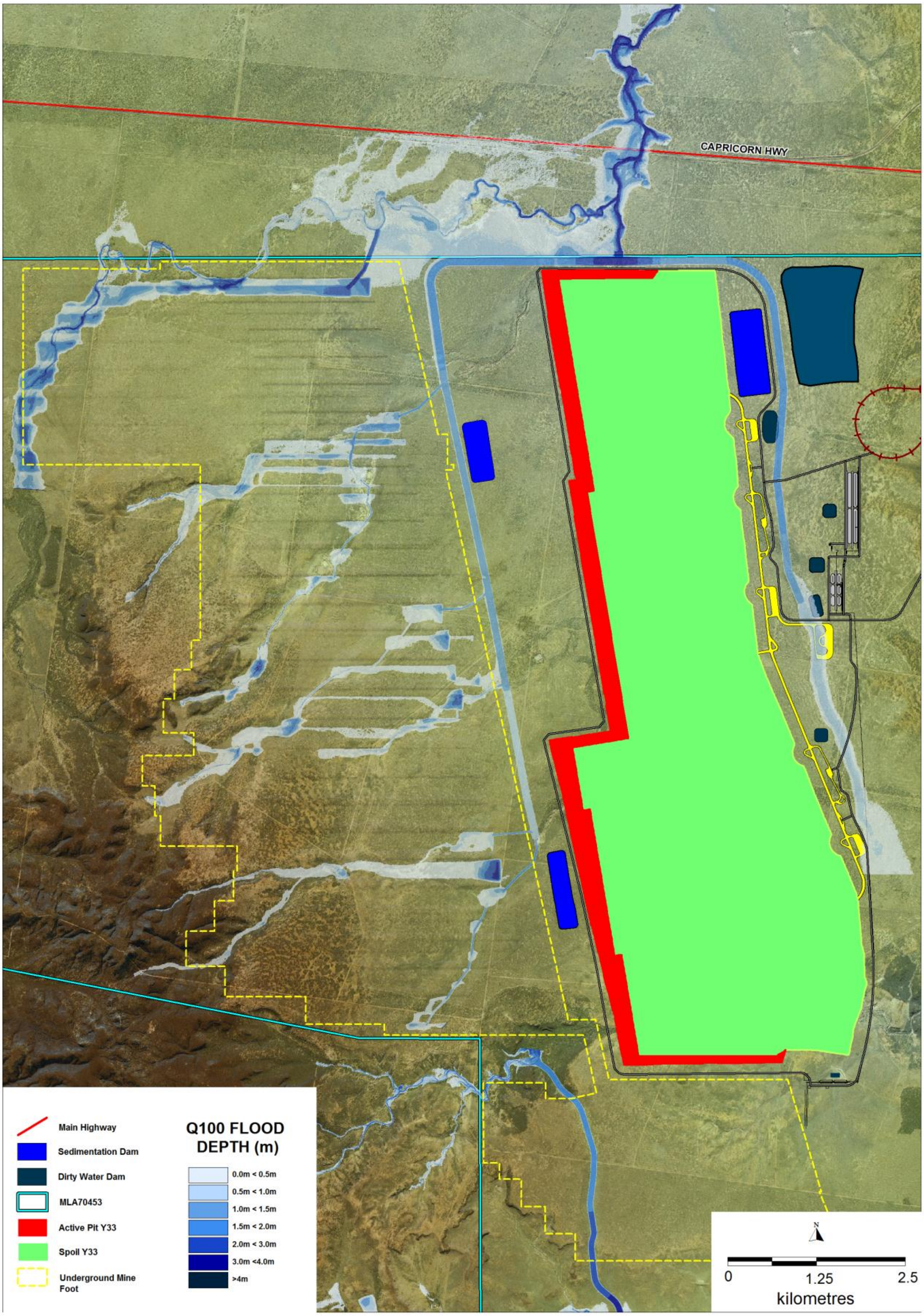


Figure 1.8 Tallarenha Creek – Post Development Conditions - 50 Year ARI Flood Depth and Extents



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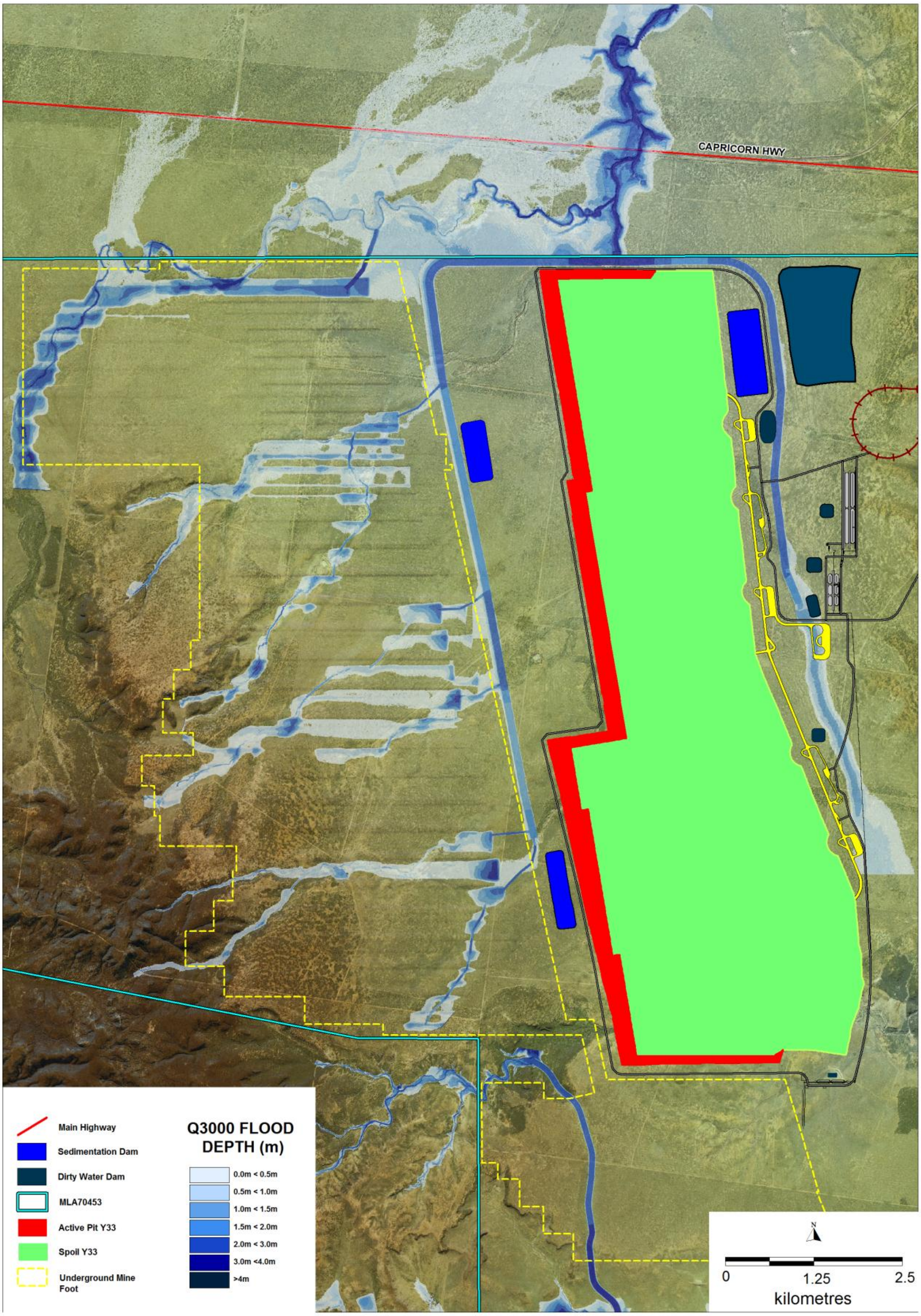

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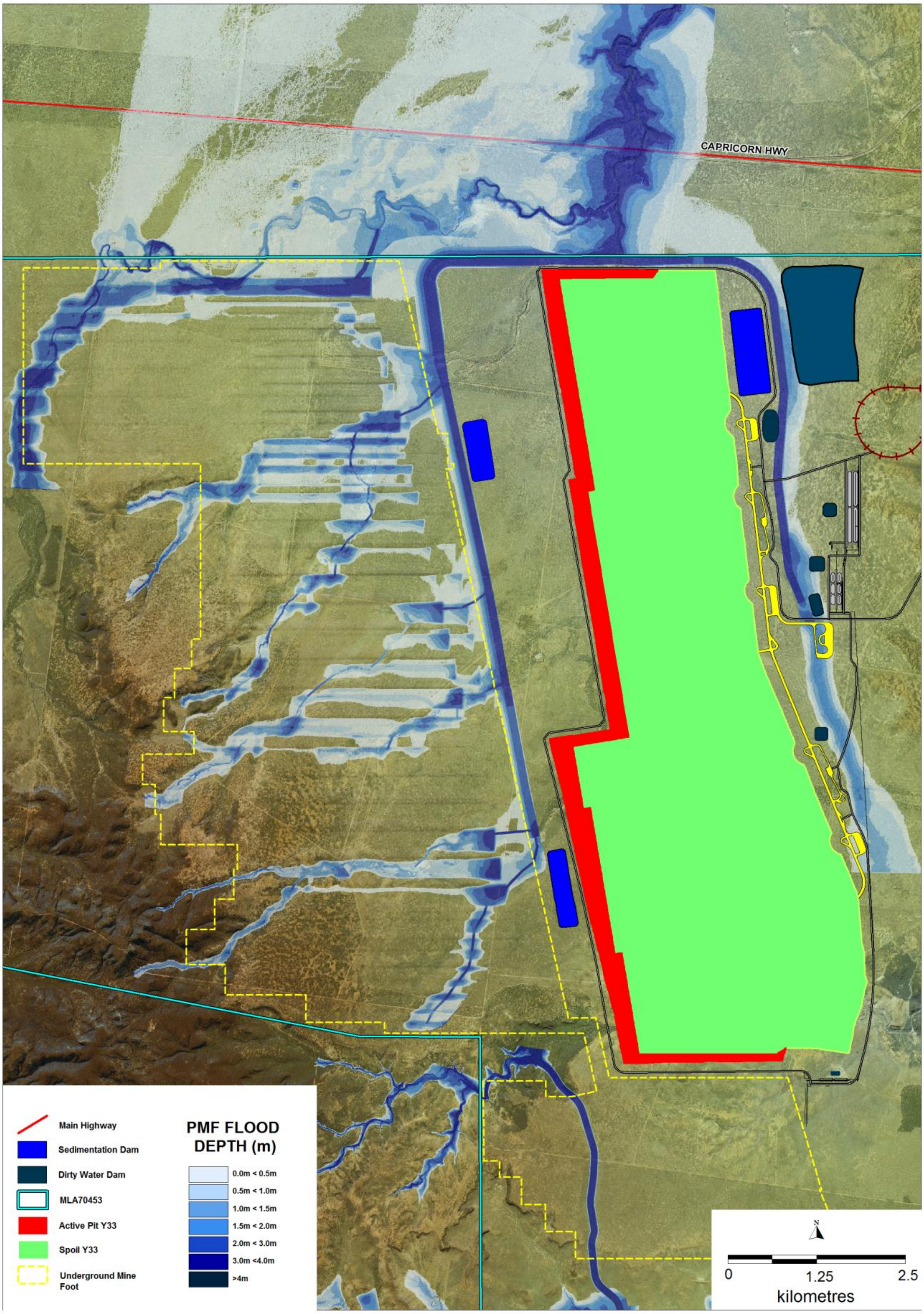

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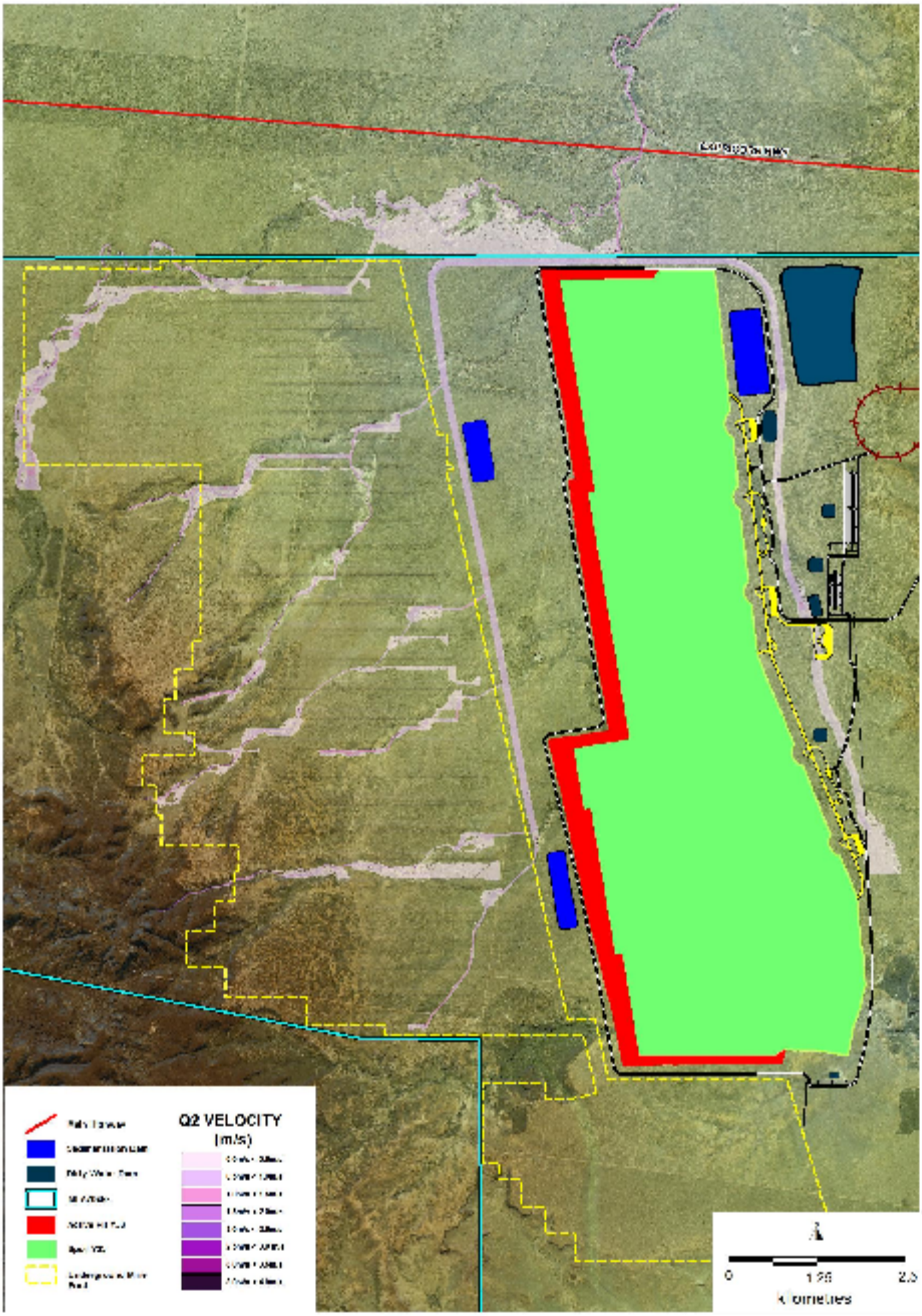


Figure 1.12 Tallarenha Creek – Post Development Conditions - 2 Year ARI Flood Velocities

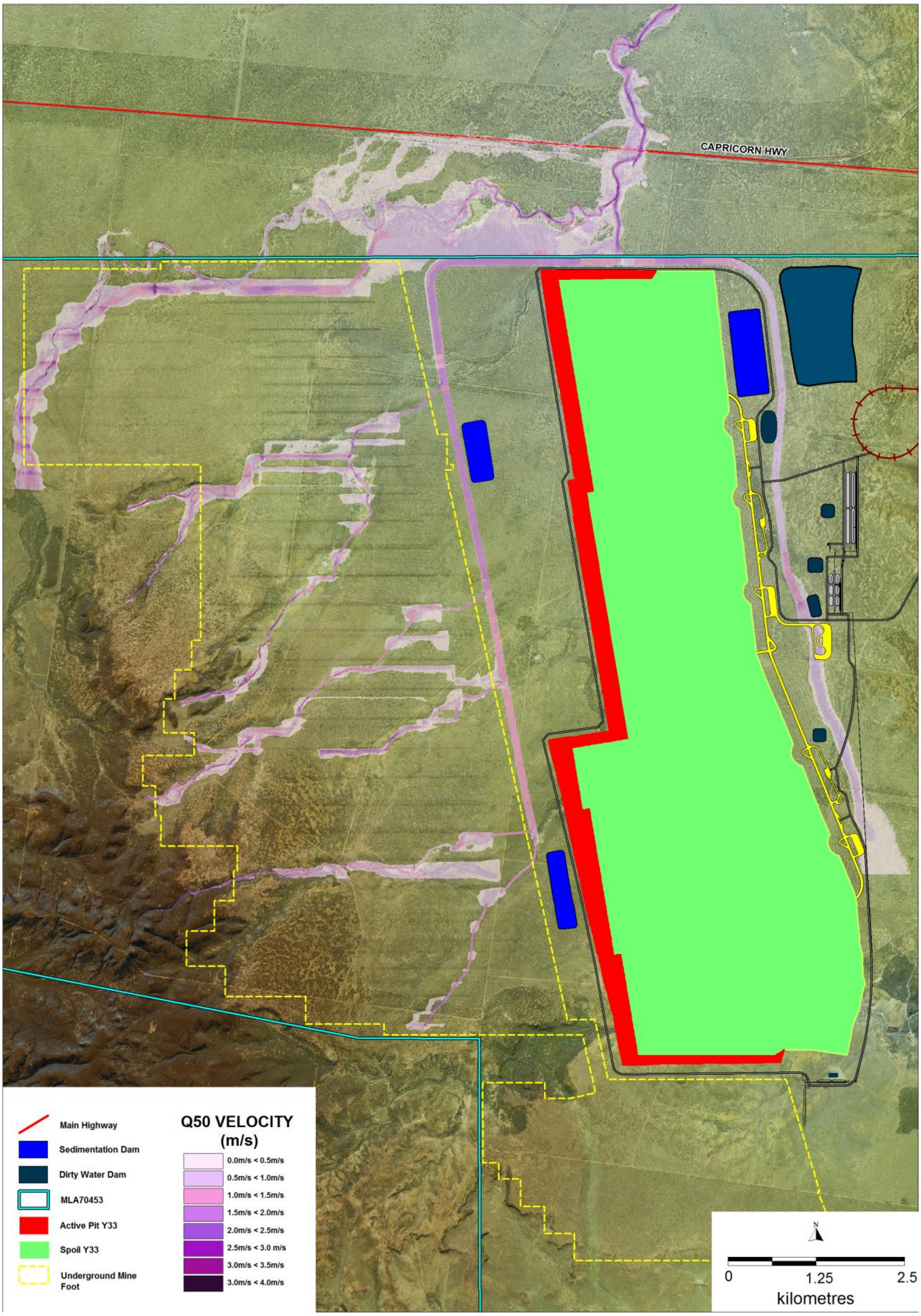
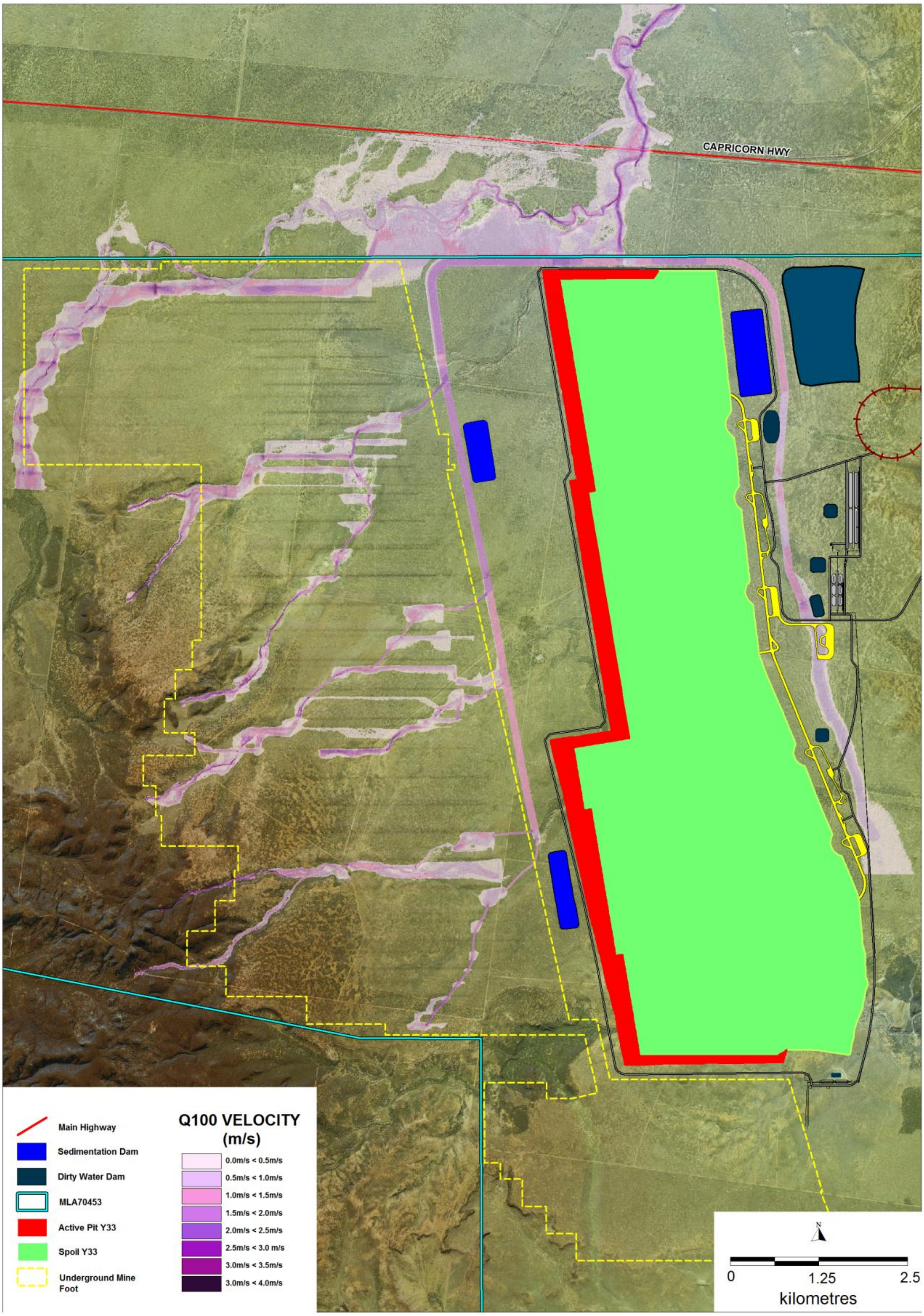


Figure 1.13 Tallarenha Creek – Post Development Conditions - 50 Year ARI Flood Velocities



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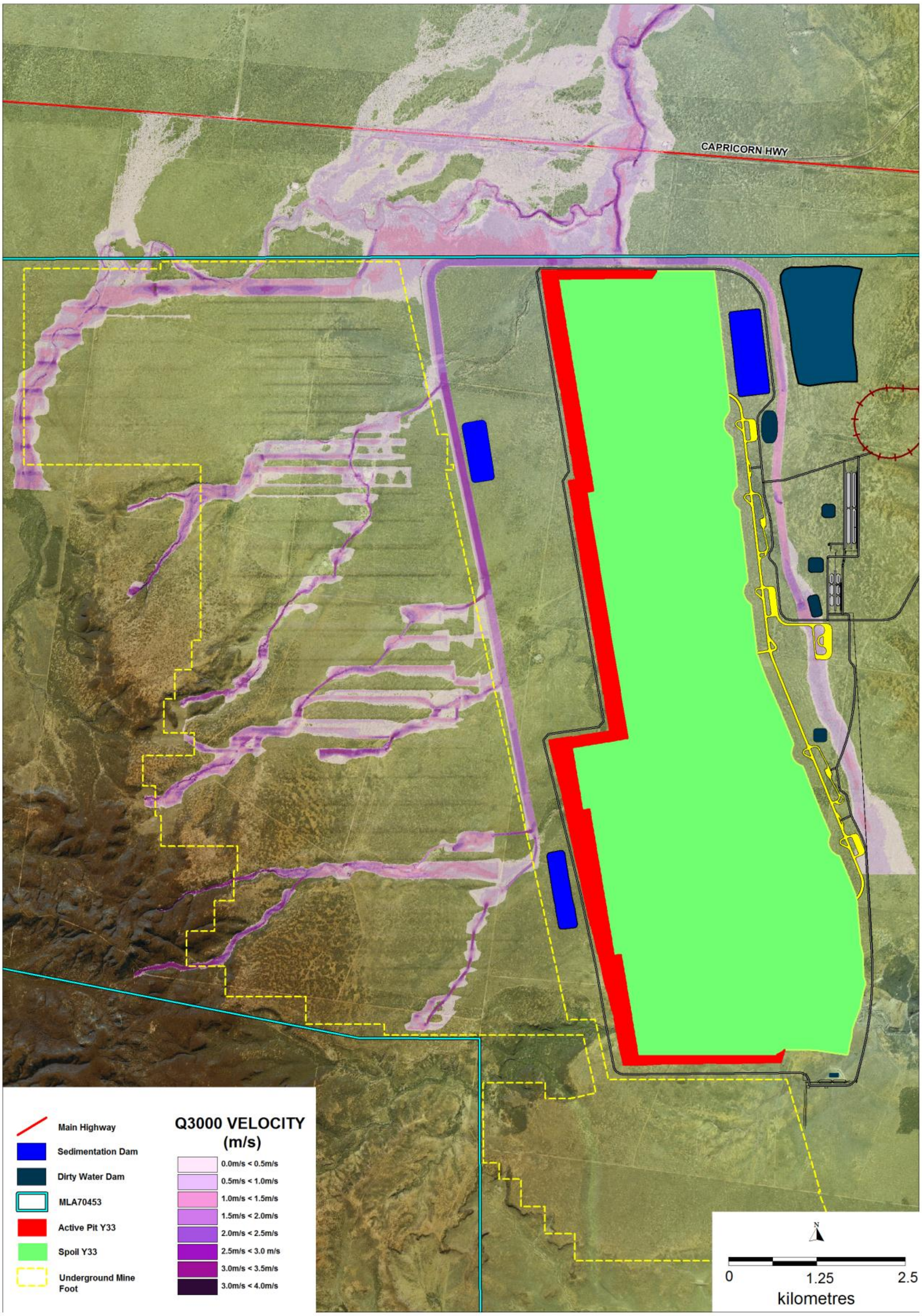
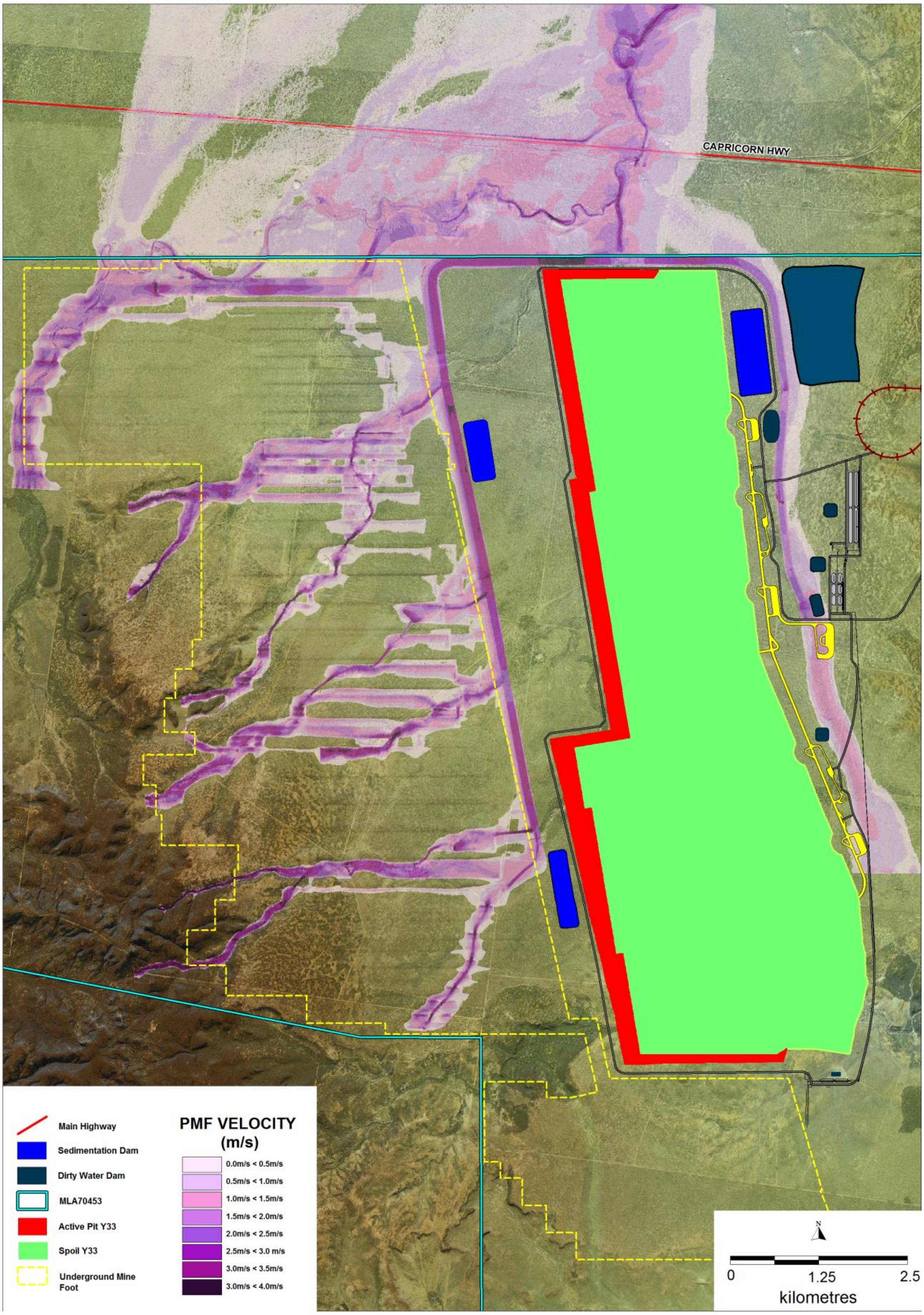


Figure 1.15 Tallarenha Creek – Post Development Conditions - 3000 Year ARI Flood Velocities



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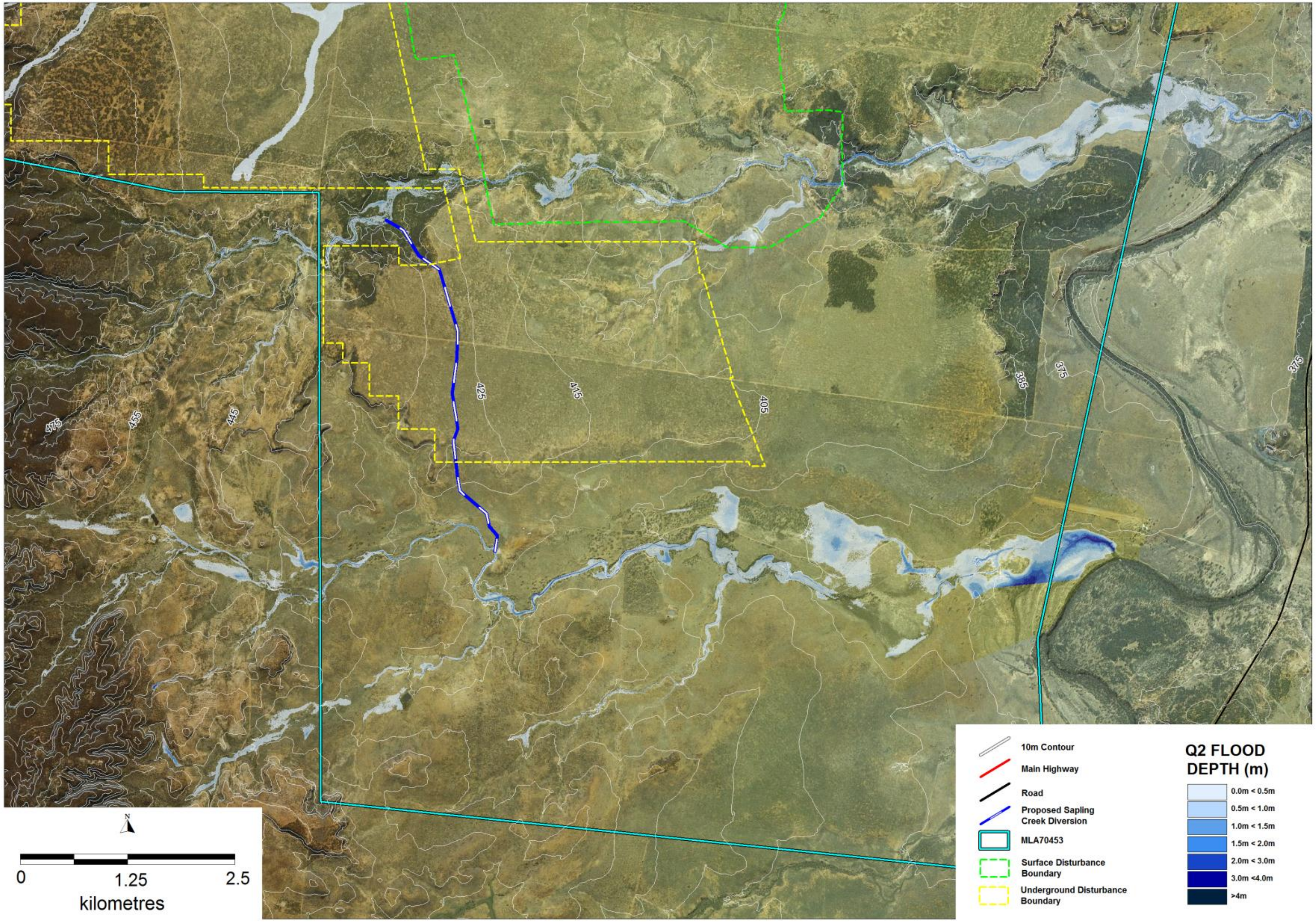
2 SAPLING CREEK AND DEAD HORSE CREEK

2.1 EXISTING CONDITIONS FLOOD DEPTHS



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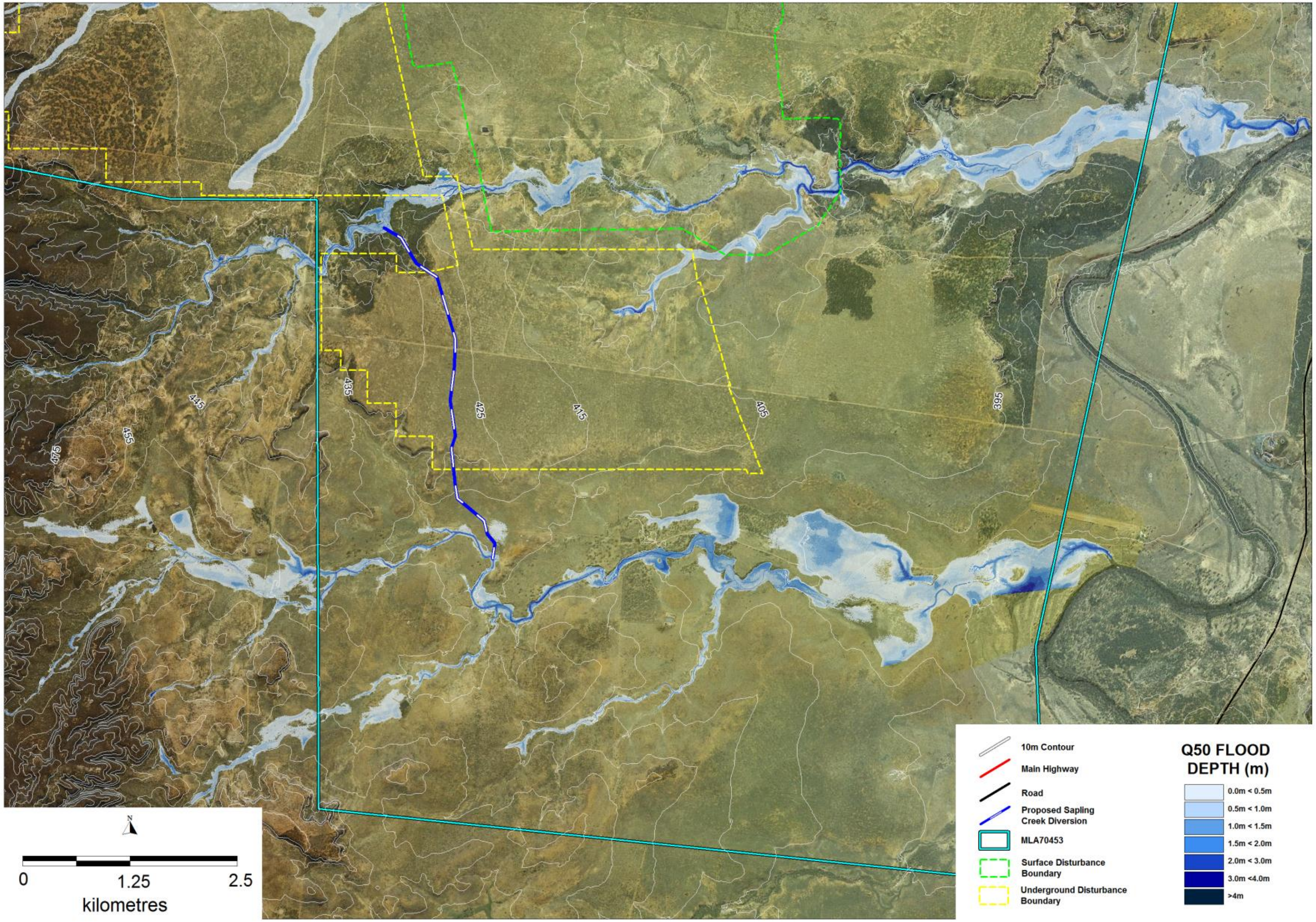

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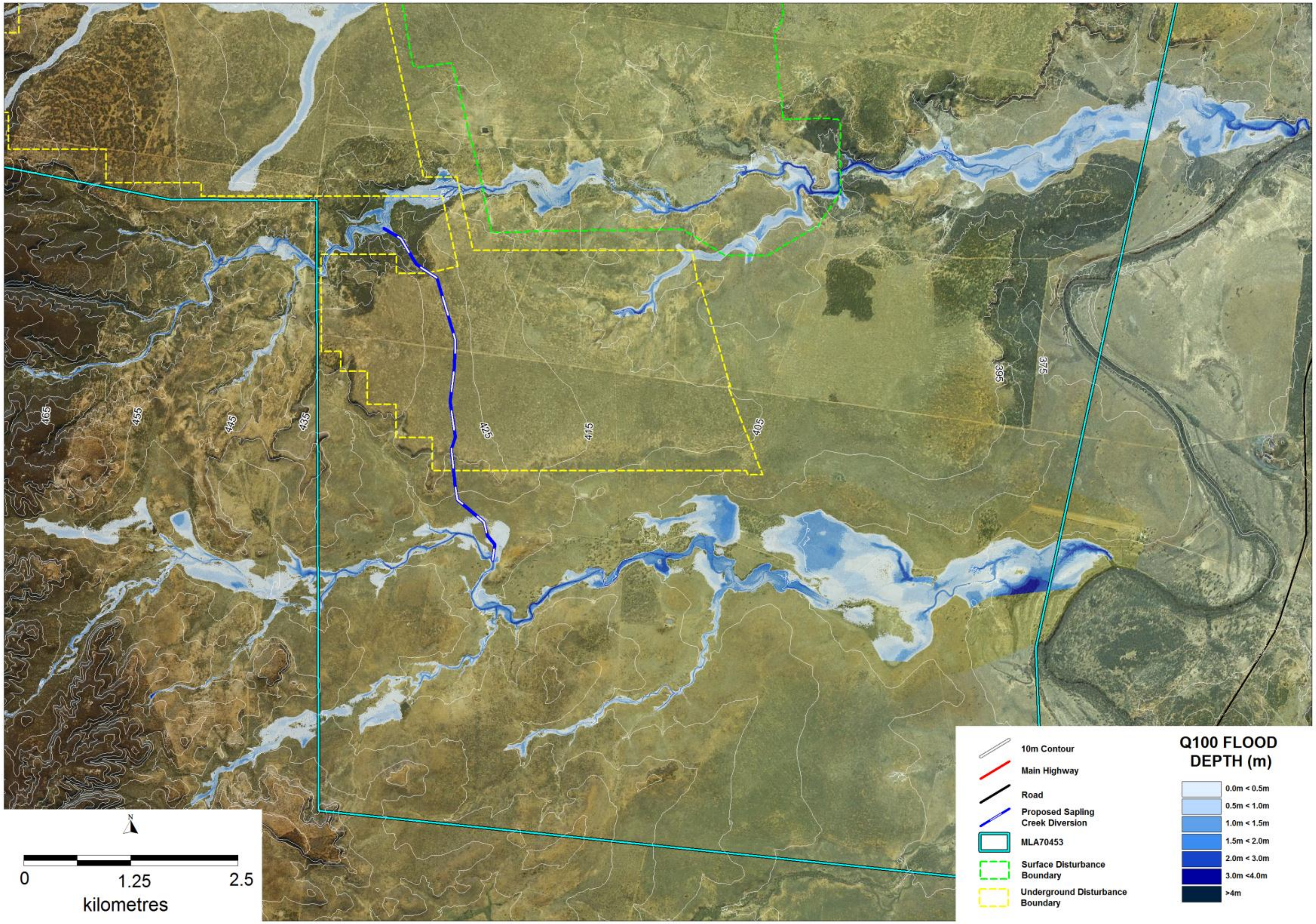

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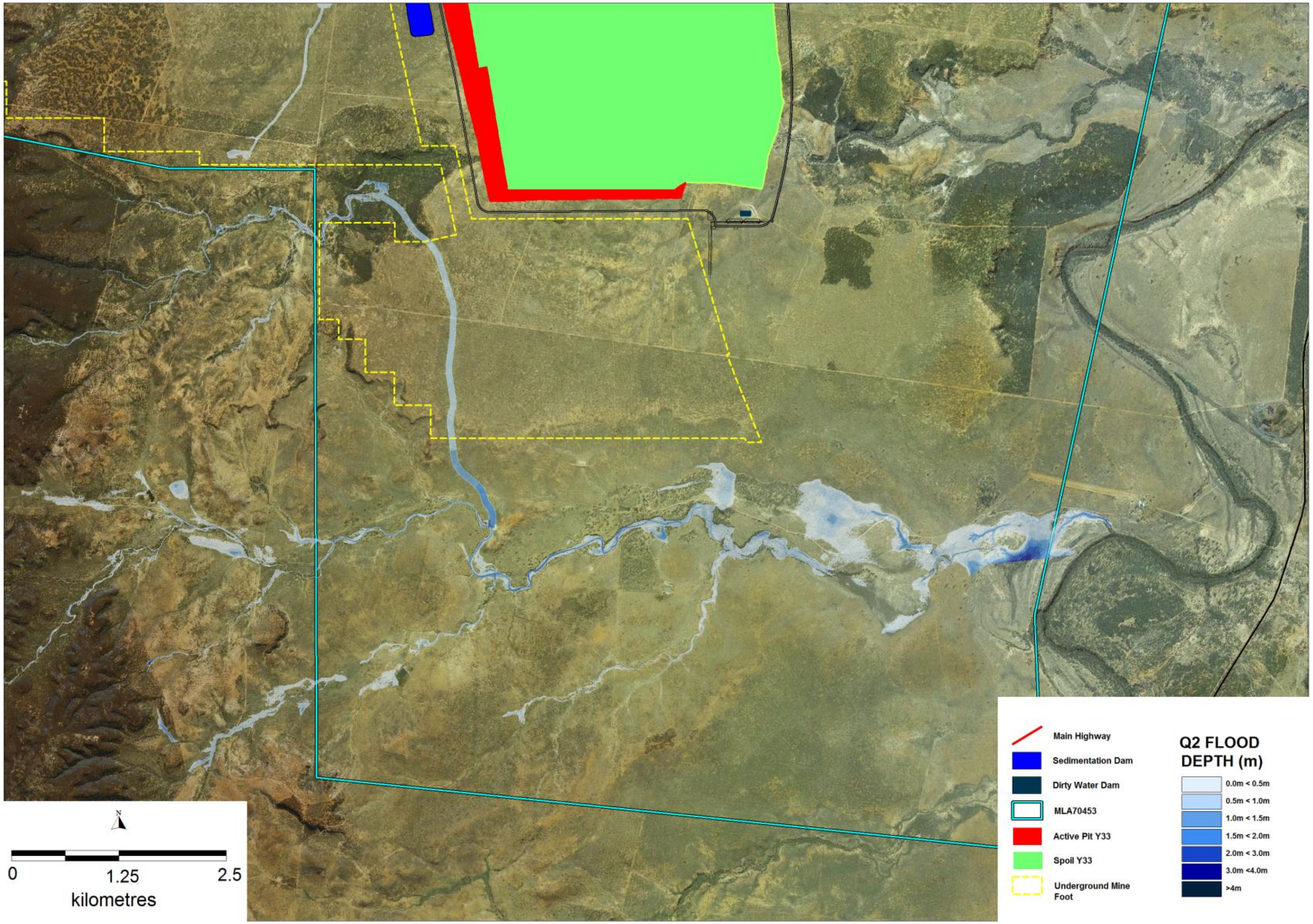

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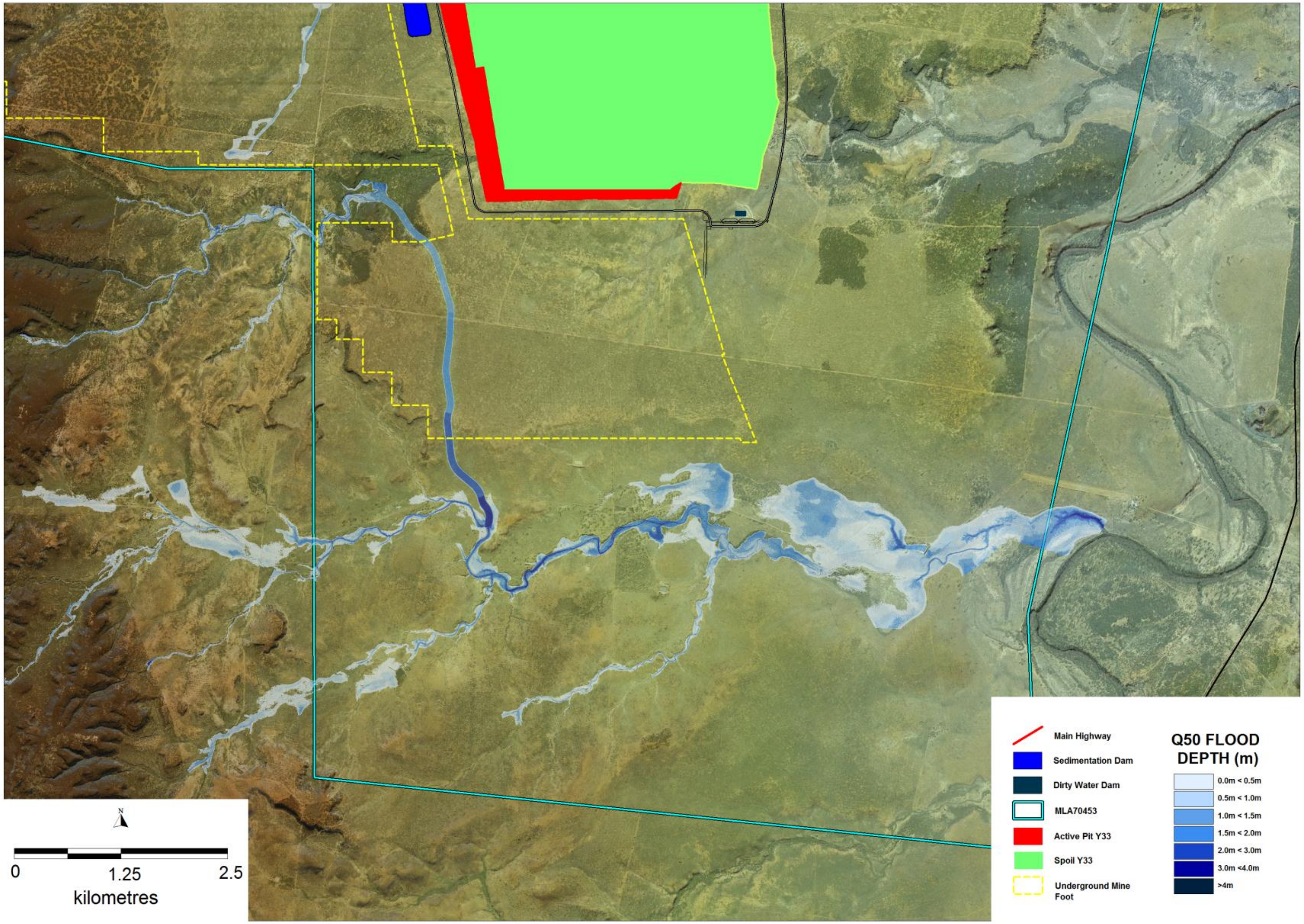

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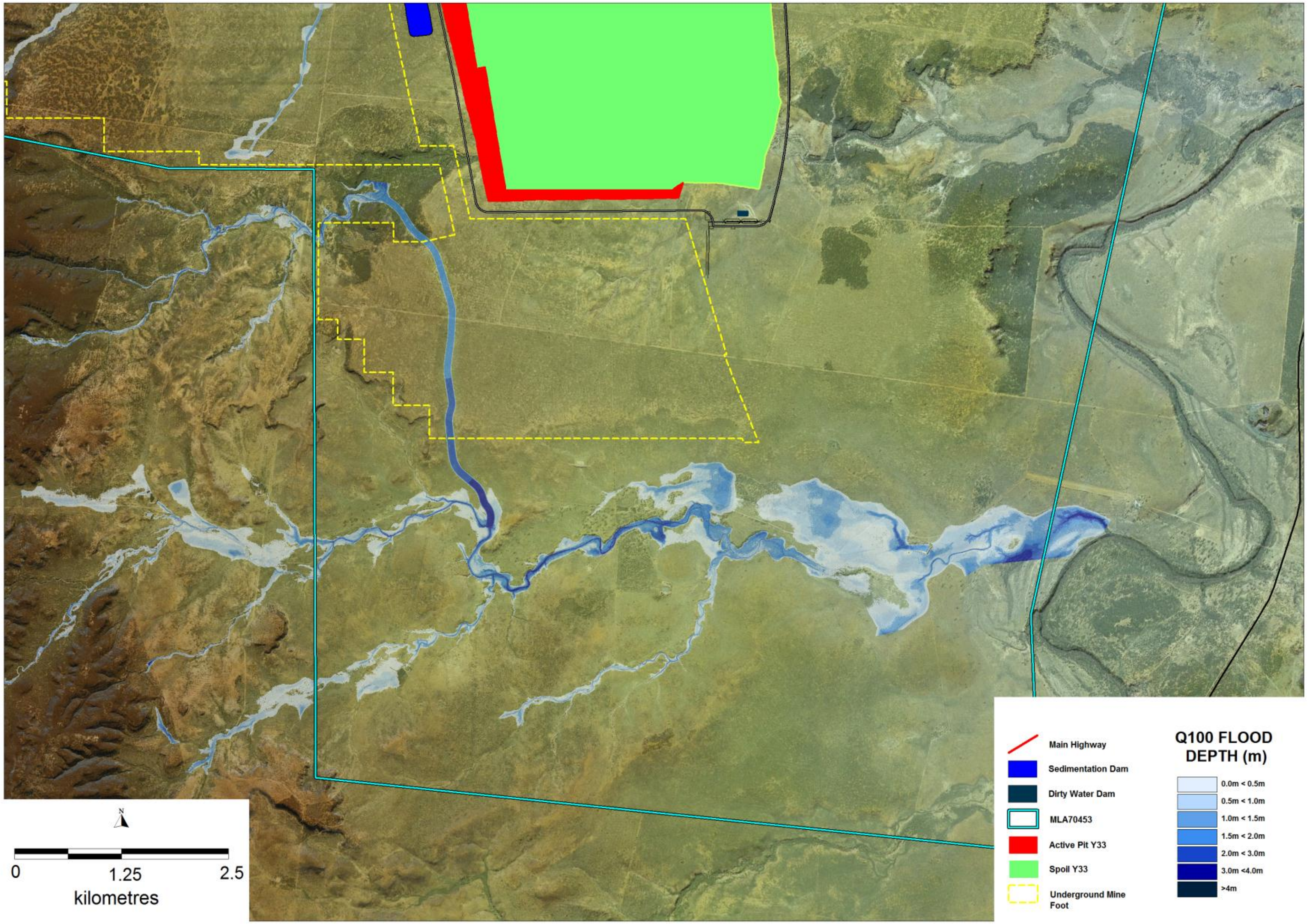

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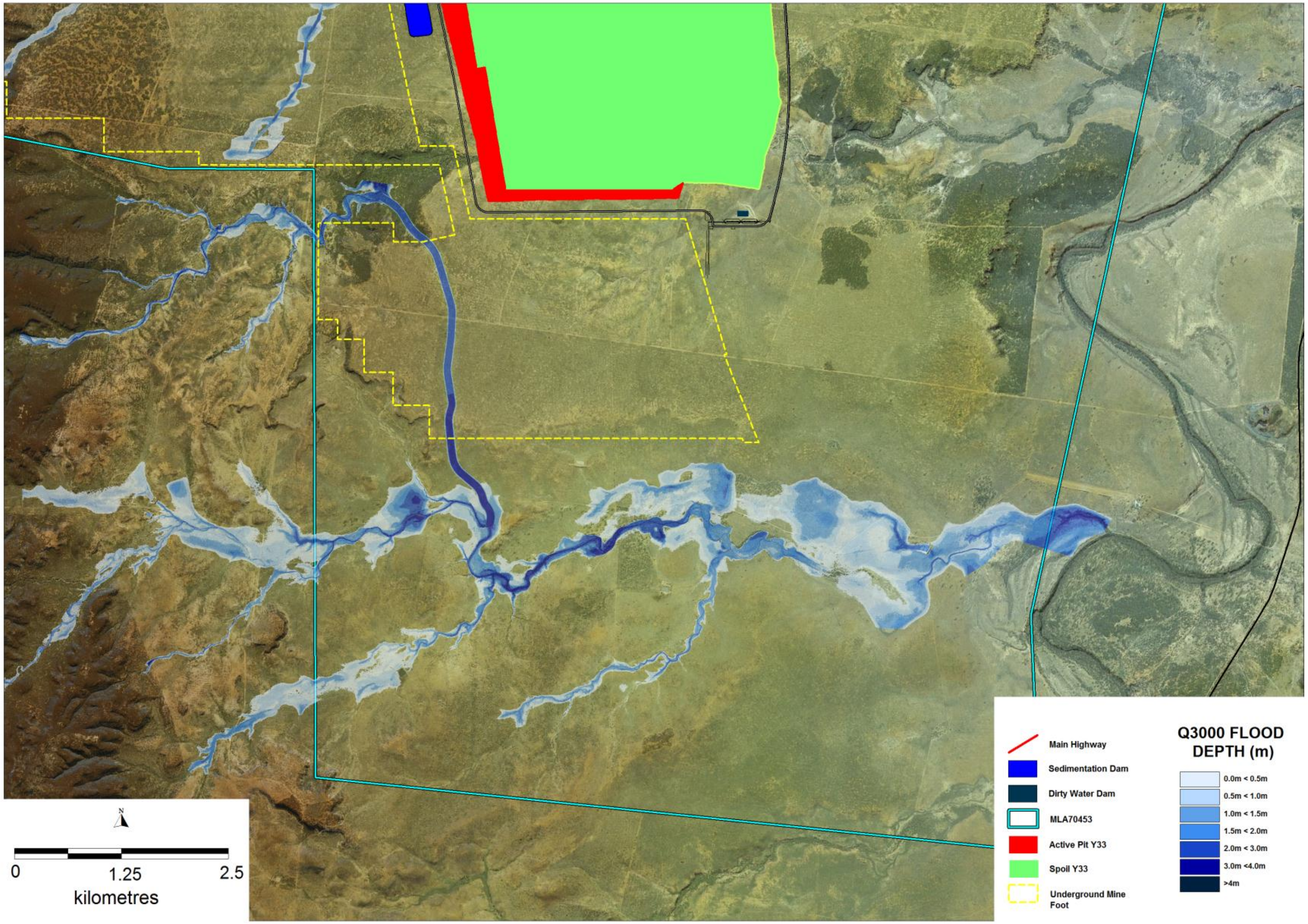

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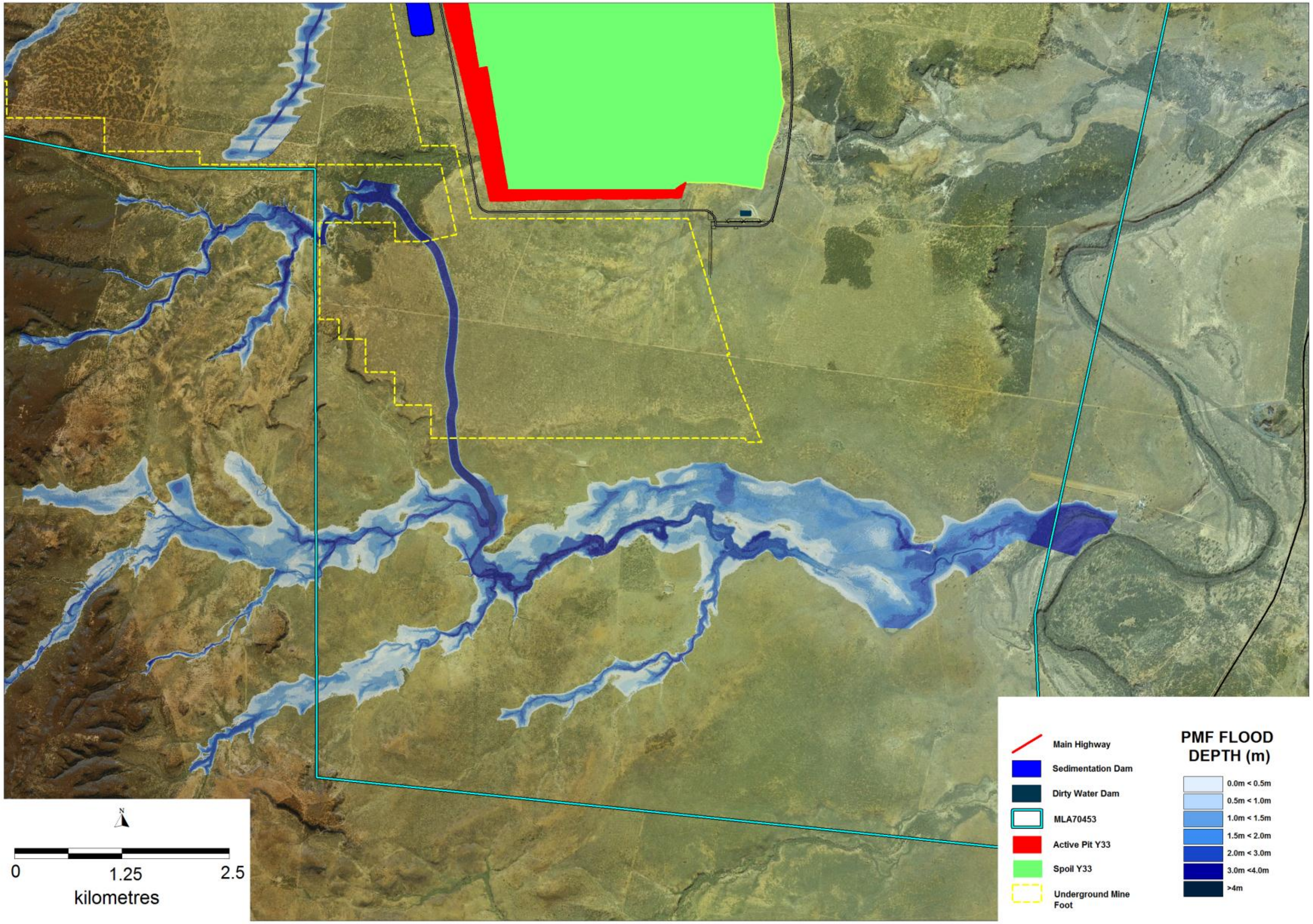

Figure 2.8 Sapling and Dead Horse Creeks – Post Development Conditions - PMF Flood Depth and Extents

3 INFRASTRUCTURE CORRIDOR

3.1 EXISTING CONDITIONS FLOOD DEPTHS AND EXTENTS

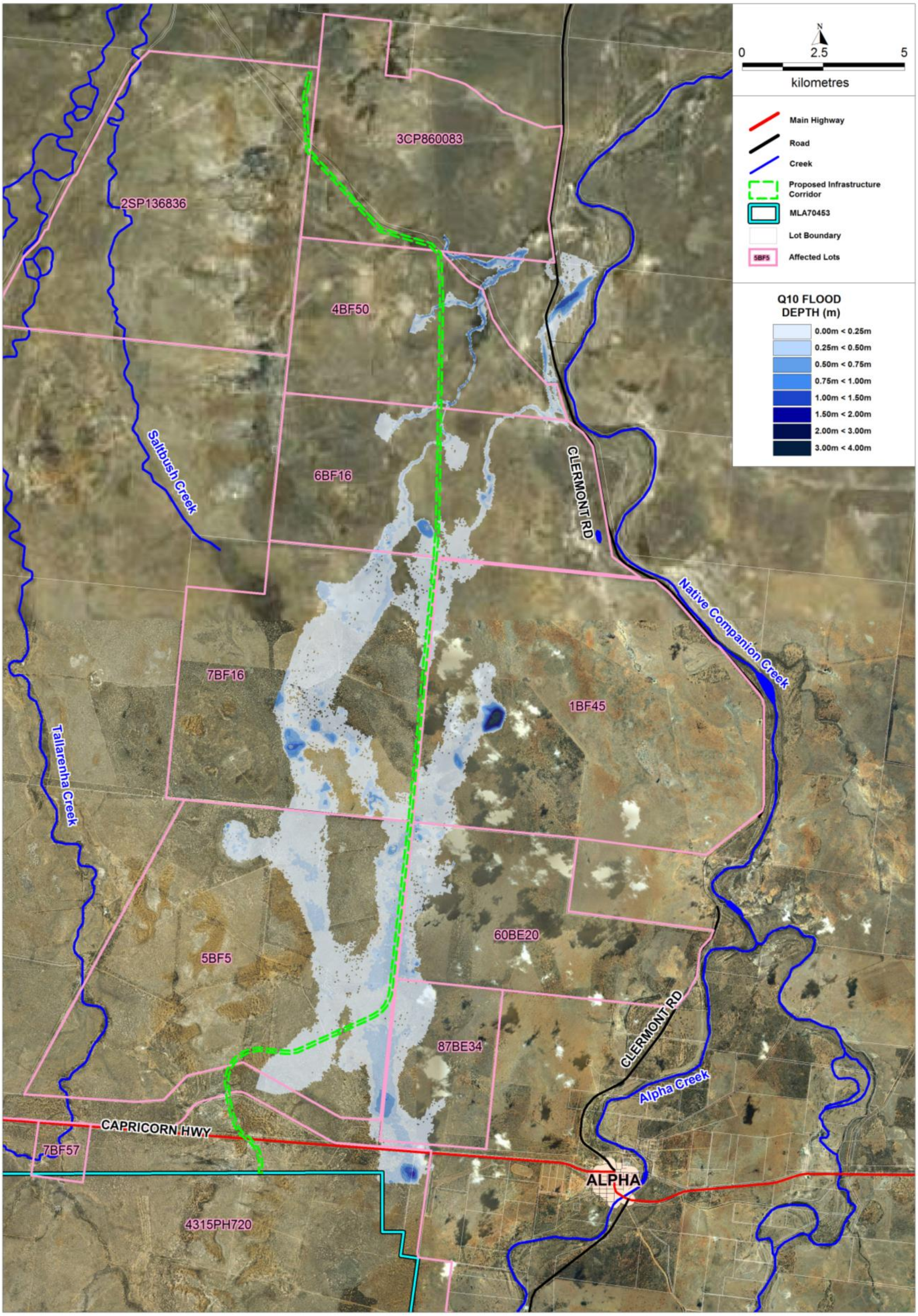

Figure 3.1 Infrastructure Corridor – Existing Conditions – 10 Year ARI Flood Depths and Extents

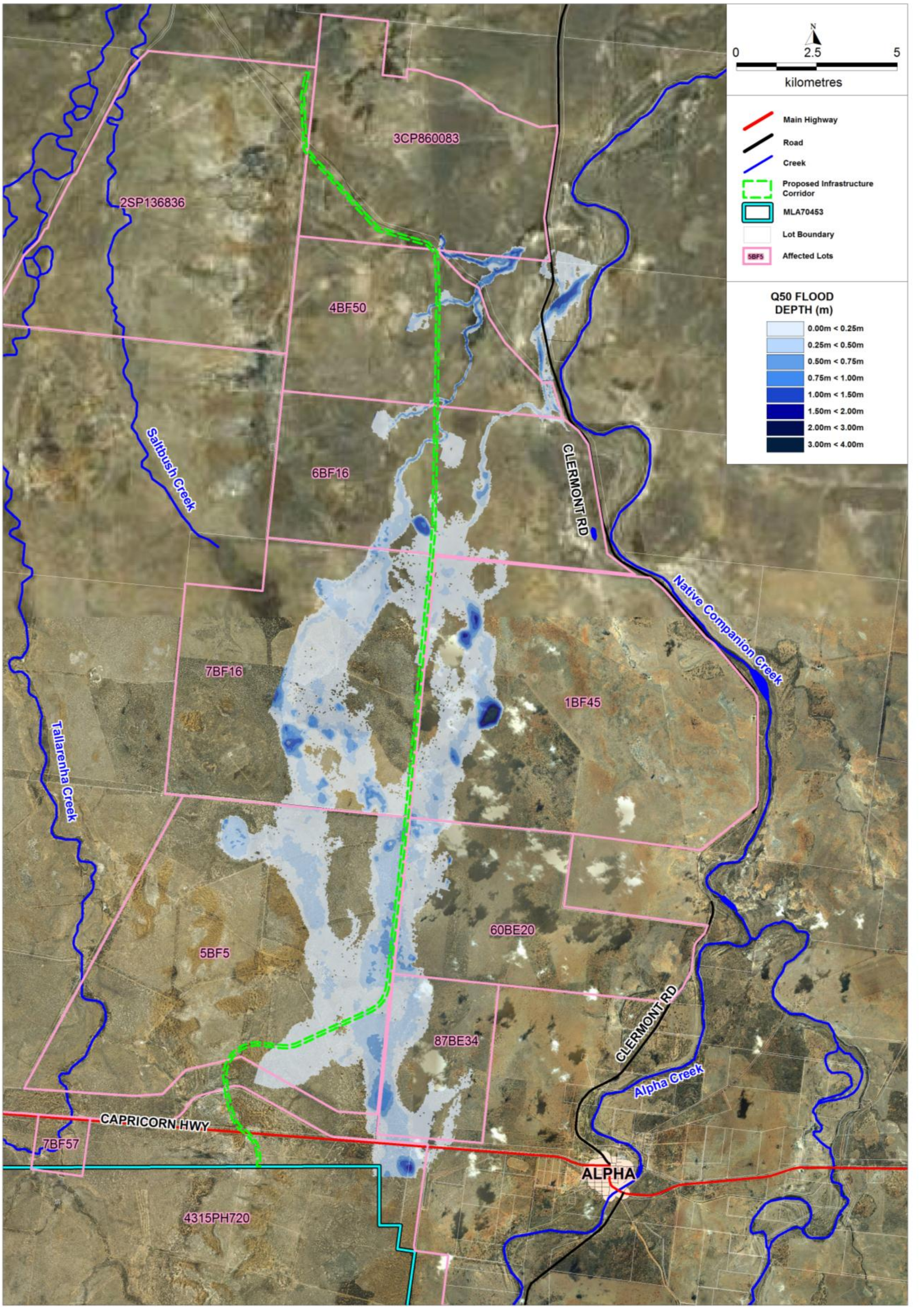

Figure 3.2 Infrastructure Corridor – Existing Conditions – 50 Year ARI Flood Depths and Extents

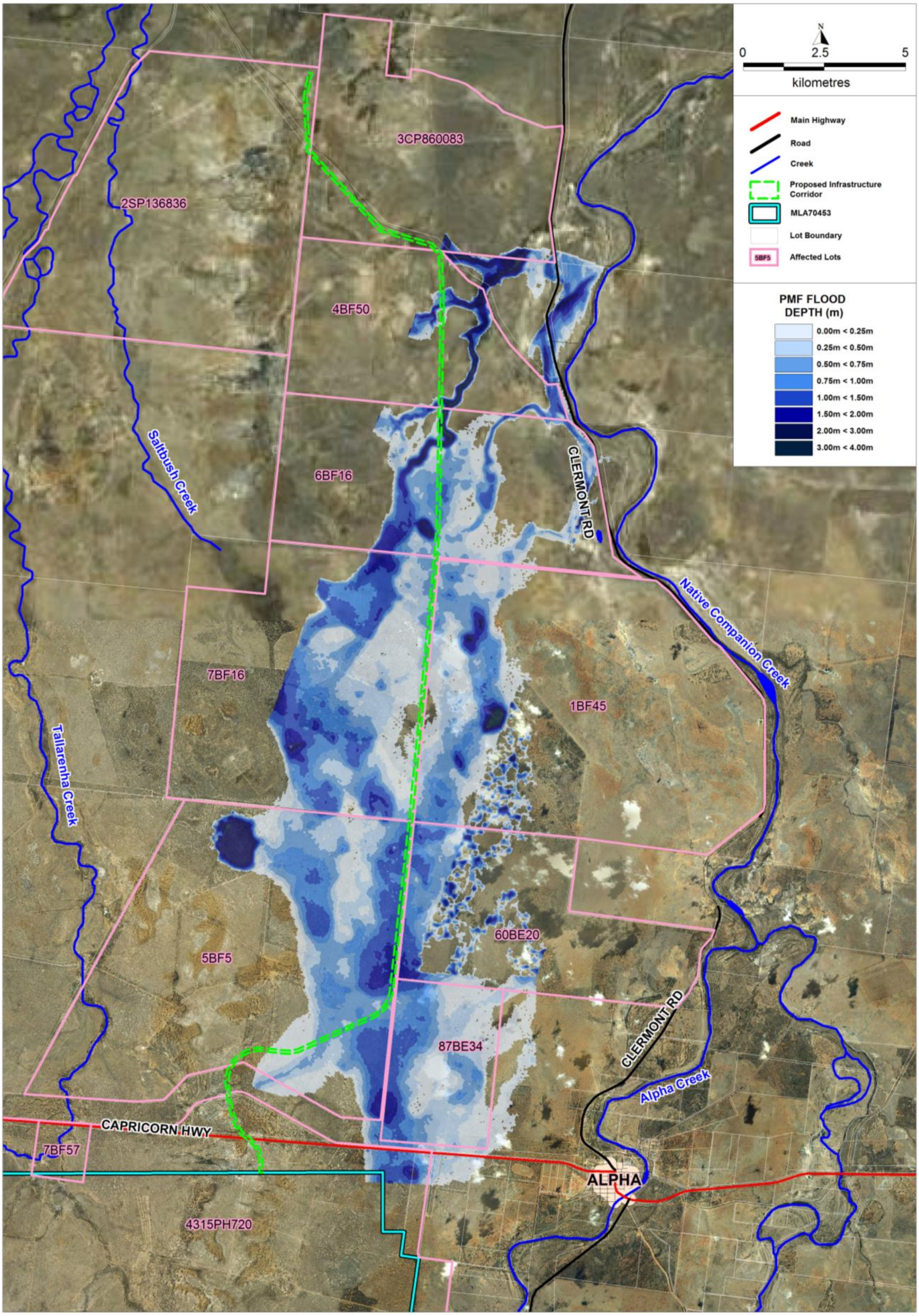

Figure 3.3 Infrastructure Corridor – Existing Conditions – PMF Flood Depths and Extents

3.2 EXISTING CONDITIONS FLOOD VELOCITIES

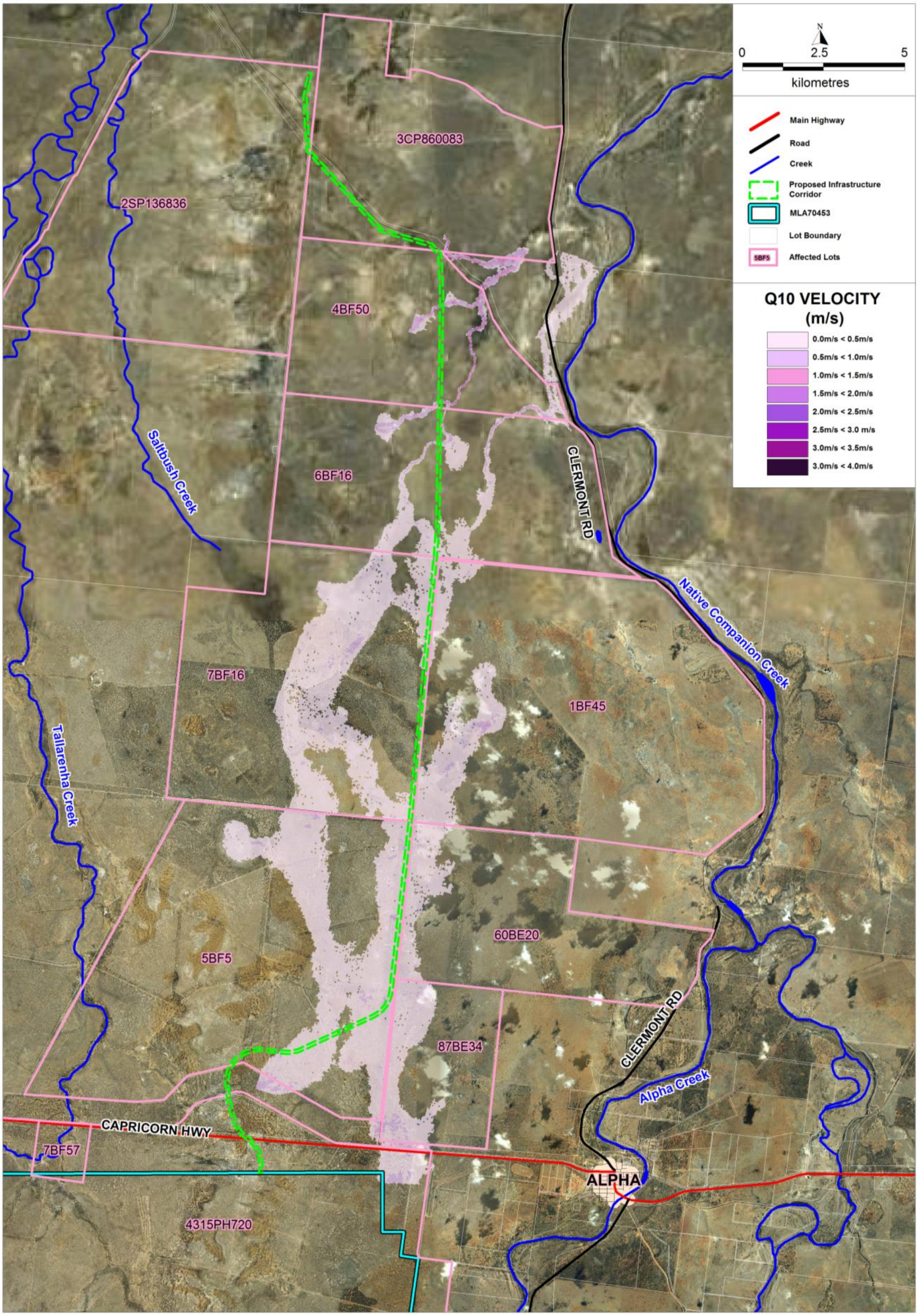

Figure 3.4 Infrastructure Corridor – Existing Conditions – 10 Year ARI Flood Velocities

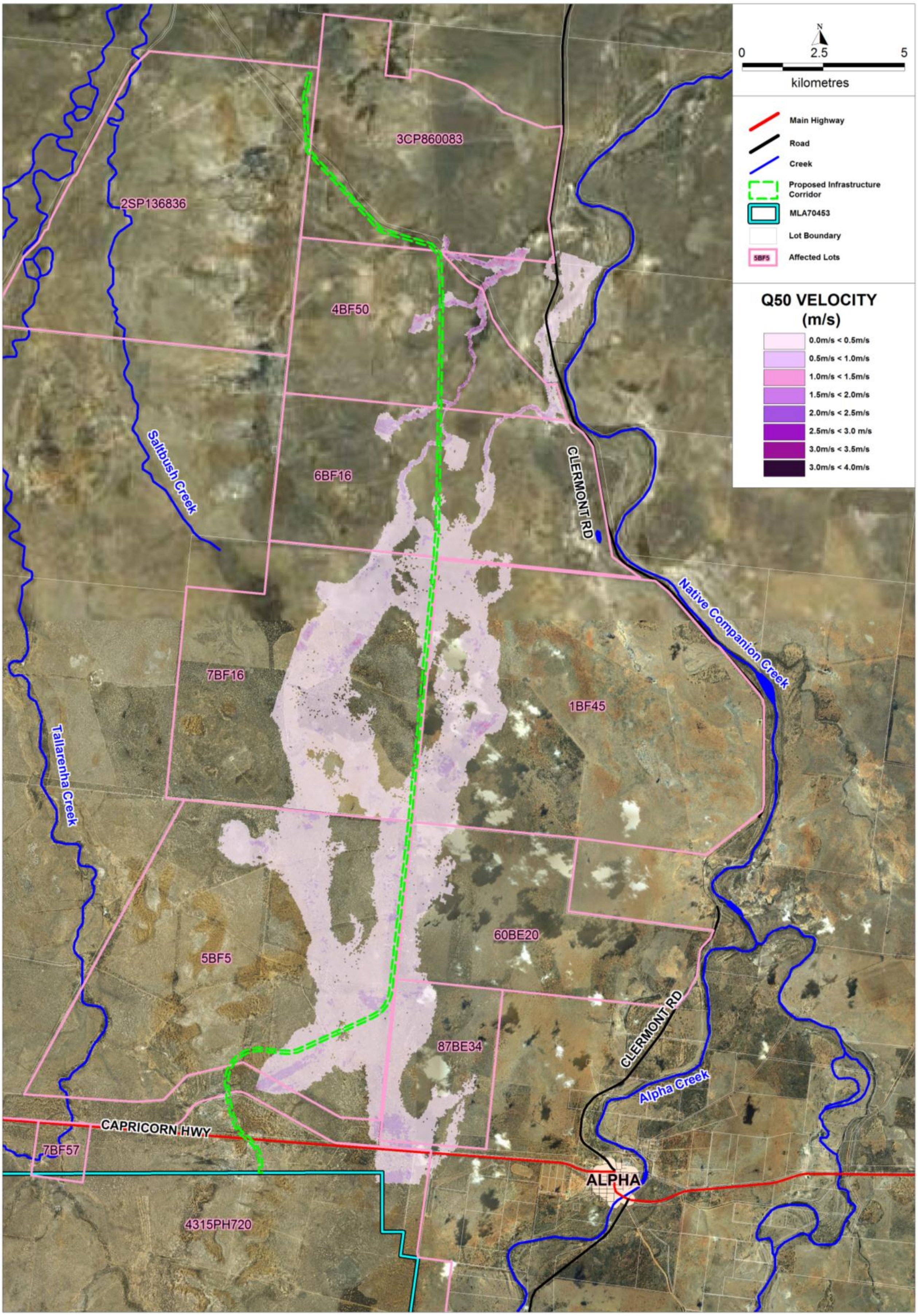

Figure 3.5 Infrastructure Corridor – Existing Conditions – 50 year Ari Flood Velocities

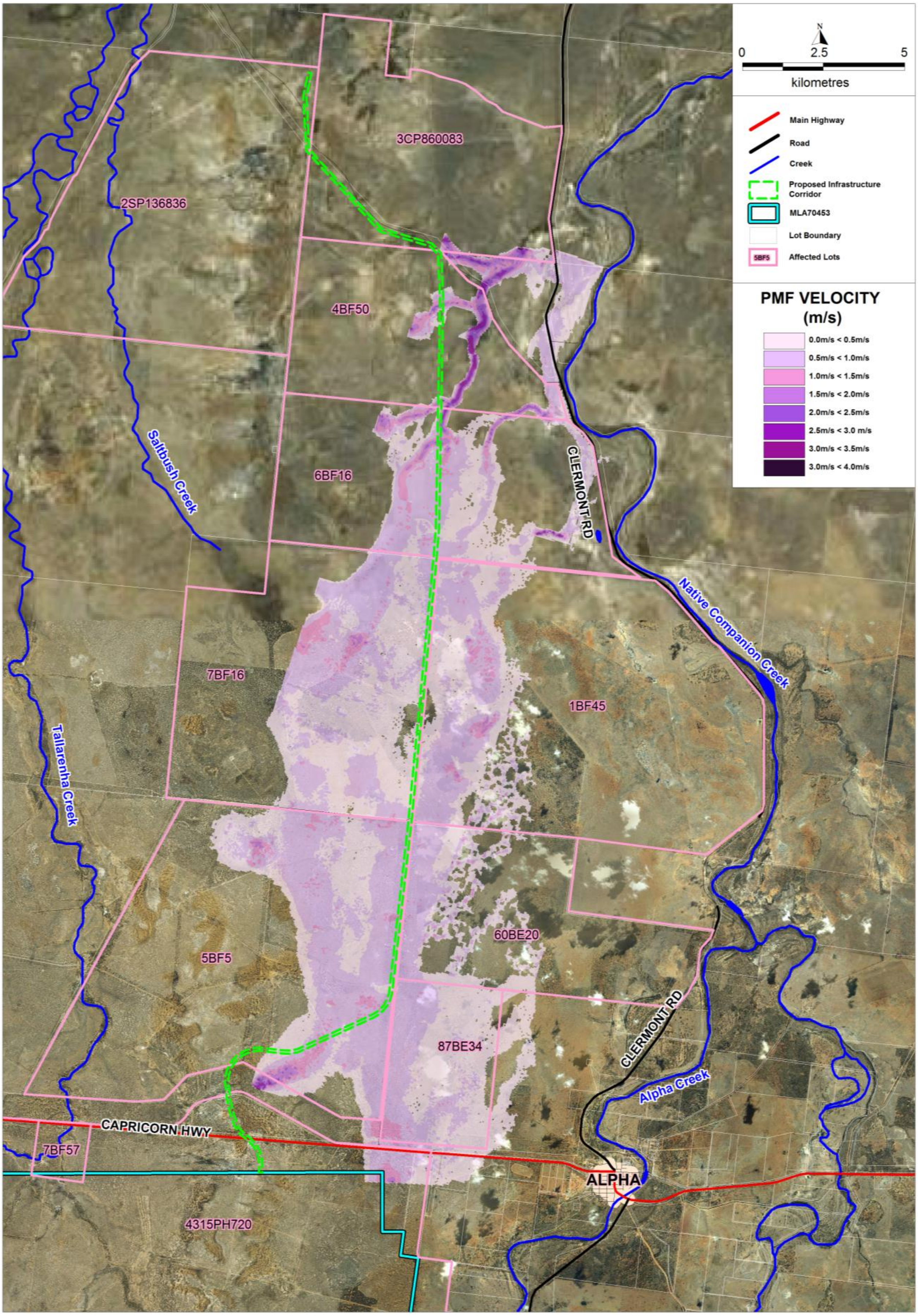

Figure 3.6 Infrastructure Corridor – Existing Conditions – PMF Flood Velocities

3.3 POST DEVELOPMENT FLOOD DEPTHS AND EXTENTS

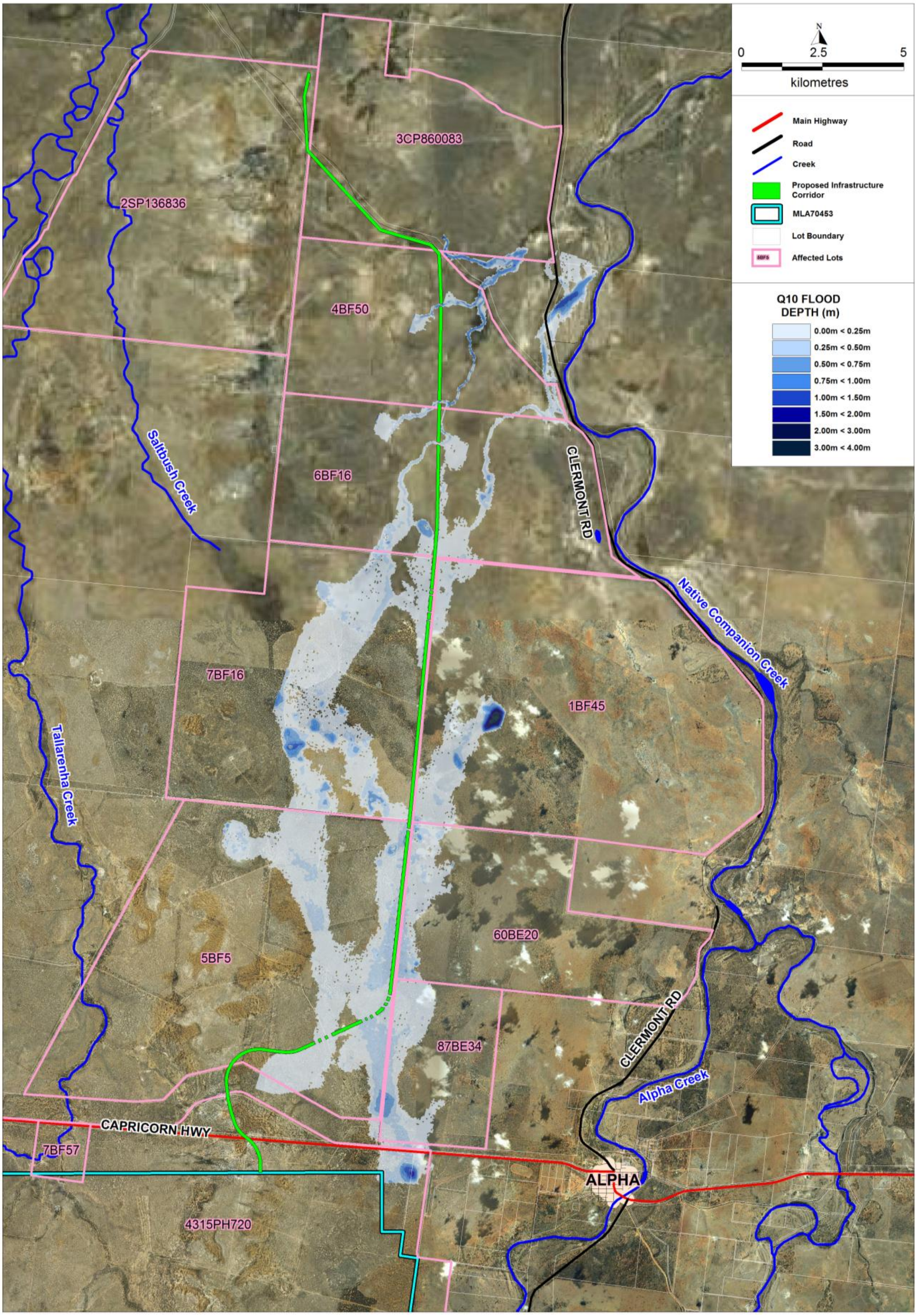

Figure 3.7 Infrastructure Corridor – Post Development Conditions – 10 Year ARI Flood Depths and Extents

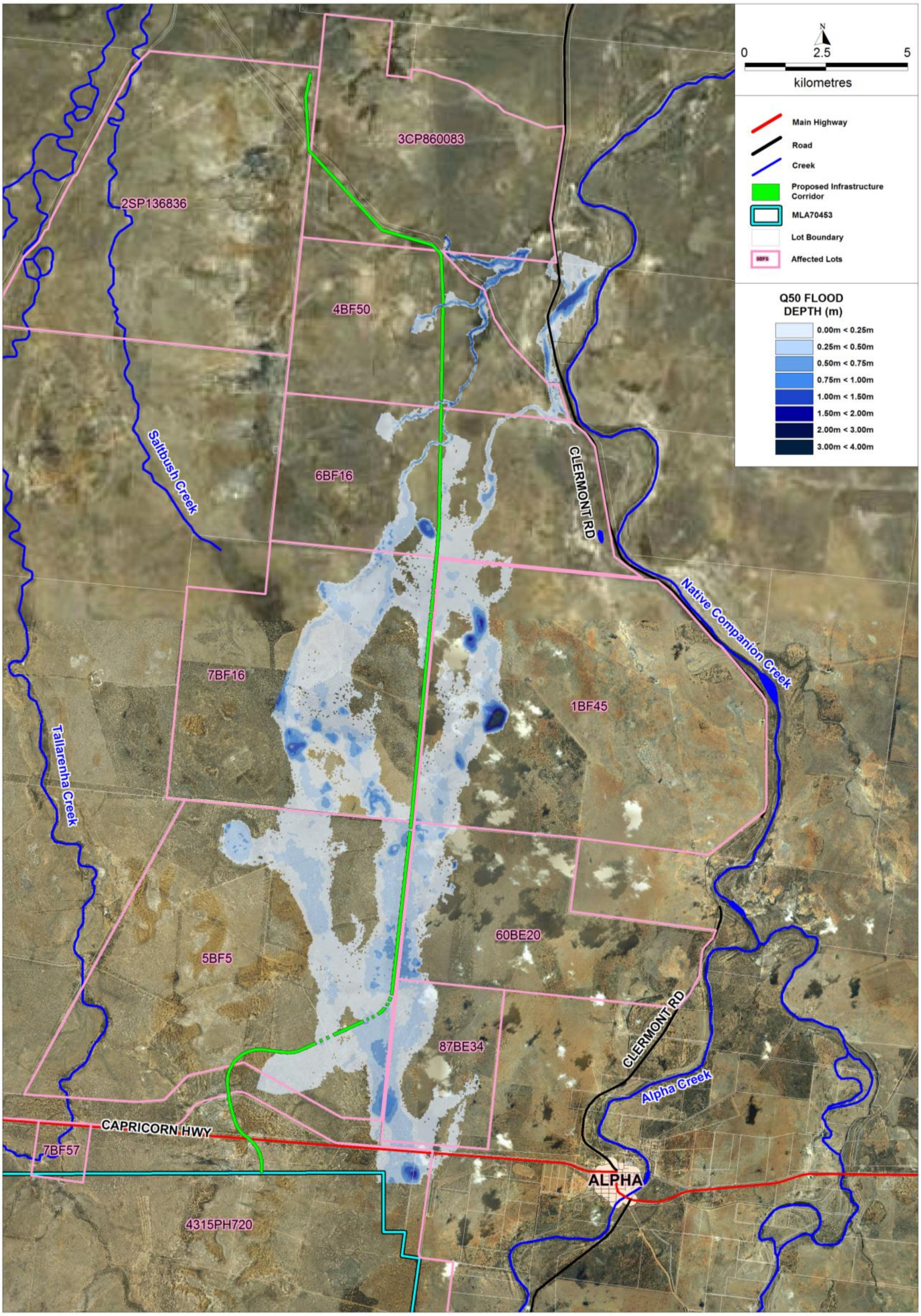

Figure 3.8 Infrastructure Corridor – Post Development Conditions – 50 year ARI Flood Depths and Extents

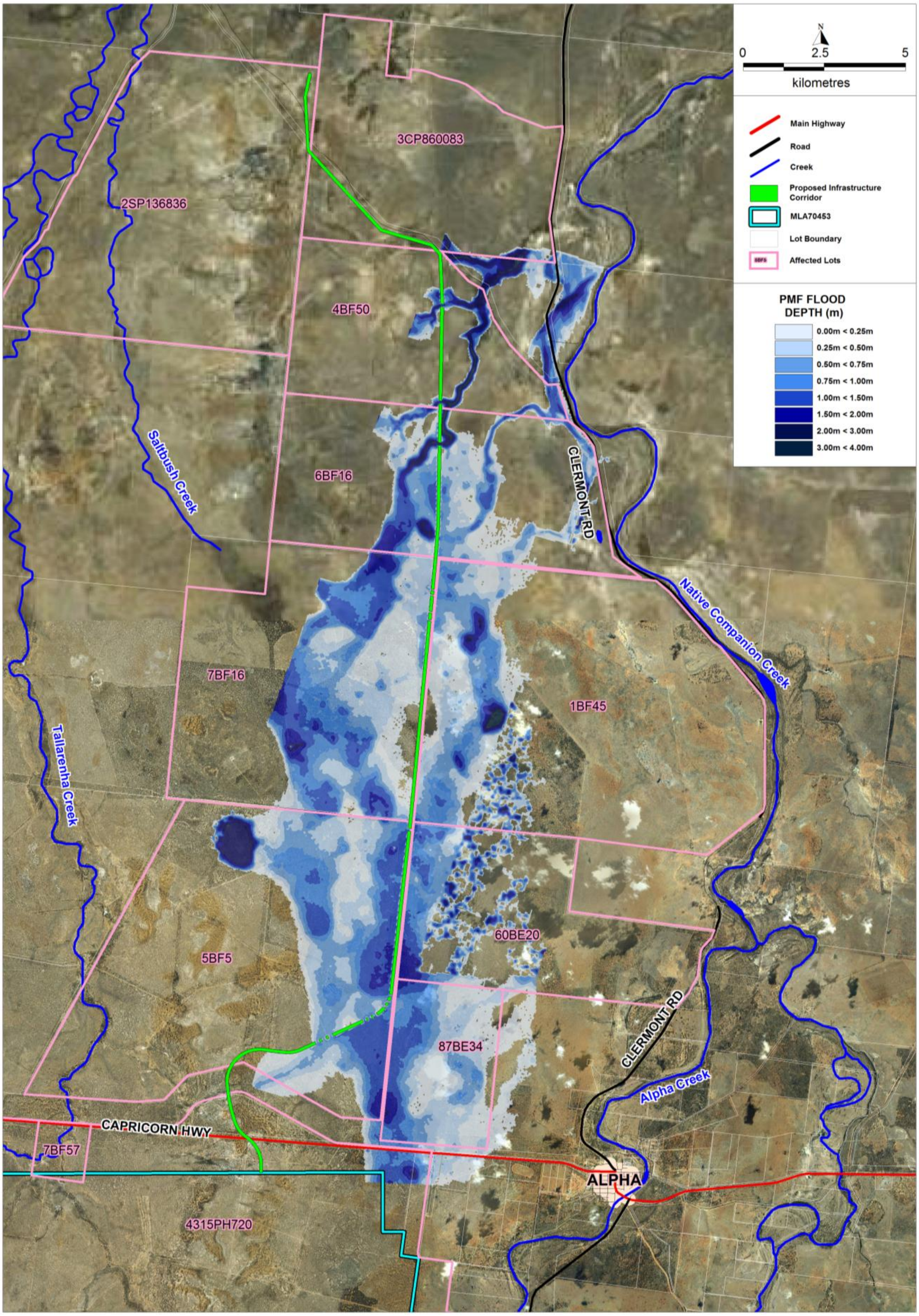

Figure 3.9 Infrastructure Corridor – Post Development Conditions – PMF Flood Depths and Extents

3.4 POST DEVELOPMENT FLOOD VELOCITIES

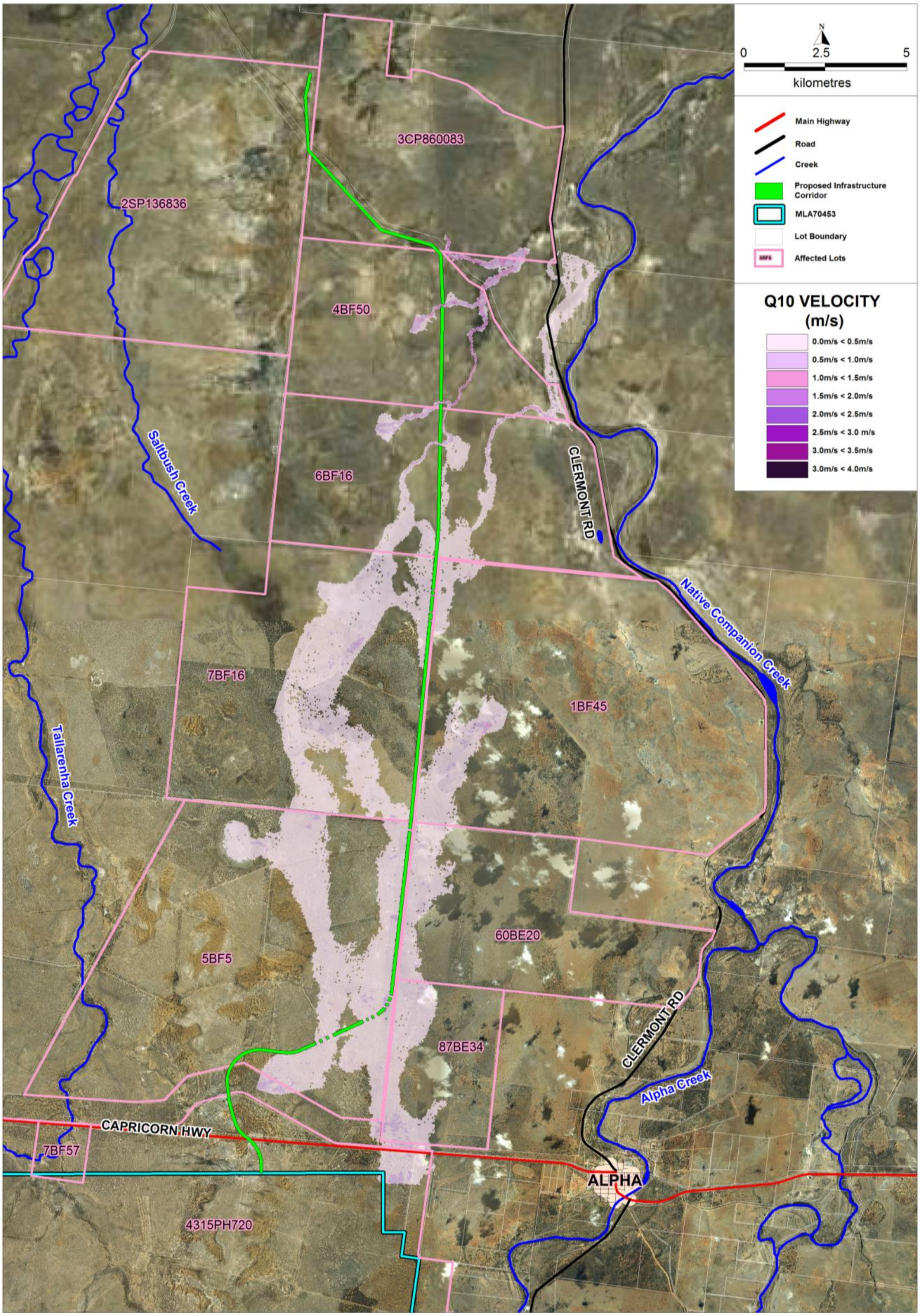

Figure 3.10 Infrastructure Corridor – Post Development Conditions – 10 Year ARI Flood Velocities

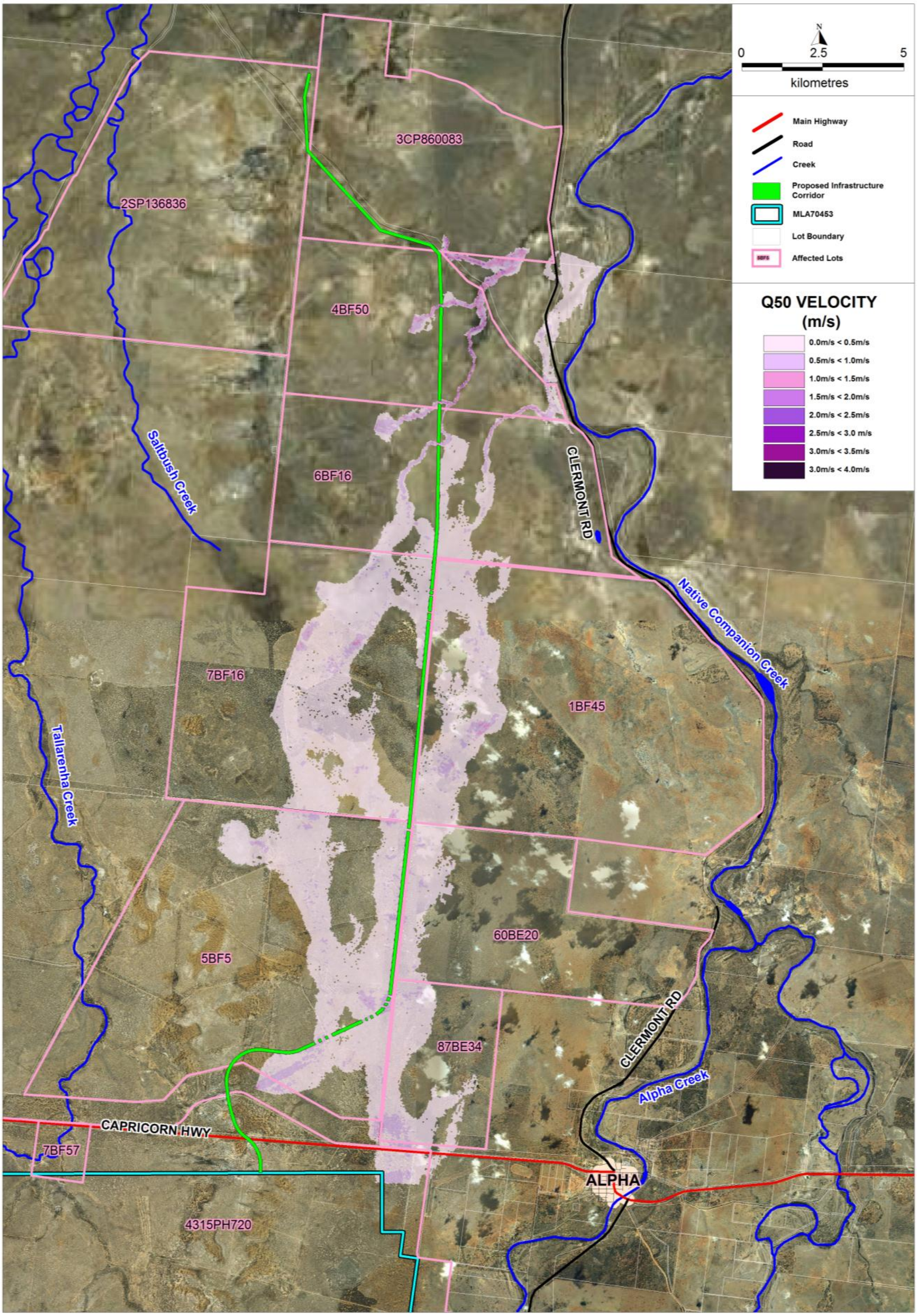

Figure 3.11 Infrastructure Corridor – Post Development Conditions – 50 Year ARI Flood Velocities

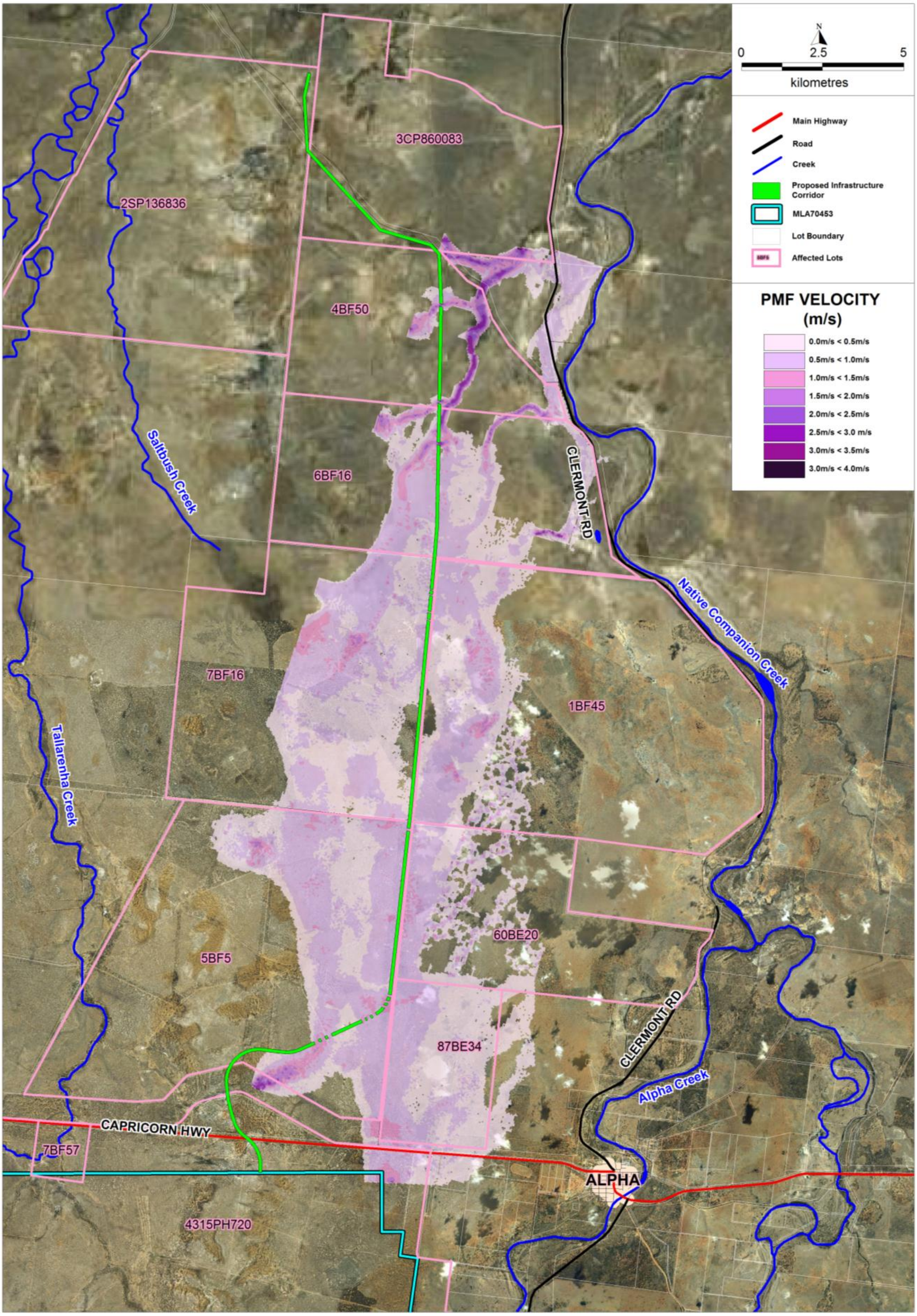

Figure 3.12 Infrastructure Corridor – Post Development Conditions – PMF Flood Velocities

3.5 POST DEVELOPMENT FLOOD DEPTH AFFLUXES

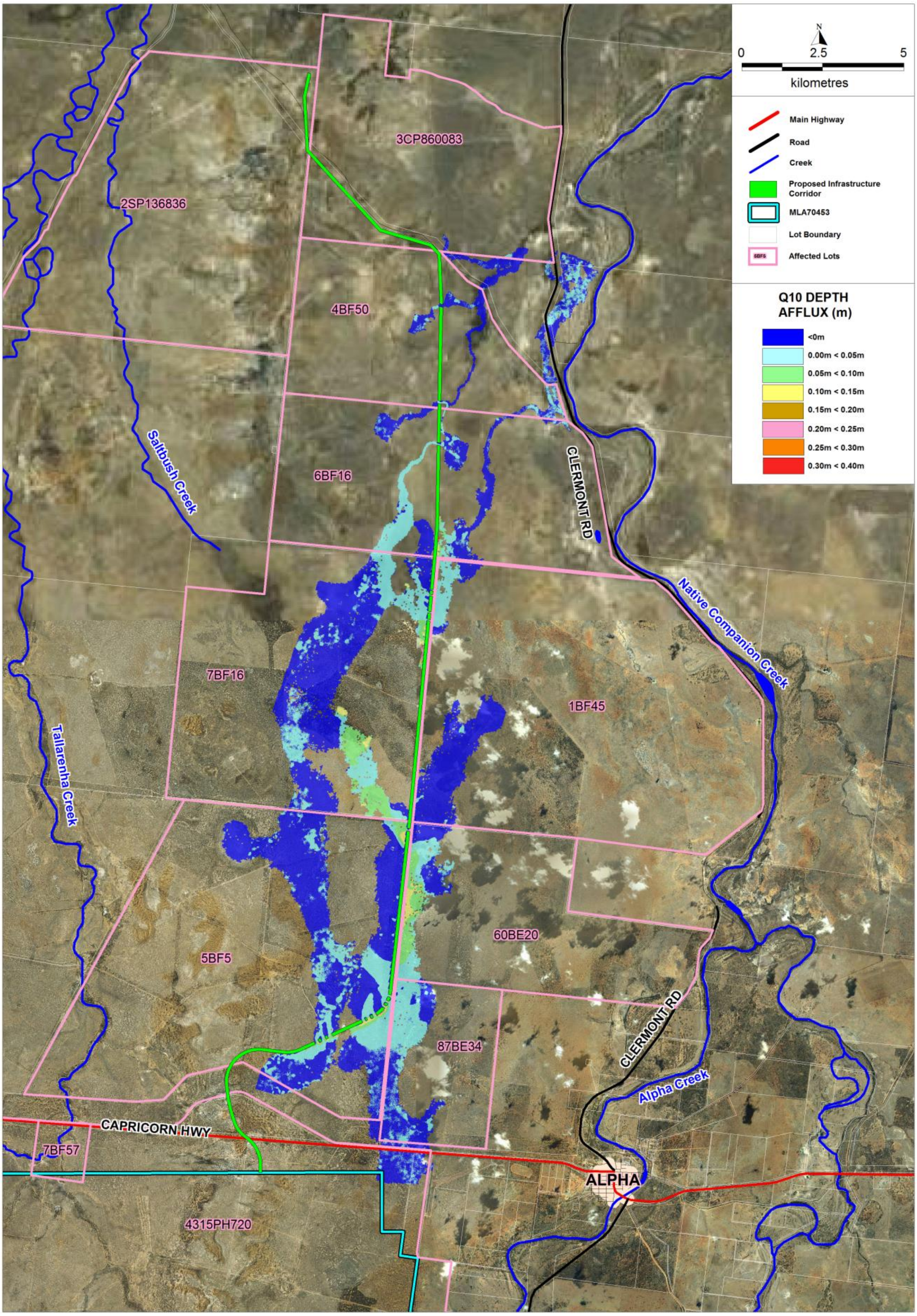

Figure 3.13 Infrastructure Corridor – 10 year ARI Flood Depth Afflux

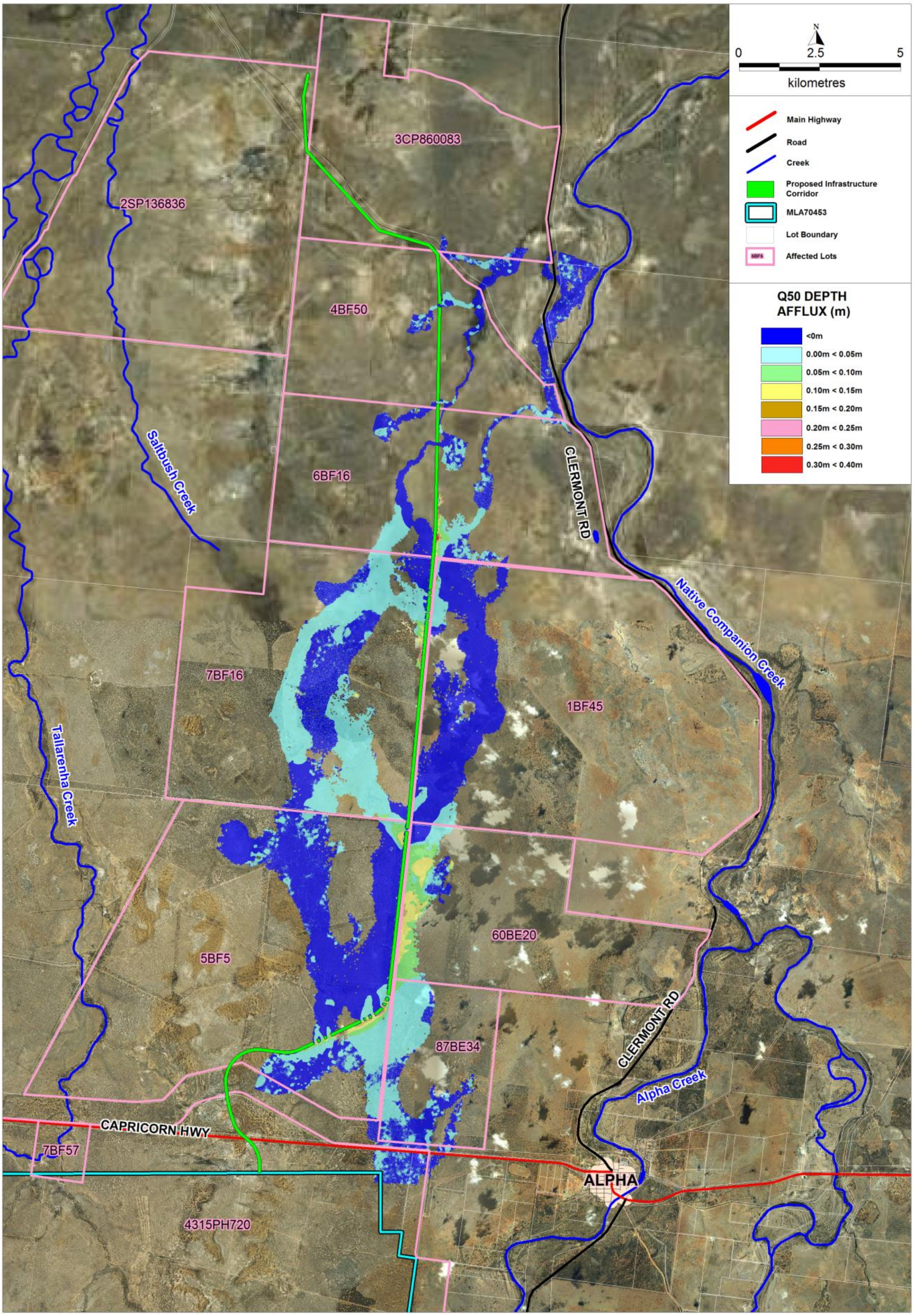

Figure 3.14 Infrastructure Corridor – 50 Year ARI Flood Depth Afflux

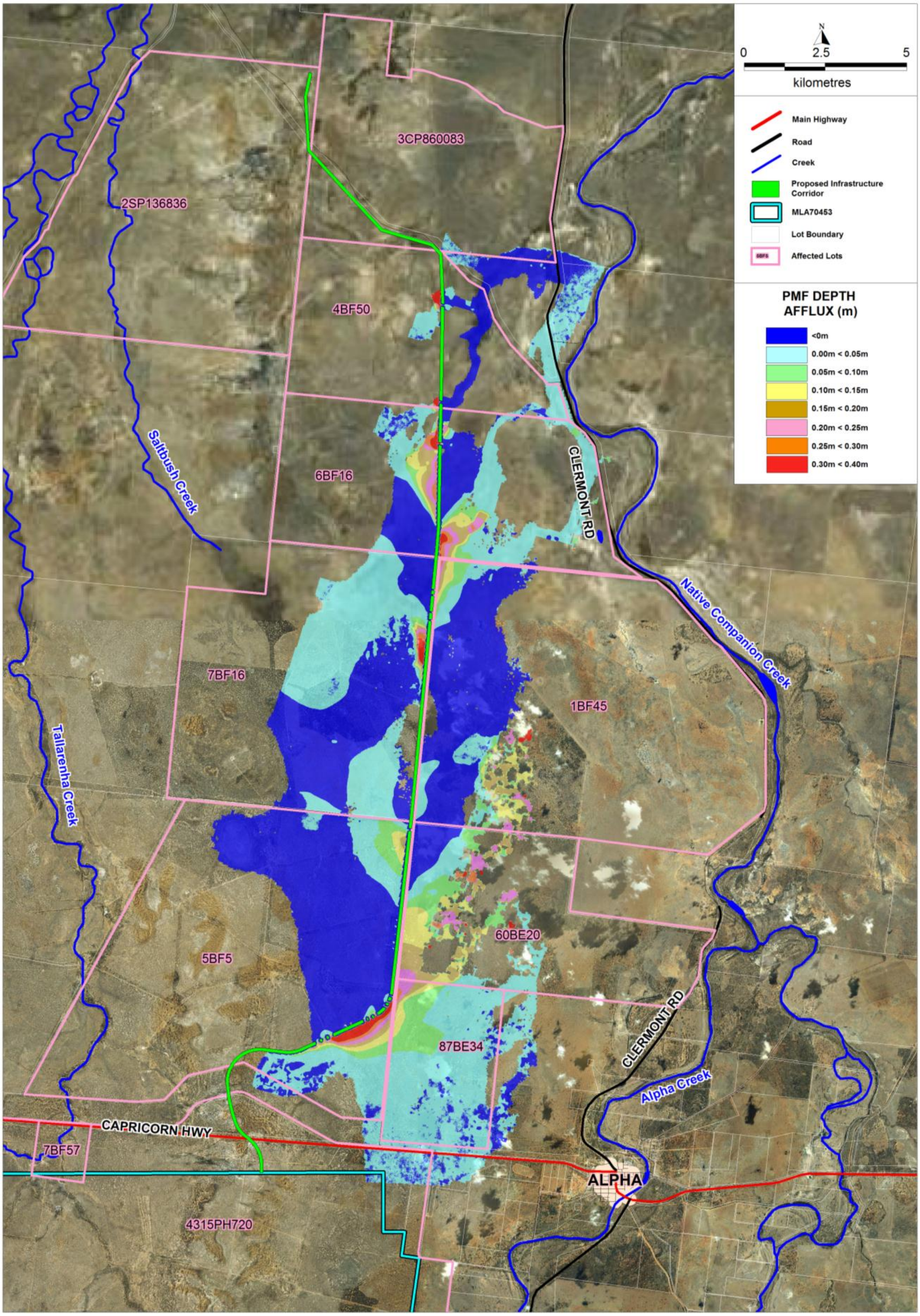

Figure 3.15 Infrastructure Corridor – PMF Flood Depth Afflux