5. Hydrologic analysis

5.1 Overview

Hydrologic modelling is the process of determining runoff generated from rainfall on a catchment. The runoff estimates are then used by the hydraulic analysis, as described in Section 6. Factors affecting the volume and peak of runoff generated include:

- size and slope of the catchment and adjoining channels
- level of development (fraction impervious) and type of catchment land use
- condition of the catchment (dry or saturated) when the rainfall starts
- intensity and temporal pattern of rainfall
- ability of the catchment and other features to store runoff.

Simplistic methods exist for estimating the amount of runoff from a catchment (i.e. peak flow methods like the Rational Method). However, modelling software is necessary to accurately predict the response of large and complex catchments to rainfall over time, and the interaction between sub-catchments. For this assessment, a hydrologic model of the Sandy Creek catchment was developed using the Watershed Bounded Network Model (WBNM) software program.

5.2 Runoff routing model

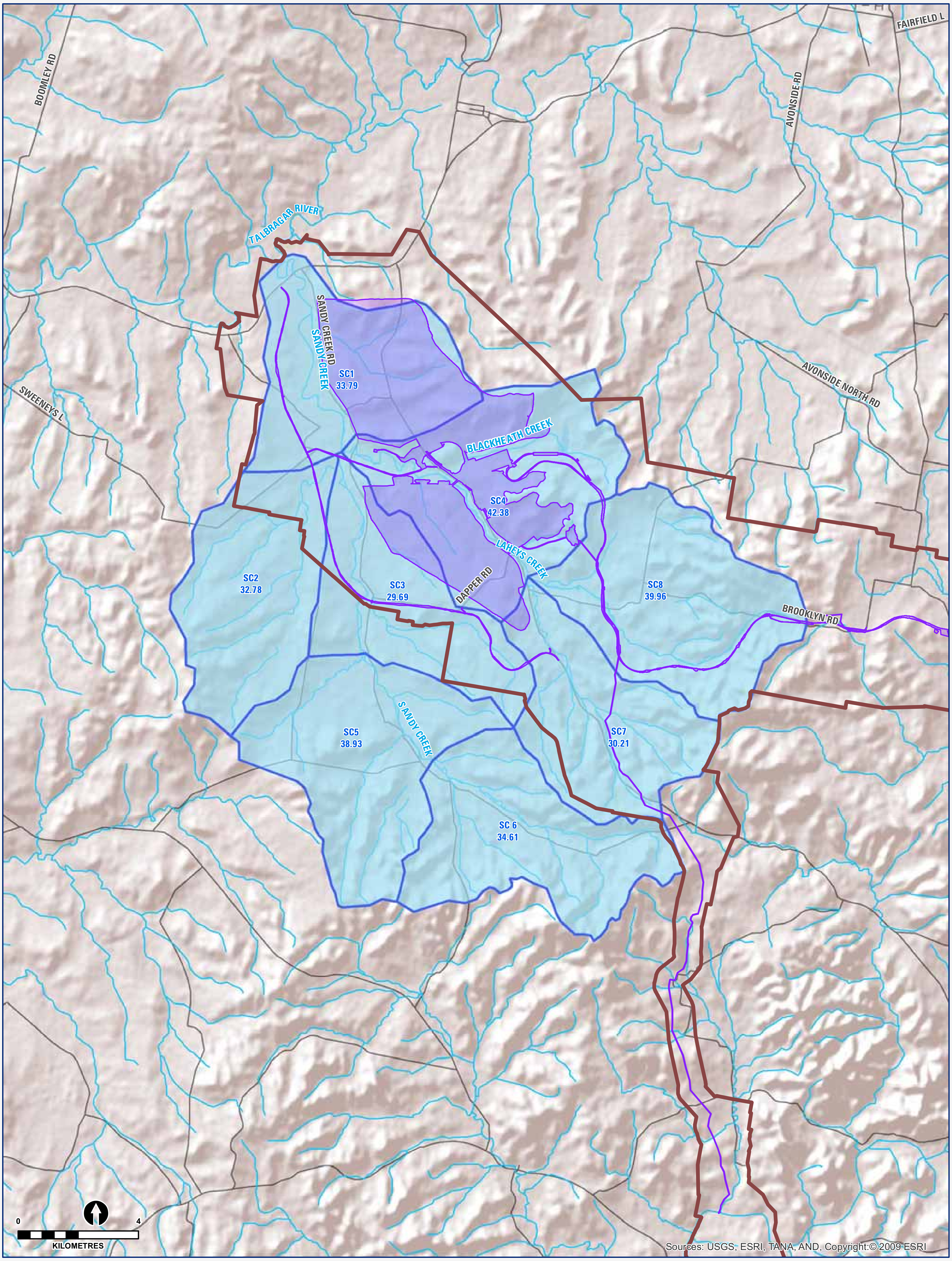
WBNM has been used extensively across NSW for urban and rural flood investigations. WBNM is an event-based hydrologic model that calculates flood hydrographs from either recorded storm rainfall hyetographs or design storm rainfall parameters. The catchment is represented in the model as a series of sub-catchments for which factors affecting runoff, such as land use (proportion of pervious versus impervious land surfaces), rainfall losses, and runoff routing through the catchment and channels, are defined.

Details of how WBNM was used to represent the Sandy Creek catchment are provided below. The model of the Sandy Creek catchment developed for this study was used to estimate flow generated from the catchment for the 2-, 5-, 100- and 2,000-year average recurrence interval (ARI) design storm events to represent a reasonable range of extreme event flood conditions.

5.2.1 Catchment area

The Sandy Creek catchment has a total area of 282.35 km^2. The catchment was divided into eight sub-catchments for greater definition of catchment parameters within the WBNM model. The sub-catchment breakdown is illustrated in Figure 5-1.

Catchment parameters, such as sub-catchment area, percentage imperviousness, sub-catchment links and channel definition within the catchment, were defined using available topographic contour data and a review of aerial photography of the area. The catchment was assumed to be completely pervious based on a review of aerial photography and knowledge of the catchment.



Mine impact area
Drainage lines
Roads
Subcatchments

Figure 5-1 Sandy Creek subcatchments

5.2.2 Design rainfall loss rates

Design rainfall loss rates adopted to represent initial and continuing losses were based on recommendations within *Australian Rainfall and Runoff* (Engineers Australia 2001). Adopted loss values are given in Table 5-1.

Catchment lag parameters and stream lag factors were based on recommendations within WBNM Theory (Boyd et al. 2007). The lag parameter for all sub-catchments was adopted as 1.8. A stream lag factor of 1 was adopted for the natural channels in the catchment.

Table 5-1 Adopted loss values

Storm event	Initial loss (mm)	Continuing loss (mm/h)
2-year ARI	30	2.5
5-year ARI	30	2.5
100-year ARI	20	2.5
2,000-year ARI	0	0

5.2.3 Estimation of design rainfall

Estimation of the design flood hydrographs, using the runoff-routing modelling technique, involves the application of the design rainfall event data. Rainfall-based design flood estimation assumes the probability of the design flood event is the same as the associated design rainfall event.

Design rainfall depths of storm events were calculated by WBNM by proportioning from storm data derived from the intensity frequency duration (IFD) method, as defined in Chapter 2 of *Australian Rainfall and Runoff*, Volume 2 (Engineers Australia 1987, 2001). The IFD parameters adopted for this study and input into the WBNM model developed for the Sandy Creek catchment are shown in Table 5-2.

Table 5-2 Sandy Creek catchment IFD parameters

Variable	Symbol	Value
Rainfall intensity (mm/h) (2-year ARI; 1-hour storm duration)	${}^{2}I_{1}$	26.6
Rainfall intensity (mm/h) (2-year ARI; 12-hour storm duration)	${}^{2}I_{12}$	4.7
Rainfall intensity (mm/h) (2-year ARI; 72-hour storm duration)	${}^{2}I_{72}$	1.18
Rainfall intensity (mm/h) (50-year ARI; 1-hour storm duration)	${}^{50}I_{1}$	50.3
Rainfall intensity (mm/h) (50-year ARI; 12-hour storm duration)	${}^{50}I_{12}$	8.9
Rainfall intensity (mm/h) (50-year ARI; 72-hour storm duration)	${}^{50}I_{72}$	2.3
Average coefficient of skewness	G	0.32
Geographical factor (2-year ARI)	F2	4.33
Geographical factor (50-year ARI)	F50	15.65

5.2.4 2,000-year ARI event

5.2.4.1 Probable maximum precipitation design rainfall

To estimate the 2,000-year ARI design rainfall, the probable maximum precipitation (PMP) rainfall depths need to be calculated. Depths were calculated using the procedures outlined in *The Estimation of Probable Maximum Precipitation in Australia: generalized short duration method* (the GSDM) (Bureau of Meteorology 2003).

Parameters used in the PMP calculations are detailed in Table 5-3. The moisture adjustment factor was determined based on Figure 3 in the GSDM. The elevation adjustment factor was adopted as 1, as the site elevation is below 1,500 mAHD. The roughness parameter used in PMP calculations is a measure of how steep the catchment is. Topographic maps were used to determine this parameter.

Table 5-3 Sandy Creek PMP calculation parameters

Parameter	Value
Moisture adjustment factor	0.71
Elevation adjustment factor	1
Percentage defined as 'rough'	95%

The temporal rainfall distribution for the PMP event was adopted from procedures outlined in the GSDM.

5.2.4.2 2,000-year ARI design rainfall

The 2,000-year ARI rainfall depths were calculated based on procedures documented in *Australian Rainfall and Runoff* (Engineers Australia 2001), which are based on log relationships between the 50-year ARI, 100-year ARI and PMP rainfall depths. The temporal rainfall distribution for the PMP was adopted for the 2,000-year ARI event.

5.3 Model validation

5.3.1 Fitting to flood frequency analysis

To assess the reasonableness of the WBNM estimates, 2-year, 5-year and 100-year ARI peak flows at Sandy Creek were estimated based on the available (albeit limited) stream flow data.

Although the preferred approach for estimating peak flow in Sandy Creek would be to carry out a flood frequency analysis, the flow record at the Medway gauge was considered unsuitable, as it only recorded average daily flows and not daily maximum flows. It was therefore necessary to develop an alternative analysis method that would use the available peak flow data, as estimated from the flood frequency analysis carried out on the Elong Elong gauge data, to validate the Sandy Creek WBNM model outputs.

To correlate a peak flow recorded in the Talbragar River to a peak flow in Sandy Creek, a relationship between annual peak flows in the Talbragar River and Sandy Creek was established. This relationship was based on the key assumption that a peak flow in Sandy Creek would be approximately equivalent to the first peak flow recorded at the Elong Elong gauge during an annual peak flow event. The relationship between first and second peaks at Elong Elong could then be used to derive annual maximum peak flow estimates in Sandy Creek. Figure 5-2 shows that a reasonable correlation can be achieved for this relationship. This relationship was used to estimate peak flows in Sandy Creek.

It should be noted that this approach would tend to overestimate peak flow in Sandy Creek, as the Talbragar River was assumed to experience negligible base flow at the time the flood peak at Sandy Creek would pass the Elong Elong gauge. Overestimation will occur when the gap between the two peaks is narrow, which would mean that flow in the Talbragar River is likely to be higher than baseflow. The shorter the gap between the two peaks, the greater the overestimation.

Thus, in addition to flow estimates using flows obtained from the flood frequency analysis curve at Elong Elong, lower bound estimates using the lower bound 95%ile confidence envelopes of the flood frequency analysis curve were also calculated for comparison. It is expected that the true peak flow in Sandy Creek would be between these two values and was likely to be lower than the estimated value.

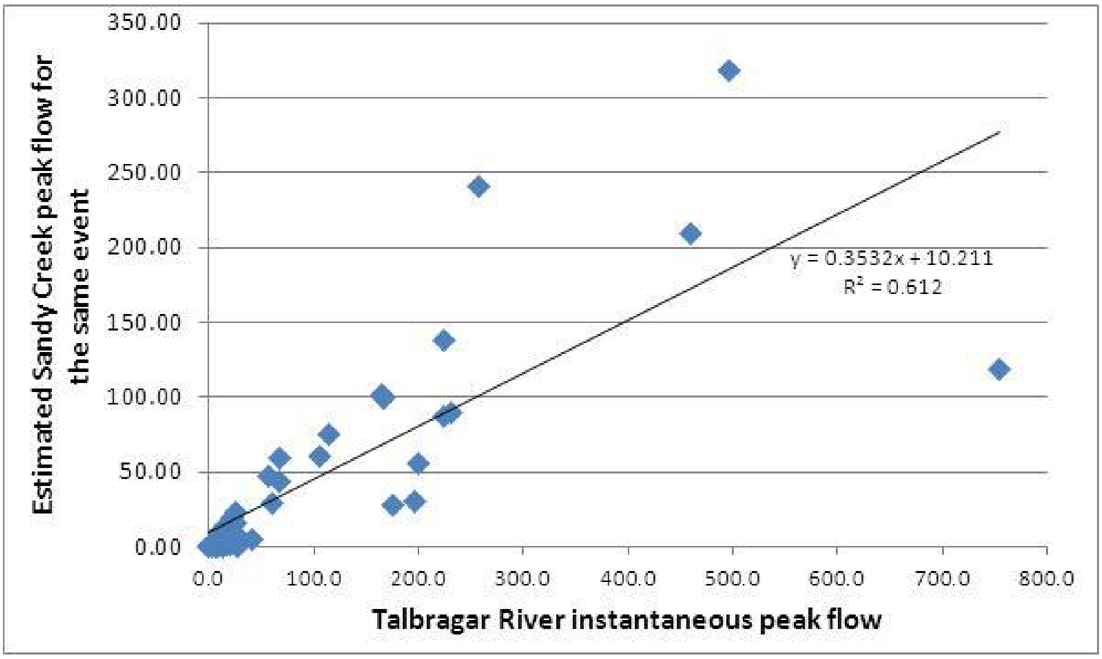


Figure 5-2 Relationship between estimated Sandy Creek peak flow and Talbragar River instantaneous peak flow during annual peak events

The model validation results are summarised in Table 5-4. The modelled peak flows are within the range of estimated peak flows from flood frequency analysis (FFA) of the recorded data on the Talbragar River transposed to Sandy Creek.

Table 5-4 Model validation summary

Event	Estimated range of Sandy Creek peak flow m^3/s (by translation from Talbragar flood frequency analysis)			WBNM modelled peak flow m^3/s
	Estimated value	95%ile lower bound	95%ile upper bound	
2-year ARI	25	20	34	29
5-year ARI	93	47	118	78
100-year ARI	446	231	1,106	429

5.3.2 December 2010 flood event analysis

Water level data was collected at three sites in the Sandy Creek catchment to obtain an understanding of the existing baseflow conditions. As discussed in Section 3.4, the use of these data for high-flow/flooding assessment is limited due to the type of data (i.e. water level, not flow data) and the relatively short time span of the record (November 2009 to February 2012).

The water level gauges at these sites did, however, capture a significant flood event that occurred in December 2010. Using the pluviograph records from the on-site meteorological stations, an attempt was made to verify the hydrologic and hydraulic models against this event.

The Bureau of Meteorology states that 2010 was the third wettest year on record for NSW and had the highest rainfall recorded in the state in over 50 years, following the very strong La Niña events in 1956 and 1950 and slightly higher than the rainfall recorded during strong La Niña events in the 1970s (Bureau of Meteorology 2011). Over the three-day period from 29 November to 1 December 2010, the total rainfall recorded at the on-site meteorological station was 217 mm, with 89 mm recorded on 1 December 2010 (see Figure 5-3).

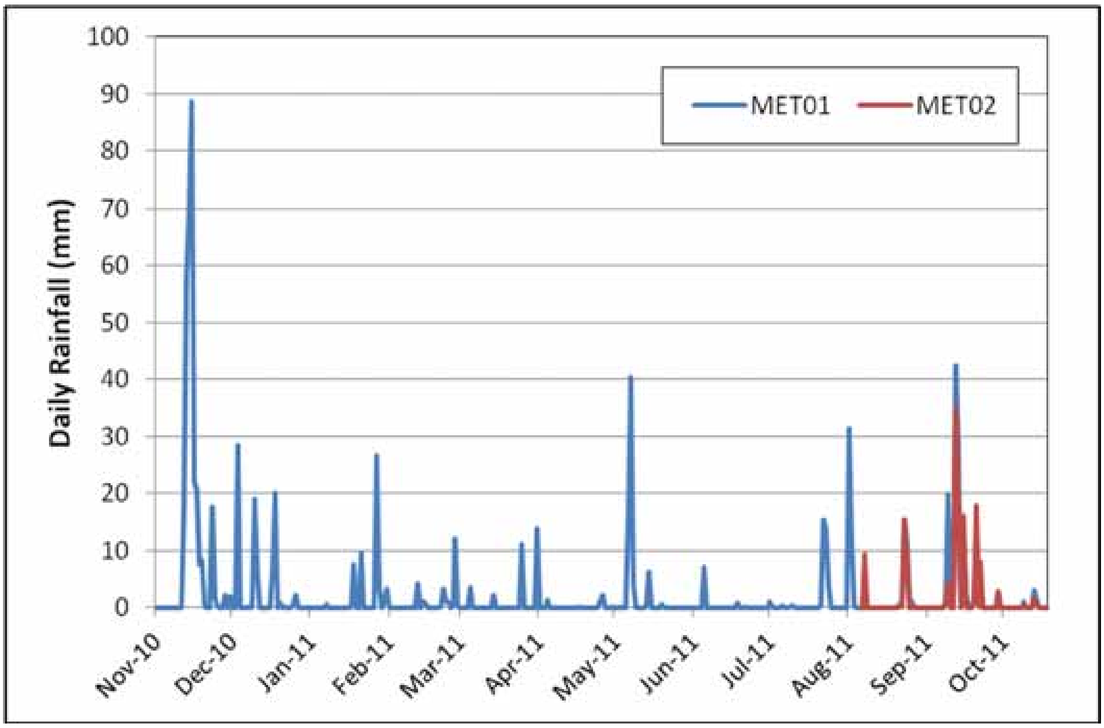


Figure 5-3 Daily rainfall recorded at on-site meteorological weather stations

An analysis of the rainfall recorded at the on-site meteorological stations for the period from 28 November to 6 December 2010 was carried out using the Wollongong University IFD Program. The analysis found that for a duration of 18 hours (which is the critical duration for the Sandy Creek catchment predicted by the WBNM model) the December 2010 event had an intensity comparable to a 20 to 50-year ARI design event. For durations longer than 36 hours and up to 72 hours, the rainfall intensity was in excess of a 100-year ARI design event.

When the WBNM model was run using the December 2010 recorded rainfall, the peak flow predicted at the outlet of the Sandy Creek catchment for this event was reasonably close to the 50-year ARI design flow estimated by the model (i.e. 374 m^3/s predicted for the December 2010 event compared with 340 m^3/s estimated for the 50-year ARI design event).

When the hydraulic model was run using the December 2010 rainfall recorded at the on-site meteorological station, the model over-predicted water levels by 0.8 m at SW2 but was within +/- 0.5 m at SW1 and SW3 compared with the recorded water levels at the gauge sites. The reasons for the model overestimation of peak flood level have not been identified, but are likely to be due to a combination of the following:

- The LiDAR data, which define channel topography in the flood model, have a vertical accuracy of +/- 0.15 m. The LiDAR data also do not always capture a detailed resolution of the low-flow portion of the channel due to the presence of vegetation or a steeply incised low-flow channel.

- The channel surveys at the water level recording stations were surveyed to temporary benchmarks. The survey data were converted to mAHD using an estimation procedure that involved correlating the survey to LiDAR contours; this method is significantly uncertain.

5.3.3 Conclusion

A flood frequency analysis was carried out for the data recorded in the Talbragar River at Elong Elong. It was not possible to perform a flood frequency analysis on the Sandy Creek recorded data due to a very short flow record and lack of suitable data. Since no suitable data are available for peak/maximum flows in Sandy Creek for model calibration (only average daily flows are available), a number of relationships were developed to relate the recorded Talbragar River instantaneous flows to the daily average flows at Sandy Creek, and to relate the first and second peaks recorded during annual flood events in the Talbragar River to the Sandy Creek gauge. It was then possible to relate the flood frequency analysis results at Talbragar River to Sandy Creek.

The WBNM model peak flow estimates lie within the range of the translated flood frequency analysis estimates. This provides a reasonable validation of the WBNM model and confirms that the model is reliable for flood flow estimation in Sandy Creek.

This conclusion is supported by the analysis undertaken for the December 2010 flood event, for which the WBNM model estimated a flow return period comparable to the return period estimated by Bureau of Meteorology based on rainfall records.

Verification of the hydraulic model to the peak water levels recorded during the December 2010 flood event was inconclusive. The model overestimated the peak flood level; however, this could be due to the combined errors in several sources: the LiDAR data used to define topography in the hydraulic model, the channel survey and the measured water level data.

Because the hydrological model has been validated and verified satisfactorily, the flood model was considered to be a reliable tool for estimating flood parameters within the modelled portion of the Sandy Creek catchment.

5.4 Existing case design flows

Peak flows generated within the Sandy Creek catchment are shown in Table 5-5.
These flows have been taken from the outlet of the model (i.e. the downstream end of Sandy Creek, immediately before the junction with the Talbragar River).

Table 5-5 Peak flows at outlet of Sandy Creek (existing case)

Event	Critical duration minutes (hours)	Peak flow (m^3/s)
2-year ARI	1,080 min (18 h)	29
5-year ARI	1,080 min (18 h)	78
100-year ARI	1,080 min (18 h)	429
2,000-year ARI	360 min (6 h)	1,354

5.5 Proposed case design flows

The proposed case hydrology was based on the surface water catchments contained in the surface water management plan developed as part of the *Water Balance and Water Management System* report (refer to Appendix E of *Surface Water Assessment* report).

The surface water management plans for Year 1, Year 4, Year 12, Year 16 and Year 20 were compared in relation to disturbed areas, active-mining area, catchment area lost, number of diversions and overflow from clean-water dams. Year 21 represents the final rehabilitated landform. In particular, the latter stages of the active mine — Years 12, 16 and 20 — were closely analysed.

Based on this assessment, it was concluded that the Year 20 mine plan represents the maximum disturbed catchment area during active mining since it has the largest area of catchment diverted from Sandy Creek into Laheys Creek and the largest rehabilitated areas.

At Year 20, parts of the lower catchment of Sandy Creek will be diverted upstream of the junction of Laheys Creek with Sandy Creek. The catchment area of the lower Sandy Creek will be decreased by this diversion and by the removal of catchment due to open-mining areas. As a result, the catchment area discharging into the middle section of Laheys Creek will be increased. However, overall the total catchment area of the Sandy Creek catchment will be decreased due to the active-mining areas and areas draining into the mine.

At Year 20, approximately 1,270 ha of catchment area will be lost to active-mining areas or diverted outside the Sandy Creek catchment. For 86 ha of the diverted catchment, surface water will be diverted north along the edge of mining area C to the catchment of Flyblowers Creek which joins the Talbragar River less than 1 km upstream of the Sandy Creek confluence. This is discussed in further detail in Section 7.1.2.

Year 20 was identified as the critical mine stage for flooding since it will have the highest flows entering the creeks due to the catchment diversions described above.

Catchment losses were adjusted based on the portion of disturbed areas, which included rehabilitated spoil, coal stockpiles, out-of-pit emplacements and cleared areas.
Percent impervious levels were also calculated based on the area of haul roads, rail spur and paved mine infrastructure areas in each of the eight existing sub-catchments.

Of these eight, only four sub-catchments experienced changes to catchment characteristics. Only two of the sub-catchments (SC1 and SC4) experienced major changes in catchment characteristics. Percent impervious increased in most of the affected sub-catchments.

Table 5-6 summarises the peak flows at the outlet of Sandy Creek based on the Year 20 catchments. It was found that the critical duration for the 2-year ARI event at some locations around the mine infrastructure area differs from the critical storm duration of 1,080 minutes under existing conditions.

The local critical duration is 360 minutes and the flows corresponding to this storm duration were used at these locations for the hydraulic analysis in parts of Laheys Creek. The overall critical duration in the Sandy Creek catchment remains the same as existing conditions (i.e. the 1,080-minute storm). For other ARI events, the critical duration for the proposed case was the same as the existing case.

Table 5-6 Proposed case peak flows at outlet of Sandy Creek

Event	Critical duration in minutes (hours)	Peak flow (m^3/s)
2-year ARI	1,080 min (18 h)	31
5-year ARI	1,080 min (18 h)	83
100-year ARI	1,080 min (18 h)	432
2,000-year ARI	360 min (6 h)	1,355

6. Hydraulic analysis

6.1 Hydaulic models

The main hydraulic model used in the assessment was a HEC-RAS model developed for Sandy Creek including its main tributaries — Laheys Creek and Blackheath Creek — as well as numerous smaller unnamed tributaries. The model extends from near the junction of Sandy Creek and the Talbragar River, about 25 km upstream along Sandy Creek and includes approximately 15 km of Laheys Creek. The model extent is shown in Figure 6-1.

In addition, two smaller localised HEC-RAS models were developed for significant creek crossings of Fords and Tallawang Creeks along the rail spur outside of the Sandy Creek catchment.

HEC-RAS is a one-dimensional hydraulic model that can simulate steady or unsteady flow in rivers and open channels. The river channel and floodplain is represented in HEC-RAS as a series of topographic cross-sections. The model can assess the effects of obstructions, such as bridges, culverts, weirs, and structures in the channel and floodplain.

6.1.1 Cross-section geometry development

6.1.1.1 Sandy Creek catchment model

A digital terrain model covering the extent of the hydraulic model was constructed using LiDAR data and 1 m contours provided by CHC (see Section 3.1).

Cross-sections of the river channel and floodplain were extracted from the digital terrain model every 300 m to 400 m along the 25 km length of Sandy Creek, Laheys Creek and minor tributaries. Cross-sections varied in width from about 300 m to 700 m depending on the depth and size of the channel and width of floodplain. Figure 6-1 shows the modelled reaches in red and model cross-sections in pink.

Cross-sections at three gauging locations (SW1, SW2 and SW3) were surveyed in November 2011. For each of the three locations, cross-section surveys were undertaken at the control (i.e. the road causeway) and at the pool where the gauge is located. These cross-section surveys aim to measure low- to medium-range flow, so their applicability to flood modelling is limited. However, the cross-sections at each of the surface water gauge locations were added into the HEC-RAS model for the purposes of model verification (see Section 5.3).

6.1.1.2 Fords Creek and Tallawang Creek models

For the localised models of the rail crossings of Fords Creek and Tallawang Creek, the cross-sections of the channels and floodplains were extracted from the same digital terrain model at intervals of 100 to 250m, with closer cross section intervals in the areas of the proposed crossing structures.

6.1.2 Boundary conditions

6.1.2.1 Sandy Creek catchment model

Inflows were assigned to reaches of the hydraulic model for each stream/tributary, based on the flow outputs of the hydrologic model.

The downstream limit of the hydraulic model is located at the confluence of Sandy Creek with the Talbragar River, approximately 600 m north-west of the Golden Highway. The model was run using a normal-depth boundary based on a downstream bed slope of 0.32%.

The use of a normal-depth boundary to create a rating curve was based on the assumption that when the Sandy Creek peak reaches the Talbragar River, the Talbragar River is flowing under normal baseflow conditions. This assumption can be justified by the observation that peak flows at the Talbragar River tend to reach the confluence of the two water bodies about one to two days after the Sandy Creek peak flow has passed through the confluence.

A sensitivity analysis was undertaken using a high water level of 348 mAHD, which generates a high water depth of 6.5 m at the downstream boundary. Under these conditions, which test the scenario of extreme flooding in the Talbragar River, it was found that water levels in the lower 2.7 km of Sandy Creek only are affected by the Talbragar River level. This indicates that flooding in the Talbragar River does not affect water levels in the middle and upper reaches of Sandy Creek. Thus, the scenario of flooding in the Talbragar River was not considered further for this study.

6.1.2.2 Fords Creek and Tallawang Creek models

The catchment area of Fords Creek draining to the rail crossing is approximately 23 km^2; this includes the Fords Creek system that flows east to west along the northern edge of the proposed railway. Although this is a significant creek system, the topography shows that there is no defined channel shape in this location. It is expected that multiple cross-drainage culverts will be required at this location depending on the extent of the floodplain in this area. Given the relatively flat topography, it is envisaged that standard box/pipe culverts and associated scour protection measures will be suitable for the cross-drainage system. A preliminary design for this crossing is presented in Section 6.3.2.

The Tallawang Creek crossing is located approximately 2 km west of the location where the proposed rail spur ties into the existing railway line. At this location, the rail spur is proposed to cross Tallawang Creek and one of its tributaries about 400 m upstream of their confluence. The total upstream catchment area draining to this location is about 44 km^2. There are two deeply incised channels at this crossing location, each with a channel depth of 4–5 m and a width of 40–50 m. Two separate bridges spanning each of the channels will be required at this location. A preliminary design for this crossing is presented in Section 6.3.2.

Design flows for Fords and Tallawang Creeks were estimated using rainfall runoff models (XP-RAFTS) and design parameters recommended in Australian Rainfall and Runoff (Engineers Australia 2001) and Bureau of Meteorology (BoM 2003). The estimated design flows are shown in Table 6-1.

Table 6-1 Design flows for Fords Creek and Tallawang Creek rail crossings

Catchment	Design peak flow (m^3/s)	
	100 year ARI	PMF
Fords Creek	53	909
Tallawang Creek west channel	40	640
Tallawang Creek east channel	45	735
Tallawang Creek total	85	1375

Downstream boundary conditions were set using a normal depth boundary at sufficient distance downstream of the crossing location so that water levels at the boundary did not affect those computed by the models at the crossing locations.

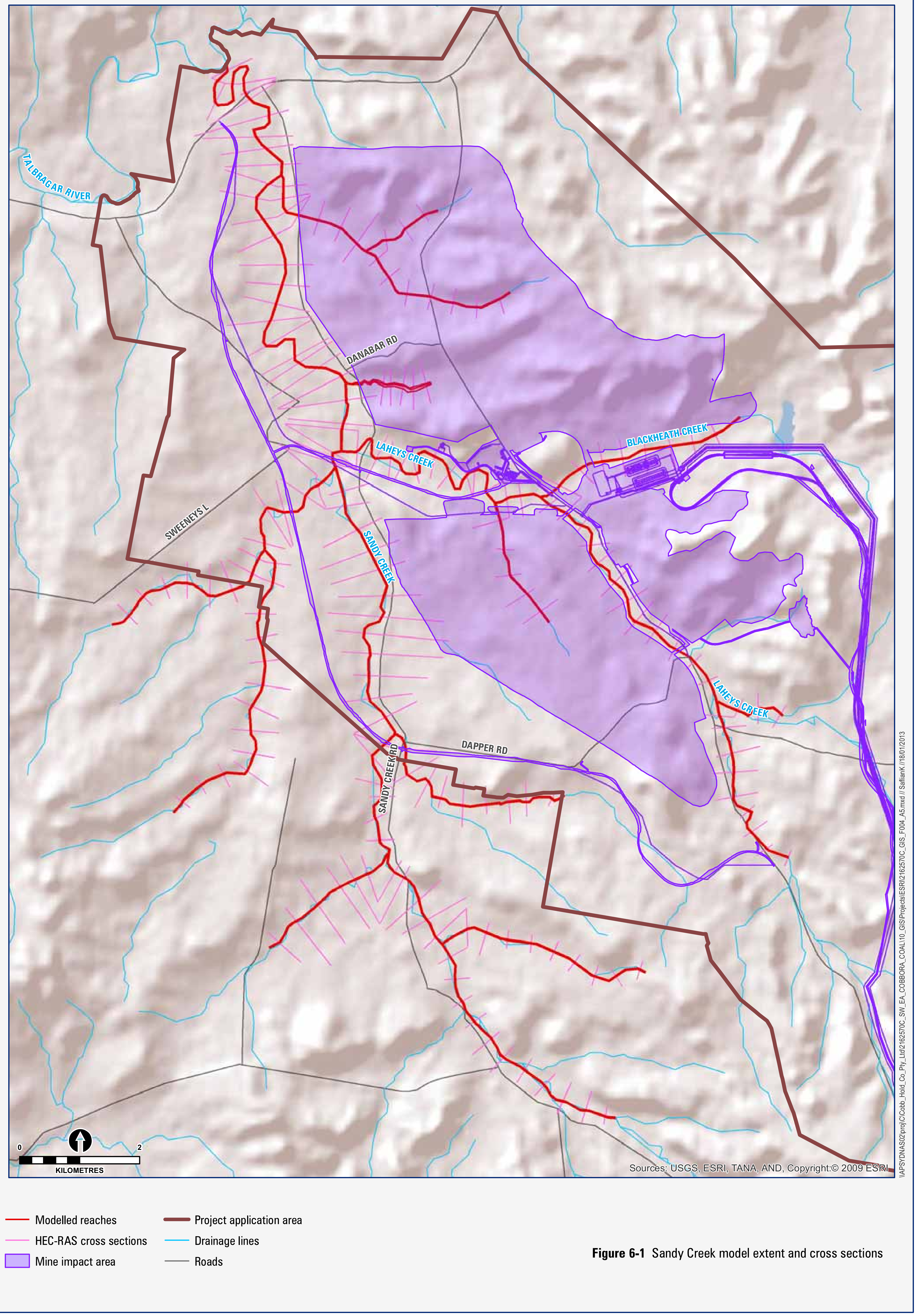


Figure 6-1 Sandy Creek model extent and cross sections

6.1.3 Roughness

Manning's *n* roughness parameters are used to represent the type of channel and varying land cover across a floodplain to allow the model to simulate changes in flow behaviour as water crosses different surfaces. Each cross-section is assigned Manning's *n* roughness values based on the channel characteristics and land cover across the floodplain.
The Manning's *n* values adopted for the modelled channels and floodplain sections were based on knowledge of the site developed during site inspections, aerial photograph interpretation and engineering judgement and experience.

The predominant Manning's *n* values adopted in the hydraulic model for the main channel and floodplain are given in Table 6-2.

Table 6-2 Adopted roughness values for Sandy Creek

Terrain type	Manning's *n*
Floodplain, short grass, meandering	0.054
Floodplain, medium grass, shrubs, trees, meandering	0.09
Existing creek channel, reeds, shrubs, meandering	0.05 to 0.08

6.2 Modelled scenarios

6.2.1 Sandy Creek catchment model

The model was run for the 2-, 5-, 100- and 2,000-year average recurrence interval (ARI) events for two catchment scenarios:

- The existing scenario, which represents the current state of the Sandy Creek catchment based on LiDAR data collected in July 2009.
- The proposed scenario, which incorporates worst-case mine development for flooding (i.e. Year 20) and associated mitigation measures. The proposed scenario represents the mine infrastructure area, mining areas, out-of-pit emplacement areas and sedimentation basins within the floodplain. The proposed scenario model has taken into account areas where levees and erosion protection will be needed to protect vulnerable infrastructure from inundation and erosion during extreme flood events.

6.2.2 Fords Creek and Tallawang Creek models

These models were run for the 100 year ARI and PMF events for:

- The existing scenario, which represents the current state of the catchments based on the LiDAR data.
- The proposed scenario, which incorporates the rail spur embankment and associated creek crossing structures.

6.3 Modelled structures

6.3.1 Existing scenario

For the existing scenario, no structures were included in the HEC-RAS hydraulic model. Although aerial photographs and site inspections indicate that structures currently do exist where watercourses intersect with roads, these are generally low-level causeways. There is limited survey information on the location, size and height of these low-level causeways and other crossing structures, such as culverts.

During flood events it is expected that the causeways and culverts will be fully submerged and small culverts will be blocked with debris and vegetation. Therefore, the hydraulic influence of these structures under extreme flood events will be insignificant, and the existing HEC-RAS model has not included them.

6.3.2 Proposed scenario

6.3.2.1 Watercourse crossings

The proposed scenario has included bridges that will be required where the proposed mine access roads and haul roads cross watercourses within the Sandy Creek catchment, and rail crossings of significant size in Fords Creek and Tallawang Creek catchments. All potential crossings have been identified and are numbered with their locations given below. Of these, all of the Zone 1 crossings and two of the Zone 3 crossings (crossings 11 and 28) have been subject to hydraulic assessment and design.

Locations of the watercourse crossings are indicated in Figure 6-6 for the Sandy Creek catchment (Zone 1) and in Figure 6-2 for the rail corridor (Zone 3). There are no watercourse crossings in Zone 2.

Appendix D.3 contains schematics, taken from the hydraulic models, of each of the watercourse crossings that were sized and modelled hydraulically.

Sandy Creek Mining Area (Zone 1)

1. Northern access road crossing Sandy Creek.
2. Northern access road crossing Laheys Creek.
3. Haul road crossing Laheys Creek.
4. Haul road crossing Blackheath Creek (east/upstream).
5. Haul road crossing Blackheath Creek (west/downstream) and CHPP access road (roads in parallel so sized together).
6. Haul road crossing Laheys Creek (south) (Year 4 only).
7. Realigned Spring Ridge Road crossing Sandy Creek (north)
8. (a) Realigned Spring Ridge Road crossing Sandy Creek (south), (b) Realigned Spring Ridge Road crossing tributary.

Rail corridor (Zone 3)

9. Rail crossing minor gully/drainage line running south-east.
10. Rail crossing tributary draining into Fords Creek running south-west.

11. Rail crossing Fords Creek, which flows along the northern edge of the proposed railway.
12. Rail crossing small gully draining north to Fords Creek.
13. Rail crossing existing dam capturing flow from small gully draining north to Fords Creek.
14. Rail crossing small gully draining north to Fords Creek.
15. Rail crossing existing dam capturing flow from small gully draining north to Fords Creek.
16. Rail crossing small gully draining north to Fords Creek.
17. Rail crossing small gully draining north to Fords Creek.
18. Rail crossing small gully draining north to Fords Creek, and crossing existing dam capturing flow from small gully draining north to Fords Creek.
19. Rail crossing existing dam capturing flow from small gully draining north to Fords Creek.
20. Rail crossing small gully draining north-west to Fords Creek.
21. Rail crossing small gully draining north-west to Fords Creek.
22. Rail crossing small gully draining north-west to Fords Creek, and crossing existing dam capturing flow from small gully draining north-west to Fords Creek.
23. Rail crossing small gully draining north-west to Fords Creek.
24. Rail crossing the upper reach of Lambing Yard Creek when it flows north-east.
25. Rail crossing Lambing Yard Creek after it loops around underneath the rail spur to then flow south-east (approximately 700 m east of crossing 25).
26. Rail crossing local low point where overland flow running north-east from upstream ridge line needs to be conveyed across the rail corridor.
27. Rail crossing small tributary running north into Tallawang Creek system.
28. Rail crossing Tallawang Creek, spanning two deeply incised channels that run close to each other and drain south.
29. Rail crossing small tributary running south into Tallawang Creek.

An iterative process was undertaken in consultation with the CHC infrastructure team to develop preliminary designs of some of the key crossings in the catchments in terms of crossing type, dimensions and invert levels of openings, clearance above flood levels and other roads, and levels of approach roads across floodplains.

Bridge spans and openings for the access and haul roads were generally sized to provide clearance (with 600mm freeboard) above the 100-year average recurrence interval (ARI) event and to avoid excessive afflux under these conditions. Afflux up to 500mm on land owned by CHC and within or adjacent to the main mining areas was considered acceptable as these areas will be protected by mine flood protection levees which will contain and control flooding under extreme events. Any significant afflux on land not owned by CHC was considered unacceptable and was avoided by ensuring the crossing designs did not cause afflux beyond CHC's land ownership.

Slightly different design criteria were adopted for three crossings on the realigned Spring Ridge Road – Crossings 7, 8a and 8b. The existing crossings at these locations are flooded at the 10 year ARI event. The modified crossings at these locations were designed to be serviceable in the 50 year ARI event rather than the 100 year ARI event to reduce the height of the road embankment and associated impacts on flood levels upstream. While the standard of flood protection for these crossings differed from the other road crossings, the same afflux criteria as described above were adopted in the design.

Crossing 6 is a high level haul road that crosses Laheys Creek and the existing Spring Ridge Road. This haul road will be constructed at a high level that will be clear of the 2,000 year ARI flood level and will therefore not have any afflux impacts.

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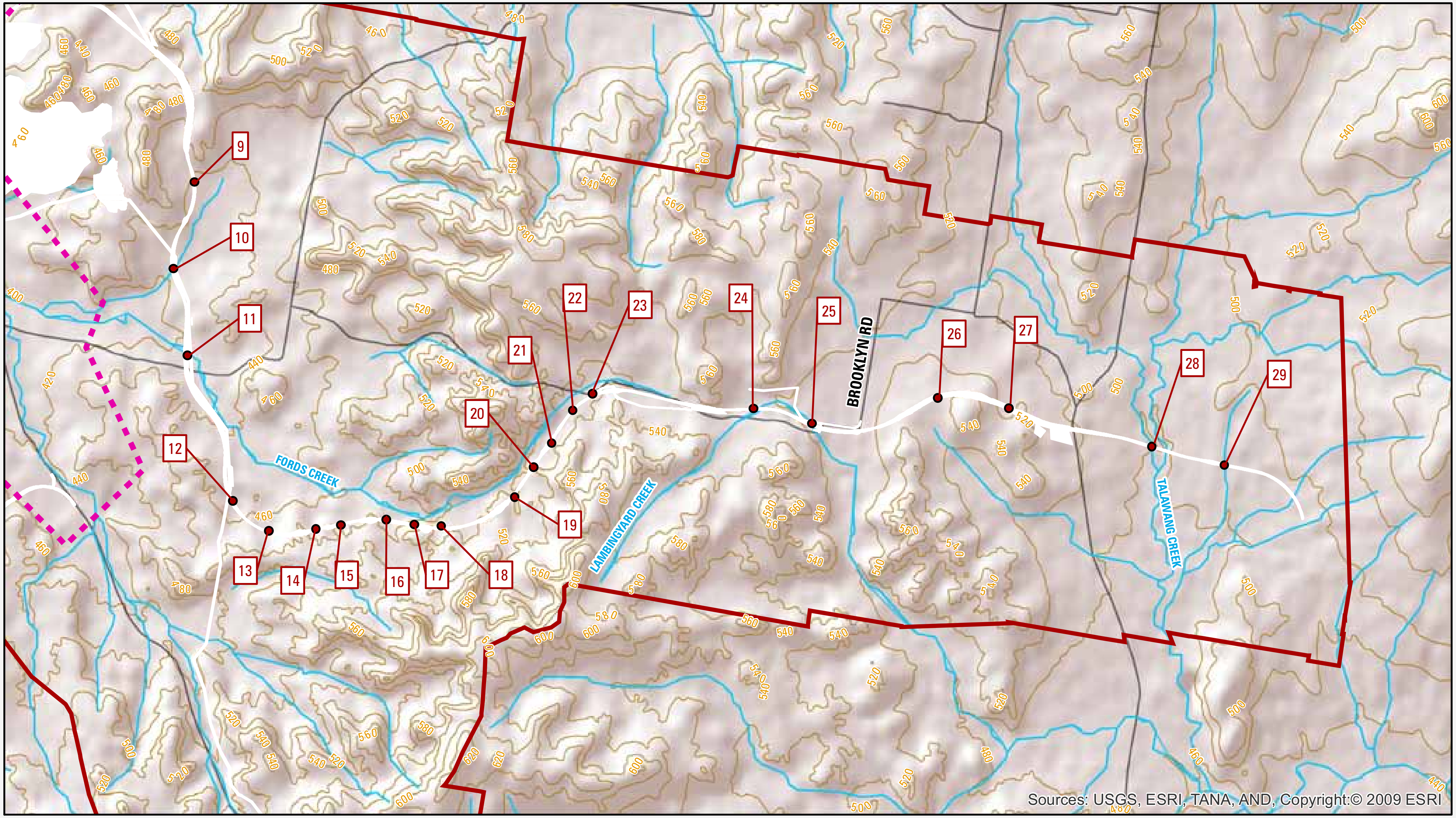
CHC PTY LTD



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Project application area
Mine impact area
Drainage lines
Roads
Elevation
1 Waterway crossings

0 3,500
Metres

Figure 6-2 Waterway crossings along rail spur

For the rail crossings the following preliminary design criteria were adopted:

- All structures are to be designed to pass 100 year ARI flood without overtopping of the rail embankment.
- Afflux under the 100 year ARI event to be limited to no more than 500 mm immediately upstream of the crossing, and reducing to no more than 100 mm within a distance of 200 m upstream of the crossing.
- Soffit of bridge structures to be a minimum of 600 mm above the 100yr ARI flood level.

Although afflux was minimised as much as possible through the preliminary design of the crossings, some degree of afflux was accepted since any increased flooding was limited to land within the mine ownership and could therefore be managed through appropriate flood mitigation measures, such as levees and scour protection.

Table 6-3 outlines watercourse crossing details and the assumptions for those included in the hydraulic model.

Table 6-3 Preliminary design of key watercourse crossings

Crossing number	Description	Assumed dimensions	Total opening width	Key levels i.e. culvert invert, soffit level, deck level
1	Bridge with piers	3 x spans	40 m	Soffit 367.6 mAHD Deck 369.1 mAHD
2	Bridge with piers with flood relief culverts	2 x spans 2 x 3m x 3m box culverts	40 m	Soffit 384.5 mAHD Deck 387.0 mAHD
3	Arch bridge with piers	5 x spans plus 1 x high-flow arch	67m	Soffit 389.9 mAHD Deck 392.0 mAHD
4	Box culverts	3 x 3.3m x 2.1m box culverts	14m	Soffit 388.7mAHD Deck 405.0 mAHD
5	Pipe culverts	8 x 1.35m dia culverts	36 m	Soffit 384.8 mAHD Deck 386.9 mAHD
6	Arch bridge	4 x arches plus 1 x high-flow arch	55 m	Soffit 402.4 mAHD Deck 406.0 mAHD
7	Pipe culverts	9 x 2.55m dia culverts	28m	Soffit 373.9 mAHD Deck 375.8 mAHD
8a	Pipe culverts	7 x 2.55m dia culverts	18 m	Soffit 386.5 mAHD Deck 389.4 mAHD
8b	Pipe culverts	4 x 1.5m dia culverts	8m	Soffit 388.5 mAHD Deck 391.5 mAHD
11	Pipe culverts	7 x 2.55m dia culverts	22 m	Soffit 430.3 mAHD Deck 432.05 mAHD
28	Bridge with piers	4 x spans	64 m	Soffit 476.8 to 477.5 mAHD Deck 479.1 to 479.9 mAHD

Table 6-4 lists the crossings identified along the proposed rail corridor, including a description and an indicative type of crossing structure. A crossing structure is required at most locations where the proposed rail spur is constructed in fill over a topographic depression to avoid flooding impacts in drainage lines and overland flow paths upstream of the rail embankment. Table 6-4 also indicates the average height of fill required at each crossing location, based on the proposed rail spur design levels.

At least two of the crossings (Crossing 20 and Crossing 21) may require special drop structures at the outlets to reduce scour impacts in the downstream channel due to the large drops across the rail corridor and associated high outlet velocities.

The remainder of the crossings are located where the proposed rail spur crosses small creeks or gullies/drainage lines. These crossings have not been modelled hydraulically as the crossings are located within small catchments and are expected to be minor crossings that can be achieved by standard box/pipe culverts. Flood flows in these areas are likely to be relatively low, with high velocity and low flow depth. Typical scour protection measures will be required at culvert inlets and outlets at these locations.

Table 6-4 Proposed watercourse crossings within rail corridor

Crossing number	Description	Type of structure required	Average height of fill (above ground level)	Additional comments/observations
9	Minor crossing of gully draining small catchment.	Standard culvert with associated erosion protection measures.	5 m	Very small farm dam located about 30 m downstream of proposed rail spur; may need to be removed.
10	Minor crossing of tributary catchment draining to Fords Creek.	Standard culvert with associated erosion protection measures.	5 m	Due to the flat topography a relatively wide and shallow floodplain may result at this location. Erosion and scour are evident at this location (based on aerial photography).
11	Crossing of Fords Creek.	Multiple cross-drainage culverts will be required to convey flow across the rail corridor in this area due to the relatively flat topography.	7 m	Due to the flat topography a relatively wide and shallow floodplain may result at this location.
12	Minor crossing of gully draining small catchment to Fords Creek.	Standard culvert with associated erosion protection measures.	1.5 m	Narrow channel, so small floodplain width and high velocities are expected at this location.
13	Rail spur located downstream of existing farm dam (within study area) draining small catchment. Dam overflows to Fords Creek.	Existing dam may need to be replaced by a standard culvert to convey flows across the rail corridor so that flows do not build up behind the rail embankment.	0.4 m	Erosion is evident downstream of the dam (based on aerial photography).
14	Minor crossing of gully draining small catchment to Fords Creek.	Multiple cross-drainage culverts are likely to be required to convey flow across the rail corridor in this area due to the relatively flat topography.	3 m	Due to the flat topography, a relatively wide and shallow floodplain may result at this location.

Crossing number	Description	Type of structure required	Average height of fill (above ground level)	Additional comments/observations
15	Rail spur located downstream of existing farm dam (within study area) draining small catchment; dam overflows to Fords Creek	Existing dam may need to be replaced by a standard culvert to convey flows across the rail corridor so that flows do not build up behind the rail embankment.	2 m	
16	Minor crossing of gully tributary draining to Fords Creek.	Standard culvert with associated erosion protection measures.	5 m	Narrow channel, so small floodplain width and high velocities are expected at this location.
17	Minor crossing of gully tributary draining to Fords Creek.	Standard culvert with associated erosion protection measures.	1.5 m	Narrow channel, so small floodplain width and high velocities are expected at this location.
18	Rail spur located upstream of existing farm dam (within study area) draining small catchment. Dam overflows to Fords Creek.	Standard culvert with associated erosion protection measures.	1 m	Existing dam may need to be removed so that flows don't build up behind the rail embankment upstream of the dam.
19	Rail spur located downstream of existing farm dam (within study area) draining small catchment. Dam overflows to Fords Creek.	Existing dam may need to be replaced by a standard culvert to convey flows across the rail corridor so that flows do not build up behind the rail embankment.	6 m	
20	Crossing of creek, with scoured banks draining small catchment to Fords Creek.	Standard culvert design may not be adequate. A drop structure may be required to reduce velocities and erosion.	0.5 m	Catchment is steep (3.5%) and velocities are expected to be high. Erosion and scour are evident at this location (based on aerial photography).
21	Crossing of creek, with scoured banks draining small catchment to Fords Creek.	Standard culvert design may not be adequate. A drop structure may be required to reduce velocities and erosion.	1.5 m	Catchment is steep (5.5%) and velocities are expected to be high. Erosion is evident at this location with cut-out gully shape (based on aerial photography).
22	Rail spur located upstream of existing farm dam (within study area) draining small catchment. Dam overflows to Fords Creek.	Standard culvert with associated erosion protection measures.	2 m	Existing dam may need to be removed so that flows don't build up behind the rail embankment upstream of the dam.

Crossing number	Description	Type of structure required	Average height of fill (above ground level)	Additional comments/observations
23	Minor crossing of gully draining small catchment to Fords Creek.	Multiple cross-drainage culverts are likely to be required to convey flow across the rail corridor in this area due to the relatively flat topography.	2 m	Due to the flat topography a relatively wide and shallow floodplain may result at this location.
24	Minor crossing of Lambing Yard Creek.	Multiple cross-drainage culverts are likely to be required to convey flow across the rail corridor in this area due to the relatively flat topography.	1.5 m	Due to the flat topography a relatively wide and shallow floodplain may result at this location.
25	Minor crossing of Lambing Yard Creek.	Multiple cross-drainage culverts are likely to be required to convey flow across the rail corridor in this area due to the relatively flat topography.	7 m	Due to the flat topography a relatively wide and shallow floodplain may result at this location.
26	Local low point.	Multiple cross-drainage culverts are likely to be required to convey flow across the rail corridor in this area due to the relatively flat topography.	2 m	This crossing will be required, to prevent local build-up of overland flow from upstream ridge line behind rail embankment.
27	Minor crossing of tributary draining to Tallawang Creek system.	Standard culvert with associated erosion protection measures.	1 m	
28	Major crossing of Tallawang Creek system.	West:3x15m spans East:4x15m spans	5 m	Two bridges will be required, with a rail embankment in between.
29	Minor crossing of small tributary draining to Tallawang Creek.	Standard culvert with associated erosion protection measures.	3 m	

6.3.2.2 Proposed mining infrastructure and mining areas

The proposed scenario hydraulic model represents a worst-case scenario. It adopted the maximum envelope of floodplain encroachment of all stages of the mine plan from spoil emplacements, access roads, haul roads, coal stockpiles and sedimentation dams. If the impacts are to be assessed based on this worst-case, then it can be assumed that during each stage of the mine plan, impacts will be equal to or less than those predicted for the worst-case scenario.

During certain stages of mining, temporary diversion channels will be required to redirect tributaries and creeks around the mine workings. Diversion channels were considered conceptually for the impact assessment but were not included in the hydraulic model. Diversion channels are discussed in Section 7.3.4.

Some reaches of Laheys Creek and Blackheath Creek will require flood protection levees to prevent flooding and damage to vulnerable infrastructure, such as coal stockpiles, infrastructure areas and spoil embankments. For the proposed scenario it has been assumed that flood levees will be provided in areas where the edge of the active-mining void is located on or within the boundary of the 2,000-year flood extent. This event was chosen as a reasonable upper limit reference event for protection of critical infrastructure. The standard of protection for critical infrastructure will be confirmed at the design stage based on a detailed risk assessment for the Project.

Based on previous experience of levee protection works, the design for the flood levee has been assumed to have a 3 m top width, with 1:3 sides and a height equal to the 2,000-year flood level plus 600 mm freeboard. This represents a design guideline only, as the levees will be subject to detailed geotechnical design during the Project's later stages.

While it is important that the proposed model accounts for floodplain lost to dirty-water channels and flood protection works such as levees, it has been assumed that the cleared area footprint around the active-mine void during the worst-case scenario will contain the levee and any dirty-water channels.

For areas where the spoil footprint is located on or within the boundary of the 100-year ARI flood extent, scour protection will be required to protect spoil emplacements from erosion. The hydraulic model was used to estimate the height of the scour protection required to resist erosion up to the 100-year ARI event (with 600 mm freeboard allowance). Typically, this scour protection would be achieved using dumped hard rock material sourced from the Project site. A potential source of suitable material for this purpose is the rail cutting. As with the flood protection levees, this represents a design guideline only, as the scour protection scheme will be subject to detailed geotechnical design during the Project's later stages.

6.4 Model results

The approach detailed in the previous sections was used to assess the flood behaviour and impacts of the mine in the Sandy Creek catchment. This section summarises the outcomes of these flooding investigations.

6.4.1 Existing scenario

6.4.1.1 Flood levels

The existing scenario, 100-year ARI flood extent is shown in Figure 6-3. The 100-year ARI event is contained within the channel in the upper tributaries of Sandy Creek and the upper reach of Laheys Creek under existing conditions. Once the catchment flattens out in the middle to lower reaches of both Sandy Creek and Laheys Creek, there is some floodplain flow in the left and right overbanks due to the meandering nature of the creeks. The existing 2-year and 5-year ARI events are mainly conveyed in-channel for the middle to lower reaches, with higher flows going out of bank in some localised areas. The 2,000-year ARI event produces mainly overbank flow, but flows are still contained by the shape of the valley in the lower reaches.

In the upper reaches of Sandy Creek, all events up to the 2,000-year ARI event are contained within the channel. As the catchment area increases and small tributaries join, the 100-year ARI event flows just over bank-full and the 2,000-year ARI event flows in the channel and floodplain. The difference in flood level between the 100-year and 2,000-year ARI events in the upper reaches of Sandy Creek is around 1–2 m. In the lower reaches of Sandy Creek there is around 1 m difference in flood levels between the 100-year and 2,000-year ARI events.

Laheys Creek experiences similar flooding behaviour. In the upper reaches, up to the 100-year ARI event, the flow is just over bank-full. As the catchment area increases and the lower parts of the catchment become flatter and the channel wider, Laheys Creek experiences floodplain flow in the 100-year ARI event in both the left and right overbanks due to its meandering nature. For the 2,000-year ARI event, the lower reaches of Laheys Creek are completely flooded and there are some interactions with flows from other tributaries (for example, Blackheath Creek). There is a 1–1.5 m difference in flood levels between the 100-year and 2,000-year ARI events.

Table 6-6 shows the existing and proposed flood levels at key locations. Existing flows and flood levels at each cross-section in the hydraulic model are given in Appendix D.1.

Water level profiles for the 100-year ARI event for the existing scenario along the modelled reaches of both Sandy Creek and Laheys Creek are shown in Figure 6-4 and Figure 6-5. The creek chainage indicated on these charts has been measured from the downstream end of each of the creeks.

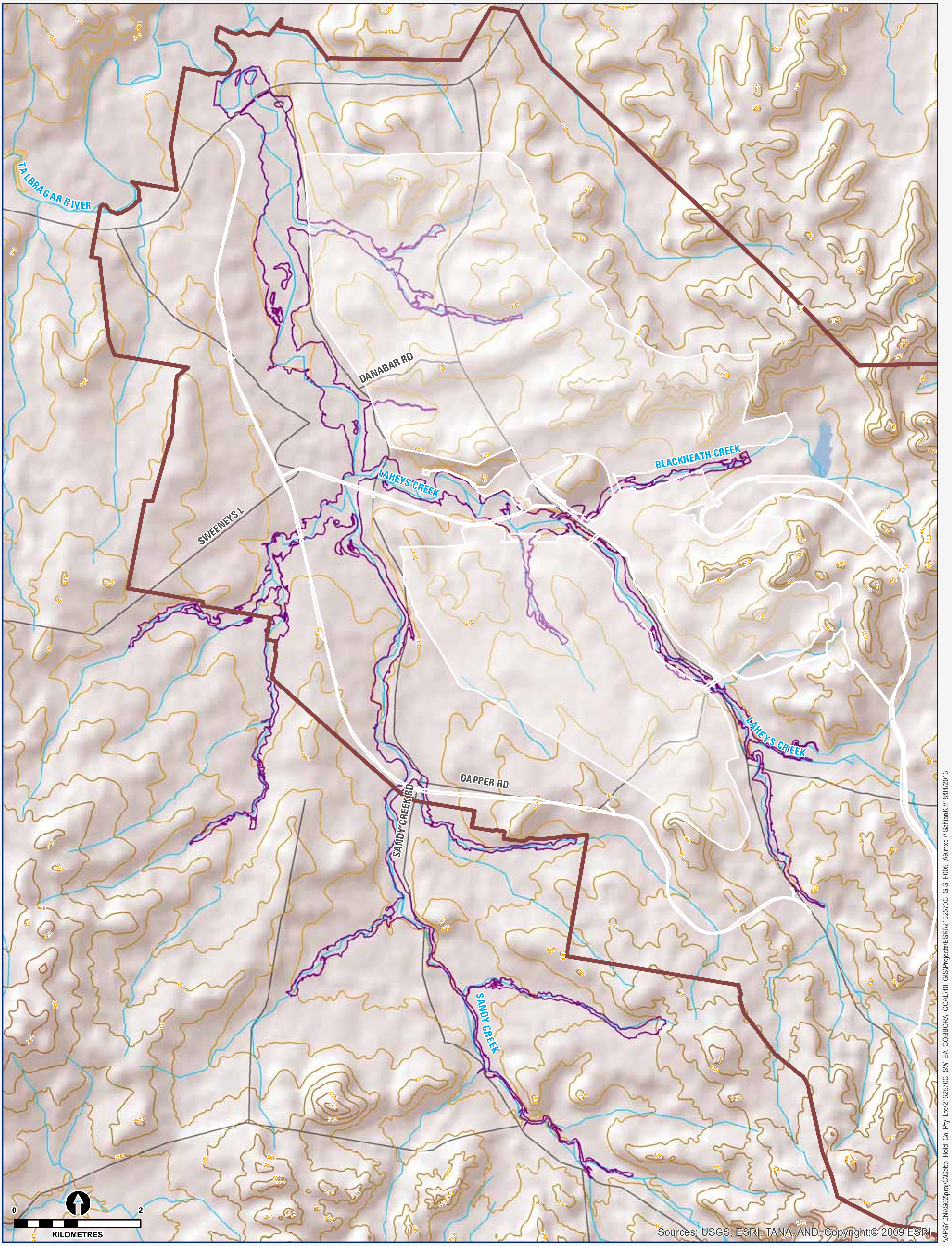
6.4.1.2 Peak velocities

Average peak velocities along Laheys Creek are generally less than 2 m/s for the 100-year ARI event, with an average velocity of 1.4 m/s. For the 100-year event, the main channel experiences a velocity range from 0.7 m/s to 2.5 m/s in the channel, and a slightly lower range of 0.1 m/s to 1.4 m/s in the floodplain. For the 2-year ARI event, peak velocities are within a range of 0.4 m/s to 2.1 m/s, with an average of 0.8 m/s.

Sandy Creek experiences a range of velocities throughout its creek system but these are lower on average than Laheys Creek due to its wider floodplain and flatter areas in its downstream reaches. For the 100-year event, Sandy Creek experiences a velocity range of 0.7 m/s to 2.3 m/s in the main channel and 0.2 m/s to 1.8 m/s in the floodplain. For the 2-year event, the velocity ranges from 0.3 m/s to 2.3 m/s and flow is mainly in-channel.

Blackheath Creek experiences lower velocities than Sandy Creek and Laheys Creek due to smaller flows. Velocities range from 0.6 m/s to 1.3 m/s for the 100-year ARI event and from 0.3 m/s to 0.8 m/s for the 2-year event.

Table 6-6 shows the existing and proposed average velocities in three key locations. Velocities stated here are averaged values across the creek cross-section. Slightly higher values will occur in the middle of the channel, and lower values near each of the overbanks.



100 year existing flood extent
Elevation (m)
Mine impact area
Drainage lines
Roads
Project application area

Figure 6.3 Existing 100 year ARI flood extent (Sandy Creek catchment)

Table 6-5 Existing and proposed flood levels at key locations

Location (cross-section ID)	2-year ARI (mAHD)			5-year ARI (mAHD)			100-year ARI (mAHD)			2,000-year ARI (mAHD)		
	Existing	Proposed	Difference	Existing	Proposed	Difference	Existing	Proposed	Difference	Existing	Proposed	Difference
Lower reach of Sandy Creek just before confluence with Talbragar River (3672)	350.79	350.86	+0.07	351.37	351.42	+0.05	352.33	352.33	0.00	353.89	353.81	-0.08
Lower reach of Sandy Creek (6066)	354.49	354.63	+0.14	355.56	355.59	+0.03	356.63	356.71	+0.08	357.72	357.63	-0.09
Sandy Creek downstream of junction with Laheys Creek (10294)	362.75	362.93	+0.18	363.95	364.01	+0.06	365.82	365.84	+0.02	366.97	366.97	0.00
Sandy Creek upstream of Laheys Creek junction (12447)	369.35	369.35	0.00	370.42	370.42	0.00	372.53	372.55	+0.02	373.75	373.75	0.00
Laheys Creek just upstream of junction with Sandy Creek (1207)	367.89	368.21	+0.32	368.62	368.69	+0.07	369.34	369.36	+0.02	370.16	370.17	+0.01[2]
Lower reaches of Blackheath Creek (4159)	385.76	385.88	+0.12[1]	385.85	385.92	+0.07	386.12	386.26	+0.14	386.45	387.20	+0.75[3]
Laheys Creek mid-reach upstream of Blackheath Creek confluence (6761)	386.30	386.34	+0.04	387.02	387.06	+0.04	388.41	388.34	-0.07	389.34	389.43	+0.09
Laheys Creek upper reach (9208)	396.31	396.66	+0.35[1]	396.95	397.20	+0.25	398.36	398.67	+0.31	399.67	399.44	-0.23

[1] The critical duration for the 2-year ARI is different for existing and proposed conditions, so the percent increase in water level is more significant.
[2] The model's accuracy is ± 0.01, so changes in water level of less than ± 0.02 should be considered negligible.
[3] High afflux in these areas is due to the presence of new watercourse crossings immediately downstream.

Table 6-6 Existing and proposed average velocities at key locations

Location (cross-section ID)	2year ARI (m/s)			5-year ARI (m/s)			100-year ARI (m/s)			2,000-year ARI (m/s)		
	Existing	Proposed	Difference	Existing	Proposed	Difference	Existing	Proposed	Difference	Existing	Proposed	Difference
Sandy Creek (6066)	1.02	1.04	+0.02	0.94	1.03	+0.09	1.23	1.51	+0.28	1.06	1.76	+0.70
Laheys Creek (1207)	0.56	0.62	+0.06	0.68	0.77	+0.09	1.10	1.42	+0.32	1.81	2.00	+0.19
Blackheath Creek (4159)	0.50	0.58	+0.08	0.62	0.57	-0.05	0.89	0.63	-0.26	1.35	0.5	-0.85

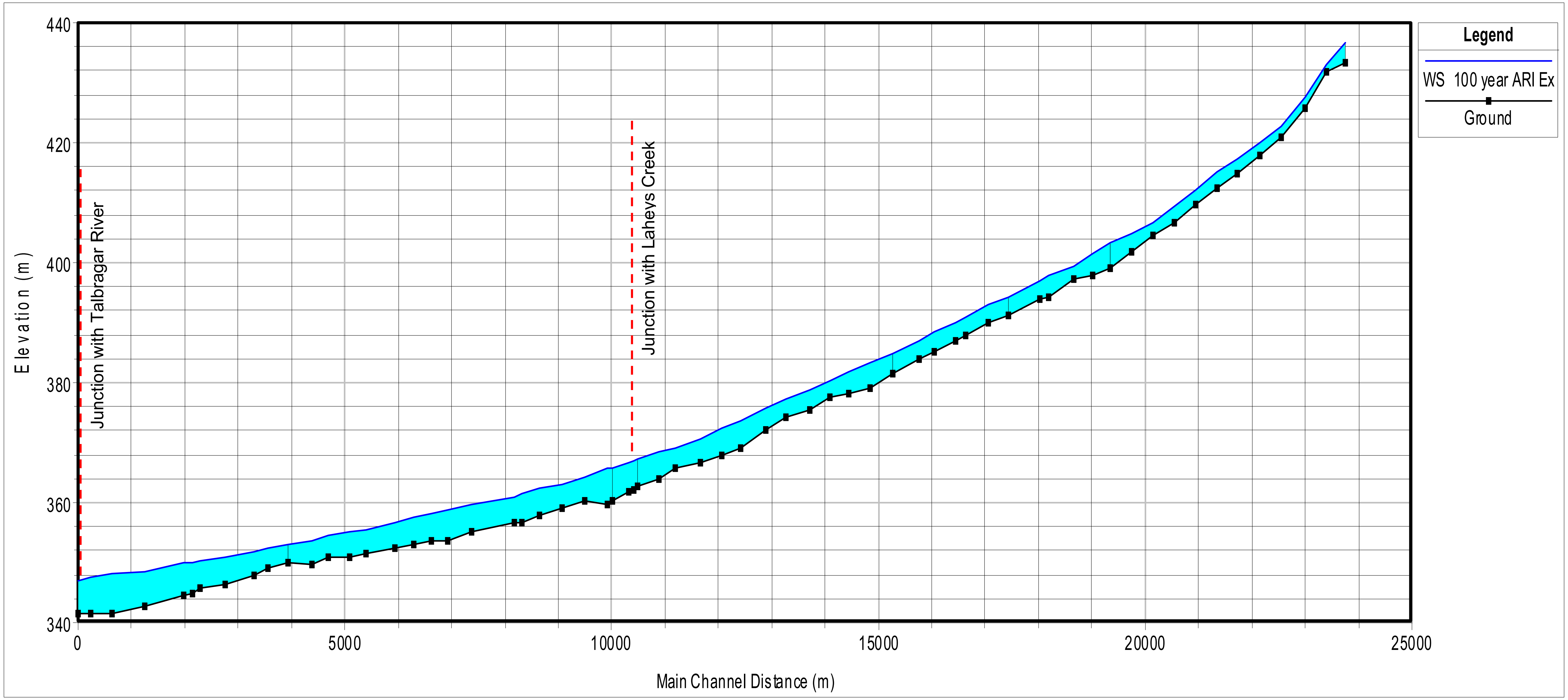


Figure 6-4 Existing longitudinal water level profile for Sandy Creek (HEC-RAS model output), 100-year ARI

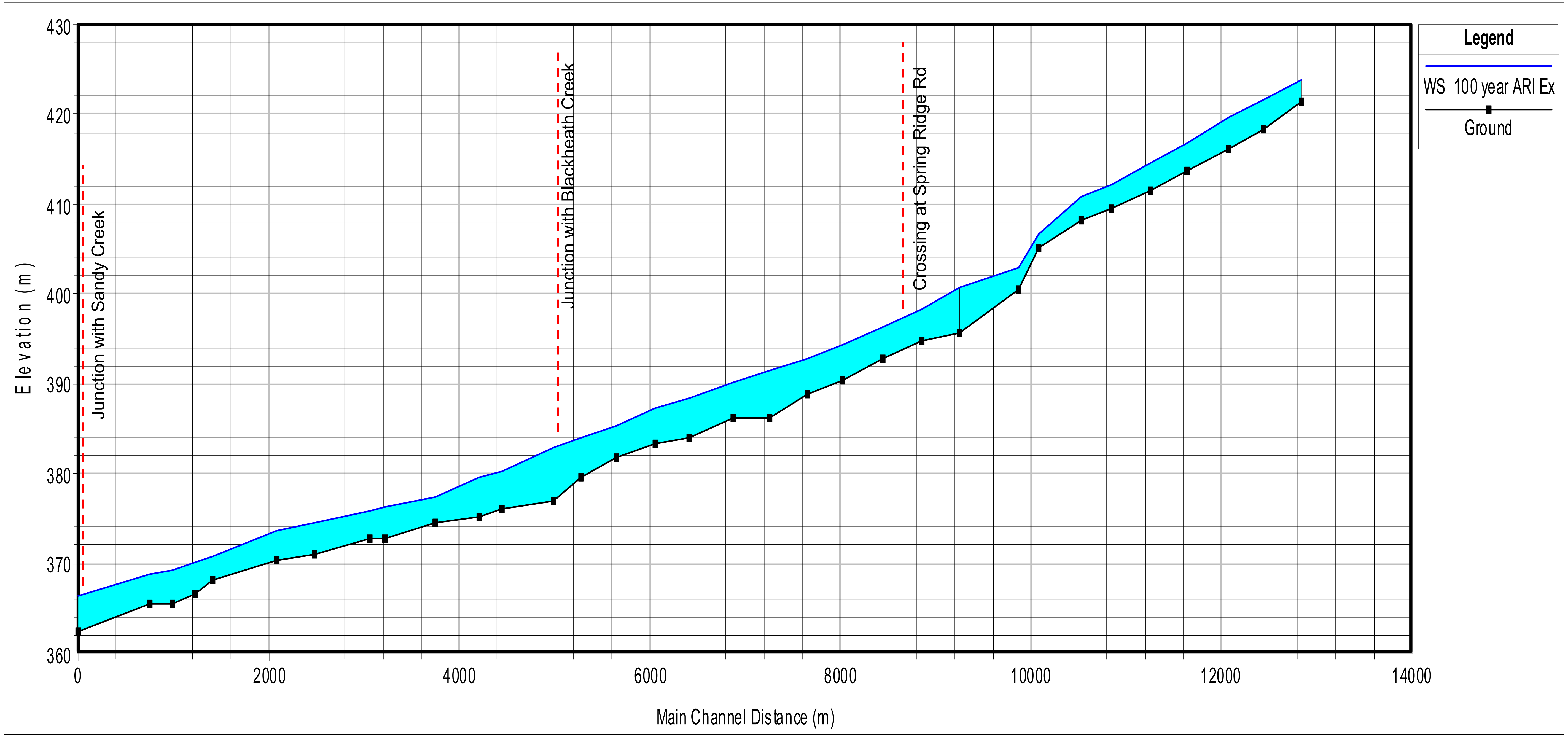


Figure 6-5 Existing longitudinal water level profile for Laheys Creek (HEC-RAS model output), 100-year ARI

6.4.2 Proposed scenario

6.4.2.1 Flood extents and levels

The proposed scenario 100-year ARI flood extent is shown in Figure 6-6. Figure 6-7 shows the flood extent on a larger scale map covering six key access and haul road crossings on Laheys Creek and Blackheath Creek. The figures show that the flood extents in Sandy Creek and Laheys Creek are generally similar to the existing scenario with the worst-case mine configuration in place. In some localised areas flood extents are affected, particularly upstream of significant crossings (see Figure 6.7, in particular Crossings 1, 3 and 5) — but flood extents and levels in the upper and lower reaches of Sandy Creek and the upper reaches of Laheys Creek and Blackheath Creek are similar for existing and proposed scenarios.

The results show that the mine footprint is mostly out of the 100-year ARI flood extent of Sandy Creek and Laheys Creek for the proposed scenario. However, some parts of the mine footprint near the workshop area, northern access road and the out-of-pit emplacement area along the upper reaches of Laheys Creek near mining area B lie on the edge of or just within the 100-year ARI flood extent for the proposed scenario.

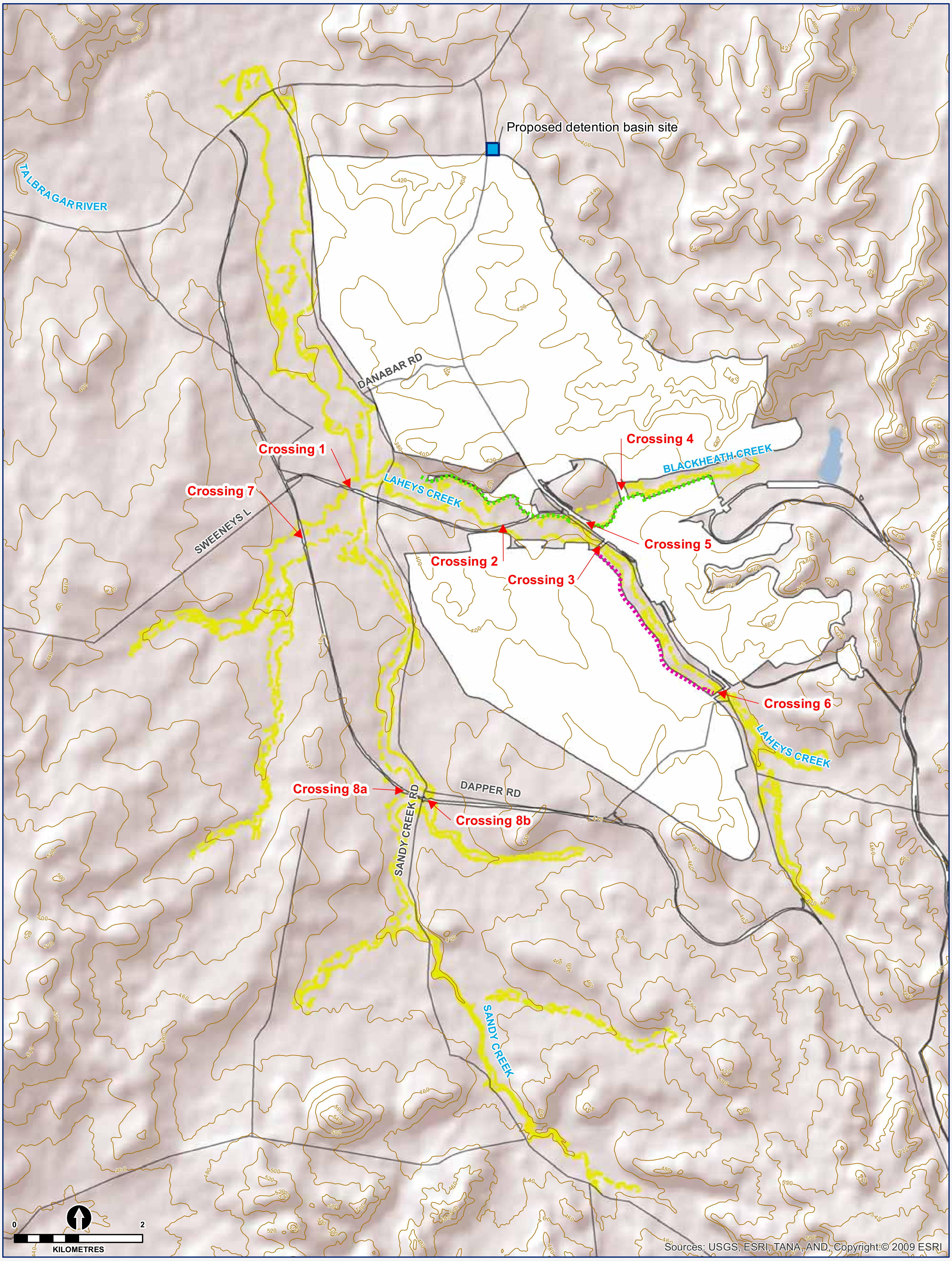
In addition to these areas, parts of the mine footprint are within the 2,000-year ARI flood extent for the proposed scenario. They are the northern bank of the mid-reaches of Laheys Creek adjacent to the workshop area, the western edge of Laheys Creek along mining area B, and the southern edge of Blackheath Creek near the CHPP.

Table 6-5 shows increases in flood levels in the middle to lower reaches of Laheys Creek and an increased flood level in Blackheath Creek due to the mine. This is expected, as there is an increased flow due to a larger catchment inflow and different timing of peak flows, flood protection levees to protect the mine act to constrain the floodplain in very extreme events (for the 100 year ARI event and beyond) and the proposed access and haul road crossings have localised impacts on flood levels. Increases in flood level (or afflux) due to watercourse crossings are discussed in more detail in Section 7.2.

Table 6-5 also shows small increases in flood levels in Sandy Creek just downstream of its junction with Laheys Creek. This is due to a slight increase in peak flow in Laheys Creek. However, by the time the peak reaches the lower section of Sandy Creek and its confluence with the Talbragar River, flood behaviour in the existing and proposed scenarios for the 100-year and 2,000-year ARI events is very similar to existing conditions.

It can be assumed that, as there is little impact to flooding behaviour for the 100-year ARI event, there is less impact to the mid-range flood events such as the 20-year and 50-year ARI events. For the lower order events (2-year and 5-year ARI events) flood levels are slightly higher at the Talbragar River confluence. Overall, the upper and lower reaches of Sandy Creek experience similar flooding behaviour in the existing and proposed scenarios. There is slightly more out-of-channel flow in the middle to lower reaches near the junction with Laheys Creek, but this impact diminishes further downstream.

Water level profiles for the 100-year ARI event for the proposed scenario along the modelled reaches of both Sandy Creek and Laheys Creek are shown in Figure 6-8 and Figure 6-9. It is noted that in Figure 6-8 a high point just upstream of the proposed access road crossing (chainage 5600) has been picked up by the LiDAR data. This area will need to be verified by site survey at the design stage to confirm whether this is the bottom of the channel or sediment build-up or an inaccuracy in the LiDAR data due to vegetation coverage. For the flood modelling, the LiDAR cross-section geometry has been used as this provides a more conservative assessment in this area. Proposed flows and flood levels at each cross-section in the hydraulic model for the proposed scenario can be viewed in Appendix D.2.



1 Waterways crossings
Proposed 100 year flood extent
Mine impact area
Elevation (m)
Drainage lines
Roads
Proposed spoil protection
Proposed levees
Proposed detention basin site

Figure 6-6 Proposed 100 year ARI flood extent

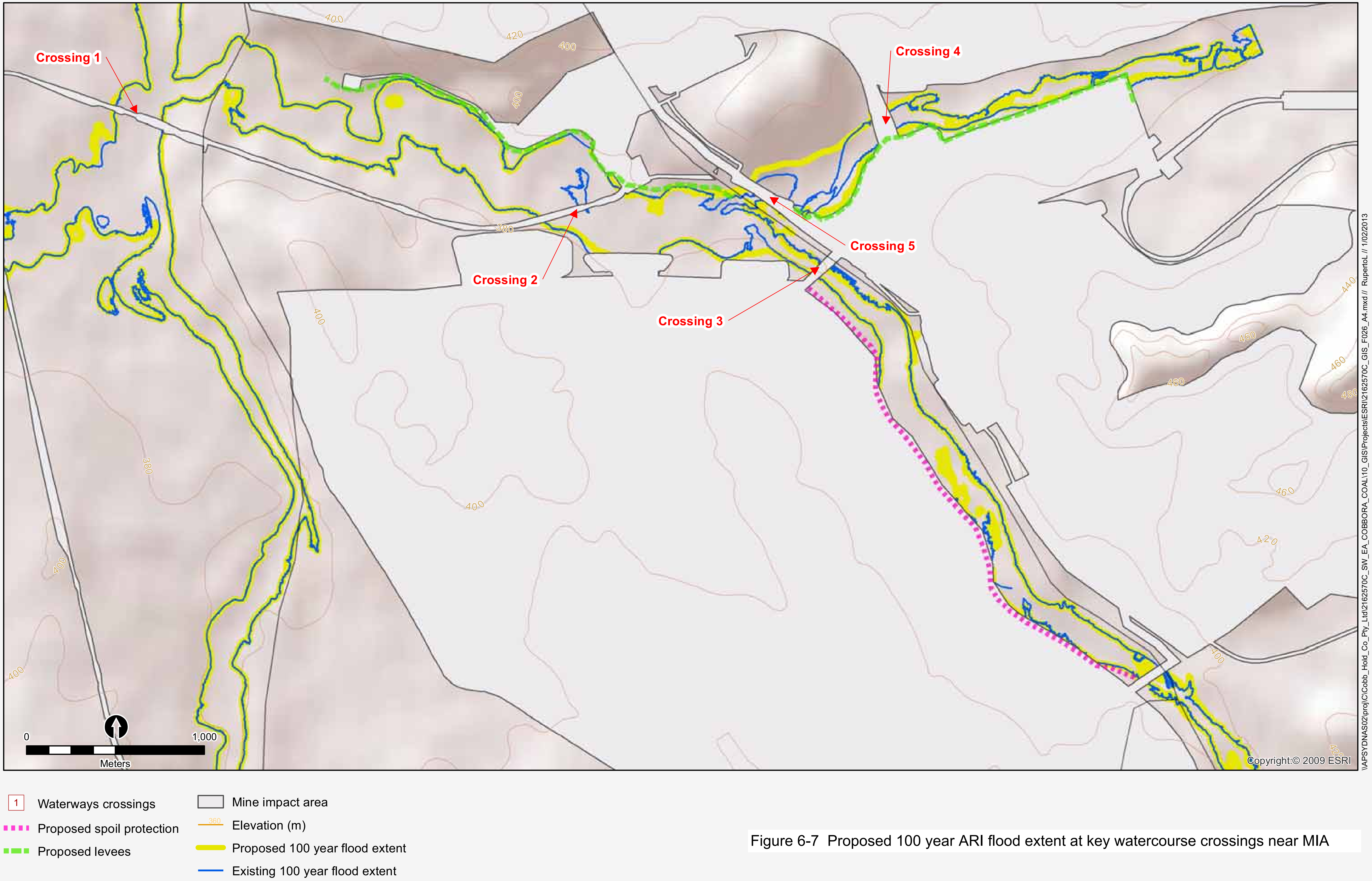


Figure 6-7 Proposed 100 year ARI flood extent at key watercourse crossings near MIA

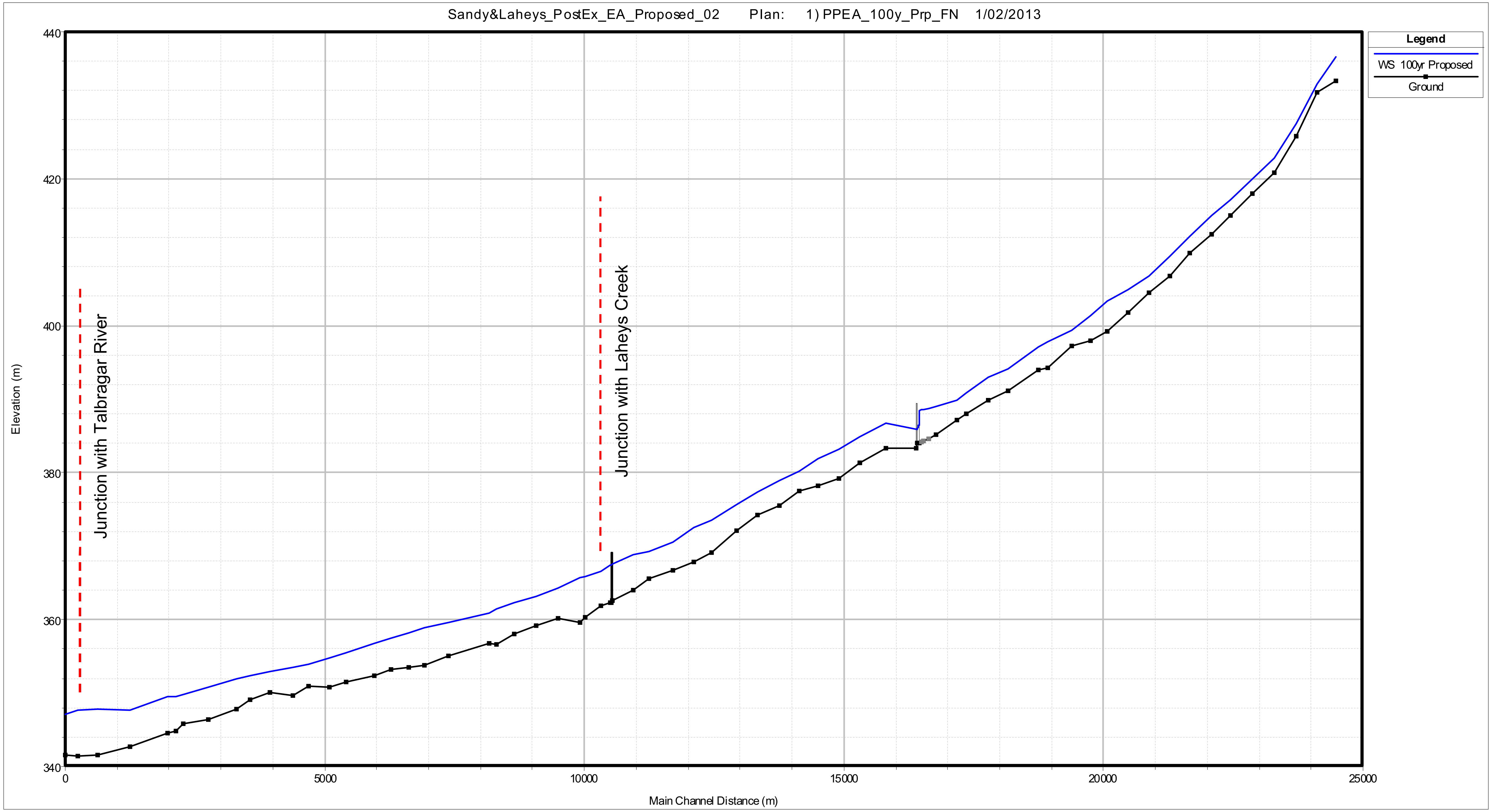


Figure 6-8 Proposed longitudinal water level profile for Sandy Creek (HEC-RAS model output), 100-year ARI

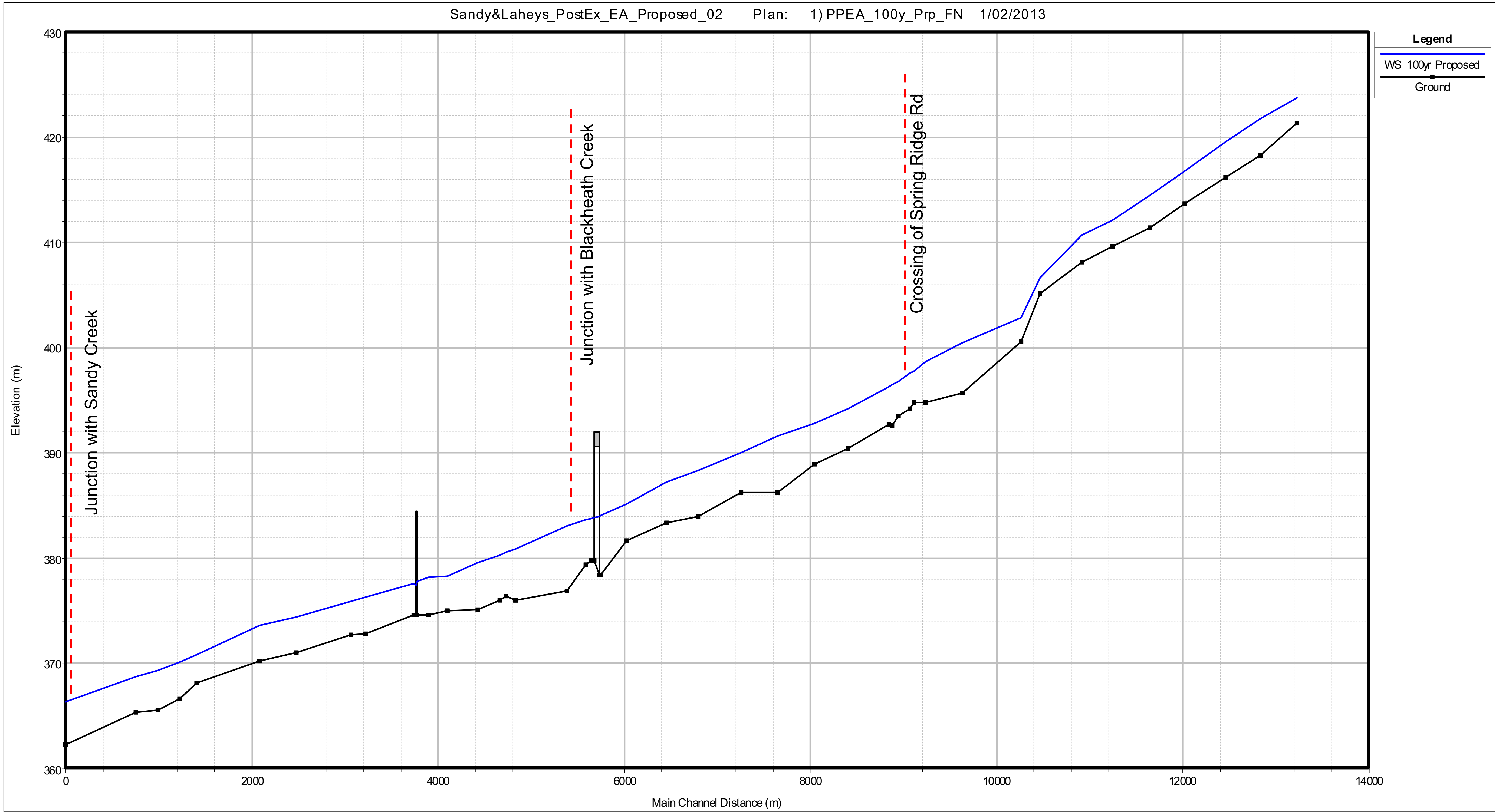


Figure 6-9 **Proposed longitudinal water level profile for Laheys Creek (HEC-RAS model output), 100-year ARI**

6.4.2.2 Peak velocities

Average peak velocities along Laheys Creek for the proposed scenario are similar to the range of existing velocities of 0.7 m/s to 2.5 m/s in the main channel and 0.1 m/s to 1.4 m/s in the floodplain for the 100-year ARI event. There are localised areas where the velocity in the channel reaches 3 m/s. These occur near proposed structures, and mitigation measures will be required in these locations to either reduce velocities or provide erosion protection to ensure channel stability.

For the 2-year ARI event, peak velocities are slightly higher than the existing velocities. The velocities are within a range of 0.4 m/s to 2.3 m/s, with the same average velocity as the existing average of 0.8 m/s.

Sandy Creek experiences the same range of velocities when compared with the existing scenario — from 0.7 m/s to 2.3 m/s in the main channel and 0.2 m/s to 1.8 m/s in the floodplain for the 100-year ARI event. For the 2-year ARI event the velocity ranges from 0.3 m/s to 2.3 m/s and flow is still mainly in-channel.

Blackheath Creek overall experiences lower velocities than the existing scenario for the 100-year ARI event, with a range of 0.6 m/s to 1.2 m/s. This could be the result of more flows in the floodplain that travel slower than the in-channel flows. This is evident by the increased flood levels shown in Table 6-5.

However, velocities at the downstream end of Blackheath Creek are slightly increased due to the presence of structures. For the 2-year ARI event, velocity ranges are slightly higher than the existing case for Blackheath Creek, with a range of 0.5 m/s to 1.4 m/s. Velocity increases in the 2-year ARI event but decreases in velocity for the 100-year ARI event because in the 2-year ARI event flow is contained in-channel resulting in higher velocities, whereas in the 100-year ARI event, flow spills out to the floodplain and has a lower velocity due to higher roughness in the overbanks.

Table 6-6 shows the existing and proposed peak velocities in three key locations. Overall, velocities are slightly increased in Sandy Creek and Laheys Creek and slightly decreased in Blackheath Creek.

7. Findings and mitigation measures

7.1 Flooding impacts

7.1.1 Sandy Creek, Laheys Creek and Talbragar River (Zone 1)

Changes in flood extent and level within Sandy Creek and Laheys Creek as a result of the proposed mine are generally minor, with some localised changes in flooding due to increased flows and proposed watercourse crossing structures. Flooding impacts due to the proposed mine are isolated to Laheys Creek and the lower reaches of Blackheath Creek around the mine footprint inside the study area. However, these are highly localised effects and do not affect any lands outside the ownership of CHC.

Flows in Laheys Creek have increased slightly overall due to the progressive diversion of the northern part of the catchment into Laheys Creek, which naturally flows into Sandy Creek. This increase in catchment area flowing to Laheys Creek is somewhat offset by the loss of catchment area to the open mining areas and so changes in peak flows are minimal.

There has been a change in the times the creeks take to peak locally around the mine footprint — namely, the disturbed catchments within the Laheys Creek and Blackheath Creek. However, by the time the peak flow reaches Sandy Creek, peak flows become closer to the existing peaks. Modelling showed only minor changes to flood levels in Sandy Creek just downstream of its junction with Laheys Creek, but these changes diminish further downstream and there is negligible change to flood levels at the downstream end of Sandy Creek. Overall, the hydraulic modelling has shown no significant change in flood levels upstream or downstream of the Sandy Creek catchment where the majority of mining area is located.

The northern boundary of the mining area is approximately 2 km away from the Talbragar River, and the top of the catchment containing the mine area is approximately 45 m above the banks of the Talbragar River. The hydraulic model shows that for all events analysed up to the 2,000-year average recurrence interval (ARI) event, water levels in the Talbragar River will not be affected by the mine because of the distance and difference in level between the mine and the river.

There will be local increases in velocities and changes to flood levels at structures where the flooding behaviour is altered; however, these impacts are highly localised. Areas where high velocities are experienced will require mitigation in the form of scour protection.

7.1.2 Flyblowers Creek (Zone 2)

Other potentially affected areas outside the Sandy Creek hydraulic model extent were assessed hydrologically or qualitatively. It was found that there would be a notable impact on the peak flow of Flyblowers Creek, north of the Sandy Creek catchment. During Year 12 to Year 20 of the mine, a significant portion of the upper, eastern Sandy Creek catchment is diverted north into the eastern arm of Flyblowers Creek. This diverted catchment reaches a maximum of 86 ha during Year 12 of the mine, as seen in the mine staging plans provided in the *Water Balance and Water Management System* report (Appendix E to the *Surface Water Assessment* report).

A hydrological assessment found that the peak flow in this creek would be increased by around 30% across a range of events (2-, 5- and 100-year ARI events) due to the diverted catchment. This increase in peak flow will have an impact on the existing watercourse crossing of the upper parts of Flyblowers Creek at Spring Ridge Road. However, this section of road will be closed and no longer in use during mining, so treatment of the road and culvert to protect them from flood damage or overtopping is not required.

The increase in peak flow also has the potential to impact on the culvert underneath the Golden Highway, which lies about 1.2 km away at the bottom of the local catchment. Mitigation of this impact would be required to ensure land not owned by CHC is unaffected and flow conditions reaching this culvert are not significantly modified. Mitigation measures at this location are recommended in Section 7.2.2.

7.1.3 Rail corridor (Zone 3)

Much of the proposed rail spur will be located on a 6 m high embankment. This has the potential to cause flooding upstream of the rail embankment. Towards the western end of the proposed rail spur, the rail corridor crosses a number of small tributaries and gullies draining into Fords Creek. Minor and localised increases to flooding at these locations may be acceptable as the entire catchments of these small tributaries lie within the study area. Towards the eastern end of the rail spur, however, where the rail line would tie into the existing rail line, the rail corridor crosses a significant channel of Tallawang Creek, where the proposed rail embankment could alter flooding behaviour.

At all of these locations, mitigation measures in the form of bridges and culverts with appropriate flow capacity would be required to prevent flows from building up behind the rail embankment and causing flooding impacts upstream. Watercourse crossings and longitudinal drainage along the rail corridor are discussed in Section 7.2.3.

7.1.4 Watercourse crossings (construction)

Watercourse crossings are required where access roads, haul roads and rail lines cross watercourses within the study area. Preliminary designs of some of the key crossings in the Sandy Creek catchment (including estimates of dimensions and invert levels of openings and clearance above flood levels) were developed as part of this assessment.

Bridge spans and openings for the access roads were sized to avoid excessive afflux under the 100-year ARI event. The openings for the haul road crossings were sized to provide sufficient freeboard above the 2,000-year ARI event flood level, on the basis that these roads are critical infrastructure and require a high standard of flood protection. Other crossings outside the Sandy Creek catchment, such as the rail spur watercourse crossings, have not been modelled hydraulically but have been identified and given an indicative type and size based on channel shape, slope and catchment area.

The impacts of building the crossings have not been specifically assessed due to the preliminary nature of the design; however, a qualitative assessment has been considered. During construction of the watercourse crossings, flood flows could be temporarily diverted around the construction works, which could have a localised impact on flood levels and sediment movement.

Standard flood mitigation measures during construction would apply at these locations. Typically, these include provision of temporary diversion channels with sufficient capacity (i.e. depending on the duration of the construction works) to convey flood flows that could occur during the construction period around works areas.

7.2 Mitigation measures

7.2.1 Sandy Creek, Laheys Creek and Talbragar River (Zone 1)

Since the impacts of the Project on flooding within Sandy Creek and Laheys Creek outside of land owned by CHC are negligible, no mitigation measures are proposed in these catchments.

The Project will cause localised impacts on flood extents, levels and velocities in Blackheath, Laheys and Sandy Creeks on land owned by CHC. These impacts are primarily due to access and haul road crossings required for the Project. Flood protection levees are also required to prevent the mining areas from flooding and will also have an impact on flood levels, although to a lesser extent than the crossings.

Preliminary designs for the access and haul road crossings of Blackheath, Laheys and Sandy Creek are presented in Section 6.3.2 based on the design criteria of safe operation under 100 year ARI flood conditions and limits on afflux in land upstream of the crossings. Table 7-1 summarises the afflux caused by these structures on adjacent land based on hydraulic models of these preliminary designs. In all cases the afflux impacts are restricted to land owned by CHC, with afflux reducing to zero outside of CHC's land ownership boundary.

Table 7-1 100 year ARI afflux results for access and haul road crossings of Blackheath Creek, Laheys Creek and Sandy Creek

Location	100 year ARI afflux (m) at crossing number								
	1	2	3	4	5	6	7	8a	8b
Approx. 500m upstream of crossing	+0.01	+0.01	-	-0.39	+0.10	0.00	+0.65	-0.07	-0.25
Approx. 200m upstream of crossing	-0.02	+0.11	+0.10	+0.31	+1.16	0.00	-	+0.27	-
Approx. 100m upstream of crossing	-	+0.37	-	+1.03	+1.51	0.00	+0.66	-	-
Just upstream of crossing	+0.1	-	+0.19	-	+2.57	0.00	+1.13	+0.16	-
Just downstream of crossing	-0.01	-0.03	+0.06	+0.11	+1.16	0.00	+0.44	-0.33	-0.13

Notes:
See Figures 6-6 and 6-7 for crossing locations.
No data provided where model data is not available or where the distance upstream coincides with the presence of another crossing structure.

Areas of significant afflux in the order of 0.5m and upwards are mainly confined to Laheys Creek and Blackheath Creek in the vicinity of the MIA and crossings 2 to 5. The mining areas will be protected by flood protection levees in this area and therefore the increased flood levels will be constrained and controlled by the levees. No significant afflux occurs beyond the extent of the flood protection levees.

Increased flood levels in these areas has the potential to impact on other environmental aspects of the creeks and floodplains, including riparian vegetation, ecological habitats (including refuge pools) and Aboriginal heritage artefacts. However, the crossings and levees do not significantly modify the regular flooding regime, i.e. flooding characteristics up to the 5 year ARI event, and therefore these features of the creeks and floodplains will not be subject to significantly different flood impacts for frequent flood events. Table 7-2 provides the afflux at the crossings for the 5 year ARI event.

Table 7-2 5 year ARI afflux results for access and haul road crossings of Blackheath Creek, Laheys Creek and Sandy Creek

Location	5 year ARI afflux (m) at crossing number								
	1	2	3	4	5	6	7	8a	8b
Approx. 500m upstream of crossing	+0.01	+0.07	-	+0.01	+0.16	0.00	+0.05	-0.01	-0.05
Approx. 200m upstream of crossing	0.00	+0.03	+0.02	+0.12	+0.59	0.00	-	+0.02	-
Approx. 100m upstream of crossing	-	+0.08	-	+0.33	+0.05	0.00	+0.16	-	-
Just upstream of crossing	+0.02	-	+0.13	-	+1.06	0.00	+0.32	-1.54	-
Just downstream of crossing	0.00	+0.03	+0.07	+0.16	+0.55	0.00	+0.22	-0.94	0.00

Notes:
See Figures 6-6 and 6-7 for crossing locations.
No data provided where model data is not available or where the distance upstream coincides with the presence of another crossing structure.

As for the 100 year ARI event, the main afflux impacts for the 5 year ARI event are confined to the reach of Laheys and Blackheath Creeks adjacent to the MIA, in particular around Crossings 4 and 5, where significant haul road crossings and flood protection levees are required. For other crossings the afflux impacts around the structures are relatively minor.

Watercourse crossings are a controlled activity under the Water Management Act 2000. The detailed design of the crossings will need to take into account NOW guidelines on design and construction of watercourse crossings which require consideration of the following:

- Minimisation of disturbance of the riparian corridor and its function.
- Preservation of native vegetation.
- Preservation of natural hydrological and geomorphological regimes.
- Rehabilitate and stabilise disturbed areas and protect against ongoing scour and erosion.

Erosion protection will be required at all structures and bridges, particularly for access roads where extreme events may overtop the bridge and local roads leading up to the structure. The appropriate erosion protection would be determined during the design phase. Typically, it would include rock revetment and scour protection to culvert inlets and outlets, bridge piers and abutments. It is essential that erosion protection measures for both the construction and operational phases of the Project be incorporated into the detailed design of these crossings.

All access and haul road crossings that have active fish movement will need to be fish-friendly and designed in accordance with *Policy and Guidelines for Fish Friendly Waterway Crossing* (NSW Fisheries 2003) and *Why Do Fish Need to Cross the Road? Fish Passage Requirements for Waterway Crossings (*Fairfull & Witheridge 2003).

7.2.2 Flyblowers Creek (Zone 2) flood detention basin

There would be a notable impact of increased peak flows in Flyblowers Creek (Zone 2), north of Sandy Creek. During Years 12 to 20, mitigation measures would be required to reduce the peak flow in this creek and avoid impacts on the Golden Highway further downstream in the catchment. It is recommended that a dry detention basin be constructed to reduce peak flows at the Golden Highway culvert to those that occur under the existing scenario.

7.2.2.1 Conceptual design

XP-RAFTS software was used to size a detention basin at this location. XP-RAFTS accurately models a detention basin with an outlet orifice and allows flexible adjustment of a range of modelling parameters and conditions. XP-RAFTS is also useful for demonstrating the feasibility of the mitigation measure.

For this catchment, the flows predicted by XP-RAFTS match reasonably well the flows predicted by Watershed Bounded Network Model (WBNM), and only minor adjustments were required. The hydrologic analysis found that a detention basin with a volume of 70,000 m^3 (70 ML) would be required to mitigate the increase in peak flow and restore peak flow at the Golden Highway culvert to the levels that occur for the existing scenario. With the dry detention basin in place, potential adverse flood impacts on the Golden Highway would be avoided.

The basin has been designed to capture the 100-year ARI flow from the diverted catchment without spilling. For the diverted catchment, the critical duration for the 100-year ARI event is 180 minutes. Figure 7-1 shows the 100-year ARI peak flow for the diverted catchment entering the detention basin. The detention basin has been designed to have a maximum depth of 3 m. During the 100-year ARI event the depth (stage) in the basin reaches 2.3 m.

With an allowance for freeboard giving an approximate total depth of 3 m, the flood level in the basin needs to exceed 401 mAHD before it spills. For events above the 100-year ARI, the spillway would need to be designed to spill safely and to minimise erosion to the embankment and the downstream areas.

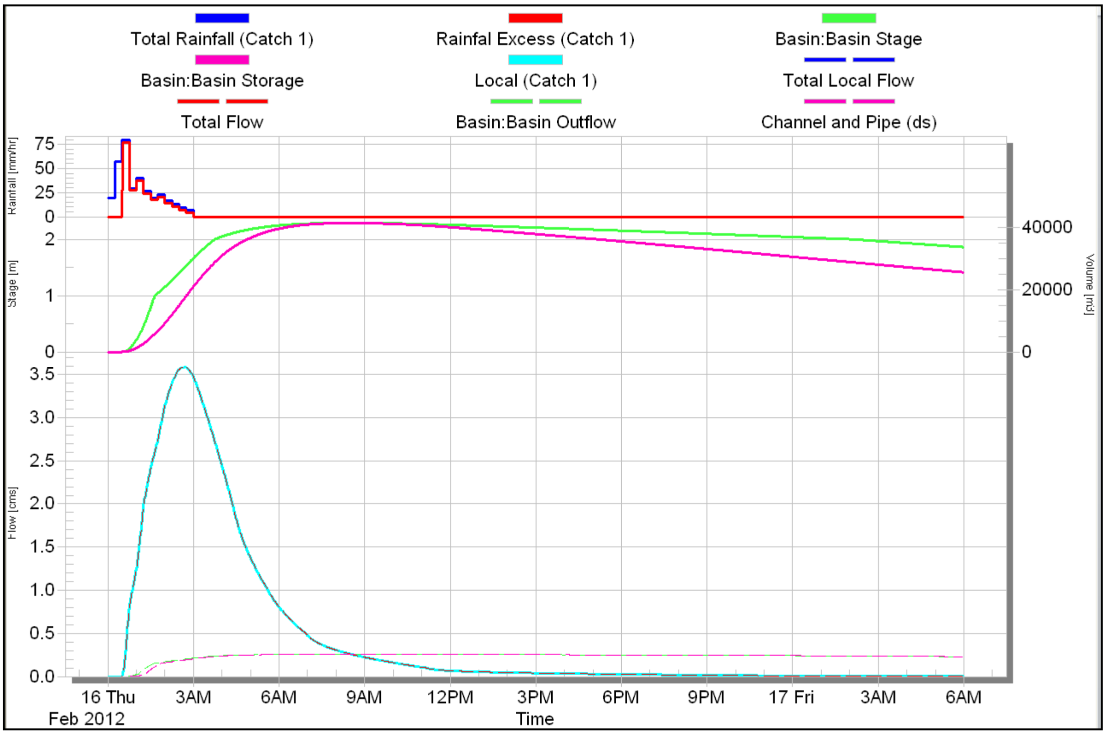


Figure 7-1 100-year ARI peak flow and stage–storage relationship for the detention basin

In the conceptual sizing exercise, a 0.3 m orifice outlet structure was modelled that allows the basin to drain slowly during and after a storm event. To test the feasibility of smaller basin sizes, a larger outlet and smaller volume basin was also tested. However, the alternative configurations did not sufficiently attenuate the flow downstream at the Golden Highway. The conceptual basin design proposed above should be investigated further at the detailed design stage. It is likely that the basin will require NOW approval as it constitutes controlled work that affects the flow of water to or from a river (under Part 8 of the Water Act 1912).

7.2.2.2 Location

The 70,000 m^3 basin can be accommodated within the study area. The proposed location for the detention basin is indicated in Figure 6-6. Although the topography in this location is relatively flat, it was chosen as it is close to the disturbed area and is further from the highway. Other locations for the detention basin were investigated, such further downstream on Flyblowers Creek, but this would disturb an area that is otherwise unaffected by mining and could have significant impacts on local ecology.

The shape of the basin has been fitted to the approximate topographic contours in the area. At this location the bottom of the detention basin would sit at 398 mAHD. The detention basin would require a 3 m embankment about 600 m in length around three edges. The 3 m embankment would to tie into the top of the basin on the eastern side at 401 mAHD.

The layout and footprint of the basin is shown in Figure 7-2. This figure shows the full basin and the total footprint of the detention basin, including the embankment. An embankment top width is 3 m and 1:3 side slopes has been assumed in the figure representation of the basin.

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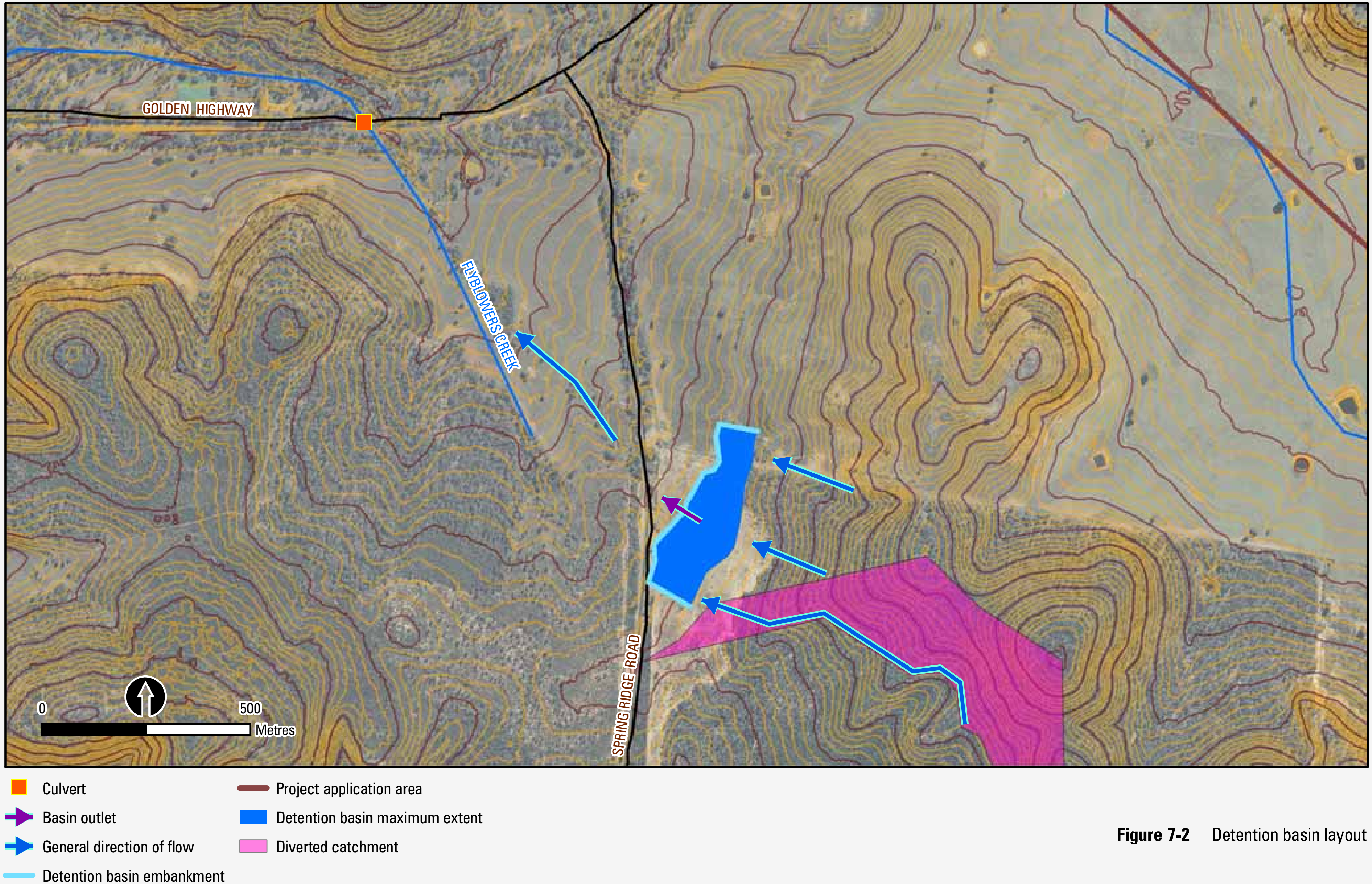

Figure 7-2 Detention basin layout

7.2.3 Rail corridor (Zone 3)

The proposed rail corridor will cross 12 watercourses and has the potential to affect local flooding behaviour at these locations. Section 6.3.2 describes the crossings and proposed structure types.

Minor watercourse crossings along the rail spur have not been designed at this stage of the Project since the flood immunity and design of the rail spur crossings depends on mine access requirements, safety requirements and other operational requirements for the rail spur. The flood design criteria for this rail spur may also have to conform to local design guidelines, which should be considered during the detailed design phase.

As a general guide, it is recommended that the rail embankment be structurally sound up to at least the 100-year ARI event, to avoid environmental impacts should the embankment fail during extreme flood events. Preliminary designs for the two significant rail line crossings of Fords Creek and Tallawang Creek are presented in Section 6.3.2 based on suggested design criteria of safe operation under 100 year ARI flood conditions and suggested limits on afflux in land upstream of the crossings. Table 7-3 summarises the flood impacts of these structures on adjacent land based on hydraulic models of these preliminary designs.

Table 7-3 Flood impacts for preliminary design of Fords Creek and Tallawang Creek crossings

Location	100 year ARI afflux (m)	
	Fords Creek	Tallawang Creek
Approx. 200m upstream of crossing	0.14	0.01
Approx. 100m upstream of crossing	0.27	0.02
Just upstream of crossing	0.40	0.04
Just downstream of crossing	0.00	0.00

Additional rail embankment treatments, such as riprap or gabions, may be needed on approaches to the crossings on the upstream and downstream faces of the embankment. Embankment treatments should protect the embankment from damage and/or reduce the extent of damage during an extreme flood event.

All cross drainage structures should be assessed for scour and appropriate scour protection measures (typically in the form of dumped rock or concrete pads) should be provided at the inlets and outlets of the crossing structures.

Watercourse crossings are a controlled activity under the Water Management Act 2000. The detailed design of the crossings will need to take into account NOW guidelines on design and construction of watercourse crossings which require consideration of the following:

- Minimisation of disturbance of the riparian corridor and its function.
- Preservation of native vegetation.
- Preservation of natural hydrological and geomorphological regimes.
- Rehabilitate and stabilise disturbed areas and protect against ongoing scour and erosion.

All rail crossings that have active fish movement will need to be fish-friendly and designed in accordance with *Policy and Guidelines for Fish Friendly Waterway Crossing* (NSW Fisheries

2003) and *Why Do Fish Need to Cross the Road? Fish Passage Requirements for Waterway Crossings (*Fairfull & Witheridge 2003).

Table 7-4 provides preliminary estimates of the lengths of rail spur that expected to be in cut and fill sections.

Table 7-4 Lengths of cut and fill along proposed rail spur

Section type	Length (km)
Fill sections	12.52
Cut sections	9.03
Total length of rail spur	**21.55**

The rail spur is 21.55 km long. This includes the rail line from the eastern end where it ties in to the existing rail line at the western end near the CHPP, but excludes the rail loop near the CHPP. The extent of cut and fill along the rail line has been assessed for the full 21.55 km length.

A typical longitudinal drainage system would be needed along the alignment of the rail spur to convey runoff intercepted by cut/fill slopes to the cross-drainage structures.
The longitudinal system should be designed to drain intercepted flows as close to natural conditions as possible. Where steep gradients are unavoidable in the system, appropriate scour protection measures in the form of rock or concrete lined channel should be provided.

7.3 Mine safety

This discussion of the mine's safety considers how flooding external to mining areas and workings affects the mine, mine infrastructure and mining operations. The proposed mine plan, as discussed above, will be located predominantly outside the 2,000-year ARI floodplain. This means that flooding in the Sandy Creek and Laheys Creek catchments will have little impact on the mine, except for very extreme low probability events larger than the 2,000-year ARI event. However, at several areas of the mine the 2,000-year ARI floodplain does encroach on the mine and mine infrastructure. These areas are discussed below.

7.3.1 Access and haul roads

The access and haul road crossings have been designed to be generally above the 100-year ARI design event with a freeboard allowance. However, most of the crossings and approach roads would be overtopped during the 2,000 year ARI event and for intermediate events above the 100 year ARI event. The overtopping potential for events exceeding the 100-year ARI will need to be considered when designing the access and haul road crossings.

Additional road embankment treatments, such as riprap or gabions, would need to be placed on approaches to the crossings on the upstream and downstream faces of the embankment. Embankment treatments should protect the embankment from damage and/or reduce the extent of damage during an extreme flood event.

The mine's site emergency plan should address the high hazard but low probability scenario of access and haul road flooding for events exceeding the 100 year ARI event.

7.3.2 Flood protection levees

Flood protection levees have been proposed at two key locations in the Laheys Creek and Blackheath Creek catchments. Although there are no proposed open-mining area along the watercourses where floodwaters can enter, there are areas that require flood protection to prevent flooding from disrupting operations and potentially causing loss of life and/or damage to working areas and equipment.

The first of the key locations is the area along the southern edge of Blackheath Creek where a flood protection levee equal to the 2,000-year ARI flood level (or other suitable, extreme-event flood level, to be determined at the design stage) would be required to protect the coal stockpile. Alternatively, the coal stockpile platform on the southern bank of Blackheath Creek could be built above the 2,000-year ARI event (or other suitable extreme event), with freeboard allowance, to protect this from flooding.

The second location extends along the edge of the workshop area located on the northern bank of the lower reaches of Laheys Creek. It is assumed that this area is required to have flood immunity up to the 2,000-year ARI event (or other suitable extreme event), plus freeboard allowance. Alternatively, the embankment for the workshop and access road could be designed to be above the 2,000-year ARI event, with scour protection added to the embankment as it would be located on a bend of Laheys Creek where scour erosion may be significant. At the design stage the levee/embankment design at this location needs to allow for the potentially very high extreme-event flood velocities at this location.

The 2,000-year ARI flood event was chosen as a reasonable upper limit reference event for protection of critical infrastructure and is not proposed as a strict design requirement.
The appropriate standard of flood protection for critical infrastructure will be confirmed at the design stage based on a detailed risk assessment for the Project.

The total length of these levees is 3.2 km, which includes a 2.2 km stretch along Blackheath Creek and 1 km stretch along Laheys Creek (see Figure 6-6). As the edge of mining area A is significantly higher than the 2,000-year ARI event level plus 600 mm freeboard, levees along this location are not required.

For the purpose of this assessment, flood levees have been assumed to have a 3 m top width with 1:3 sides and a height equal to the 2,000-year ARI flood level plus 600 mm freeboard. These parameters are proposed as an initial guide and should be reviewed at the design stage of the Project. There should also be an allowance on the inside edge at the toe of the levee for a collection drain to collect runoff from the embankments. The levees should be designed to prevent localised failure points and have a surface that minimises erosion.

The levees prevent land from being flooded and therefore constitute controlled work under Part 8 of the Water Act 1912. The design of the levees will need to conform to NOW guidelines for such works.

7.3.3 Spoil emplacement protection

One of the spoil emplacement areas is to be located on or within the boundary of the 100-year ARI flood extent. It would require scour protection to protect it from erosion. It is proposed that instead of providing a levee, the toe of the spoil emplacement itself be engineered similarly to a levee. During the area’s formation, a suitable hard rock material sourced locally from the site (e.g. from the rail cutting) should be used to construct the bottom section of the emplacement up to the level of the 100-year ARI flood level, plus a freeboard of 600 mm. The area is located along the western edge of the upper reaches of Laheys Creek, adjacent to mining area B.

The 100-year ARI flood levels are presented in Appendix D.1, but these levels should be checked after detailed design of the spoil emplacements. In addition to engineering the toe of the spoil emplacement, erosion and sediment control measures would be required to ensure that, during a flood event, sediment-laden runoff does not enter any downstream watercourses.

The spoil protection areas will modify flood behaviour in high order events and will prevent land from being flooded. They will therefore constitute controlled work under Part 8 of the Water Act 1912 and the design will need to conform to NOW guidelines for such works.

7.3.4 Temporary diversion channels

As indicated earlier, mining areas and infrastructure are predominately out of the floodplain in both the existing and proposed scenarios. In the proposed scenario, however, the mine plan requires temporary and permanent diversion/removal of some of the smaller tributaries. In some cases the upstream portion of an existing creek will be unaffected by the mine but the downstream section will need to be temporarily diverted. As the mine expands, the entire creek will be incorporated into the mining area and the temporary diversion no longer required.

A review of the proposed mine plan shows that two temporary diversions of existing watercourses are required. For this assessment, basic feasibility calculations were undertaken to ensure these diversion channels could be provided. It is recommended that these diversion channels be designed to follow the shape of the existing channel as closely as possible, with a low-flow channel and a high-flow channel above the low-flow channel. The average channel slope of the proposed diversion channels should be similar to that of the existing channel, and hydraulic checks of the channel should be completed to ensure velocities in the channel are not greater than 2 m/s to reduce the erosion potential in the diverted channel.

These diversion channels are intended to prevent clean water from the unaffected catchment flowing into mining area B (channel 1, Years 1–4) and mining area B (expanded) (channel 2, Years 4–8). The indicative locations of these channel diversions can be found in Appendix D.4. The following sections provide conceptual design for these channel diversions.

7.3.4.1 Temporary creek diversion 1

The purpose of this diversion channel is to divert flows around mining area B during years 1-4. It is an existing tributary of Laheys Creek and will divert flows back into Laheys Creek.

The approximate length of the diversion channel is 1.1km with a slope of 1.0%, which matches the existing average channel slope. This channel will tie into the existing channel upstream of mining area B and into the proposed northern access road crossing at the northern end of mining area B. From here the diversion channel will continue to join Laheys Creek at a meander point downstream of its existing confluence with Laheys Creek. Scour protection is likely to be required where the channel intersects with the access road and with the new connection to Laheys Creek.

XP-RAFTS has been used to estimate design flows for this tributary sub-catchment and Mannings Equation calculations have been used to determine a conceptual trapezoidal channel design for the diversion, which is shown below in Figure 7-3.

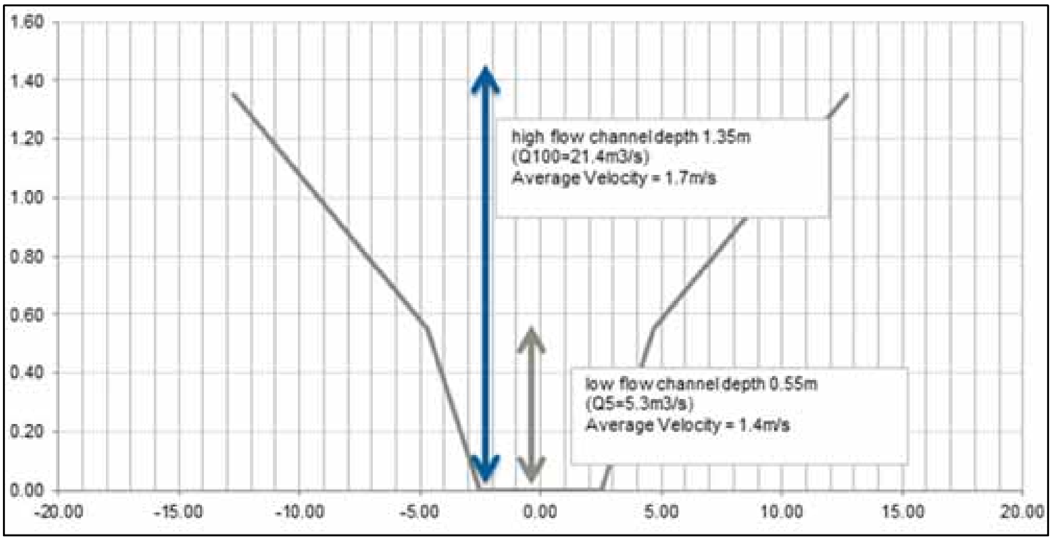

Figure 7-3 Conceptual cross section for temporary creek diversion 1

7.3.4.2 Temporary creek diversion 2

The purpose of this diversion channel is to divert flows around mining area B during years 4-8. It is an existing tributary of Laheys Creek and will divert flows west into Sandy Creek.

This channel has an approximate length of 1.1km and should be constructed with a minimum slope of 0.5%. The proposed channel will generally follow a similar elevation to a saddle in the catchment divide. At the end of the diversion channel an energy dissipater will be required to prevent localised erosion where it drops more steeply into Sandy Creek.

XP-RAFTS has been used to estimate design flows for this tributary sub-catchment and Mannings Equation calculations have been used to determine a conceptual trapezoidal channel design for the diversion, which is shown below in Figure 7-4. The channel has a similar catchment area and design flows to that of temporary creek diversion 1.

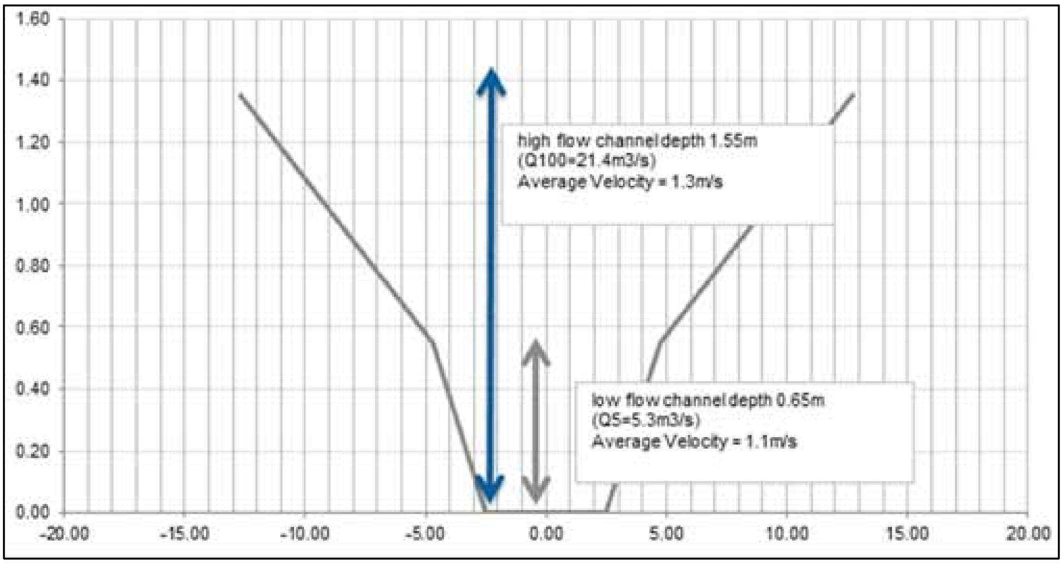

Figure 7-4 Conceptual cross section for temporary creek diversion 2

8. Conclusions and recommendations

A detailed flood investigation was undertaken to assess the impacts of the proposed mine on flooding of the watercourses within, upstream and downstream of the study area. Three crucial zones were identified where flooding impacts could be significant. The Sandy Creek catchment was subject to hydrologic and hydraulic modelling so that the impacts of the mine on Sandy Creek and Laheys Creek could be quantified. Areas outside the Sandy Creek catchment were assessed separately.

A detailed flood investigation, including hydrologic and hydraulic modelling, was undertaken to understand the existing flooding behaviour of Sandy Creek and Laheys Creek and how it affects the main mining area (Zone 1), and to determine how the proposed mine and associated infrastructure will affect flood behaviour in the catchment. Investigations were also undertaken to identify whether flooding within Sandy Creek and Laheys Creek would affect the Project, the proposed mine development and its operation.

Areas outside the Sandy Creek mining area, such as Flyblowers Creek (Zone 2) north of Sandy Creek catchment and the proposed rail spur (Zone 3) watercourse crossings, were assessed separately using other methods.

It was found that flows in Laheys Creek would increase slightly overall due to the progressive diversion of the northern part of the catchment (which naturally flows into Sandy Creek) into Laheys Creek. However, this increase in catchment area flowing to Laheys Creek would be counteracted by the loss of catchment area to the mine, so changes in peak flows overall would be minimal.

A change in the timing of peaks would occur locally around the mine footprint, but by the time the peak flow reaches Sandy Creek, the flood behaviour would remain very similar to the existing scenario. The minor changes to flood levels in Sandy Creek just downstream of its junction with Laheys Creek would be negligible at the downstream end of Sandy Creek.

The northern boundary of the mining area is approximately 2 km from the Talbragar River, and the top of the catchment containing the mine area is approximately 45 m above the banks of the Talbragar River. For all events analysed, including the 2,000-year average recurrence interval (ARI) event, flood levels in the Talbragar River will not be affected by the mine due to the distance and difference in elevation between the two locations.

There will be local increases in velocities and changes to flood levels at structures where the flooding behaviour is changed, but these impacts will be localised and will only affect land under CHC’s ownership. Areas experiencing high velocities will require channel erosion protection measures.

A feature of the mine infrastructure is the crossing of watercourses through the stages of the mine workings. These crossings have undergone preliminary sizing to check and reduce their impact on design flood levels. It is essential that erosion protection measures for both the construction and operational phases of the Project be incorporated into the detailed design of these crossings. Such measures include rock revetments and scour protection for bridge piers and abutments.

In areas where the mine footprint encroaches on the 2,000-year ARI flood extent, flood protection works will be required to protect critical infrastructure, such as the workshop area on Laheys Creek, the northern access road and the stockpile along the southern edge of Blackheath Creek.

Where the spoil emplacement encroaches on the 100-year ARI flood extent, spoil protection will be required, to prevent erosion and collapse of spoil embankments. Small diversion channels will be required temporarily throughout the earlier stages of the mine; however, the catchments of these tributaries slowly decrease as the mine progresses.

The flood protection levees and the spoil protection areas will modify flood behaviour in high order events and will prevent land from being flooded. They will therefore need to be designed in accordance with NOW guidelines for controlled work under Part 8 of the Water Act 1912.

Overall, the hydraulic modelling shows no significant change in flood levels along Sandy Creek and Laheys Creek and upstream and downstream of the main mining area. Therefore, no mitigation measures are required in the Sandy Creek catchment to reduce flooding impacts within the catchment or downstream in the Talbragar River.

However, there would be an impact on Flyblowers Creek (Zone 2), north of the Sandy Creek catchment. During years 12–20, mitigation measures would be required, to reduce the peak flow in Flyblowers Creek and avoid impacts to the Golden Highway. A dry detention basin with a capacity of 70,000 m^3 is recommended to mitigate this impact. The detention basin can be accommodated within the study area, and should be close to the disturbed area. With this detention basin, it is estimated that peak flows reaching the Golden Highway should be close to those of the existing scenario, thereby avoiding adverse impacts on the highway.

In the rail corridor, the proposed rail spur will cross local watercourses at 21 locations, including Fords Creek, Lambing Yard Creek and Tallawang Creek. Because some of the rail spur will be located on a 6 m high embankment, local flooding impacts, particularly on the upstream side of the embankment, could be significant. Therefore, waterway crossings, cross-drainage culverts and rail corridor longitudinal drainage need to be designed so that local flooding can be managed and impacts to the local environment and rail infrastructure minimised.

Watercourse crossings are a controlled activity under the Water Management Act 2000. The detailed design of the crossings will therefore need to take into account NOW guidelines on design and construction of watercourse crossings which require consideration of the following:

- Minimisation of disturbance of the riparian corridor and its function.
- Preservation of native vegetation.
- Preservation of natural hydrological and geomorphological regimes.
- Rehabilitate and stabilise disturbed areas and protect against ongoing scour and erosion.

The crossings where active fish movement occurs will need to be fish-friendly and designed in accordance with *Policy and Guidelines for Fish Friendly Waterway Crossing* (NSW Fisheries 2003) and *Why Do Fish Need to Cross the Road? Fish Passage Requirements for Waterway Crossings (*Fairfull & Witheridge 2003).

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Appendix D.1

HEC-RAS results – existing scenario

HEC-RAS results – existing scenario

Hydraulic results for 2-, 5-, 100- and 2,000-year ARI design rainfall event for all creeks and tributaries

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
T9	5076.31	2.18	420.97	4.97	421.06	20	421.31	63.55	421.64
T9	4691.95	2.18	416.72	4.97	416.8	20	417	63.55	417.24
T9	4272.49	2.18	411.03	4.97	411.12	20	411.35	63.55	411.4
T9	3882.82	2.18	403.84	4.97	404.02	20	404.5	63.55	404.97
T9	3477.03	2.18	401.98	4.97	402.1	20	402.47	63.55	403.02
T9	3037.1	2.18	399.29	4.97	399.37	20	399.6	63.55	399.95
T7	3634.5	2.18	468.87	4.97	468.97	20	469.29	63.55	469.75
T7	3195.38	2.18	453.71	4.97	453.81	20	453.99	63.55	454.24
T7	2804.41	2.18	445.16	4.97	445.26	20	445.5	63.55	445.82
T7	2391.43	2.18	437.99	4.97	438.1	20	438.41	63.55	438.81
T7	2003.62	2.18	431.14	4.97	431.24	20	431.53	63.55	432.02
T7	1596.85	2.18	425.09	4.97	425.38	20	425.8	63.55	426.28
T7	1198.37	2.18	418.61	4.97	418.77	20	419.19	63.55	419.57
T7	810.58	2.18	415.23	4.97	415.31	20	415.52	63.55	415.83
T7	400.38	2.18	410.78	4.97	410.95	20	411.16	63.55	411.41
T6	3554.34	3.67	415.61	9.95	415.75	31.96	416.06	102.27	416.54
T6	3085.28	3.67	411.32	9.95	411.48	31.96	411.74	102.27	412.18
T6	2803.38	3.67	408.4	9.95	408.54	31.96	408.87	102.27	409.41
T6	2397.17	3.67	404.52	9.95	404.93	31.96	405.52	102.27	406.29
T6	1991.35	3.67	400.95	9.95	401.08	31.96	401.35	102.27	401.62
T6	1603.5	3.67	396.63	9.95	397.07	31.96	397.58	102.27	398.15

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
T6	1197.85	3.67	394.43	9.95	394.57	31.96	394.66	102.27	395.15
T6	804.92	3.67	390.84	9.95	391.13	31.96	391.76	102.27	392.28
T6	340.67	3.67	387.87	9.95	388.08	31.96	388.1	102.27	388.54
T4	1210.26	8.87	408.91	20.26	409.28	81.75	410.17	259.55	412.05
T4	798.94	8.87	406.48	20.26	406.79	81.75	407.62	259.55	407.63
T4	383.84	8.87	402.42	20.26	402.65	81.75	403.41	259.55	404.6
T3	6530	4.09	402.49	9.38	402.77	38.12	403.07	120.88	403.39
T3	6129.97	4.09	398.93	9.38	399.09	38.12	399.6	120.88	400.19
T3	5742.66	4.09	396.39	9.38	396.63	38.12	397.07	120.88	397.47
T3	5322.14	4.09	393.82	9.38	394	38.12	394.42	120.88	394.96
T3	4908.86	4.09	390.29	9.38	390.47	38.12	390.97	120.88	391.6
T3	4564.44	4.09	388.09	9.38	388.27	38.12	388.61	120.88	389.06
T3	4399	4.09	387.03	9.38	387.18	38.12	387.52	120.88	387.97
T3	4254	4.09	386.15	9.38	386.31	38.12	386.64	120.88	387.07
T3	4159.99	4.09	385.8	9.38	385.9	38.12	386.16	120.88	386.58
T3	4000	4.09	385.09	9.38	385.22	38.12	385.6	120.88	386.08
T3	3910	4.09	384.55	9.38	384.69	38.12	385.04	120.88	385.45
T3	3810	4.09	384.34	9.38	384.45	38.12	384.69	120.88	384.96
T3	3728.48	4.09	383.32	9.38	383.38	38.12	383.61	120.88	384.02
T3	3700	4.09	382.39	9.38	382.48	38.12	382.74	120.88	383.06
T3	3650	4.09	382.1	9.38	382.17	38.12	382.32	120.88	382.76
T3	3600	4.09	381.25	9.38	381.89	38.12	382.22	120.88	382.62
T3	3540	4.09	381.1	9.38	381.86	38.12	382.17	120.88	382.51
T3	3538	Culvert		Culvert		Culvert		Culvert	
T3	3536.64	4.09	380.56	9.38	380.79	38.12	381.85	120.88	382.42

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
T2	5247.05	1.04	401.54	2.34	401.61	9.26	401.87	29.48	401.93
T2	4832.14	1.04	394.38	2.34	394.47	9.26	394.53	29.48	395.09
T2	4428.3	1.04	388.89	2.34	388.95	9.26	389.24	29.48	389.34
T2	4039.51	1.04	383.55	2.34	383.63	9.26	383.67	29.48	384.18
T2	3636.26	1.04	377.58	2.34	377.66	9.26	378.17	29.48	378.27
T2	3188.01	1.04	372.5	2.34	372.62	9.26	372.66	29.48	372.8
T2	2826.34	1.04	366.93	2.34	367	9.26	367.23	29.48	367.48
T2	2383.54	3.99	362.83	9.01	362.71	31.33	363.4	113.4	363.57
T2	2017.59	3.99	360.31	9.01	360.35	31.33	360.58	113.4	361.36
T2	1622.68	3.99	357.55	9.01	357.78	31.33	358.05	113.4	358.04
T2	1278.99	3.99	354.81	9.01	354.79	31.33	354.96	113.4	355.98
T2	1056.59	1.29	353.6	9.01	353.84	85.94	354.49	463.79	355.65
T2	825.92	1.29	352.51	17.85	352.96	304.01	354.04	862.61	355.27
T2	627.51	1.29	352.04	27.1	352.62	274.6	353.33	889.8	354.64
T1	1186.85	1.99	377.39	4.51	377.53	17.8	377.56	56.7	377.88
T1	789.36	1.99	371.52	4.51	371.38	17.8	372.31	56.7	372.63
T1	398.39	1.99	367.34	4.51	367.78	17.8	367.71	56.7	368.57
SC	24396.1	8.1	435.21	18.34	435.85	72.63	436.54	231.24	437.88
SC	24007.48	8.1	432.53	18.34	432.65	72.63	432.93	231.24	433.5
SC	23590.68	8.1	426.31	18.34	426.65	72.63	427.47	231.24	428.18
SC	23210.61	8.1	421.67	18.34	422.03	72.63	422.76	231.24	423.91
SC	22791.34	8.1	418.88	18.34	419.24	72.63	419.94	231.24	420.91
SC	22393.29	8.1	415.8	18.34	416.14	72.63	417.18	231.24	418.53
SC	22000.34	8.1	413.53	18.34	413.96	72.63	415.05	231.24	416.08
SC	21594.63	8.1	410.73	18.34	411.14	72.63	412.2	231.24	413.22

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
SC	21198.78	8.1	407.69	18.34	408.2	72.63	409.45	231.24	410.71
SC	20800.59	8.1	405.52	18.34	405.88	72.63	406.78	231.24	408.14
SC	20397.18	8.1	402.87	18.34	403.43	72.63	404.89	231.24	406.57
SC	20002.84	9.56	400.58	24.41	401.52	114.04	403.4	362.69	405.19
SC	19608.18	9.56	399.38	24.41	399.84	114.04	401.39	362.69	403.09
SC	19193.98	9.56	397.9	24.41	398.34	114.04	399.41	362.69	400.84
SC	18799.33	9.56	395.87	24.41	396.75	114.04	397.87	362.69	398.84
SC	18575.84	9.56	395.09	24.41	395.89	114.04	397.06	362.69	397.79
SC	17995.23	10.04	392.63	26.44	393.09	127.84	394.15	406.5	395.31
SC	17620.46	10.04	391.16	26.44	391.71	127.84	392.95	406.5	394.19
SC	17191.31	10.04	389.32	26.44	389.98	127.84	390.84	406.5	391.8
SC	16799.9	10.04	388.48	26.44	388.86	127.84	389.9	406.5	391.04
SC	16394.36	10.04	386.52	26.44	387.2	127.84	388.68	406.5	390.23
SC	16131.95	10.04	386	26.44	386.8	127.84	388.23	406.5	389.8
SC	16050	10.04	385.25	26.44	386.05	127.84	387.09	406.5	387.98
SC	15628.48	10.38	382.53	27.85	383.53	144.3	384.92	457.11	385.75
SC	15208.91	10.38	380.96	27.85	381.98	144.3	383.21	457.11	384.32
SC	14819.5	10.38	380.4	27.85	381.21	144.3	381.88	457.11	382.73
SC	14444.44	10.38	378.56	27.85	379.39	144.3	380.25	457.11	380.97
SC	14079.1	10.38	376.96	27.85	377.69	144.3	378.85	457.11	379.71
SC	13642.13	10.38	375.6	27.85	376.26	144.3	377.38	457.11	378.31
SC	13251.15	10.71	373.06	29.27	373.75	160.75	375.7	507.73	376.97
SC	12789.38	10.71	370.31	29.27	371.29	160.75	373.6	507.73	375.16
SC	12447.42	10.71	369.35	29.27	370.42	160.75	372.57	507.73	373.77
SC	12044.6	10.71	368.48	29.27	369.36	160.75	370.49	507.73	371.52

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
SC	11560.14	10.71	366.76	29.27	367.88	160.75	369.3	507.73	370.94
SC	11222.89	10.71	365.84	29.27	366.98	160.75	368.83	507.73	370.71
SC	10856.9	18.54	364.75	46.92	365.97	230.18	367.5	729.08	368.62
SC	10745.43	18.54	364.64	46.92	365.89	230.18	367.41	729.08	368.41
SC	10595.95	18.54	363.8	46.92	365.17	230.18	366.58	729.08	368.06
SC	10294.08	30.1	362.75	81.69	363.95	417.24	365.82	1315.88	366.97
SC	10032.72	30.1	362.63	81.69	363.72	417.24	365.67	1315.88	366.56
SC	9613.45	30.1	362.01	81.69	362.99	417.24	364.26	1315.88	365.19
SC	9193.18	30.1	360.97	81.69	362	417.24	363.15	1315.88	364.52
SC	8774.07	30.1	359.67	81.69	360.93	417.24	362.31	1315.88	363.53
SC	8431.69	30.1	358.85	81.69	359.88	417.24	361.46	1315.88	362.3
SC	8290.61	30.1	358.61	81.69	359.61	417.24	360.9	1315.88	361.93
SC	7497.25	30.1	356.58	81.69	357.87	417.24	359.61	1315.88	360.92
SC	7046.07	30.1	356.06	81.69	357.11	417.24	358.82	1315.88	360.21
SC	6742.61	30.1	355.74	81.69	356.67	417.24	358.15	1315.88	359.22
SC	6406.71	30.1	355.17	81.69	356.2	417.24	357.44	1315.88	358.32
SC	6066.35	30.1	354.5	81.69	355.55	417.24	356.7	1315.88	357.63
SC	5524.29	30.1	353.8	81.69	354.51	417.24	355.42	1315.88	356.41
SC	5203	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	5202.72	32.8	353.48	81.69	354.12	362.62	354.81	965.48	355.83
SC	4814	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	4813.22	32.8	352.52	72.86	353.27	144.55	353.88	566.67	355.09
SC	4499.5	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	4499.48	32.8	352.14	63.61	352.83	173.97	353.49	539.47	354.82
SC	4062.73	28.53	351.75	77.76	352	429.05	352.92	1354.35	354.34

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
SC	3672.02	28.53	350.79	77.76	351.37	429.05	352.33	1354.35	353.81
SC	3418.04	28.53	349.97	77.76	351.17	429.05	351.96	1354.35	353.43
SC	2875.94	28.53	348.35	77.76	349.36	429.05	350.84	1354.35	352.21
SC	2398.41	28.53	347.6	77.76	348.44	429.05	349.75	1354.35	351.13
SC	2270	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	2262.77	28.53	347.54	76.89	348.32	401.08	349.51	1227.73	350.81
SC	2000	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	1963	28.53	347.43	30.46	348.21	21.92	349.44	328.97	350.69
SC	1400	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	1213.63	28.53	345.14	76.89	346.09	386.7	347.68	899.06	349.72
SC	1100	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	767.56	28.53	344.2	77.76	345.36	82.84	347.86	126.63	349.65
SC	500	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	412.38	28.53	343.77	77.76	344.92	407.2	347.67	899.44	349.48
SC	350	28.53	343.22	77.76	344.27	429.05	347.03	1354.35	348.92
LC	13198.68	2.43	421.83	16.67	422.51	64.85	423.75	207.4	425.1
LC	12806.65	2.43	419.07	16.67	420.4	64.85	421.72	207.4	422.98
LC	12424.45	2.43	417.19	16.67	418.66	64.85	419.6	207.4	420.29
LC	11995.3	2.43	414.46	16.67	415.62	64.85	416.76	207.4	417.46
LC	11629.37	2.43	412.07	16.67	413.07	64.85	414.49	207.4	415.05
LC	11211.95	2.43	410.16	16.67	411.02	64.85	412.14	207.4	413.24
LC	10825.65	2.43	408.82	16.67	409.55	64.85	410.76	207.4	412.07
LC	10440.81	2.43	406.12	16.67	406.44	64.85	406.63	207.4	407.3
LC	10236.23	2.43	400.94	16.67	401.37	64.85	402.9	207.4	404.08
LC	9608.36	16.3	397.82	36.92	398.69	146.6	400.46	466.57	401.84

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
LC	9208.76	16.3	396.63	36.92	397.18	146.6	398.64	466.57	399.44
LC	8920	16.3	395.74	36.92	396.31	146.6	397.77	466.57	399.13
LC	8900	16.3	395.49	36.92	396.1	146.6	397.5	466.57	398.67
LC	8870	16.3	394.65	36.92	395.34	146.6	396.73	466.57	397.97
LC	8820	16.3	394.25	36.92	394.89	146.6	396.45	466.57	397.55
LC	8811.17	16.04	394.1	37	394.74	151.77	396.32	482.82	397.39
LC	8377.35	16.04	392.36	37	393.09	151.77	394.24	482.82	395.33
LC	7992.23	16.04	390.44	37	391.26	151.77	392.79	482.82	393.97
LC	7639.78	16.04	389.24	37	389.95	151.77	391.5	482.82	392.56
LC	7204.96	16.04	388.06	37	388.64	151.77	390.14	482.82	390.84
LC	6761.61	16.04	386.27	37	387.02	151.77	388.32	482.82	389.29
LC	6420.26	16.04	385.43	37	386.13	151.77	387.25	482.82	388.16
LC	6004.29	16.04	382.59	37	383.15	151.77	385.02	482.82	386.37
LC	5660	16.04	381.14	37	381.92	151.77	383.87	482.82	385.2
LC	5644.39	16.04	380.89	37	381.71	151.77	383.67	482.82	384.86
LC	5500	16.04	380.74	37	381.62	151.77	383.58	482.82	384.7
LC	5352.7	16.04	380.19	37	381.2	151.77	383.09	482.82	384.02
LC	5003	14.99	378.63	37.3	379.35	172.45	380.85	547.86	381.73
LC	4823	14.99	378.4	37.3	379.07	172.45	380.49	547.86	381.27
LC	4808.16	14.99	378.19	37.3	378.86	172.45	380.17	547.86	380.94
LC	4571.76	14.99	377.59	37.3	378.32	172.52	379.61	557.01	380.41
LC	4326	13.94	376.89	37.6	377.41	193.13	378.16	612.89	378.96
LC	4253	13.94	376.58	37.6	377.12	193.13	377.86	612.89	378.56
LC	4103.04	13.94	376.28	37.6	376.91	193.13	377.67	612.89	378.32
LC	3586.81	13.94	374.92	37.6	375.68	193.13	376.33	612.89	377.13

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
LC	3424.42	13.94	374.47	37.6	375.24	193.13	375.93	612.89	376.77
LC	2838.58	13.94	373.13	37.6	373.68	193.13	374.49	612.89	375.36
LC	2440.29	13.94	372.51	37.6	372.87	193.13	373.59	612.89	374.54
LC	1771.3	13.94	369.07	37.6	369.81	193.13	370.82	612.89	371.62
LC	1592.37	13.94	368.3	37.6	369.08	193.13	370.09	612.89	371.02
LC	1207.16	13.94	367.9	37.6	368.62	193.13	369.34	612.89	370.16
LC	967.02	13.94	367.41	37.6	368.02	193.13	368.76	612.89	369.62
LC	216.38	13.94	363.48	37.6	364.76	193.13	366.33	612.89	367.9
12	2022.95	2.3	398.62	5.27	398.68	21.44	398.87	67.99	399.44
12	1656.23	2.3	395	5.27	395.15	21.44	395.38	67.99	395.44
12	1242.83	2.3	390.02	5.27	390.29	21.44	390.76	67.99	391.27
12	789.38	2.3	385.1	5.27	385.25	21.44	385.5	67.99	385.65
12	700	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
12	379.26	2.3	379.99	5.27	380.08	21.37	380.59	58.85	381.05
11	5219.22	3.91	401.19	8.82	401.43	34.71	402.03	110.68	402.64
11	4811.29	3.91	398.17	8.82	398.32	34.71	398.61	110.68	399.01
11	4399.31	3.91	394.29	8.82	394.45	34.71	394.79	110.68	395.28
11	3971.94	3.91	390.48	8.82	390.6	34.71	390.89	110.68	391.24
11	3575.7	3.91	386.14	8.82	386.34	34.71	386.85	110.68	387.39
11	3195.69	3.91	383.56	8.82	383.65	34.71	383.83	110.68	384.12
11	2763.45	3.91	380.05	8.82	380.12	37.13	380.42	125.45	380.81
11	2443.27	3.91	378.17	8.82	378.3	45.1	378.67	165.4	379.24
11	2168.85	7.83	376.94	17.65	377.08	69.42	377.45	221.35	377.82
11	1544.85	7.83	373.9	17.65	374.03	69.42	374.34	221.35	374.73
11	1230.99	7.83	372.29	17.65	372.42	69.42	372.78	221.35	373.17

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
11	1160	7.83	371.85	17.65	371.99	69.42	372.17	221.35	372.52
11	1075	7.83	371.07	17.65	371.16	69.42	371.58	221.35	371.84
11	1000	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
11	701.66	7.83	369.51	17.65	369.81	69.42	370.06	221.35	370.75
11	700	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
11	347.2	7.83	368.46	17.65	368.54	69.42	369.05	221.35	370.55
10	5220.59	3.91	424.46	8.82	424.59	34.71	425.01	110.68	425.66
10	4804.14	3.91	421.08	8.82	421.25	34.71	421.69	110.68	422.08
10	4389.26	3.91	413.66	8.82	413.81	34.71	414.2	110.68	414.57
10	3960.26	3.91	406.95	8.82	407.17	34.71	407.47	110.68	407.89
10	3651.41	3.91	402.28	8.82	402.57	34.71	403.47	110.68	404.23
10	3229.31	3.91	399.1	8.82	399.38	34.71	399.83	110.68	400.46
10	2805.48	3.91	396.25	8.82	396.31	34.71	396.64	110.68	397.19
10	2410.96	3.91	392.72	8.82	393.1	34.71	393.56	110.68	394.23
10	2010.26	3.91	390.6	8.82	390.67	34.71	391.01	110.68	391.58
10	1606.27	3.91	388.19	8.82	388.36	34.71	388.7	110.68	389.15
10	1197.16	3.91	385.33	8.82	385.52	34.71	385.92	110.68	386.33
10	1000	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
10	617.5	3.91	381.52	8.82	381.72	32.29	382.11	95.91	382.36
10	300	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
10	287.64	3.91	379.53	8.82	379.66	24.33	379.77	55.95	379.92

Appendix D.2

HEC-RAS results – proposed scenario

HEC-RAS results – proposed scenario

Hydraulic results for 2-, 5-, 100- and 2,000-year ARI design rainfall event for all creeks and tributaries

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
T9	5076.31	2.2	420.97	5	421.06	20	421.31	63.5	421.64
T9	4691.95	2.2	416.72	5	416.8	20	417	63.5	417.24
T9	4272.49	2.2	411.03	5	411.12	20	411.35	63.5	411.4
T9	3882.82	2.2	403.85	5	404.02	20	404.5	63.5	404.97
T9	3477.03	2.2	401.98	5	402.11	20	402.47	63.5	403.02
T9	3037.1	2.2	399.29	5	399.37	20	399.6	63.5	399.95
T7	3634.5	2.2	468.87	5	468.97	20	469.29	63.5	469.75
T7	3195.38	2.2	453.71	5	453.81	20	453.99	63.5	454.24
T7	2804.41	2.2	445.16	5	445.26	20	445.5	63.5	445.82
T7	2391.43	2.2	437.99	5	438.1	20	438.41	63.5	438.81
T7	2003.62	2.2	431.14	5	431.24	20	431.53	63.5	432.02
T7	1596.85	2.2	425.09	5	425.38	20	425.8	63.5	426.28
T7	1198.37	2.2	418.61	5	418.77	20	419.19	63.5	419.58
T7	810.58	2.2	415.23	5	415.31	20	415.52	63.5	415.82
T7	400.38	2.2	410.78	5	410.95	20	411.16	63.5	411.41
T6	3554.34	4.5	415.57	7.9	415.69	30.3	416.04	96.6	416.53
T6	3085.28	4.5	411.37	7.9	411.44	30.3	411.73	96.6	412.14
T6	2803.38	4.5	408.34	7.9	408.47	30.3	408.85	96.6	409.4
T6	2397.17	4.5	404.66	7.9	404.85	30.3	405.49	96.6	406.23
T6	1991.35	4.5	400.88	7.9	401.02	30.3	401.33	96.6	401.64
T6	1603.5	4.5	396.79	7.9	396.98	30.3	397.46	96.6	397.99
T6	1197.85	4.5	394.38	7.9	394.51	30.3	394.94	96.6	395.5

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
T6	804.92	4.5	390.99	7.9	391.08	30.3	391.51	96.6	392.01
T6	702.69*	4.5	390.16	7.9	390.28	30.3	390.72	96.6	391.25
T6	600.46*	4.5	389.34	7.9	389.48	30.3	390.03	96.6	391.05
T6	498.23*	4.5	388.54	7.9	388.75	30.3	389.96	96.6	391.02
T6	460	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
T6	447.115*	4.5	388.24	7.9	388.52	22.11	389.96	27.88	391.02
T6	396	4.5	388	7.9	388.28	22.11	389.89	27.88	390.97
T6	350	Culvert		Culvert		Culvert		Culvert	
T6	340.67	4.5	387.9	7.9	388.08	22.11	387.97	27.88	388.08
T4	1210.26	8.9	408.91	20.3	409.29	81.7	410.17	259.6	412.05
T4	798.94	8.9	406.48	20.3	406.79	81.7	407.62	259.6	407.63
T4	383.84	8.9	402.41	20.3	402.65	81.7	403.41	259.6	404.6
T3	6530	10.89	402.77	12.85	402.81	39.5	402.85	108.3	403.48
T3	6129.97	10.89	399.16	12.85	399.21	39.5	399.75	108.3	399.91
T3	5742.66	10.89	396.64	12.85	396.68	39.5	396.94	108.3	397.7
T3	5322.14	10.89	394.09	12.85	394.13	39.49	394.57	108.27	394.49
T3	4908.86	10.89	390.44	12.85	390.48	39.49	390.57	108.27	393.82
T3	4564.44	10.89	388.36	12.85	388.39	39.49	388.92	108.27	393.82
T3	4399	10.89	387.42	12.85	387.51	39.49	388.55	108.27	393.76
T3	4300	Culvert		Culvert		Culvert		Culvert	
T3	4254	10.89	386.44	12.85	386.47	39.49	386.75	108.27	387.24
T3	4159.99	10.89	385.88	12.85	385.92	39.49	386.26	108.27	387.2
T3	4000	10.89	385.32	12.85	385.33	39.49	386.2	108.27	387.18
T3	3910	10.89	385.28	12.85	385.28	39.49	386.19	108.27	387.17
T3	3810	10.89	384.49	12.85	384.5	39.49	386.19	108.27	387.17
T3	3728.48	10.89	384.35	12.85	384.44	32.12	386.18	108.27	387.15

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
T3	3710	Culvert		Culvert		Culvert		Culvert	
T3	3700	10.89	382.93	12.85	383.03	32.12	383.89	108.27	385.68
T3	3650	10.89	382.89	12.85	382.98	32.12	383.86	108.27	385.65
T3	3610	Culvert		Culvert		Culvert		Culvert	
T3	3600	10.89	381.93	12.85	381.95	32.12	382.19	108.27	382.62
T3	3540	10.89	381.89	12.85	381.93	32.12	382.14	108.27	382.49
T3	3538	Culvert		Culvert		Culvert		Culvert	
T3	3536.64	10.89	380.82	12.85	380.87	32.12	381.89	108.27	382.44
T2	5247.05	0.1	401.46	0.1	401.42	0.1	401.42	0.1	401.42
T2	4832.14	0.1	394.17	0.1	394.19	0.1	394.19	0.1	394.2
T2	4428.3	0.1	388.73	0.1	388.68	0.1	388.68	0.1	388.65
T2	4039.51	0.1	383.4	0.1	383.42	0.1	383.42	0.1	383.39
T2	3636.26	0.1	377.47	0.1	377.45	0.1	377.45	0.1	377.49
T2	3188.01	0.1	372.28	0.1	372.3	0.1	372.3	0.1	372.25
T2	2826.34	0.1	366.88	0.1	366.85	0.1	366.85	0.1	366.9
T2	2383.54	0.1	362.29	0.1	362.33	0.1	362.33	0.1	362.25
T2	2017.59	0.1	360.08	0.1	360.05	0.1	360.05	0.1	360.09
T2	1622.68	0.1	357.28	0.1	357.31	0.1	357.31	0.1	357.24
T2	1278.99	0.1	354.63	0.1	354.56	0.1	354.56	0.1	355.65
T2	1056.59	0	353.33	1.64	353.74	54.89	354.41	339.97	355.44
T2	825.92	0	352.37	12.17	352.84	281.11	353.94	785.9	355.03
T2	627.51	0	352.08	23.25	352.63	221	353.3	613.51	354.62
T1	1186.85	0.1	377.25	0.1	377.25	0.1	377.25	0.1	377.3
T1	789.36	0.1	371.15	0.1	371.16	0.1	371.16	0.1	371.12
T1	398.39	0.1	367.18	0.1	367.16	0.1	367.16	0.1	367.21
SC	24396.1	8.1	435.21	18.3	435.85	72.6	436.54	231.2	437.88

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
SC	24007.48	8.1	432.53	18.3	432.66	72.6	432.93	231.2	433.5
SC	23590.68	8.1	426.31	18.3	426.65	72.6	427.47	231.2	428.18
SC	23210.61	8.1	421.67	18.3	422.02	72.6	422.76	231.2	423.91
SC	22791.34	8.1	418.88	18.3	419.24	72.6	419.94	231.2	420.91
SC	22393.29	8.1	415.8	18.3	416.14	72.6	417.18	231.2	418.53
SC	22000.34	8.1	413.53	18.3	413.96	72.6	415.05	231.2	416.08
SC	21594.63	8.1	410.73	18.3	411.14	72.6	412.2	231.2	413.22
SC	21198.78	8.1	407.69	18.3	408.2	72.6	409.45	231.2	410.71
SC	20800.59	8.1	405.52	18.3	405.88	72.6	406.78	231.2	408.14
SC	20397.18	8.1	402.87	18.3	403.43	72.6	404.89	231.2	406.57
SC	20002.84	9.6	400.59	24.4	401.52	114	403.4	362.7	405.19
SC	19608.18	9.6	399.38	24.4	399.84	114	401.39	362.7	403.09
SC	19193.98	9.6	397.9	24.4	398.34	114	399.4	362.7	400.84
SC	18799.33	9.6	395.87	24.4	396.75	114	397.87	362.7	398.84
SC	18575.84	9.6	395.09	24.4	395.89	114	397.05	362.7	397.79
SC	17995.23	10	392.64	26.4	393.08	127.8	394.15	406.5	395.31
SC	17620.46	10	391.15	26.4	391.71	127.8	392.93	406.5	394.18
SC	17191.31	10	389.34	26.4	389.98	127.8	390.87	406.5	391.82
SC	16799.9	10	388.38	26.4	388.85	127.8	389.83	406.5	391.26
SC	16394.36	10	386.6	26.4	387.22	127.8	388.95	406.5	390.79
SC	16281.9*	10	386.13	26.4	386.78	127.8	388.73	406.5	390.65
SC	16206.9*	10	385.69	26.4	386.3	127.8	388.59	406.5	390.57
SC	16169.4*	10	384.98	26.4	385.5	135.99	388.53	475.22	390.53
SC	16131.95	10	384.75	26.4	385.26	135.99	388.39	475.22	390.49
SC	16100	Culvert		Culvert		Culvert		Culvert	
SC	16050	10	384.35	26.4	385.11	135.99	386.76	475.22	388.12

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
SC	15628.48	10.4	382.53	27.8	383.53	143.5	384.92	454.1	385.75
SC	15208.91	10.4	380.96	27.8	381.98	143.5	383.21	454.1	384.31
SC	14819.5	10.4	380.4	27.8	381.17	143.5	381.88	454.1	382.73
SC	14444.44	10.4	378.56	27.8	379.46	143.5	380.24	454.1	380.96
SC	14079.1	10.4	376.96	27.8	377.81	143.5	378.85	454.1	379.71
SC	13642.13	10.4	375.6	27.8	376.25	143.5	377.38	454.1	378.3
SC	13251.15	10.7	373.06	29.1	373.74	159.1	375.69	501.7	376.95
SC	12789.38	10.7	370.31	29.1	371.28	159.1	373.59	501.7	375.15
SC	12447.42	10.7	369.35	29.1	370.42	159.1	372.55	501.7	373.75
SC	12044.6	10.7	368.48	29.1	369.35	159.1	370.48	501.7	371.5
SC	11560.14	10.7	366.76	29.1	367.87	159.1	369.3	501.7	370.92
SC	11222.89	10.7	365.85	29.1	366.97	159.1	368.83	501.7	370.7
SC	10856.9	18.6	364.79	46.8	365.99	228.7	367.6	723	370.14
SC	10837	Bridge		Bridge		Bridge		Bridge	
SC	10745.43	18.6	364.67	46.8	365.89	228.7	367.4	723	368.4
SC	10595.95	18.6	363.92	46.8	365.19	228.7	366.59	723	368.06
SC	10294.08	34.5	362.93	86	364.01	422.8	365.84	1317.5	366.97
SC	10032.72	34.5	362.8	86	363.78	422.8	365.68	1317.5	366.57
SC	9613.45	34.5	362.17	86	363.03	422.8	364.26	1317.5	365.19
SC	9193.18	34.5	361.08	86	362.08	422.8	363.16	1317.5	364.52
SC	8774.07	34.5	359.81	86	361.03	422.8	362.31	1317.5	363.52
SC	8431.69	34.5	358.97	86	359.93	422.8	361.47	1317.5	362.46
SC	8290.61	34.5	358.72	86	359.65	422.8	360.92	1317.5	362.2
SC	7497.25	34.5	356.75	86	357.93	422.8	359.63	1317.5	361.07
SC	7046.07	34.5	356.19	86	357.16	422.8	358.84	1317.5	360.21
SC	6742.61	34.5	355.85	86	356.72	422.8	358.16	1317.5	359.22

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
SC	6406.71	34.5	355.29	86	356.25	422.8	357.45	1317.5	358.32
SC	6066.35	34.5	354.63	86	355.59	422.8	356.71	1317.5	357.63
SC	5524.29	34.5	353.86	86	354.54	422.8	355.43	1317.5	356.43
SC	5203	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	5202.72	34.6	353.51	84.46	354.14	368.01	354.82	977.63	355.84
SC	4814	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	4813.22	34.6	352.61	73.93	353.27	141.79	353.95	531.7	355.21
SC	4499.5	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	4499.48	34.6	352.23	62.85	352.85	201.9	353.52	704.09	354.87
SC	4062.73	31.4	351.81	82.6	351.94	431.5	352.92	1355.3	354.34
SC	3672.02	31.4	350.86	82.6	351.42	431.5	352.33	1355.3	353.81
SC	3418.04	31.4	350.05	82.6	349.58	431.5	351.96	1355.3	353.43
SC	2875.94	31.4	348.43	82.6	349.44	431.5	350.84	1355.3	352.21
SC	2398.41	31.4	347.71	82.6	348.48	431.5	349.75	1355.3	351.13
SC	2270	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	2262.77	31.4	347.66	81.49	348.36	403.31	349.51	1228.42	350.81
SC	2000	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	1963	31.4	347.54	28.68	348.25	22.35	349.45	328.58	350.69
SC	1400	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	1213.63	31.4	345.25	81.49	346.15	388.43	347.69	901.15	349.72
SC	1100	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	767.56	31.4	344.31	82.6	345.44	82.72	347.87	127	349.65
SC	500	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
SC	412.38	31.4	343.86	82.6	345.01	409.19	347.68	901.15	349.48
SC	350	31.4	343.3	82.6	344.35	431.5	347.04	1355.3	348.92
LC	13198.68	2.4	421.83	16.7	422.51	64.9	423.75	207.4	425.1

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
LC	12806.65	2.4	419.06	16.7	420.4	64.9	421.73	207.4	422.98
LC	12424.45	2.4	417.19	16.7	418.67	64.9	419.6	207.4	420.29
LC	11995.3	2.4	414.46	16.7	415.63	64.9	416.77	207.4	417.46
LC	11629.37	2.4	412.07	16.7	413.07	64.9	414.49	207.4	415.05
LC	11211.95	2.4	410.15	16.7	411.03	64.9	412.14	207.4	413.24
LC	10825.65	2.4	408.81	16.7	409.55	64.9	410.75	207.4	412.05
LC	10440.81	2.4	406.12	16.7	406.44	64.9	406.64	207.4	407.33
LC	10236.23	2.4	400.94	16.7	401.37	64.9	402.88	207.4	404.05
LC	9608.36	16.3	397.86	36.9	398.7	146.6	400.49	467	401.86
LC	9208.76	17.3	396.66	37.9	397.2	152	398.67	479	399.44
LC	8920	17.3	395.78	37.9	396.33	152	397.81	479	398.94
LC	8900	17.3	395.54	37.9	396.12	152	397.54	479	398.71
LC	8870	17.3	394.69	37.9	395.37	152	396.75	479	398.02
LC	8820	17.3	394.31	37.9	394.91	152	396.45	479	397.63
LC	8811.17	17.3	394.18	37.9	394.76	152	396.33	479	397.5
LC	8377.35	17.3	392.48	37.9	393.11	152	394.25	479	395.56
LC	7992.23	17.4	390.53	38.1	391.28	154.6	392.82	486.9	394.25
LC	7639.78	17.4	389.31	38.1	389.98	154.6	391.65	486.9	392.77
LC	7204.96	17.4	388.12	38.1	388.67	154.6	390.06	486.9	391.12
LC	6761.61	17.6	386.34	38.3	387.06	157.3	388.34	494.7	389.43
LC	6420.26	17.6	385.51	38.3	386.18	157.3	387.29	494.7	388.15
LC	6004.29	18.2	382.64	39	383.17	165.4	385.12	518.2	386.62
LC	5660	18.2	381.32	39	382.05	165.4	384.06	518.2	386.05
LC	5659	Bridge		Bridge		Bridge		Bridge	
LC	5644.39	18.2	381.02	39	381.78	165.4	383.73	518.2	384.9
LC	5500	18.2	380.91	39	381.7	165.4	383.63	518.2	384.78

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
LC	5352.7	18.2	380.5	39	381.28	165.4	383.11	518.2	384.05
LC	5003	19.6	378.88	40.5	379.42	184.2	380.91	573	381.78
LC	4823	19.6	378.59	40.5	379.15	184.2	380.53	573	381.36
LC	4808.16	19.6	378.37	40.5	378.94	184.2	380.25	573	381.1
LC	4571.76	19.6	377.84	40.5	378.39	184.2	379.6	573	380.66
LC	4326	19.7	377.12	40.7	377.44	186.6	378.27	580.9	380.48
LC	4253	19.7	376.81	40.7	377.2	186.8	378.23	580.9	380.46
LC	4150	Bridge		Bridge		Bridge		Bridge	
LC	4103.04	19.7	376.56	40.7	376.94	186.8	377.64	580.9	378.27
LC	3586.81	19.7	375.15	40.7	375.76	186.8	376.32	580.9	377.09
LC	3424.42	19.7	374.7	40.7	375.32	186.8	375.91	580.9	376.72
LC	2838.58	19.7	373.36	40.7	373.71	186.8	374.47	580.9	375.32
LC	2440.29	19.7	372.7	40.7	372.9	186.8	373.58	580.9	374.51
LC	1771.3	19.9	369.36	40.9	369.88	189.5	370.79	588.7	371.57
LC	1592.37	19.9	368.63	40.9	369.15	189.5	370.1	588.7	371.02
LC	1207.16	20.7	368.21	41.7	368.69	200.3	369.36	620	370.17
LC	967.02	20.7	367.68	41.7	368.13	200.3	368.78	620	369.63
LC	216.38	20.7	363.74	41.7	364.85	200.3	366.35	620	367.91
12	2022.95	0.1	398.48	0.1	398.48	0.1	398.52	0.1	398.52
12	1656.23	0.1	394.77	0.1	394.77	0.1	394.69	0.1	394.69
12	1242.83	0.1	389.59	0.1	389.59	0.1	389.74	0.1	389.74
12	789.38	0.1	384.79	0.1	384.79	0.1	384.75	0.1	384.75
12	700	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
12	379.26	0.1	379.81	0.1	379.81	0.1	379.91	0.1	380.56
11	5219.22	3.9	401.37	8.8	401.7	34.7	401.93	110.7	402.49
11	4811.29	3.9	398.07	8.8	398.15	34.7	398.65	110.7	399.07

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow	Flood Level	Peak Flow	Flood Level	Peak Flow	Flood Level	Peak Flow	Flood Level
		(m^3/s)	(mAHD)	(m^3/s)	(mAHD)	(m^3/s)	(mAHD)	(m^3/s)	(mAHD)
11	4399.31	3.9	394.46	8.8	394.64	34.7	394.71	110.7	395.09
11	3971.94	3.9	390.37	8.8	390.43	34.7	390.93	110.7	391.33
11	3575.7	3.9	386.41	8.8	386.66	34.7	386.7	110.7	387.2
11	3195.69	3.9	383.42	8.8	383.5	34.7	383.87	110.7	384.23
11	2763.45	3.9	380.18	8.8	380.3	34.7	380.29	110.98	380.6
11	2443.27	3.9	378.01	8.8	378.08	34.74	378.83	169.37	379.5
11	2168.85	7.8	377.11	17.7	377.34	69.4	377.13	221.4	377.48
11	1544.85	7.8	374.6	17.7	374.64	69.4	374.99	221.4	375.7
11	1230.99	7.8	372.39	17.7	372.58	69.4	373.44	221.4	375.67
11	1160	7.8	371.99	17.7	372.31	69.4	373.3	221.4	375.67
11	1150	Culvert		Culvert		Culvert		Culvert	
11	1075	7.8	371.26	17.7	371.38	69.4	372.02	221.4	372.59
11	1000	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
11	701.66	7.8	369.51	17.7	369.82	69.4	370.07	221.4	370.76
11	700	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
11	347.2	7.8	368.46	17.7	368.54	69.4	369.03	221.4	370.57
10	5220.59	3.9	424.46	8.8	424.59	34.7	425.02	110.7	425.68
10	4804.14	3.9	421.07	8.8	421.25	34.7	421.68	110.7	422.05
10	4389.26	3.9	413.66	8.8	413.82	34.7	414.2	110.7	414.61
10	3960.26	3.9	406.95	8.8	407.17	34.7	407.4	110.7	407.85
10	3651.41	3.9	402.28	8.8	402.57	34.7	403.62	110.7	404.28
10	3229.31	3.9	399.1	8.8	399.38	34.7	399.72	110.7	400.4
10	2805.48	3.9	396.25	8.8	396.31	34.7	396.75	110.7	397.26
10	2410.96	3.9	392.74	8.8	393.1	34.7	393.39	110.7	394.1
10	2010.26	3.9	390.58	8.8	390.66	34.7	391.18	110.7	391.76
10	1606.27	3.9	388.2	8.8	388.36	34.7	388.47	110.7	388.95

Reach	Chainage	2 year		5 year		100 year		2000 year	
		Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)	Peak Flow (m^3/s)	Flood Level (mAHD)
10	1197.16	3.9	385.21	8.8	385.51	34.7	386.14	110.7	386.54
10	1000	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
10	617.5	3.9	381.54	8.8	381.73	34.7	381.73	110.42	382.19
10	300	Lat Struct		Lat Struct		Lat Struct		Lat Struct	
10	287.64	3.9	379.33	8.8	379.65	34.66	380.07	52.03	380.33

Appendix D.3

Watercourse crossing schematics from HEC-RAS

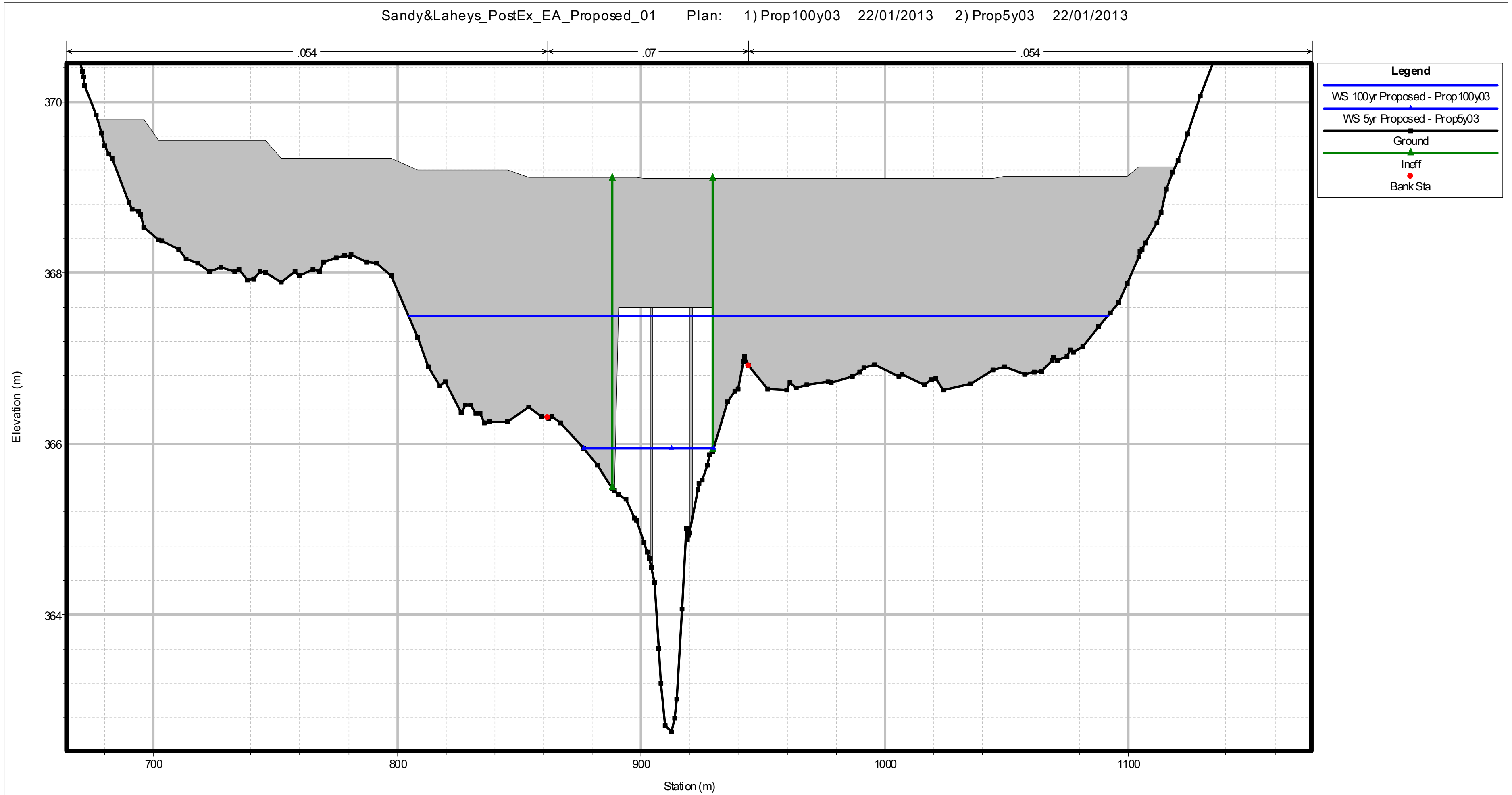


Figure D3.1 Crossing 1 schematic

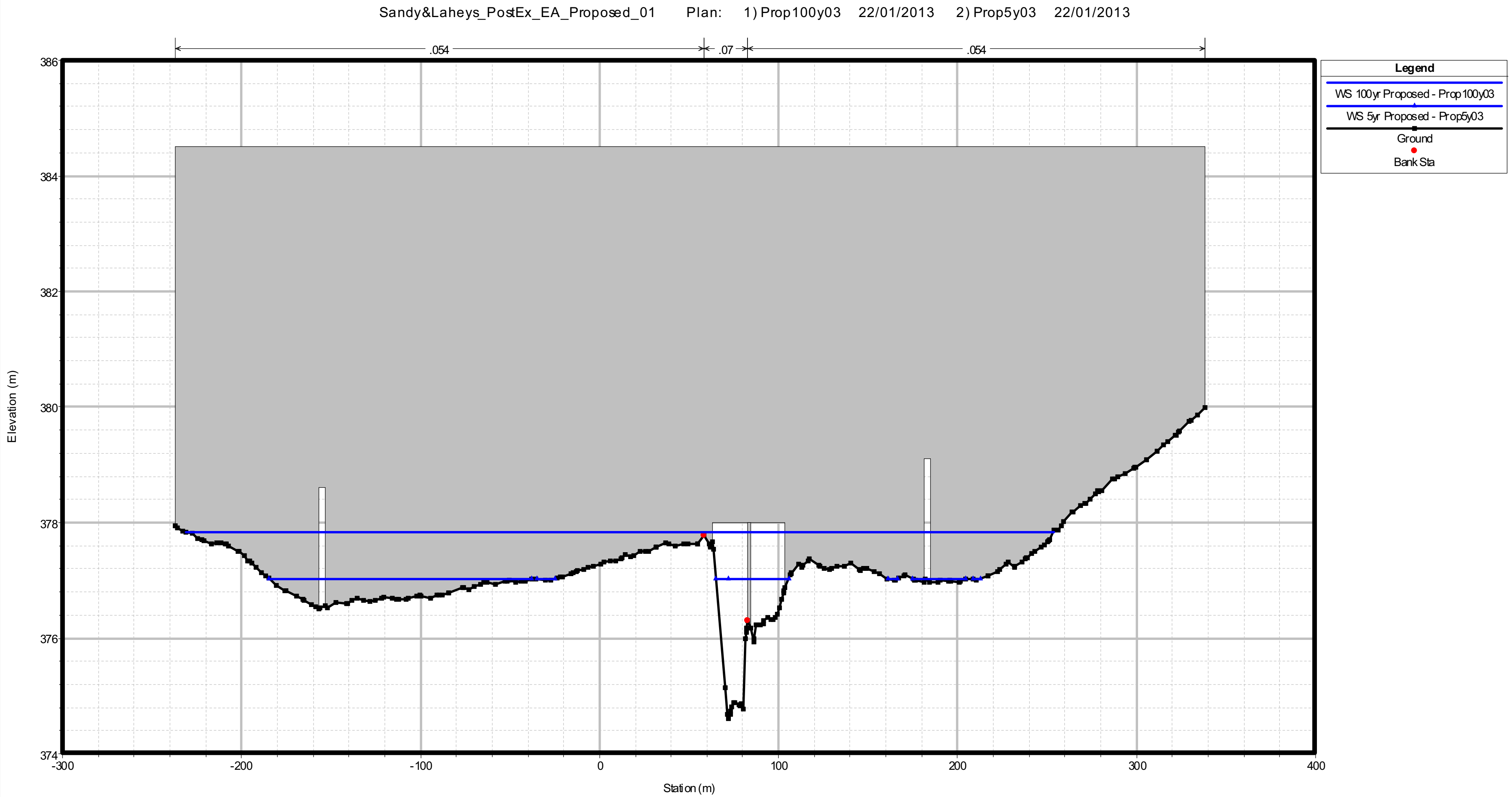


Figure D3.2 Crossing 2 schematic

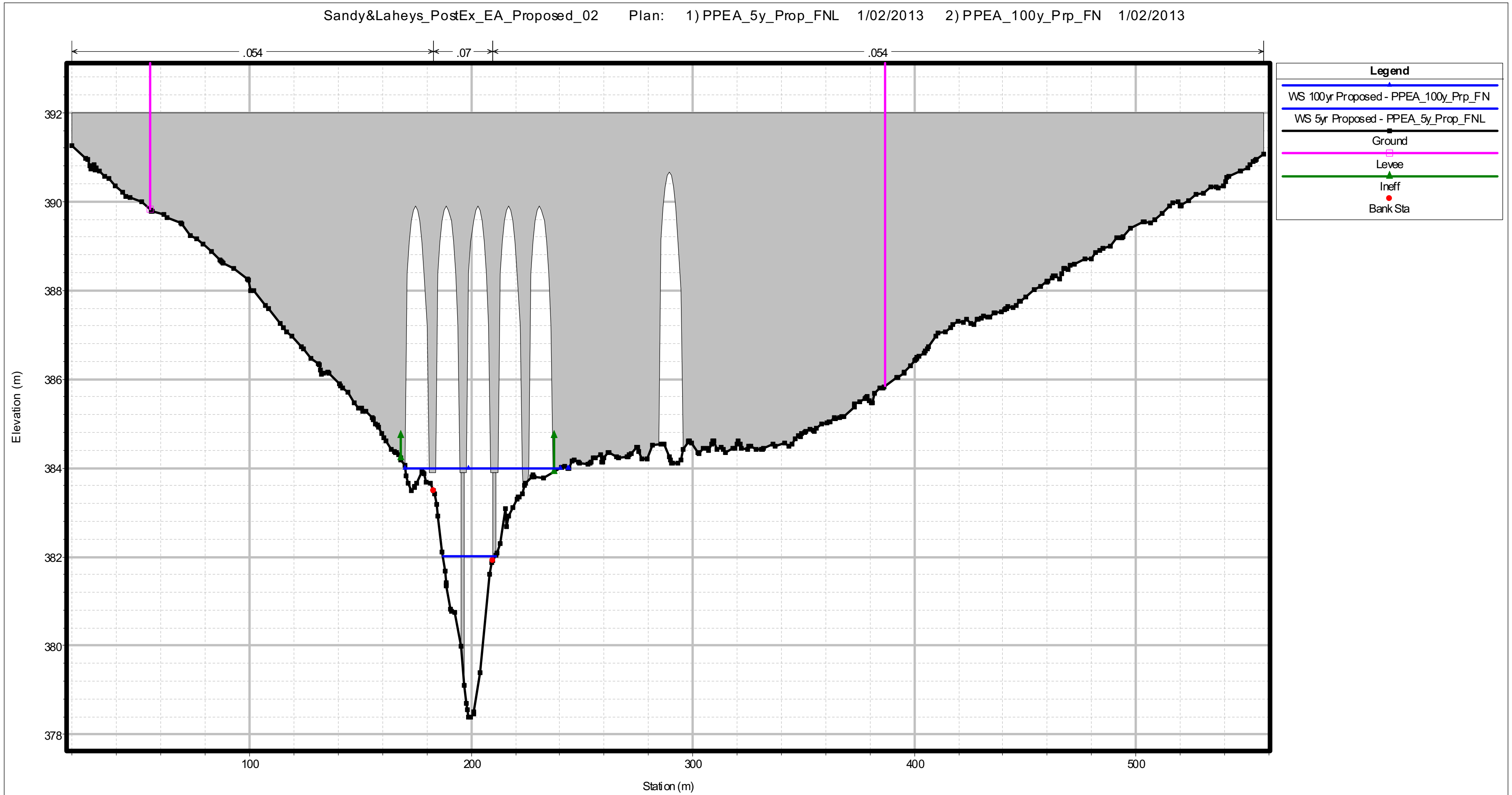


Figure D3.3 Crossing 3 schematic

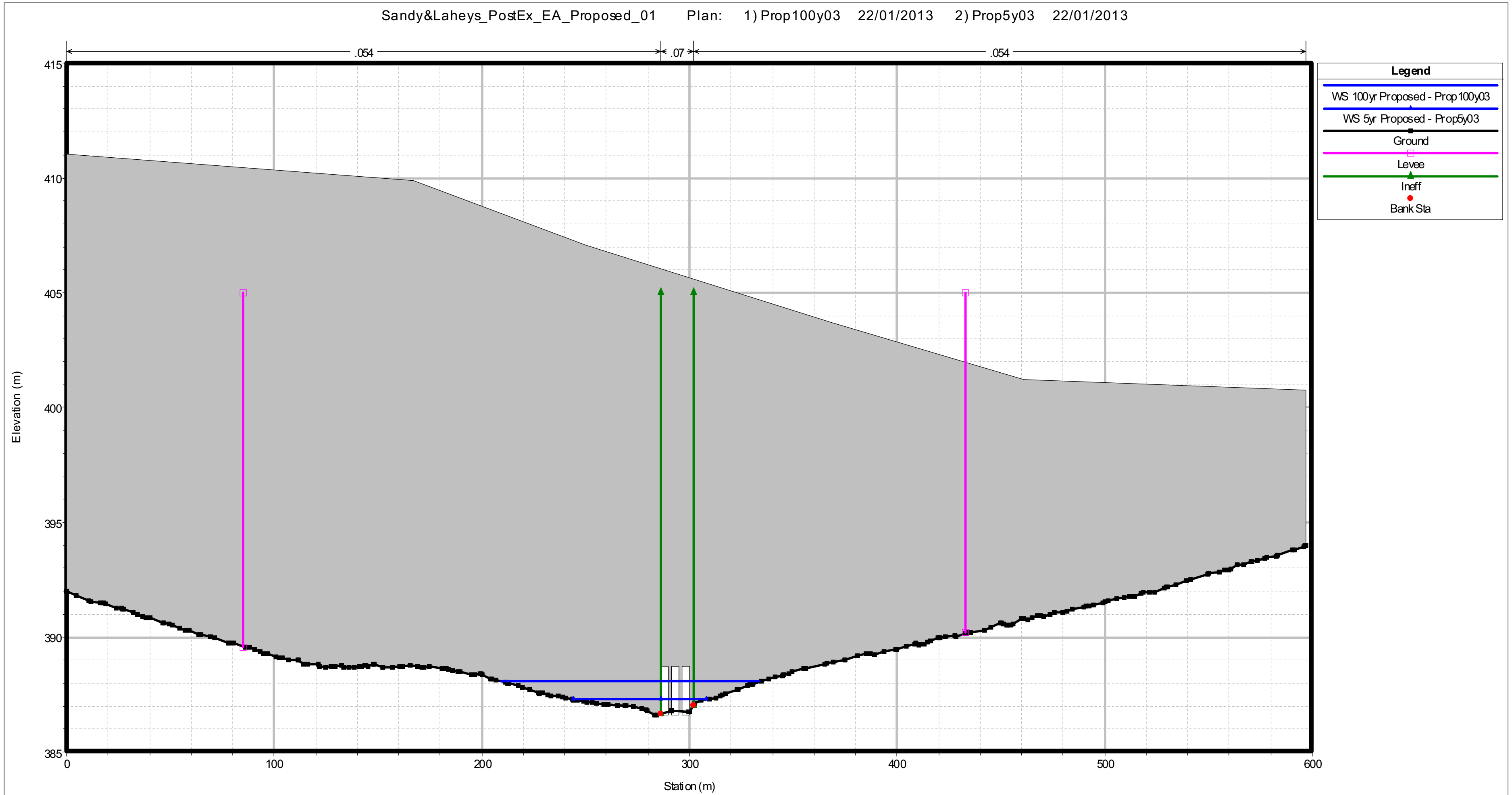


Figure D3.4 Crossing 4 schematic

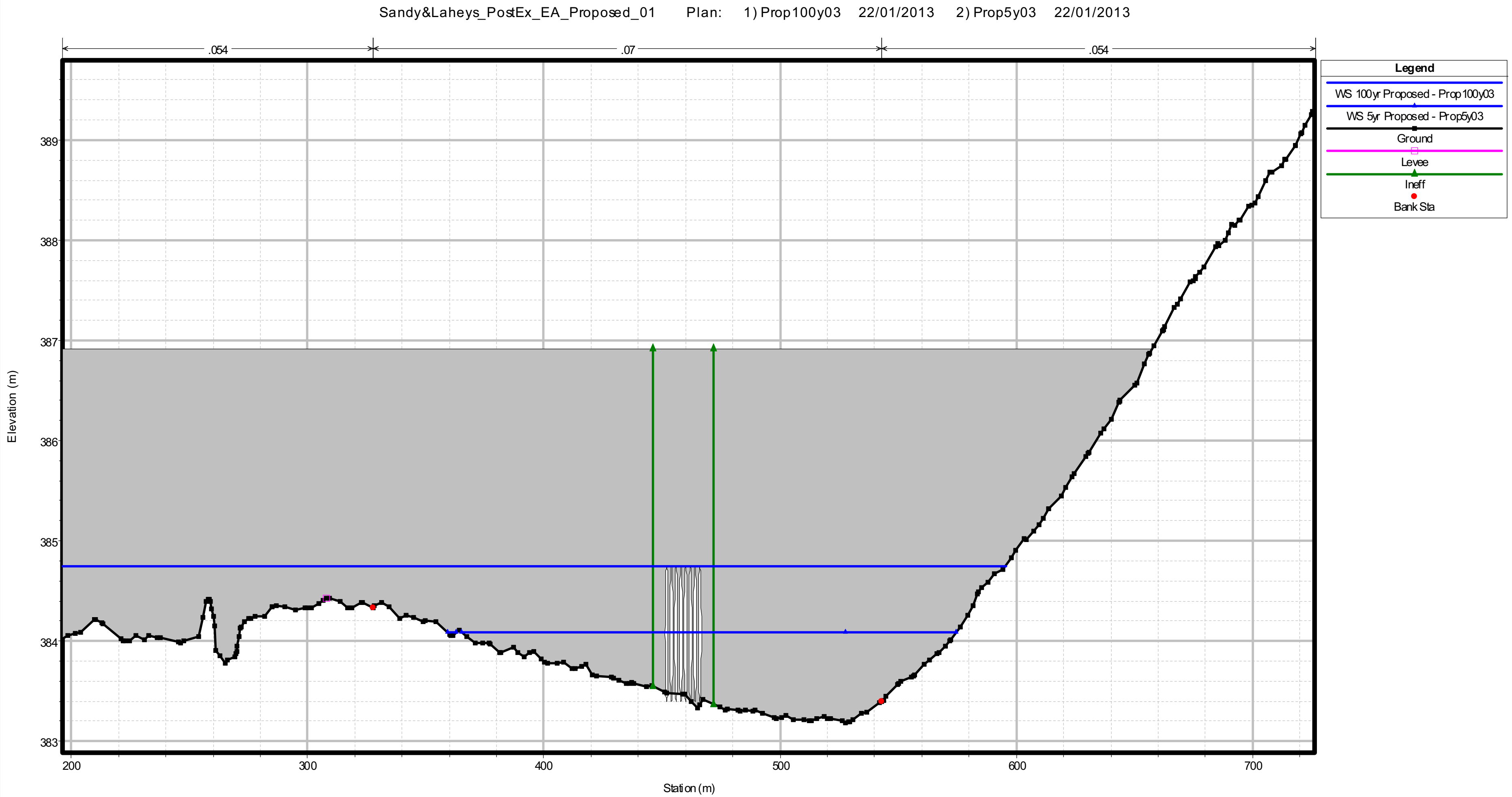


Figure D3.5 Crossing 5 schematic

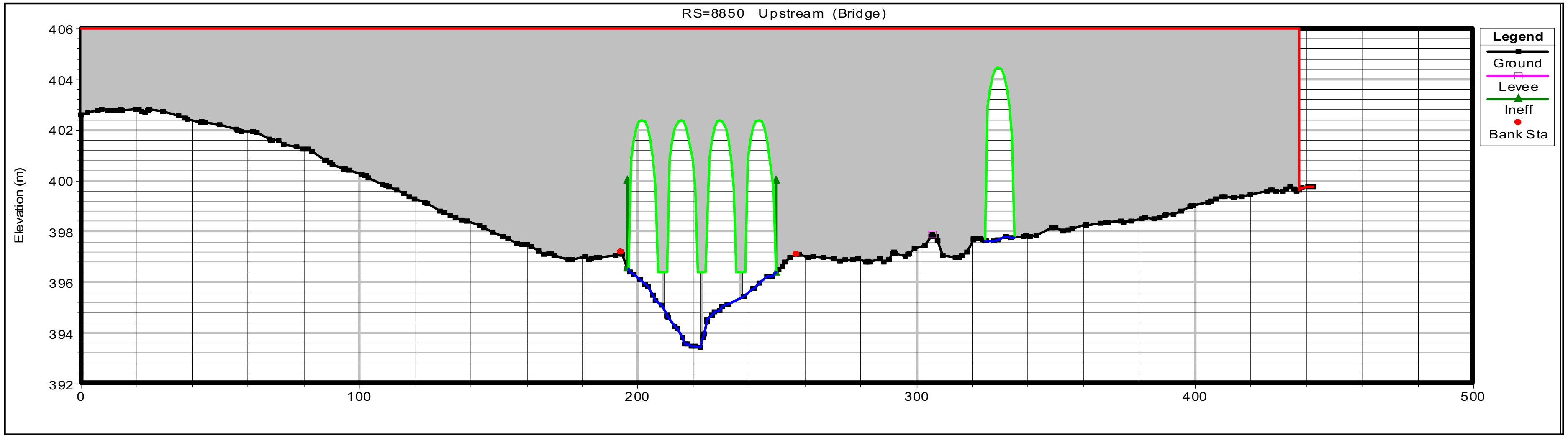


Figure D3.6 Crossing 6 schematic

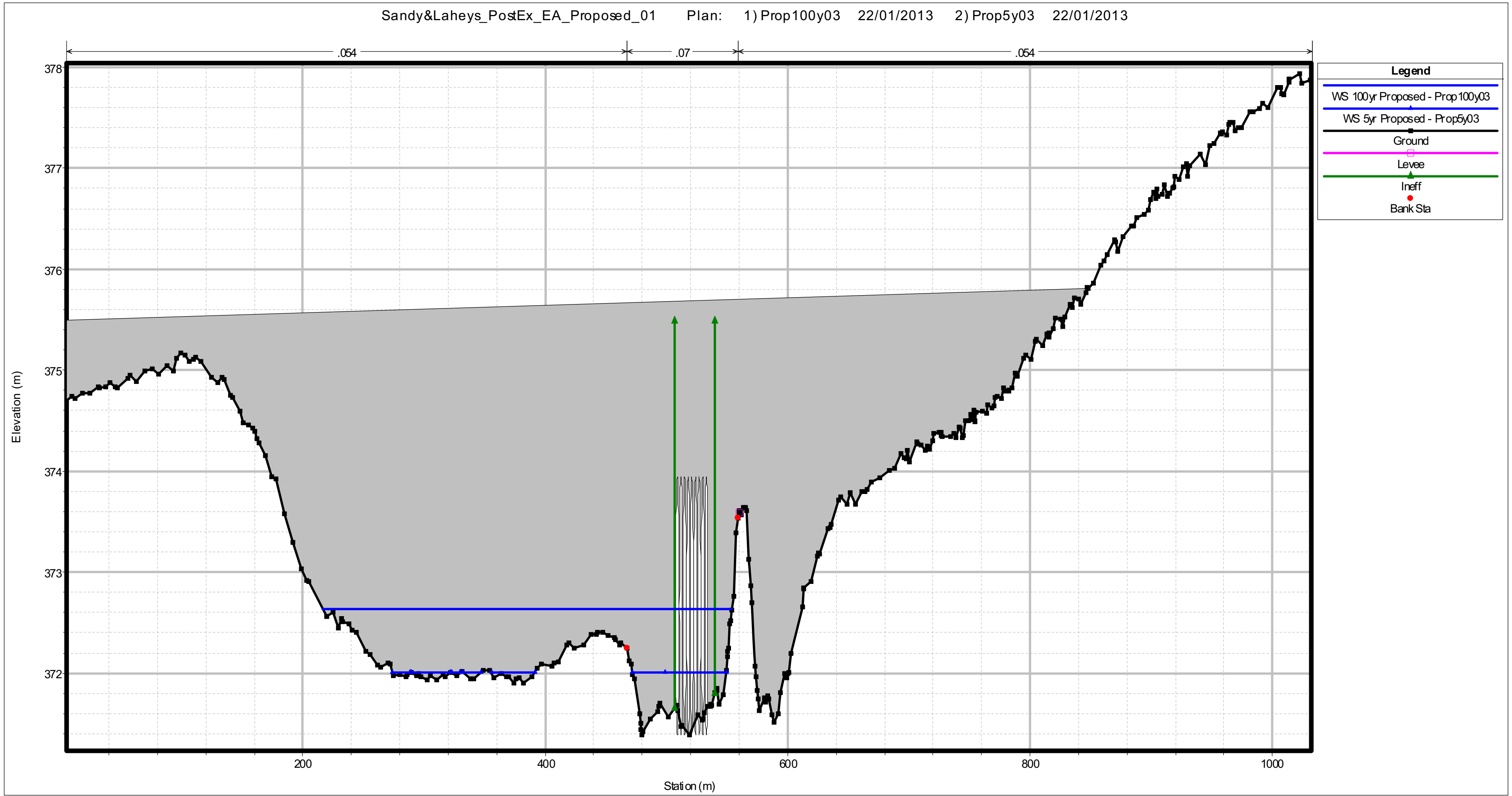


Figure D3.7 Crossing 7 schematic

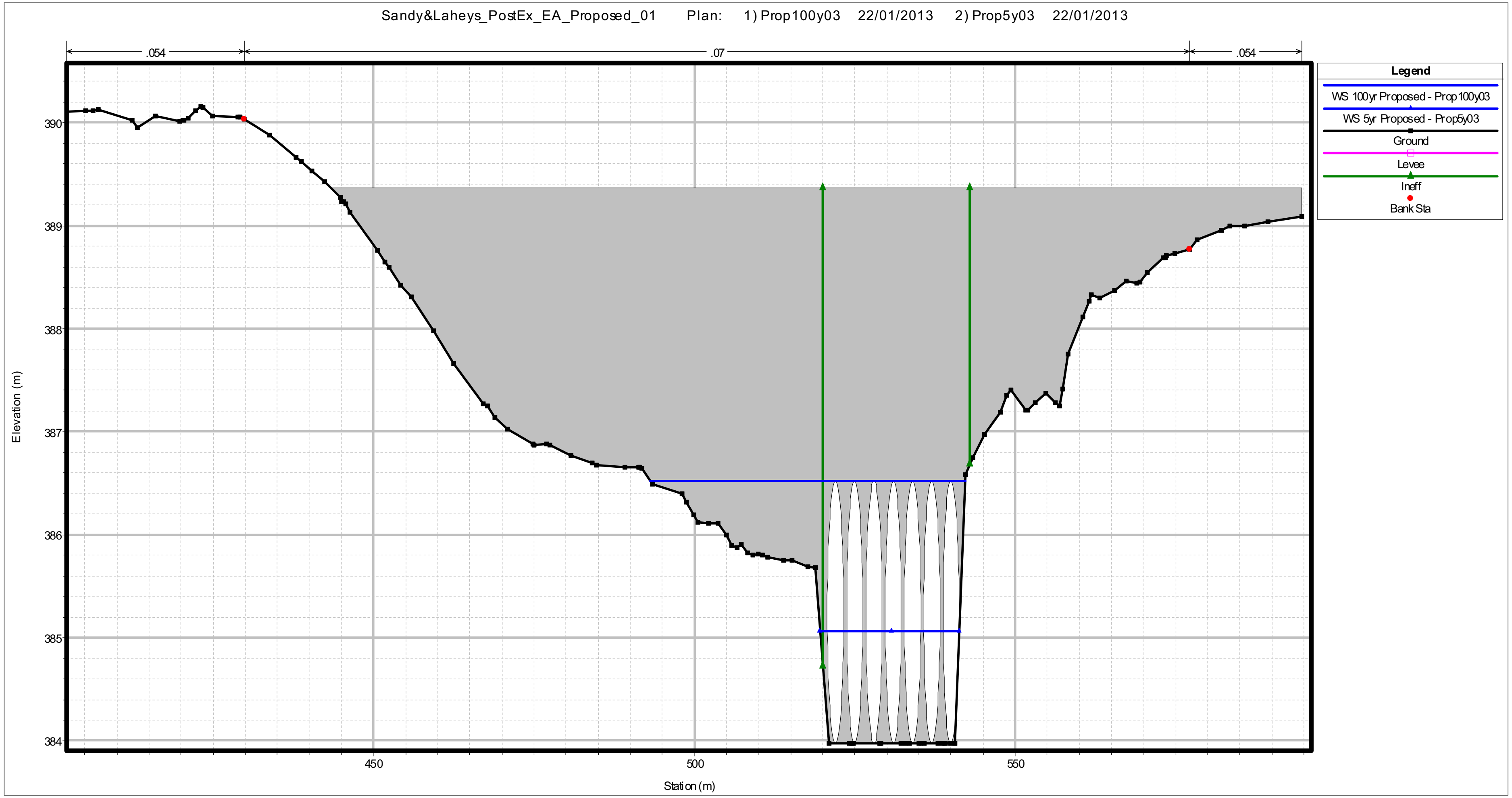


Figure D3.8a Crossing 8a schematic

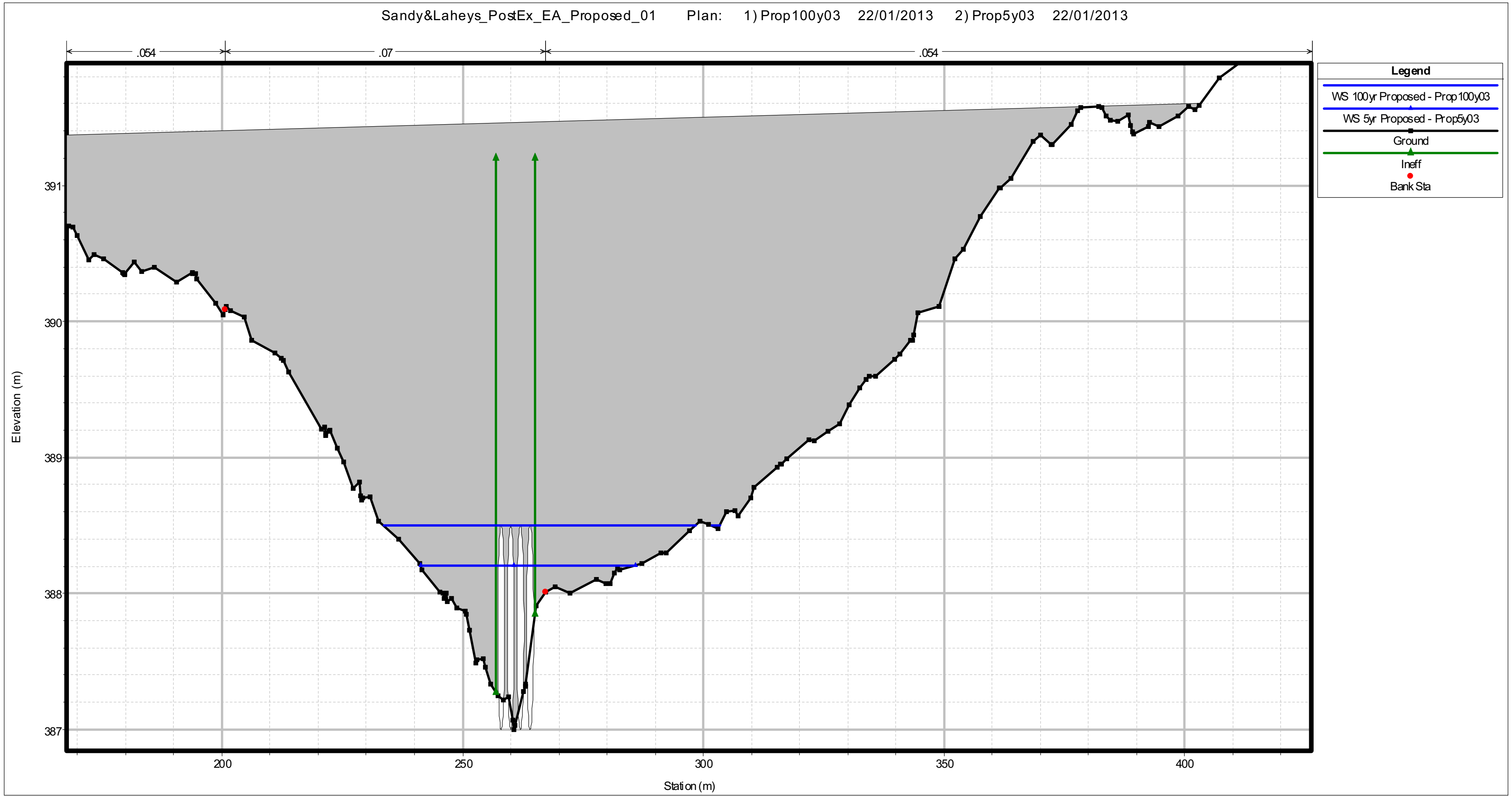


Figure D3.8b Crossing 8b schematic

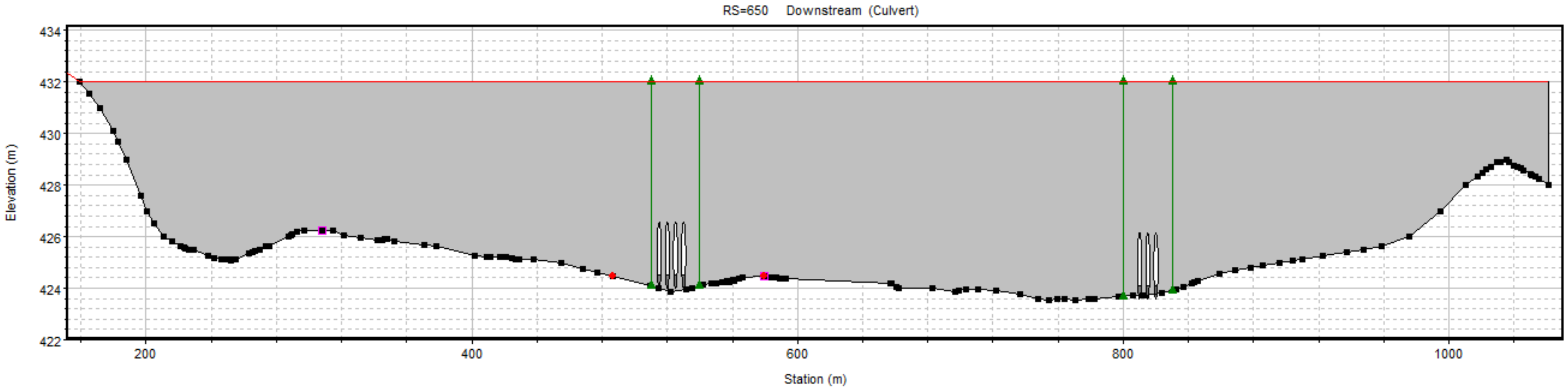


Figure D3.8 Crossing 11 schematic

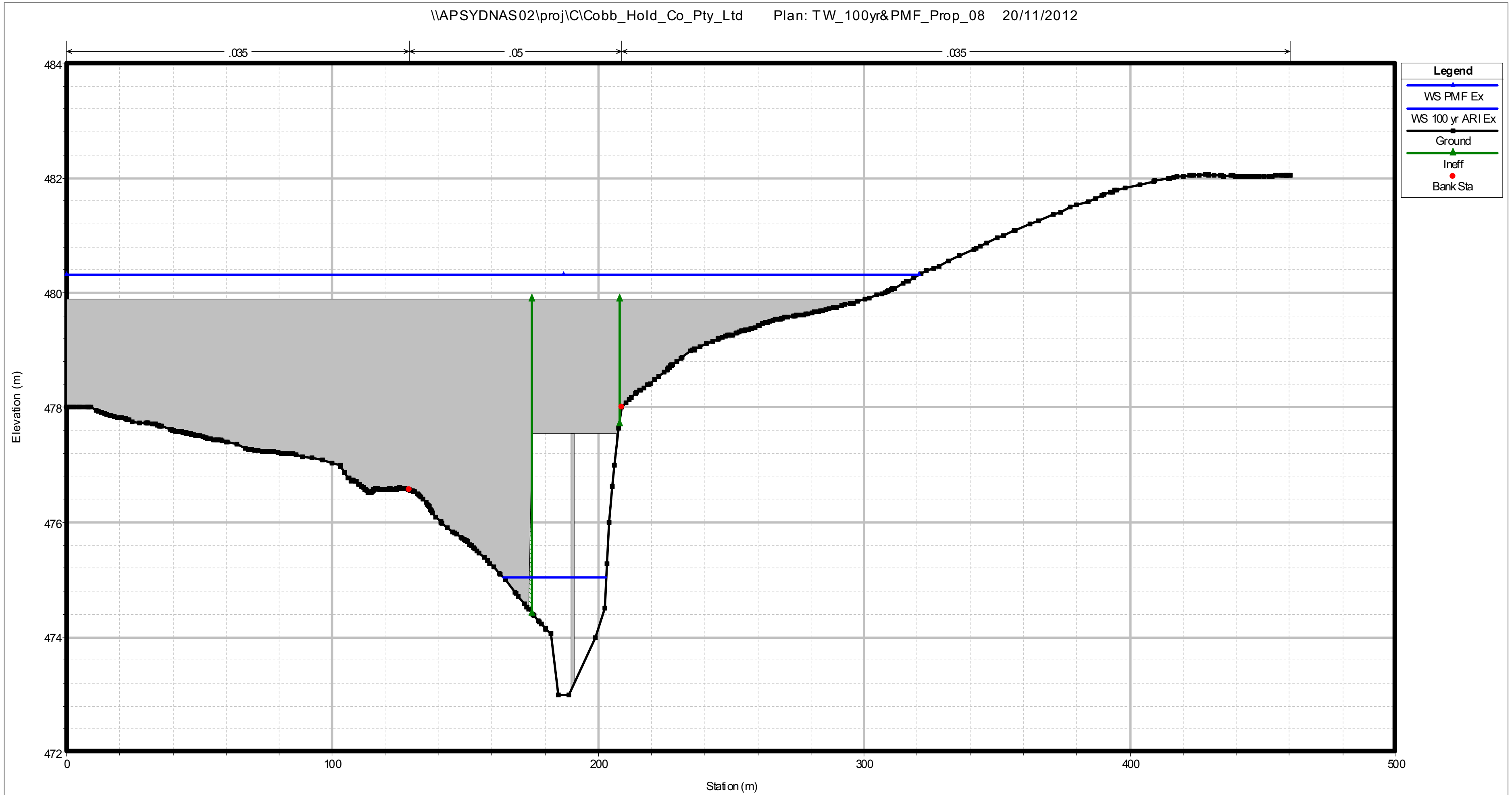


Figure D3.9 Eastern channel of Crossing 28 schematic

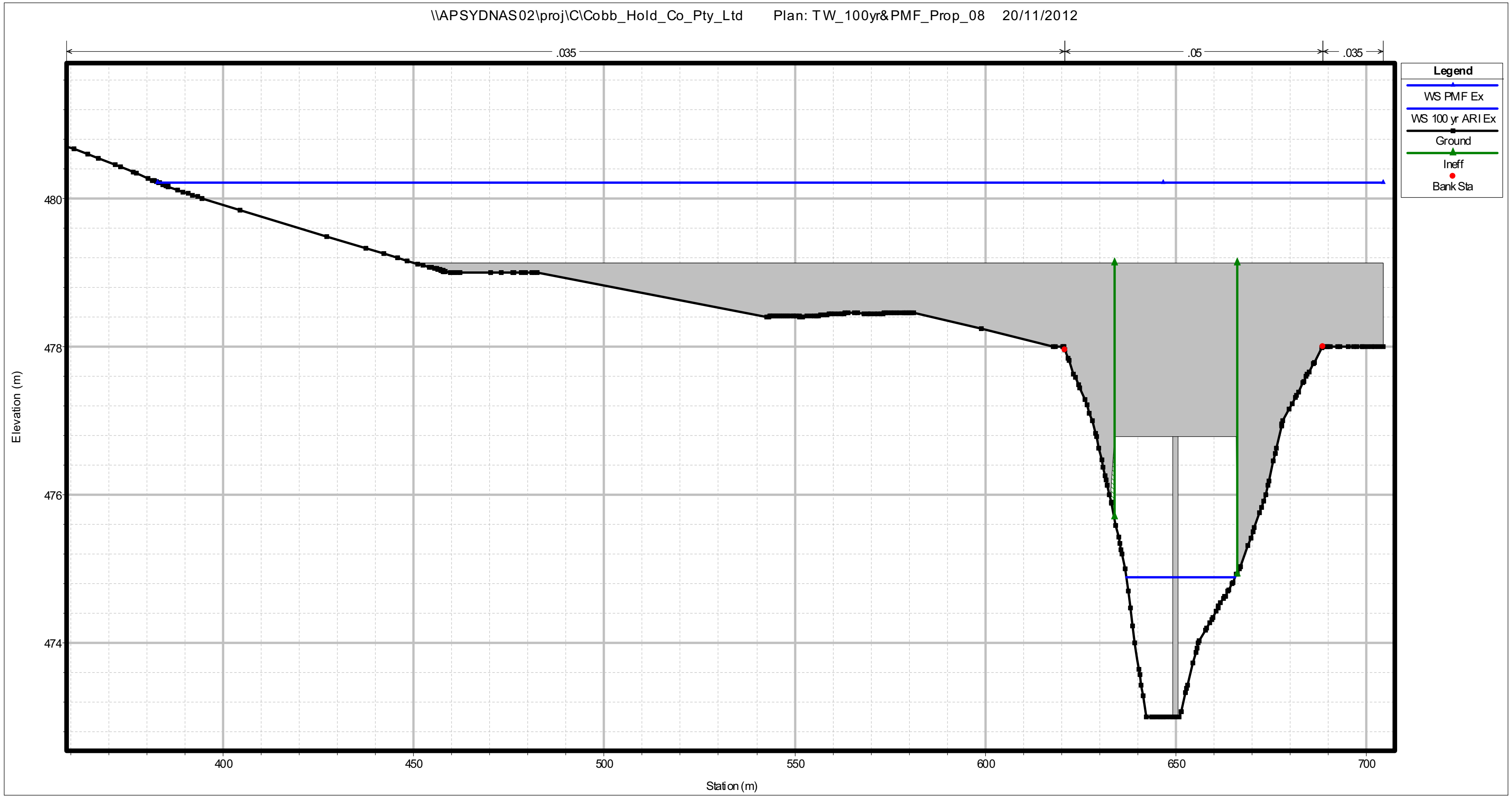
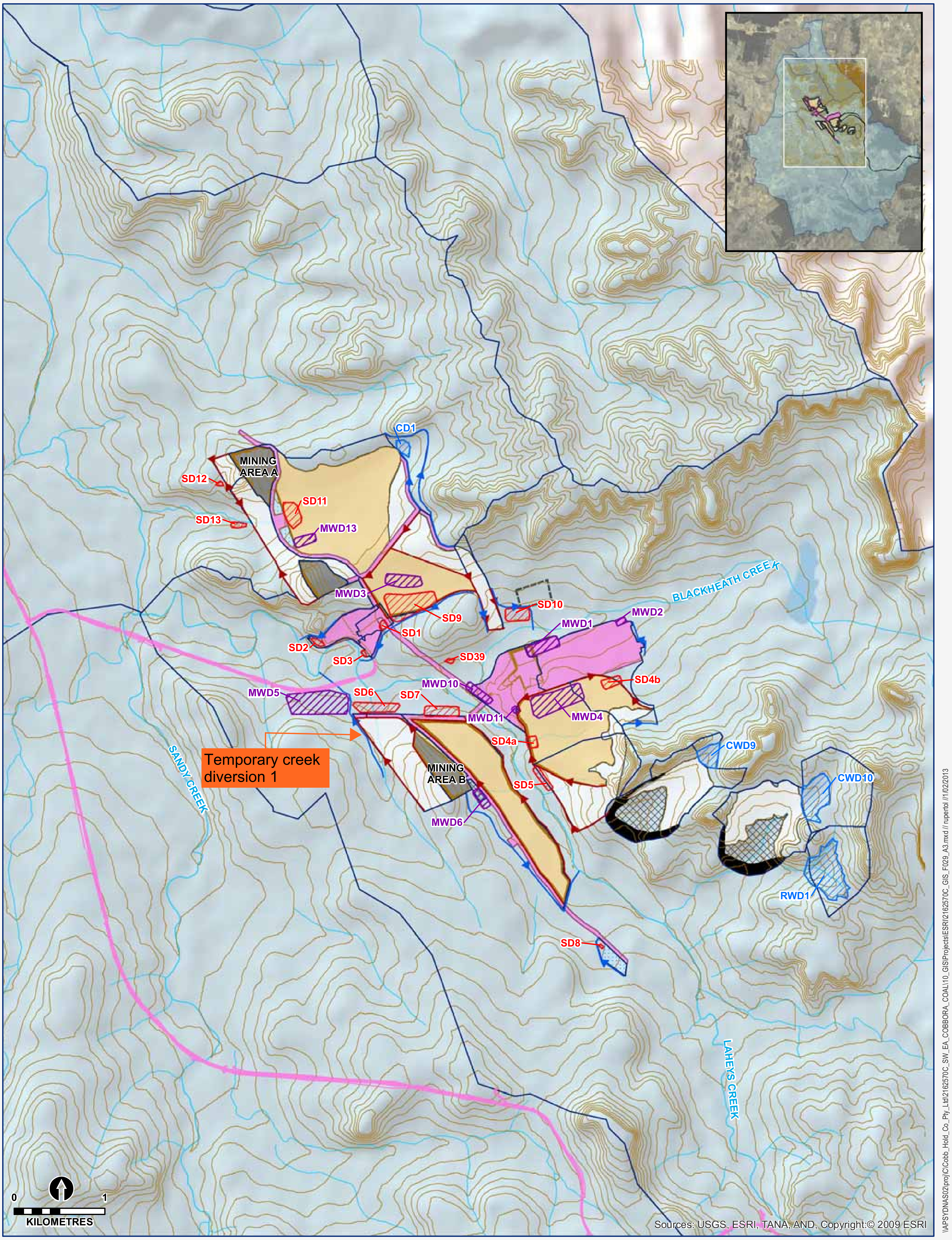


Figure D3.10 Western channel of Crossing 28 schematic

Appendix D.4

Temporary diversion channel locations



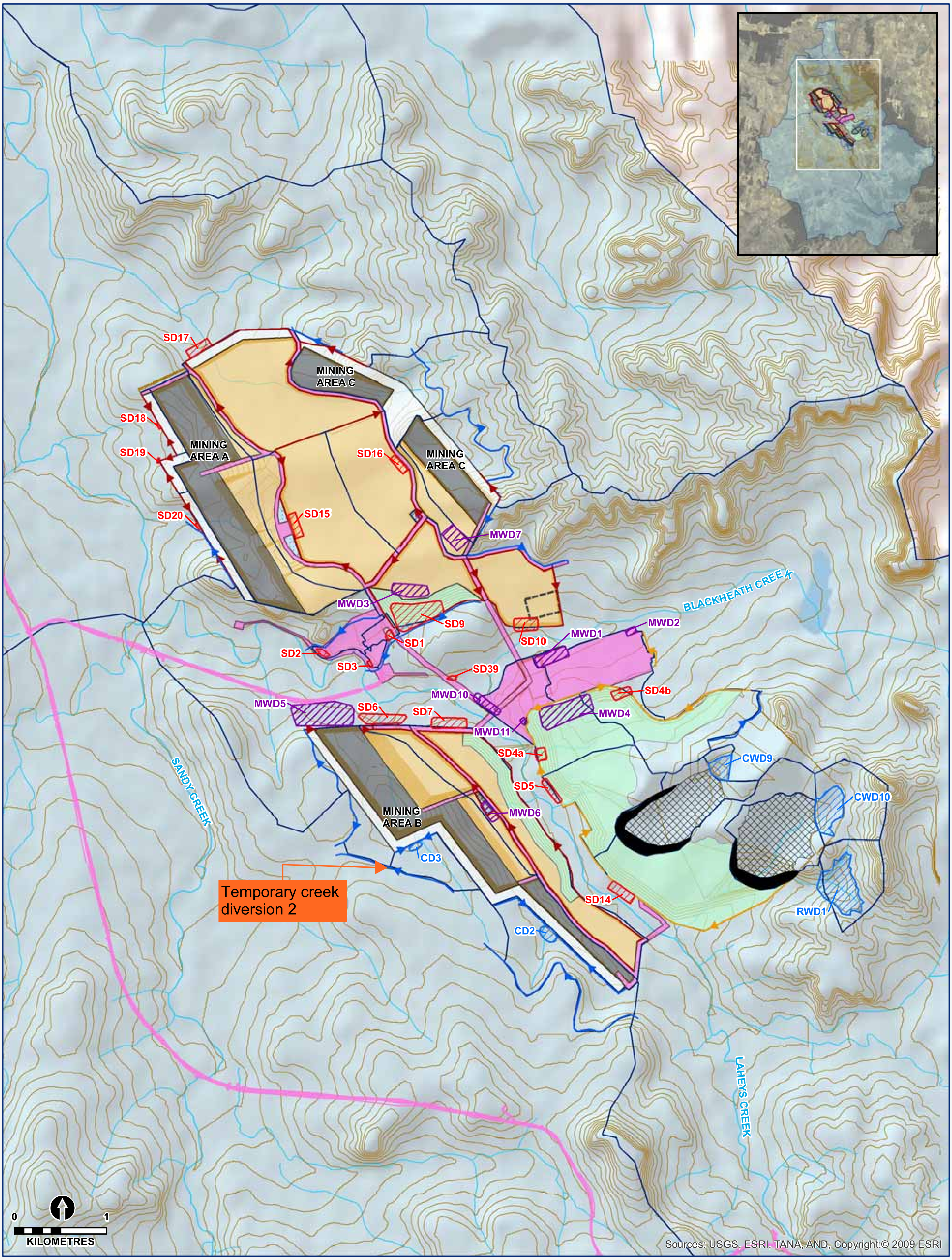
Year 1 surface water drainage
- Clean water diversion drain
- Dirty water diversion drain
- Year 1 mine elevation model 2m contours

- Year 1 catchment boundary

Year 1 dams
- Clean water dam
- Mine water dam
- Sediment dam

Year 1 mine landuse
- Relocated ROM pad
- Active emplacement
- Active mine
- Cleared area
- Haul road / infrastructure
- Rehabilitated emplacement area
- Tailings emplacement area
- Topsoil stockpile

Figure 4.1 Water management system concept for Year 1



Year 4 surface water drainage
- Clean water diversion drain
- Dirty water diversion drain
- Dirty water diversion drain (established rehabilitation released)
- Year 4 mine elevation model 2m contours
- Year 4 catchment boundary

Year 4 dams
- Clean water dam
- Mine water dam
- Sediment dam
- Dam walls

Year 4 mine landuse
- Relocated ROM pad
- Active emplacement
- Active mine
- Cleared area
- Haul road / infrastructure
- Rehabilitated emplacement area
- Tailings emplacement area
- Topsoil stockpile

Figure 4.2 Water management system concept for Year 4

Appendix E

Cobbora Coal Project – Water Balance and Surface Water Management System

Cobbora Coal Project – Water Balance and Surface Water Management System

January 2013

Cobbora Holding Company Pty Limited

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A+ GRI Rating: Sustainability Report 2010

PR_0082D-2122570C_Appendix E

Revision	Details	Date	Amended By
A	Draft for EMM/CHC review	24 February 2012	LD
B	Final draft for Adequacy Review	31 August 2012	RL
C	Final for Public Exhibition	6 September 2012	RL
D	Updated final post exhibition	31 January 2013	LD, NH, RL

Author: ....Leigh Doeleman

Signed: ....L Doeleman.....................

Reviewer: ....Aditya Jha

Signed: ....AJha.....................

Approved by: ....Rob Leslie

Signed: .... Rob Leslie ....

Date: .... 31 January 2013

Distribution: ....EMM, Parsons Brinckerhoff

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Contents

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Glossary

AEP	Annual exceedance probability
AMD	Acid and metalliferous drainage
ARI	Average recurrence interval
BOM	Bureau of Meteorology
CHPP	Coal-handling and preparation plant
DERM	Department of Environment and Resource Management (Queensland)
DSC	Dams Safety Committee
EA	Environmental assessment
ESCP	Erosion and Sediment Control Plan
MIA	Mine infrastructure area
NOW	NSW Office of Water
ROM	Run of mine
WMS	Water management system

1. Introduction

Parsons Brinckerhoff has been commissioned by Cobbora Holding Company Pty Limited (CHC) to prepare a site water balance and surface water management system concept for the Cobbora Coal Project (the Project).

This report contributes to the surface water assessment that Parsons Brinckerhoff undertook for the Project's environmental assessment (EA) report.

1.1 Background

The Project comprises a new open-cut coal mine to be developed near Dunedoo in the central west of New South Wales (NSW). The Project Application Area is approximately 274 square kilometres (km^2). The primary purpose of the Project is to provide coal for five major NSW power stations.

The mine will extract around 20 million tonnes per annum (Mt/a) of run-of-mine (ROM) coal. From this, approximately 9.5 Mt/a of product coal will be sold to Macquarie Generation, Origin Energy and Delta Electricity under long-term contract. In addition, approximately 2.5 Mt/a will be produced for export or for the spot domestic market.

The Project's key elements are:

- an open-cut mine
- a coal-handling and preparation plant (CHPP)
- a train-loading facility and rail spur
- a mine infrastructure area
- supporting infrastructure, including access roads, water supply and storage, and electricity supply.

Construction is expected to commence in mid-2013, with coal being supplied to customers from the first half of 2015. The mine life will be 21 years.

1.2 Scope of works

This report is based on the following scope of works:

- Develop surface water management system concepts that are representative of the mine's development over its 21-year life. For the purpose of this assessment, concepts were developed for five mine stages, corresponding to Year 1, 4, 12, 16 and 20 mine landforms.
- Develop a comprehensive daily site water balance model for the five water management system concepts that correspond to Year 1, 4, 12, 16 and 20 mine landforms.

- Undertake operational water balance assessments to:
  - estimate runoff volumes within the Project site and their reuse
  - identify potential overflows from proposed water management dams
  - estimate volumes of imported water required and assess the adequacy of CHC's existing water entitlements over the 21-year mine life
  - assess the likelihood of mine pits flooding from above-average wet years
  - assess changes to the surface water regime downstream from the Project site.
- Develop a water balance and salt balance model to assess potential impacts to surface water resources downstream of the Project site following mine closure.

2. Design objectives and criteria

2.1 Water management system design objectives

The objectives of the surface water management system for the Project include:

- protect the environmental values of downstream watercourses and local aquifers
- maintain the quality and quantity of surface waters to protect existing beneficial downstream uses
- provide a sustainable water supply for use in the mine.

Downstream water quality will be protected by carefully managing mine water, process water, coal stockpile runoff and unrehabilitated overburden runoff. This will preserve downstream environmental values.

In line with leading industry practice, the key objectives of the water management system design for the Project are to:

- limit dependence on external water sources
- preferentially reuse mine water for coal washing and dust suppression
- provide sufficient on-site storage to avoid releases that could affect the quality of downstream watercourses
- treat runoff from unrehabilitated overburden areas to settle coarse suspended solids for the nominated design storm event.

2.2 Geochemical characteristics of overburden and washery waste materials

Design criteria for the surface water management system have been set with consideration to the likely geochemical characteristics of overburden and washery waste materials at the site.

A geochemical assessment was undertaken for the Project by GeoTerra in March 2012. The geochemical assessment concluded that:

- overall, overburden and the pit floor are likely to be non-acid forming and should not require any special handling for acid and metalliferous drainage control
- overburden has low salinity and neutral acidity
- mixing mining waste will mitigate isolated acid and metalliferous drainage (AMD) leachate
- water coming into contact with overburden will meet relevant ANZECC criteria for protecting downstream watercourses.

Based on these conclusions, overburden runoff will need treatment for suspended solids, but not for salinity or acidity.

2.3 Adopted design criteria

2.3.1 Sedimentation dams

Sedimentation dams are split into two categories based on the runoff source:

- overburden water sedimentation dams receiving runoff from the overburden areas (and also runoff from topsoil stockpiles and cleared areas)
- infrastructure water sedimentation dams receiving water from the MIA only.

Overburden water sedimentation dams

Sedimentation dams capturing runoff from overburden areas have been sized based on criteria recommended in the guidelines *Managing Urban Stormwater: Soils and Construction* (Landcom 2004) and *Managing Urban Stormwater: Soils and Construction — Mines and Quarries* (DECCW 2008). The guidelines recommend providing Type F/D sedimentation dams for catchments with fine or dispersible soils. These are 'wet basins', comprising a 'settling zone' for temporary treatment storage and a 'sediment zone' for storage of sediment.

The guidelines recommend that for a sensitive receiving environment, the 'settling zone' be sized to capture runoff from a 95th percentile 5-day duration storm event. This means that sedimentation dams would on average overflow once or twice a year when the 95th percentile 5-day design storm event is exceeded.

The guidelines recommend volumetric runoff coefficients for disturbed catchments in urban areas. For design rainfall depths of 61 mm to 80 mm, the recommended runoff coefficients range from 0.37 for soils with very low runoff potential to 0.79 for soils with high runoff potential (refer to Table F2 of the guidelines). It should be noted, however, that these values relate to urban disturbed areas (e.g. urban construction sites with compacted bare soil).

For the purpose of sizing sedimentation dams at the site, a runoff coefficient of 0.4 has been adopted for overburden areas, which corresponds to the lower values in the range recommended for urban disturbed areas in the guidelines. The lower value was adopted because overburden areas at the site are typically gently sloping on the top surface and uncompacted, and are expected to have lower runoff than urban disturbed areas.

Typical design features of proposed sedimentation dams are as follows:

- dams configured as Type F/D dams, as described in the guidelines *Managing Urban Stormwater: Soils and Construction* (Landcom 2004)
- 'sediment zone' for sediment storage sized at 50% of 'settling zone'
- 'settling zone' for temporary treatment storage
- slotted riser and discharge pipe with valve arrangement to allow manual operation of pipe
- slotted riser sized to drawdown 'settling zone' over five days
- select clay fill embankment with 1:3 (V:H) slopes
- embankment crest 4 m wide with gravel capping and 3% cross-fall

- spillway at top water level to safely convey the 100-year average recurrence interval (ARI) peak flow
- freeboard between top water level and top of bank
- scour protection at sedimentation dam outlet
- pump and pipeline system to transfer water to downstream watercourses where a gravity-fed outlet is not practical.

The above design criteria have been based on the conclusions of the geochemical assessment (Section 2.2). Water from sedimentation dams will be reused in the mine's water management system where practical. Surplus water that cannot be reused will be treated in the sedimentation dams before being released to downstream watercourses.

Infrastructure water sedimentation dams

Sedimentation dams capturing runoff from mine infrastructure areas (i.e. SD1, SD2 and SD3) have been sized to capture runoff from a 20-year ARI 72-hour storm event, with an additional allowance of 20% for sediment or water storage. A runoff coefficient of 0.85 has been adopted for these areas, which comprise mainly hardstand surfaces. These sizing criteria result in larger sedimentation dams for mine infrastructure areas compared to overburden areas.

2.3.2 Mine water dams

Mine water dams are split into three categories based on the runoff source:

- infrastructure water storage dams receiving water from the CHPP, coal stockpiles and train loading facility
- pit water dams receiving water pumped from the pit. These dams are typically 'turkeys nest' dams and have minimal local catchment
- process water dam receiving a mix of water types from other site dams for reuse on-site. This dam will be centrally located near the CHPP.

Infrastructure water storage dams receiving water from the CHPP, coal stockpiles and train loading facility have been initially sized to capture local catchment runoff from a 100-year ARI 72-hour storm event. For sizing purposes, a runoff coefficient of 0.85 has been adopted for the CHPP and mine infrastructure areas. The capacity of these dams was then confirmed by water balance modelling, with the dam being sized to achieve no discharge when operated as part of the overall site water management system under historical climate conditions. In most cases, this involved increasing the capacity above the 100-year ARI 72-hour storm event to cater for extended wet periods.

Pit water dams receiving water pumped from the open cuts were sized based on water balance modelling to achieve a reasonable level of pit dewatering when operated as part of the overall site water management system under historical climate conditions.

The process water dam has been sized at a nominal capacity of 100 ML. The capacity of this dam will be confirmed during detailed design. The nominal 100 ML capacity is adequate to supply approximately 8 days of site demands at peak coal production.

All dams storing mine water will be provided with appropriate freeboard between the top water level and top of bank and will be designed not to overflow under historical climate conditions. A spillway will be provided for emergency overflow. Mine water dam volumes may need to be reassessed during detailed design when the mine dewatering plan is finalised.

2.3.3 Clean water highwall dams

Clean water highwall dams will be provided at selected locations upslope of the pit to reduce peak flow rates and velocities from undisturbed catchments. Highwall dams will capture clean runoff that will be returned to the creek system, with large storm events overflowing to the pit.

Highwall dams have been sized to capture runoff from the 100-year ARI 24-hour storm event, assuming a runoff coefficient of 0.4 for undisturbed areas.

2.3.4 Prescribed dams

'Prescribed' dams are those listed in Schedule 1 of the *Dams Safety Act 1978*. The Dams Safety Committee (DSC) can require owners of prescribed dams to implement measures that will ensure the safety of their dams.

The final configuration of the site dams will be established during later design stages, and will depend on the availability of construction materials and relative costs of excavation and embankment construction. The currently proposed water management system for the Project includes a number of dams that may meet the criteria of a prescribed dam and will therefore need to be referred to the DSC for assessment and approval before construction of these dams can commence.

3. Existing environment

This section provides an overview of the existing surface water environment at the Project site, and focuses on the climate and rainfall-runoff characteristics that are relevant to the development of the Project's water balance model and water management system.

For a more detailed description of the existing surface water environment, refer to Appendix A of the *Surface Water Assessment* report.

3.1 Catchment description

The Project is located in the catchment of Sandy Creek and Laheys Creek. Both waterways initially flow north approximately parallel to each other, until Laheys Creek joins Sandy Creek approximately 6 km upstream of the Golden Highway. Sandy Creek and Laheys Creek are both ephemeral waterways, meaning that they are usually dry for part of the year.

Sandy Creek joins the Talbragar River approximately 1 km downstream of the Golden Highway, and 7 km downstream of the confluence of Sandy Creek and Laheys Creek. The Talbragar River is a tributary of the Macquarie River, which is a tributary of the Barwon River, which in turn is a tributary of the Darling River.

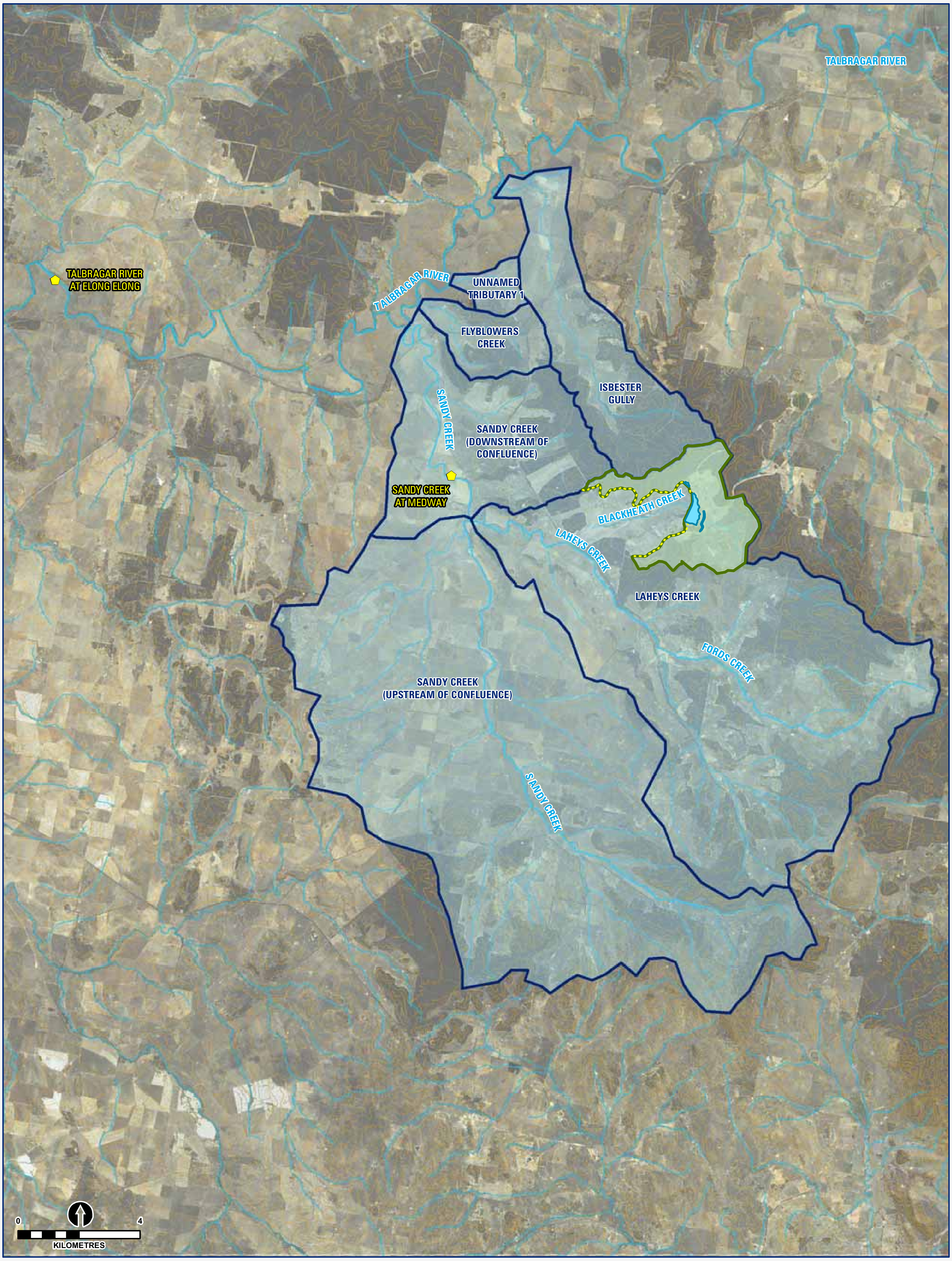
An existing catchment plan for the Project site is provided in Figure 3-1, and a summary of the sub-catchment areas is provided in Table 3-1. The total catchment area for Sandy Creek at its confluence with the Talbragar River is 28,218 ha.

Table 3-1 Existing catchment areas

Sub-catchment	Area (ha)
Sandy Creek (upstream of Laheys Creek confluence)	13,901
Sandy Creek (downstream of Laheys Creek confluence)	3,221
Laheys Creek *	11,095
Flyblowers Creek	738
Isbester Gully	2,454
Unnamed Tributary 1	278
Total	31,688

* Laheys Creek catchment area of 11,095 ha includes 1,034 ha to the existing west 'Woolandra' farm dams located in the upper reaches of Blackheath Creek.

Land use within the catchment of Sandy Creek and Laheys Creek is mainly rural, with some areas of native vegetation in the upper reaches of the catchment. Numerous small farm dams exist on drainage lines within the catchment. Two existing farm dams with a combined capacity of approximately 1,485 ML are located at 'Woolandra' in the upper reaches of Blackheath Creek, which is a tributary of Laheys Creek. The larger Woolandra West Dam has a capacity of about 1,470 ML. The smaller dam is commonly referred to as the 'Sausage Dam'. A network of existing contour drains diverts runoff from a catchment of approximately 1,034 ha to these two farm dams.



Flow gauges
Existing contour drains
Drainage lines
Topographic contours (20m interval)
Catchment boundary
Existing farm dam subcatchment
Existing farm dam (1,460ML capacity)

Figure 3-1 Existing study catchment

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3.2 Climate data

Climate data used in the water balance model was based on 111 years (1900–2010) of daily data sourced from the Data Drill database, developed by the Queensland Department of Environment and Resource Management (DERM). The Data Drill accesses grids of data derived by interpolating the Bureau of Meteorology's station records. Interpolations are calculated by splining and kriging techniques. The data in the Data Drill are all synthetic and there are no original meteorological station data in the calculated grid fields. However, the Data Drill does have the advantage of being available for any set of coordinates in Australia (DERM 2011).

The Data Drill is considered superior to individual Bureau of Meteorology station records and site observations for water balance modelling purposes because it draws on a greater dataset, both spatially and in time. The Data Drill is also considered superior for modelling purposes as it does not contain gaps.

A plot of annual rainfall for the site is provided in Figure 3.2, and a plot of annual lake evaporation for the site is provided in Figure 3.3. Summary statistics for rainfall and evaporation are provided in Table 3-2.

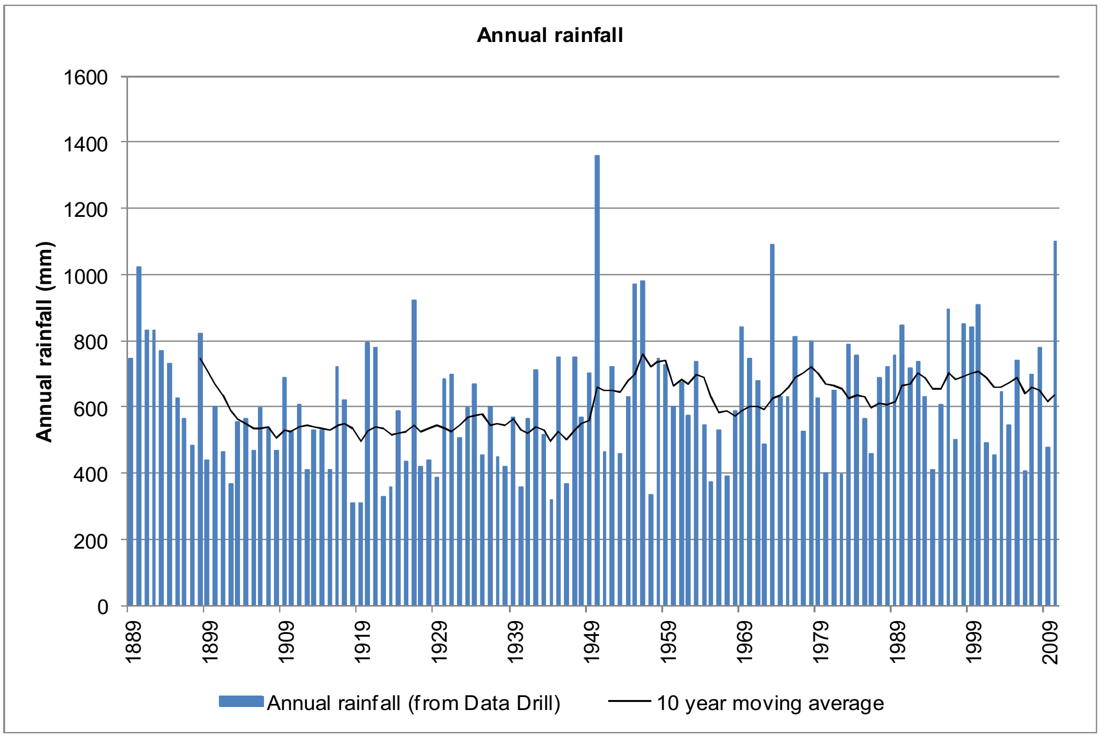

Figure 3.2 Annual rainfall for Cobbora site — Data Drill (1889 to 2010)

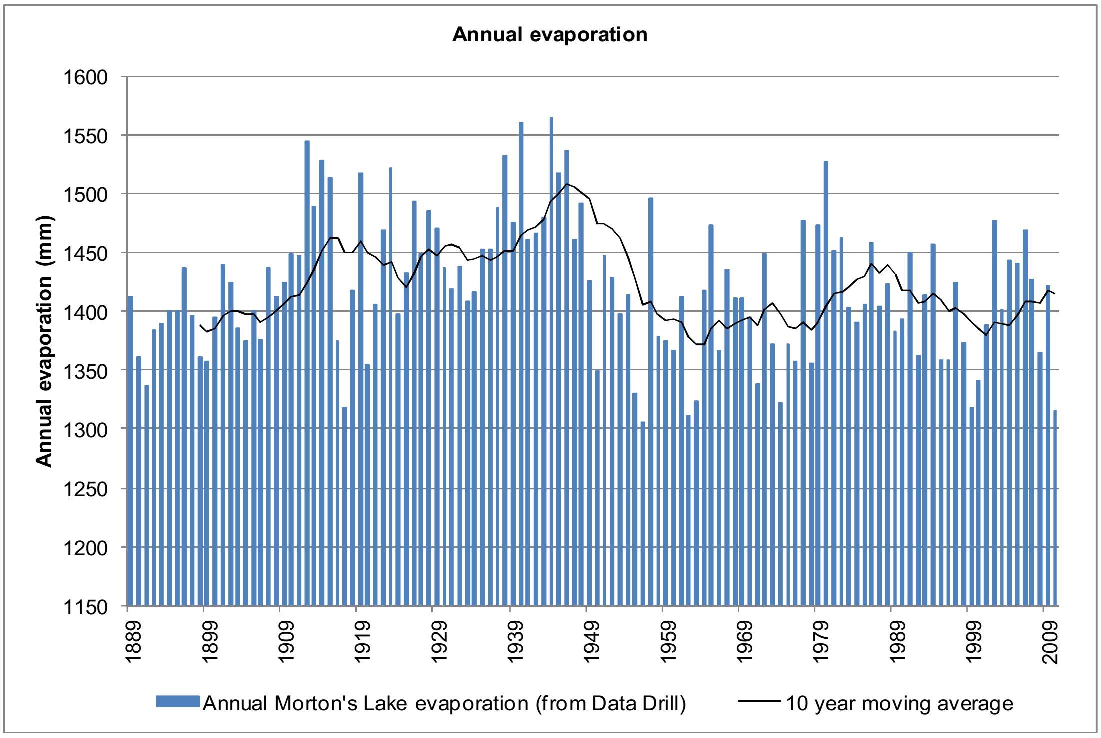

Figure 3.3 Annual lake evaporation for Cobbora site — Data Drill (1889 to 2010)

Table 3-2 Summary climate statistics for Cobbora site (1889 to 2010)

Statistic	Annual rainfall (mm)	Annual potential evapotranspiration[1] (mm)	Annual lake evaporation[2] (mm)
5th percentile	360	1,203	1,324
10th percentile	397	1,223	1,355
50th percentile (median)	604	1,321	1,416
90th percentile	839	1,422	1,493
95th percentile	921	1,456	1,526
99th percentile	1,099	1,489	1,557
Mean	620	1,321	1,421
Minimum	310	1,147	1,306
Maximum	1,360	1,502	1,564
Standard deviation	187	76	58

1 Potential evapotranspiration calculated using the Penman-Monteith formula (source: Food and Agriculture Organization of the United Nations 1998).

2 Lake evaporation calculated using the Morton formula for shallow lakes (source: Morton 1983).

A comparison has been made between the Data Drill rainfall data and the rainfall recorded at the two on-site meteorological stations. Quality checked rainfall data is available from November 2010. The rainfall depth recorded at the on-site meteorological stations was 645 mm for the 11-month period from November 2010 to October 2011. This compares to 649 mm for the same period obtained from the Data Drill, which is only a 1% difference and gives confidence in the Data Drill.

As discussed above, the on-site meteorological data has not been adopted for the purposes of long-term water balance modelling because of the short dataset available. However, the on-site meteorological data will be useful in the future to verify and update the water balance model during mine operation.

3.3 Design rainfall data

3.3.1 Rainfall intensity-frequency-duration data

Design rainfall intensity-frequency-duration (IFD) data was prepared for the site in accordance with the method outlined in *Australian Rainfall and Runoff* (Engineers Australia 2001). The 100-year ARI 24-hour duration rainfall depth is 157 mm. The 20-year and 100-year ARI 72-hour duration rainfall depths are 146 mm and 200.2 mm respectively.

3.3.2 Five-day rainfall depths

Five-day rainfall depths for the site have been estimated using a procedure outlined in the guidelines *Managing Urban Stormwater: Soils and Construction* (Landcom 2004). Five-day rainfall depths (see Table 3-3) are based on the linear relationship between five-day rainfall and average annual rainfall given in Figure 6.11 of the guidelines, and assume an average annual rainfall of 620 mm.

Table 3-3 Five-day rainfall depths for Cobbora site

Percentile	Five-day rainfall depth (mm)
75th percentile	20.3
85th percentile	31.1
95th percentile	63.3

3.4 Stream gauge data

Table 3-4 provides a summary of NOW stream gauging stations in the vicinity of the study area.

Table 3-4 NOW streamflow gauges

Location	Gauge number	Period of record
Operational gauges		
Talbragar River at Elong Elong	421042	1964–present
Discontinued gauges		
Talbragar River at Cobbora	421028	1950–1954
Talbragar River at Naranmore	421037	1955–1976
Sandy Creek at Medway	421064	1966–1985

Source: NOW website and PINEENA database

Two NOW stream gauging stations have operated near the Project site (numbers 421042 and 421064), and two other NOW gauging stations have operated in the wider region (numbers 421028 and 421037).

The Sandy Creek at Medway gauging station (no. 421064) is located within the Sandy Creek catchment; however, it ceased operation in 1985 and there are no active gauging stations operating within the Sandy Creek catchment.

The Talbragar River at Elong Elong gauging station (no. 421042) is located on the Talbragar River approximately 20 km downstream of where Sandy Creek joins the Talbragar River, and is active.

Stream gauge records were obtained from NOW for Sandy Creek at Medway and Talbragar River at Elong Elong for the period of available record.

A daily flow duration curve for Sandy Creek at Medway is provided in Figure 3.4 for the period 1966 to 1985. The contributing catchment area is 260 km^2. The flow duration curve shows that while the highest recorded mean daily flow was 8,937 ML/d (in February 1971), for 50% of the time flows were less than 0.04 ML/d, and for 45.8% of the time there was no flow.

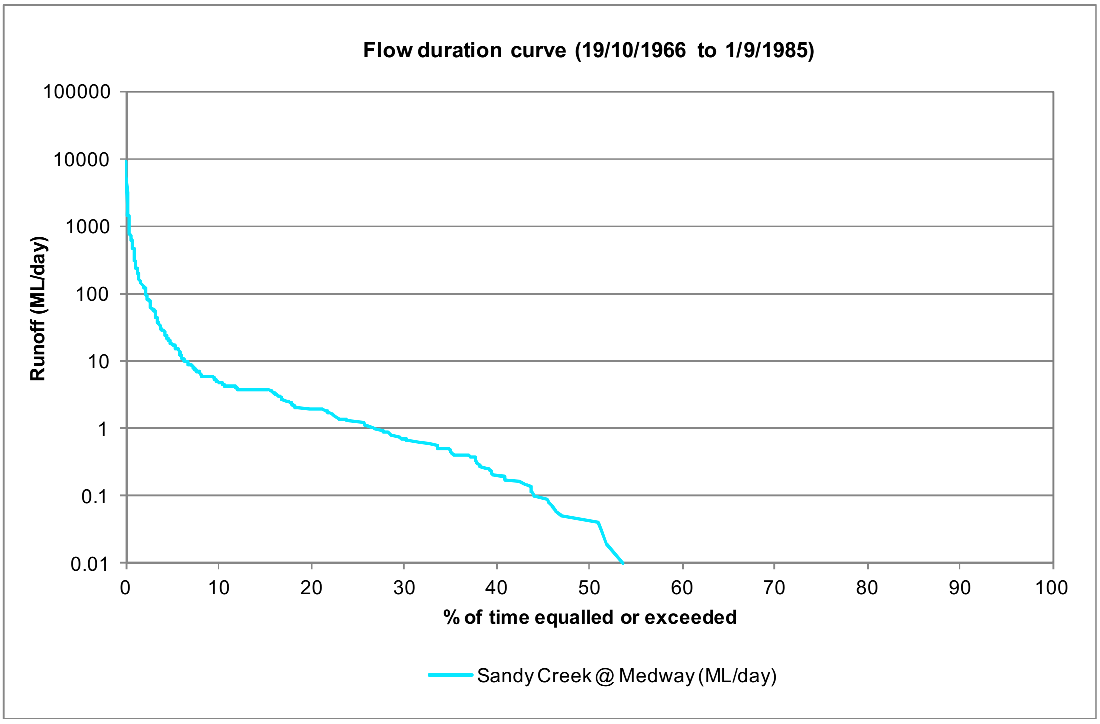

Figure 3.4 Flow duration curve for Sandy Creek at Medway (no. 421064)

A daily flow duration curve for Talbragar River at Elong Elong is provided in Figure 3.5 for the period 1970 to 2011. The contributing catchment area is 3,050 km^2. The flow duration curve shows the highest recorded mean daily flow was 40,835 ML/d (in December 2010), for 50% of the time flows were less than 11.9 ML/d, and for 31.8% of the time there was no flow.

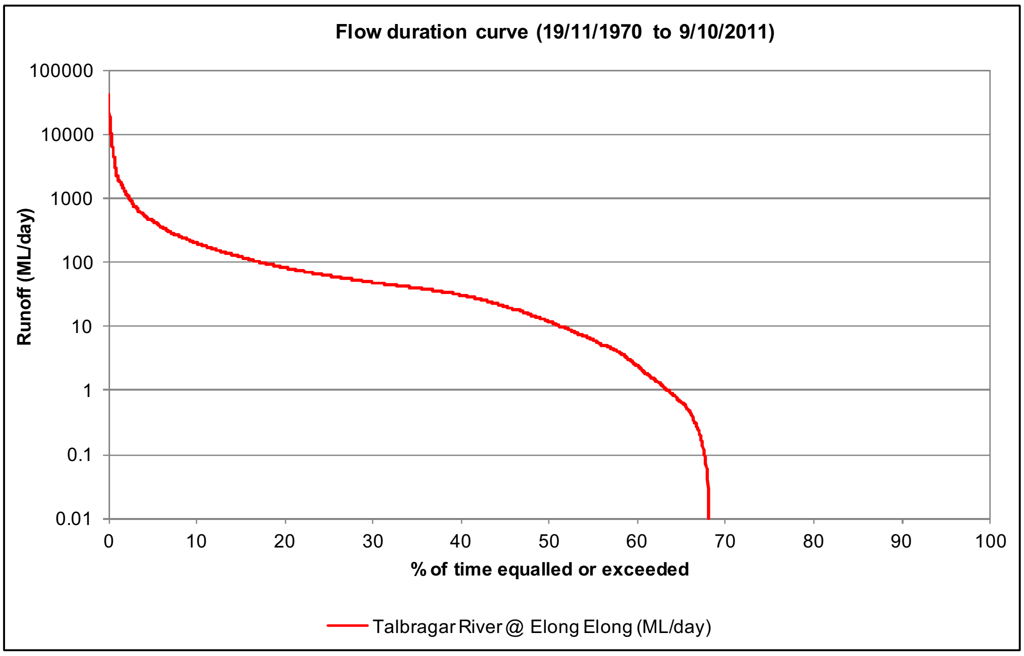

Figure 3.5 Flow duration curve for Talbragar River at Elong Elong (no. 421042)

Figure 3.6 shows a comparison of the Sandy Creek at Medway and Talbragar River at Elong Elong daily flow duration curves from 1970 to 1985, when gauging data is available for both stations (note this period is different to the plots in Figure 3.4 and Figure 3.5). Flows are given as runoff depth per unit area to allow comparison between the two gauging stations. Figure 3.6 shows that the flow duration curve for Sandy Creek at Medway has longer dry periods and higher runoff depths at the high flow end of the curve than the Talbragar River at Elong Elong, which is typical of a watercourse with significantly larger catchment area.

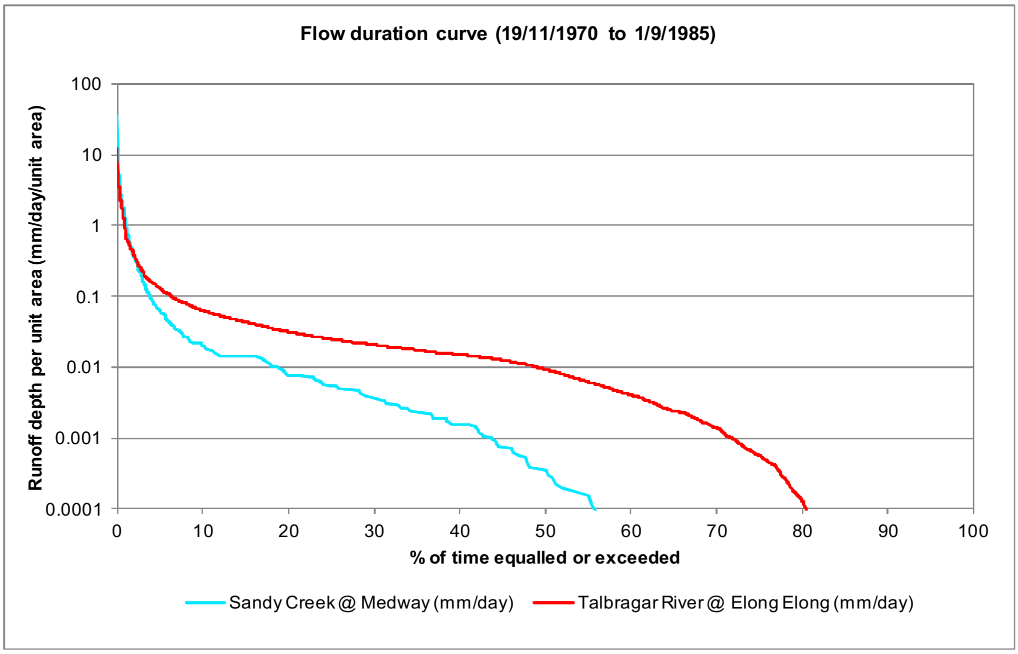

Figure 3.6 Comparison of Sandy Creek and Talbragar River flow duration curves

3.5 Surface runoff

The volume of surface water runoff has been estimated using the Australian Water Balance Model (AWBM) rainfall-runoff model that has been incorporated into the water balance model. The rainfall-runoff model has been calibrated using recorded streamflow data.

3.5.1 Australian Water Balance Model

The Australian Water Balance Method (AWBM) (Boughton 1993) was used to derive catchment runoff time series from undisturbed, disturbed and rehabilitated catchments for use in the water balance model.

The AWBM is a partial area saturation overland flow model. The use of partial areas divides the catchment into regions (contributing areas) that produce runoff during a rainfall-runoff event and those that do not. These contributing areas vary within a catchment according to antecedent catchment conditions, and allow for the spatial variability of surface storage in a catchment. The use of the partial area saturation overland flow approach is simple and provides a good representation of the physical processes occurring in most Australian catchments (Boughton 1993). This is because daily infiltration capacity is rarely exceeded, and the major source of runoff is from saturated areas. Figure 3.7 shows a schematic layout of the AWBM.

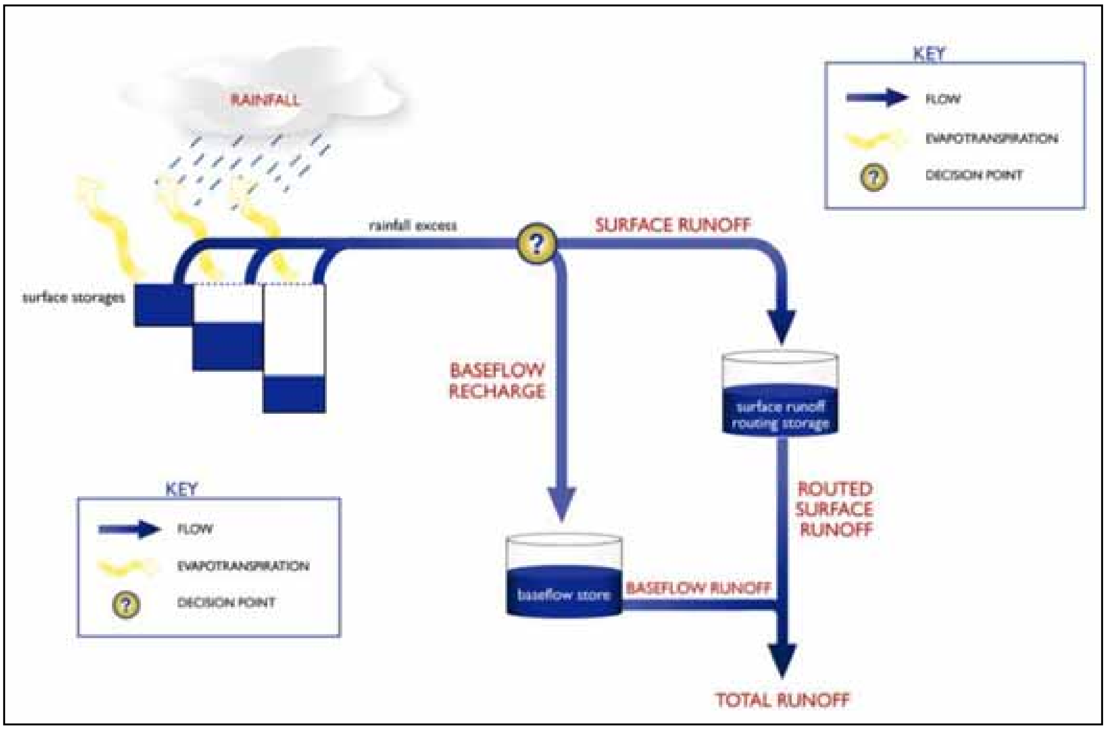

Figure 3.7 Schematic layout of the AWBM rainfall-runoff model (source: CRC for Catchment Hydrology 2004

To implement the AWBM in a given catchment, a set of nine parameters must be defined as summarised in Table 3-5. These parameters define the generalised model for a particular catchment. The parameters are usually derived for a gauged catchment by a process of calibration where the recorded streamflows are compared with calculated streamflows.

The parameters are adjusted to produce the best match between the means and standard deviations of the daily streamflows, to match the difference in peak flow discharges.

Table 3-5 Description of AWBM parameters

Parameter	Description
A1, A2, A3	Partial areas represented by surface storages
C1, C2, C3	Surface storage capacities
Ks	Daily surface flow recession constant
BFI	Baseflow index
K_{base}	Daily baseflow recession constant

3.5.1.1 AWBM calibration for undisturbed catchments

AWBM parameters adopted for the portions of catchments not affected by mining are provided in Table 3-6. These parameters simulated flows that closely matched the average annual volume and flow duration curve from the historically gauged Sandy Creek streamflows at Medway (no. 421064). Note that the pre-mining catchment is largely rural, however, is referred to as ‘undisturbed’ for the purposes of this study.

The Sandy Creek at Medway record was selected as it is located within the same catchment as the Project. While the Sandy Creek at Medway record is shorter than the Talbragar River at Elong Elong record, the Sandy Creek record is considered superior for calibration purposes as the Talbragar River catchment is significantly larger than the Sandy Creek and Laheys Creek catchments and has different runoff characteristics as demonstrated by the flow duration curve in Figure 3.8.

Table 3-6 Calibrated AWBM parameters for pre-mining ‘undisturbed’ Sandy Creek at Medway catchment

Catchment	AWBM parameters								
	Ks	BFI	K_{base}	A1	A2	A3	C1 (mm)	C2 (mm)	C3 (mm)
Sandy Creek at Medway	0.400	0.150	0.955	0.134	0.433	0.433	50	150	350

A comparison of predicted and observed flow duration curves is provided in Figure 3.8 for the common data period from 1970 to 1985. Flows are given as runoff depth per unit area to allow comparison between the two gauging stations and the AWBM simulated flow.
The curve shows that the flow duration curve simulated from the AWBM compares reasonably well to that observed for Sandy Creek at Medway.

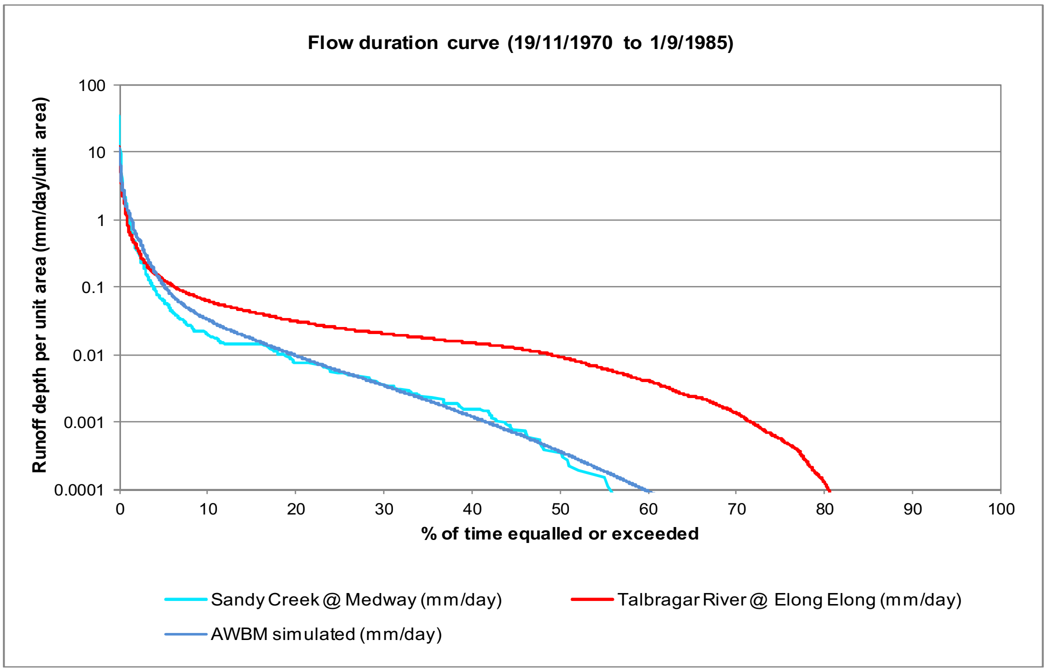

Figure 3.8 Comparison of predicted and observed flow duration curves

The mean annual runoff depth predicted by the AWBM model was 20.0 mm/a (3.0% of mean annual rainfall) for the period 1970 to 1985. The mean annual runoff depth observed at the Sandy Creek at Medway gauging station was 20.8 mm/a (3.1% of mean annual rainfall) for the period 1970 to 1985. The mean annual runoff depth observed at the Talbragar River at Elong Elong gauging station was 19.2 mm/a (2.9% of mean annual rainfall) for the period 1970 to 1985.

Relationships between annual rainfall and annual runoff in the major drainage divisions of Australia are published in the document Estimating Runoff in Ungauged Catchments from Rainfall, PET and the AWBM Model (Boughton and Chiew, 2006). Boughton and Chiew developed multiple linear regressions to relate average annual runoff to average annual rainfall and areal potential evapotranspiration using data from 213 catchments grouped according to location in six of the major drainage divisions. The regression relationship for catchments with less than 700 mm average annual rainfall in the Murray-Darling drainage division is:

$$Q = 0.276P - 0.139E + 47$$

where:
Q = average annual runoff (mm/a)
P = average annual rainfall (mm/a)
E = average annual areal potential evapotranspiration (mm/a)

Based on the above regression relationship, and adopting an average annual rainfall of 620 mm/a and potential evapotranspiration of 1,321 mm/a for the site, the calculated average annual runoff is 34.5 mm/a (5.6% of mean annual rainfall). The mean annual runoff depths observed at the Sandy Creek at Medway gauging station of 20.8 mm/a (3.1% of mean annual rainfall) and the Talbragar River at Elong Elong gauging station of 19.2 mm/a (2.9% of mean annual rainfall) for the period 1970 to 1985 are less than that calculated from the Boughton and Chiew regression relationship, but are still considered to compare reasonably well.

3.5.1.2 Disturbed catchments

AWBM parameters for mine site overburden and rehabilitation areas were estimated based on calibrated parameters published in the Australian Coal Association Research Program (ACARP) document *ACARP Project No. C7007: Water Quality and Discharge Prediction for Final Void and Spoil Catchments* (PPK Environment & Infrastructure 2001). The aim of the ACARP study was to develop a methodology, based on computer modelling and field monitoring, for predicting the hydrology and water quality of final spoil–void systems. The ACARP study included establishing a network of eight monitoring sites representing individual spoil–void catchments, including six mine sites from Queensland and two mine sites from New South Wales. The monitoring data was used to calibrate the Spoil Hydrology Lumped Parameter Model (SHLPM) rainfall-runoff model. The SHLPM differs in some ways to the AWBM, but has the same basic structure and parameters as the AWBM. Published calibrated parameters from the ACARP study are summarised in Table 3-7.

Table 3-7 Summary of calibrated parameters published in ACARP study

	Parameter						
	C1 (mm)	C2 (mm)	C3 (mm)	A1	A2	A3	BFI
Bare dragline spoil							
Range	50–90	15–70	5	0.4–0.6	0.2–0.4	0.2	0
Average [1]	70.0	42.5	5.0	0.5	0.3	0.2	0
Vegetated dragline spoil							
Range	55–120	25–60	10–50	0.6	0.3	0.1	0–0.97
Average [1]	88.3	39.0	19.2	0.6	0.3	0.1	0.2
Rehabilitated dragline spoil							
Range	55–170	15–165	7–45	0.3–0.65	0.3–0.33	0.02–0.4	0–0.95
Average [1]	115.0	68.5	18.7	0.6	0.3	0.1	0.1
Rehabilitated truck and shovel spoil							
Range	160	60	25	0.6	0.3	0.1	0.7
Average [1]	160	60	25	0.6	0.3	0.1	0.7

Source: adapted from *ACARP Project No. C7007: Water Quality and Discharge Prediction for Final Void and Spoil Catchments* (PPK Environment & Infrastructure 2001).

Notes:
1 The average values presented in the table are averages of the site values and not the averages of the ranges.

As the overburden areas at the site are typically gently sloping on the top surface, the upper range of the surface storage capacity parameters for bare dragline spoil from the ACARP study were typically adopted for the 'active overburden' land use.

The surface storage capacity parameters adopted for the 'recent rehabilitation' land use were within the range of the rehabilitated dragline spoil and rehabilitated truck and shovel spoil parameters from the ACARP study. Based on the parameters from the ACARP study, higher runoff is expected from established rehabilitation areas compared to the natural catchment. However, runoff characteristics from established rehabilitation areas are expected to move closer to the natural case over time.

There is limited published information available on calibrated rainfall-runoff parameters for pit areas. In the absence of data for calibration, the lower range of the surface storage capacity parameters for bare dragline spoil from the ACARP study were typically adopted for the 'pit' land use, comprising the pit floor and in-pit overburden areas. This resulted in a simulated long term average annual runoff coefficient of approximately 27% for pit areas (refer to Section **Error! Reference source not found.** Table 3-9).

AWBM parameters for industrial areas with hardstand surfaces were estimated by modifying the storage capacity and partial area parameters to achieve a similar average annual runoff coefficient to urban catchments (around 40%).

3.5.1.3 Summary

A summary of the adopted AWBM parameters is provided in Table 3-8.

Table 3-8 Adopted AWBM parameters

Land use	AWBM parameters								
	Ks	BFI	K_{base}	A1	A2	A3	C1 (mm)	C2 (mm)	C3 (mm)
Undisturbed	0.400	0.150	0.955	0.134	0.433	0.433	50	150	350
Established rehabilitation (> 4 years)	0	0.200	0	0.100	0.300	0.600	40	100	170
Recent rehabilitation (< 4 years)	0	0.200	0	0.100	0.300	0.600	20	70	170
Active overburden	0	0.200	0	0.100	0.300	0.600	5	70	90
Pit (pit floor and in-pit overburden)	0	0	0	0.200	0.300	0.500	5	15	50
Industrial	0	0	0	0.333	0.333	0.333	2	10	40

The AWBM rainfall-runoff model was used to generate a daily time series of runoff from undisturbed, disturbed and rehabilitated catchments using the parameters in Table 3-8. The average runoff coefficients estimated from the AWBM rainfall-runoff model for the 111 year period from 1900 to 2010 are summarised in Table 3-9 for various modelled land uses.

Table 3-9 Calculated average runoff coefficients from adopted AWBM parameters for modelled land uses (1900 to 2010)

Land use	Volumetric runoff coefficient
Undisturbed	3.1%
Established rehabilitation (> 4 years)	4.7%
Recent rehabilitation (< 4 years)	6.9%
Active overburden	11.9%
Pit (pit floor and in-pit overburden)	27.3%
Industrial	40.0%

3.6 Harvestable rights

In most rural areas, land owners can collect in a farm dam (or dams) up to 10% (Department of Water & Energy 2008) of the average regional catchment runoff without a licence and use for any purpose - referred to as 'harvestable rights'. A landholder can construct, without a licence, farm dams to capture their harvestable right provided they are located on hillsides, gullies or minor, non-permanent watercourses. The total capacity of all dams on a property allowed under the harvestable right is the Maximum Harvestable Right Dam Capacity (MHRDC). Previously there was a 7 ML limit on dams, however this has been revoked, and the only limit on the dam size is the MHDRC, and what can practically be built on a hillside, gully etc.

A licence is required to capture more water than the harvestable right.

Harvestable rights dams are generally permitted on 1st and 2nd order streams anywhere in NSW consistent with the harvestable rights order by which the area is constituted. 1st and 2nd order streams are defined as:

- starting at the top of a catchment, any watercourse that has no other watercourses flowing into it is classed as a 1st order watercourse
- where two 1st order watercourses join, the watercourse becomes a 2nd order watercourse
- if a 2nd order watercourse is joined by a 1st order watercourse - it remains a 2nd order watercourse.

This MHRDC is determined via a runoff coefficient provided by NOW and the property's area. For the project site the MHRDC is 0.065 ML/ha (NOW 2008). Therefore based on a property area of 32,538 ha (as of January 2013), there is the potential to construct dams of capacity up to 2,115 ML.

A desktop mapping assessment by CHC identified 811 unlicensed farm dams located on the CHC property area. The total capacity of the existing unlicensed farm dams is estimated at 1,545 ML.

The capacity of harvestable right currently not accounted for by unlicensed dam capacity is estimated at 570 ML. Additional storage capacity would require licensing.

4. Proposed site water management system

4.1 Water segregation

Where practical, it is proposed to segregate water within the mine site according to its quality to minimise the stored volumes of water with high concentrations of dissolved ions or suspended solids. This would allow containment of water requiring treatment (e.g. settling suspended sediment) and the diversion of clean water away from mining activities (e.g. undisturbed catchments).

Five water classifications have been nominated for the mine site:

- **Clean water** — runoff from undisturbed areas that are expected to have pre-mining water quality and that can be diverted to the creek system.
- **Overburden water** — runoff from overburden emplacements, topsoil stockpiles and other disturbed areas. This water contains elevated suspended solids, which will be settled in sedimentation dams before release to the creek system or on-site reuse.
- **Pit water** — runoff from the open pits and groundwater seepage into the mine. This water can potentially contain suspended solids, salts and heavy metals etc. This water will be stored on-site and will be reused.
- **Infrastructure water** — runoff from the areas around the coal preparation plant, stockpiles and infrastructure. This water will be directed to the process water circuit.
- **Process water** — water that is utilised in the CHPP, including return water from the reject emplacement areas and refuse disposal ponds. This water is continuously recycled within the system.

Water management system concepts for the various classes of water are discussed in the following sections.

4.2 Clean water system

Clean water runoff from undisturbed catchments will be diverted around the mine site as much as practical to minimise the site water inventory and maintain pre-development releases into Sandy Creek and Laheys Creek. Clean water runoff that is not diverted around the site and is captured within the water management system will be reused on-site.

The clean water system comprises:

- Clean water catch drains to divert minor catchments around the mine site, where practical. Catch drains have been considered when delineating catchments, but have not yet been designed as part of the water management system. The size of catch drains will be considered further during detailed design.
- Clean water highwall dams and levees upslope of the pit to reduce peak flow rates and velocities from undisturbed catchments. Highwall dams will be pumped out to the creek system. Runoff from very large storm events will overtop highwall dams and flow into the pit. For the purposes of conceptual design, highwall dams have been provided at selected key locations only. However, it may be possible to provide additional highwall dams to further reduce inflows to the pit, and this will be considered during detailed design.

- A pump and pipeline system to pump clean water stored in clean water highwall dams to Sandy Creek and Laheys Creek.
- Raw water dam to store water imported to the site from the Macquarie and Cudgegong Regulated River Water Source to supplement on-site water requirements. The proposed raw water dam is located on a drainage line within the Laheys Creek catchment, and has a maximum storage volume of 1,000 ML. The proposed raw water dam has a local natural catchment area of approximately70 ha, and clean runoff from this catchment would also flow to this dam.
- A pump and pipeline system from the raw water dam to deliver stored water to either:
  - process water dam (for use in the CHPP and mine infrastructure area)
  - truck fill stations (for haul road dust suppression)
  - water treatment plant (for potable applications).
- Flood mitigation works. Levees will be provided along the edge of the mining areas to help control flow and prevent flood water from Sandy Creek and Laheys Creek entering the mine working area. Flood mitigation works are described in the *Flood Impact Assessment* report.

Clean water runoff from established rehabilitated overburden emplacements will be returned directly to Sandy Creek and Laheys Creek. Water from established rehabilitation areas will be returned once rehabilitation success criteria are met.

4.3 Overburden water system

Runoff from disturbed areas will be captured in sedimentation dams for environmental protection and on-site reuse, thus minimising the need to use raw water supplies when the mine is short of water.

Sedimentation dams will allow time for coarse sediments to settle and, if necessary, allow a suitable flocculent to be added to remove fine or dispersive sediment to meet licensed water quality discharge limits.

Treated runoff from large storm events will flow through sedimentation dams and typically pass to Sandy Creek and Laheys Creek. Captured water would either be released to Sandy Creek and Laheys Creek following settling, or transferred to water management dams for on-site storage and reuse.

To ensure sufficient capacity is available in the ‘settling zone’ for water from storm events, sedimentation dams would be maintained in a drawn-down state, as much as practical. If reuse opportunities are not available, releases would only be permitted when licence limits have been met.

The overburden water management system comprises:

- A drainage system to convey runoff from overburden emplacements, topsoil stockpiles and other disturbed areas to the nearest sedimentation dam.
- Several sedimentation dams strategically located throughout the mine site to capture water from the above sources.

- Pump and pipeline systems to transfer water stored in sedimentation dams where the dams do not allow runoff to return freely to downstream watercourses.
- Facilities to monitor water quality and quantity at nominated surface water release and monitoring points.
- Streamflow gauging stations to determine and record streamflows in Sandy Creek and Laheys Creek upstream and downstream of the site.
- A pump and pipeline system to transfer water stored in sedimentation dams to the nearest mine water management dam when the water can be reused on-site.

For the purpose of conceptual design, the majority of sedimentation dams have their own release points to the creek system. It may be possible, however, to minimise the number of such points by combining several sedimentation dam outlets into a common point. This would require additional drains and possibly pumps and pipelines and should be considered during detailed design.

4.4 Pit, infrastructure and process water systems

While water will be carefully managed to minimise the volume collected in the open mine pits, some water will make its way into the pits either by direct rainfall, groundwater seepage, runoff from and seepage through overburden emplacements, or runoff from undisturbed catchments upslope of pits that cannot be practically diverted around or captured in highwall dams.

For the purpose of this assessment, the pit, infrastructure and process water systems have been combined into a single system, referred to as the mine water system.

The mine water management system comprises:

- Small sumps in the pit floor to collect and contain local surface water runoff from the pit floor, high wall, low wall and end walls, as well as groundwater seepage.
- Pit dewatering pumps and associated dewatering pipelines to transfer pit water to the nearest mine water dam, if necessary via a small staging dam.
- Treated effluent from the on-site wastewater treatment plant.
- A drainage system to convey runoff from disturbed areas to the nearest mine water storage dam.
- Mine water dams to store and contain water from the above sources. Care has been taken in the location of storages and the layout of the drainage system to minimise the areas draining to these dams, so as to minimise storage requirements and maintain environmental compliance during rainfall events.
- A return water pump and pipeline system from each mine water dam to deliver stored water to either:
    - CHPP (for processing of ROM coal into product coal)
    - mine infrastructure area (for vehicle wash, workshop and dust-suppression sprayers etc.)
    - truck fill stations (for haul road dust suppression).

There will be no releases of water captured in the mine water system to the natural creek system. This system will be used as a priority to meet on-site water demands and imported raw water will only be used when there is a water deficit or high quality water is required for uses such as potable applications.

During extended wet periods, surplus mine water will be stored in-pit once the mine water dams have reached their capacity. The scale of the mine with multiple pits allows pit water to be pumped between individual open cuts. This gives good flexibility, allowing the mine to continue to operate without flooding active workings or relying on unscheduled releases of surplus water.

4.5 Tailings management system

The proposed tailings management system is described in the Tailings Storage Facilities Management Plan (CHC, 2013). The system will involve the construction and operation of a series of tailings emplacements collectively known as Tailing Storage Facilities over the life of the project to store fine-grained slurry waste from the CHPP. Each emplacement will have a design life incorporating a final decommissioning and rehabilitation phase. The overriding strategy will be to use a combination of multiple in-pit and out-of-pit storage emplacements to manage rise rates and hence settled densities. Rehabilitation will be an ongoing process throughout the life of the mine.

Tailings will be mainly deposited from the back of the out of pit emplacements with a decant pond forming at the embankment wall. For the in-pit emplacements the reverse will apply; i.e. beaching at the embankment with decant at the back.

The tailings decant water will be one of the main sources of water for the CHPP. Tailings emplacement decant water will be pumped from the decant pond into the mine water system. It is intended to operate pumps manually, and to minimise the stored volume in the decant pond, including after rainfall.

The recovery of water from the Tailings Storage Facilities is a priority of the mine operation. Ground conditioning of floor and embankment areas within the out-of-pit Tailings Storage Facilities will be undertaken to limit seepage rates, and maximise water available for decant. Conditioning would typically take the form of a clay liner. Seepage will be managed using seepage channels and ponds downstream of the embankment.

Based on experience with similar out of pit tailings emplacements elsewhere, ATC Williams Pty. Limited as advised CHC that recoverable process water will be about 30% of the water contained in the tailings received. This includes an allowance for evaporative and seepage losses. ATC Williams Pty. Limited has advised CHC that for in-pit emplacements the more permeable walls and embankments are expected to result in higher seepage losses. Recoverable process water is estimated to be about 15% of the water contained in the tailings received.

4.6 Staging of the water management system

The water management system will evolve as the Project expands, to be compatible with the mine landform and schedule.

Development of the water management system concepts over the mine's 21-year life have been illustrated through snapshots at five mine stages, corresponding to Year 1, 4, 12, 16 and 20 mine landforms. For the purpose of the surface water assessment, the five mine stages selected provide a reasonable representation of how the mine will be developed.

The five mine stages also illustrate the design flow paths of different classes of water and locations of water management dams during respective stages of the mine's development. Schematic diagrams showing the general connectivity between water sources, demands and storages are provided in Figure 4.6 to

Figure 4.10.

A total of 12 mine water dams, 39 sedimentation dams, one raw water dam and nine highwall dams are proposed to manage surface water runoff and water supply of the mine over its 21-year life. Dam staging is summarised in Table 4-1.

Table 4-1 Dam staging over life of the Project

Year	Number of active dams			
	Mine water dam	**Sedimentation dam**	**Raw water dam**	**Clean water/ highwall dam**
Year 1	9	15	1	3
Year 4	11	18	1	5
Year 12	9	18	1	7
Year 16	9	15	1	3
Year 20	9	13	1	2
Total no. of dams (over life of project)	*12*	*39*	*1*	*10*

Note: Excludes tailings emplacements.

It has been assumed that dams will be constructed to their maximum capacity when they are first commissioned. In practice, there may be opportunities for staging storage capacities without compromising the system's performance when catchment areas increase as the mine develops. This will be considered during detailed design.

Runoff from rehabilitation areas will be returned directly to the creek system once rehabilitation success criteria have been met. For the purposes of conceptual design of sedimentation dams and environmental impact assessment, it has been assumed that rehabilitated areas would be established four-years after final shaping, topsoil spreading and revegetation.

It is also assumed that runoff from these areas would contain low levels of suspended solids and would be suitable for direct return to the creek system (i.e. without passing through a sedimentation dam). This would be confirmed by water quality monitoring.

Where it is not practical to divert runoff from areas of active spoil or recent rehabilitation away from downstream established rehabilitation, runoff from established rehabilitation will not be included in the sedimentation dam network.

4.7 Erosion and sediment controls during construction

An Erosion and Sediment Control Plan (ESCP) should be prepared and implemented during the construction of all mine infrastructure. Erosion and sediment controls should be established to a standard consistent with the following guidelines:

- *Managing Urban Stormwater – Soils and Construction – Volume 1* (Landcom 2004).
- *Managing Urban Stormwater – Soils and Construction – Volume 2E Mines and Quarries* (DECCW 2008).

Erosion and sediment controls should include:

- minimising forward clearing, particularly areas around flow lines, drainage lines and watercourses

- minimising site disturbance by containing machinery access to areas required for approved construction works, access tracks or material stockpiles
- staging construction activities where practical, so that land disturbance is confined to the minimum possible area
- completing work and rehabilitating disturbed areas quickly and progressively
- minimising erosion from drainage lines that can be vulnerable to the erosive effects of concentrated flow
- intercepting and diverting clean water runoff from undisturbed areas so that it does not flow onto disturbed areas
- passing clean water through the site without mixing it with runoff from disturbed areas
- treating highly dispersive soils with gypsum to reduce the potential for tunnel erosion and surface rilling of disturbed areas
- limiting erosion potential within earthworks areas by managing runoff fetches and velocities, with measures such as diversion banks
- locating sediment traps, such as silt fences and check dams, downstream of disturbed areas
- treating runoff from large construction areas (greater than 2,500 m^2) in sedimentation basins or dams, before water is displaced to watercourses
- providing shaker ramps and rock pads at construction exits to remove excess mud from truck tyres and under bodies.

Management strategies to strip, handle and respread topsoil post-disturbance, and prepare seedbeds, should also be implemented during the construction of mine infrastructure.

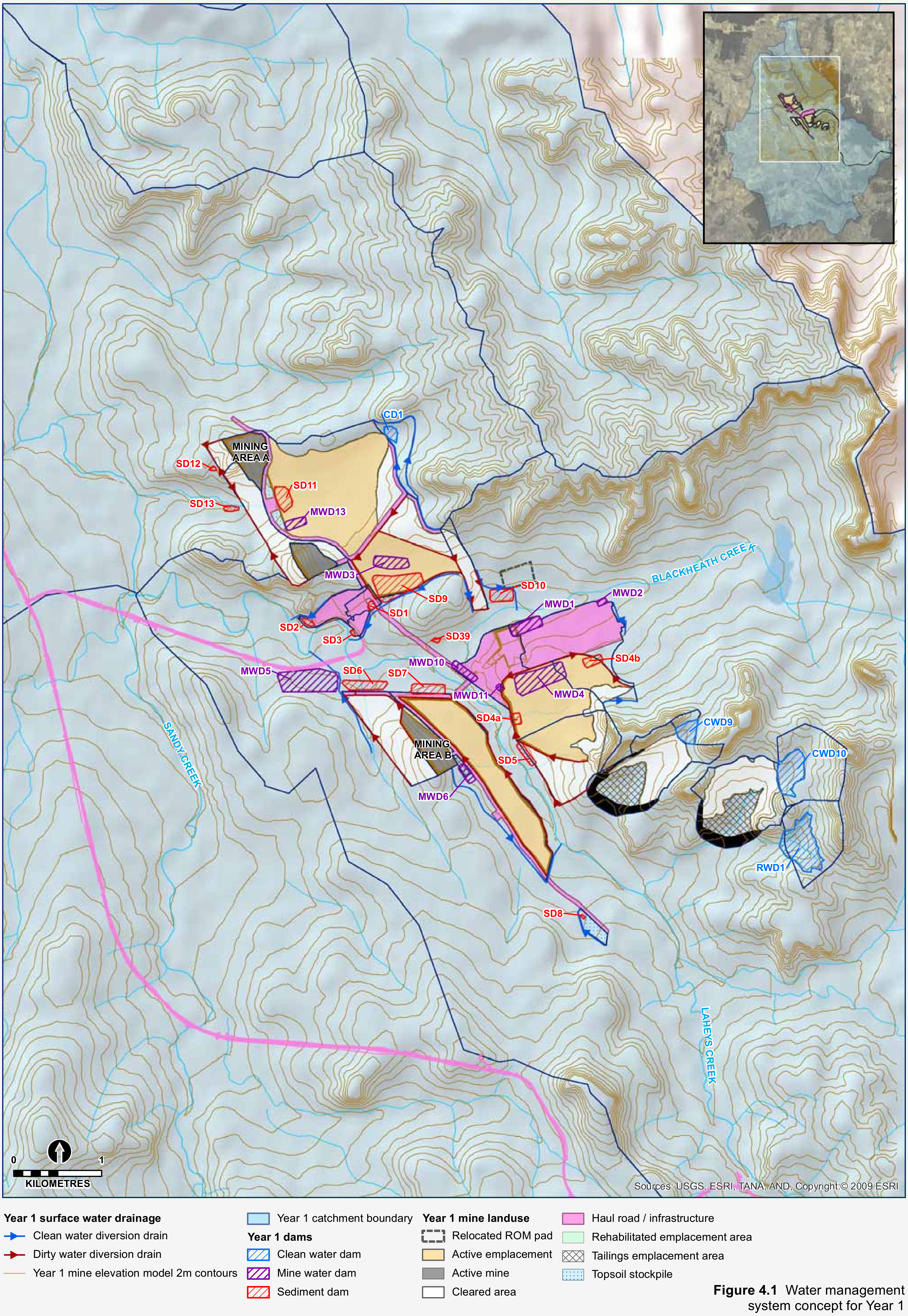

Figure 4.1 Water management system concept for Year 1

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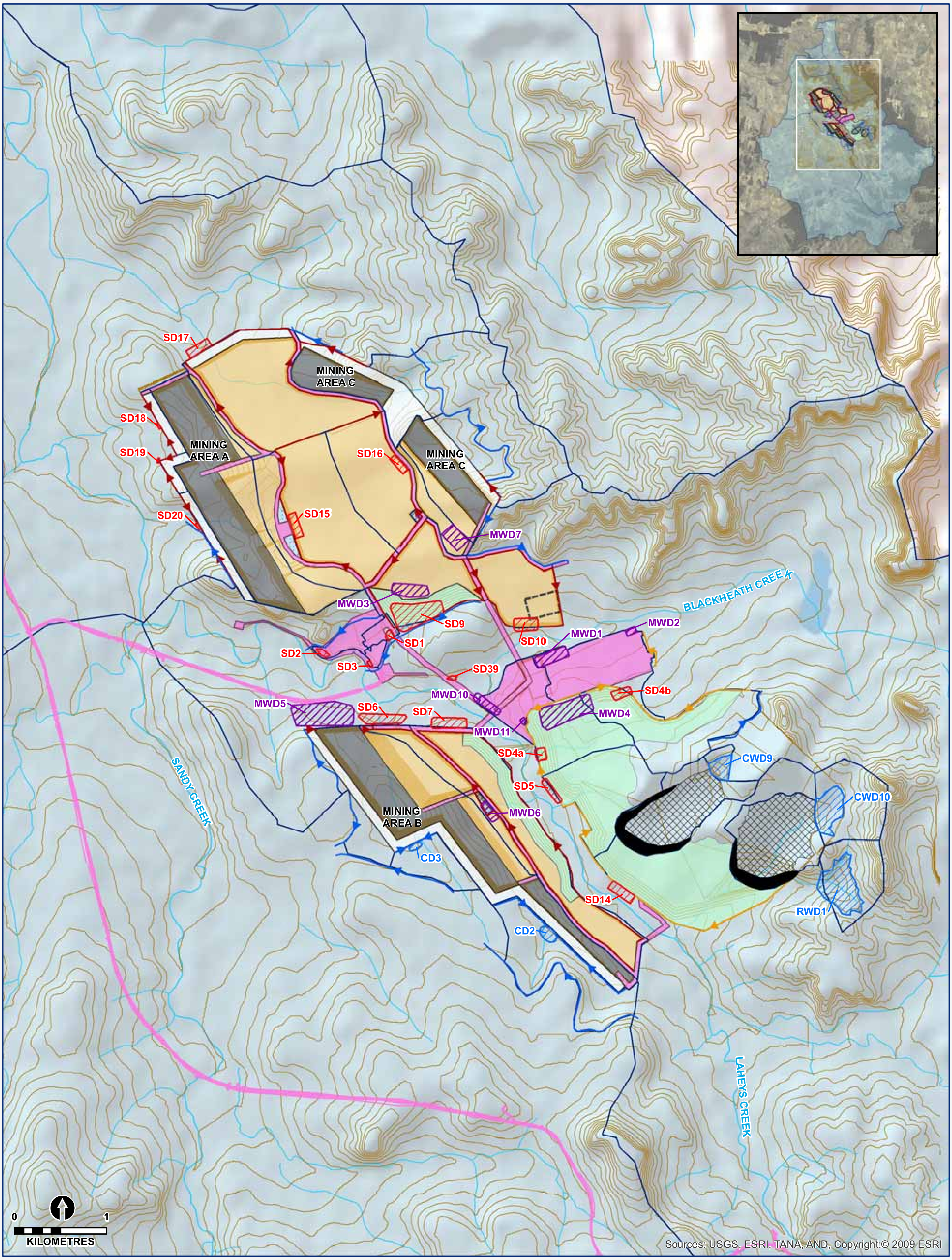

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Year 4 surface water drainage
- Clean water diversion drain
- Dirty water diversion drain
- Dirty water diversion drain (established rehabilitation released)
- Year 4 mine elevation model 2m contours

Year 4 catchment boundary

Year 4 dams
- Clean water dam
- Mine water dam
- Sediment dam
- Dam walls

Year 4 mine landuse
- Relocated ROM pad
- Active emplacement
- Active mine
- Cleared area
- Haul road / infrastructure
- Rehabilitated emplacement area
- Tailings emplacement area
- Topsoil stockpile

Figure 4.2 Water management system concept for Year 4

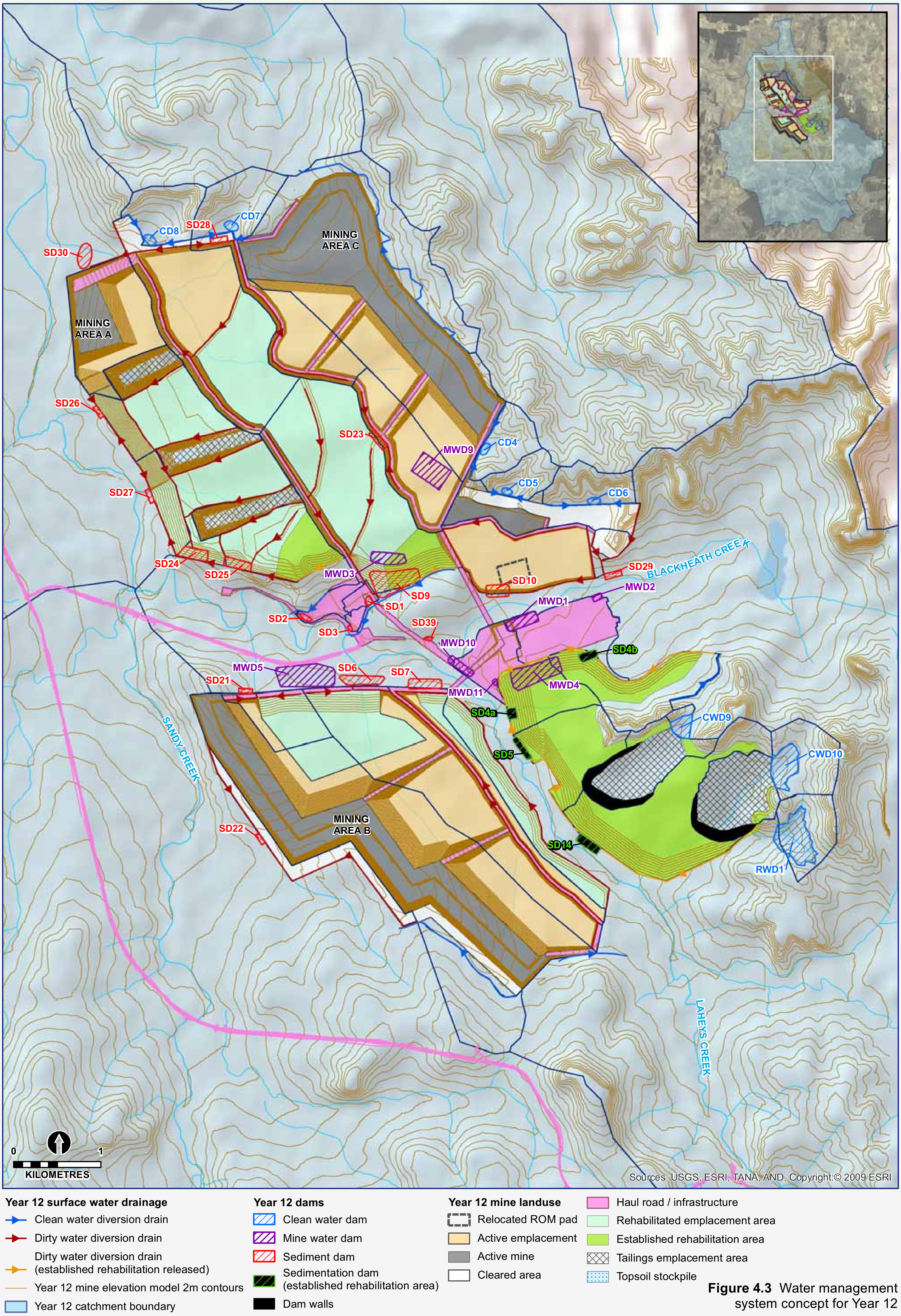
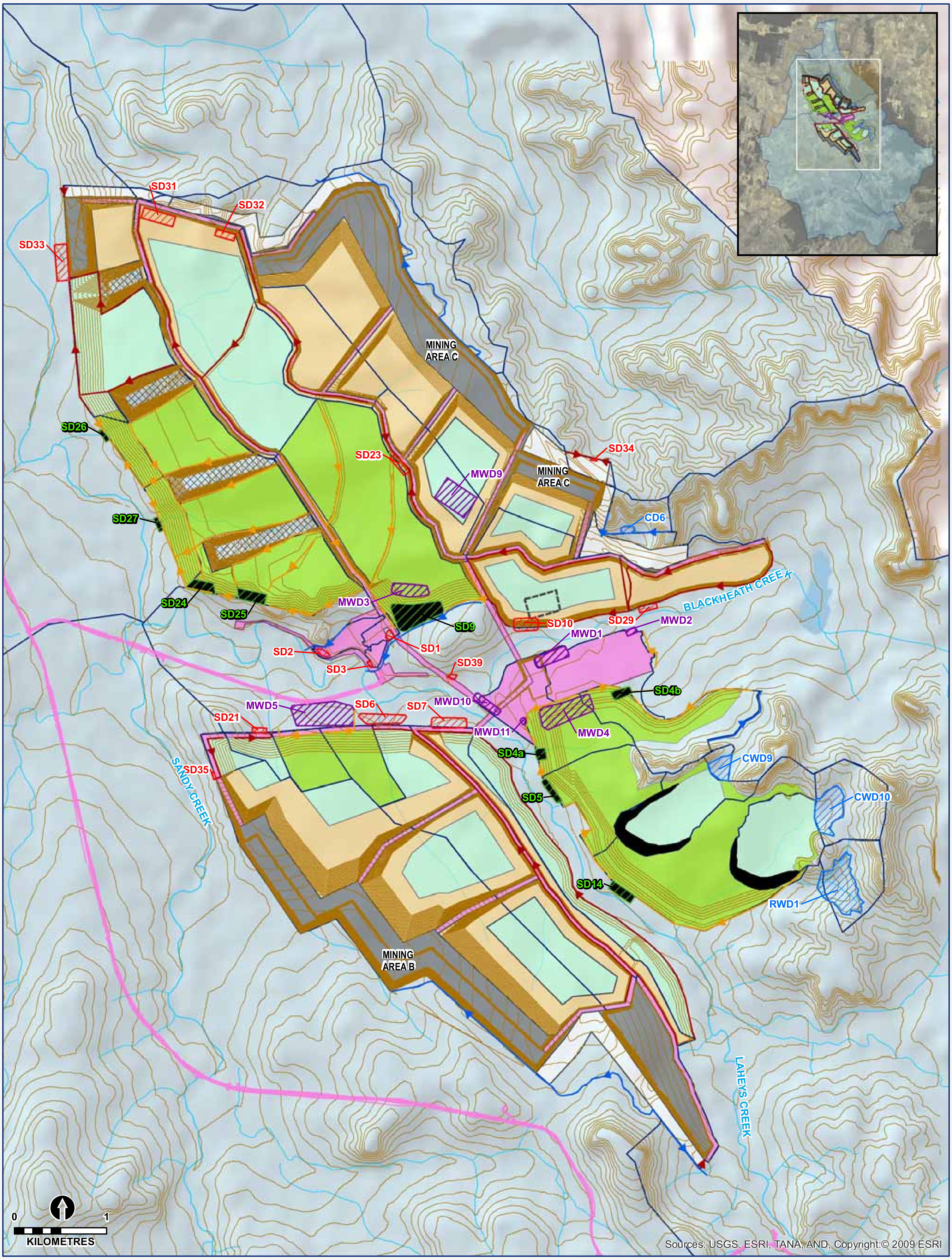


Figure 4.3 Water management system concept for Year 12



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Year 16 surface water drainage
- Clean water diversion drain
- Dirty water diversion drain
- Dirty water diversion drain (established rehabilitation released)
- Year 16 mine elevation model 2m contours
- Year 16 catchment boundary

Year 16 dams
- Clean water dam
- Mine water dam
- Sediment dam
- Sedimentation dam (established rehabilitation area)
- Dam walls

Year 16 mine landuse
- Relocated ROM pad
- Active emplacement
- Active mine
- Cleared area
- Haul road / infrastructure
- Rehabilitated emplacement area
- Established rehabilitation area
- Tailings emplacement area
- Topsoil stockpile

Figure 4.4 Water management system concept for Year 16

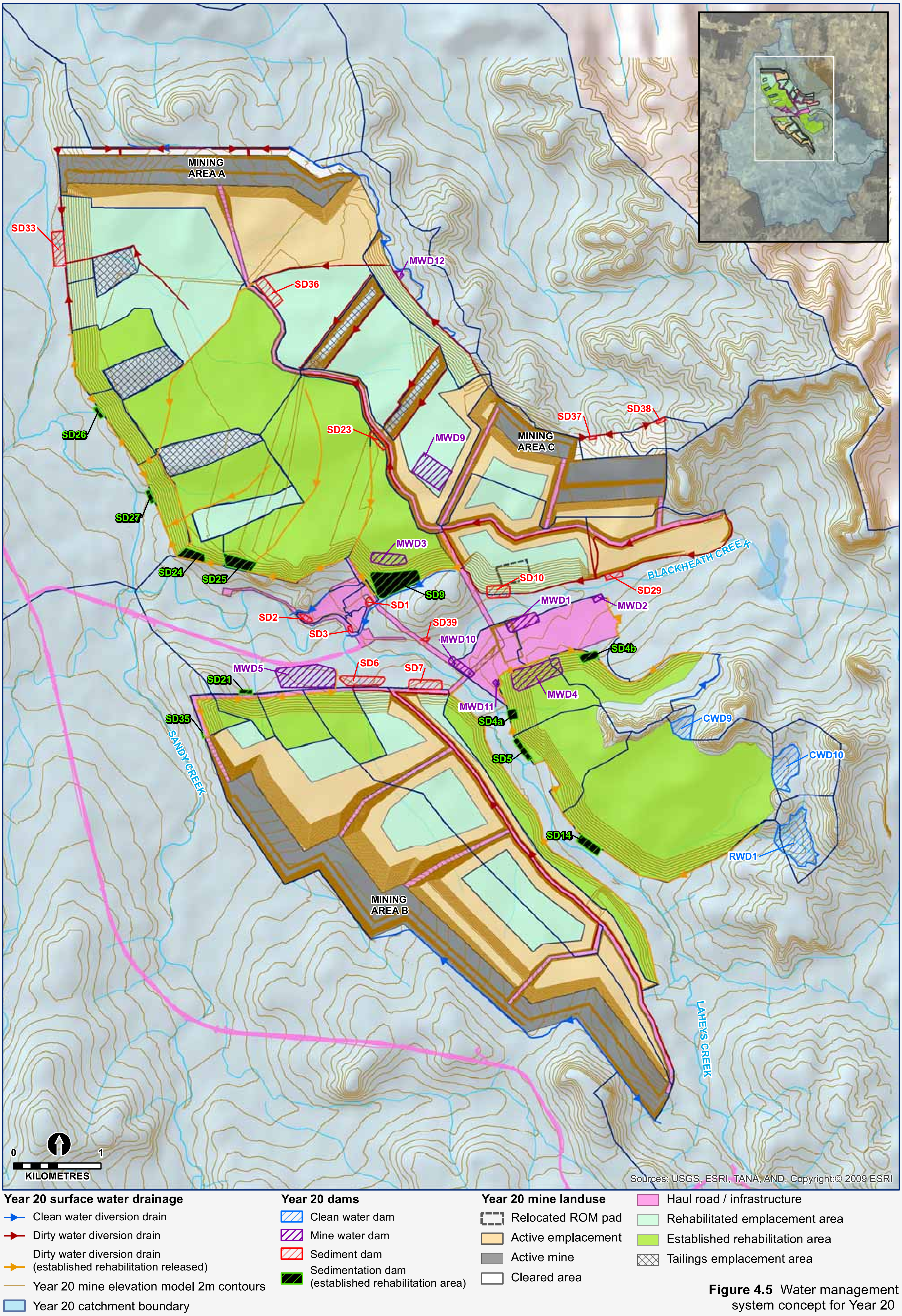

Figure 4.5 Water management system concept for Year 20

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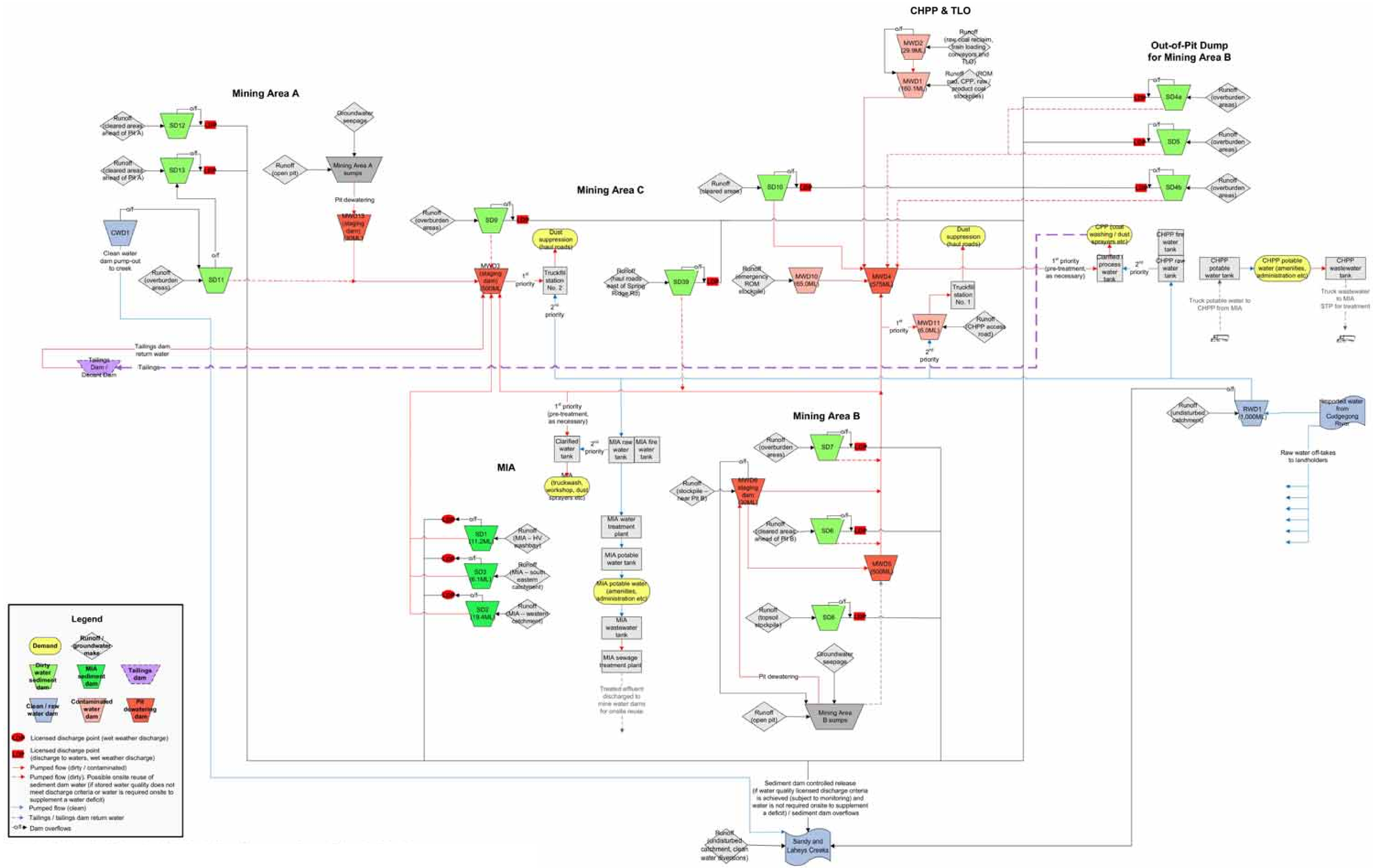

FIGURE 4.6
SCHEMATIC WATER MANAGEMENT SYSTEM CONCEPT FOR YEAR 1

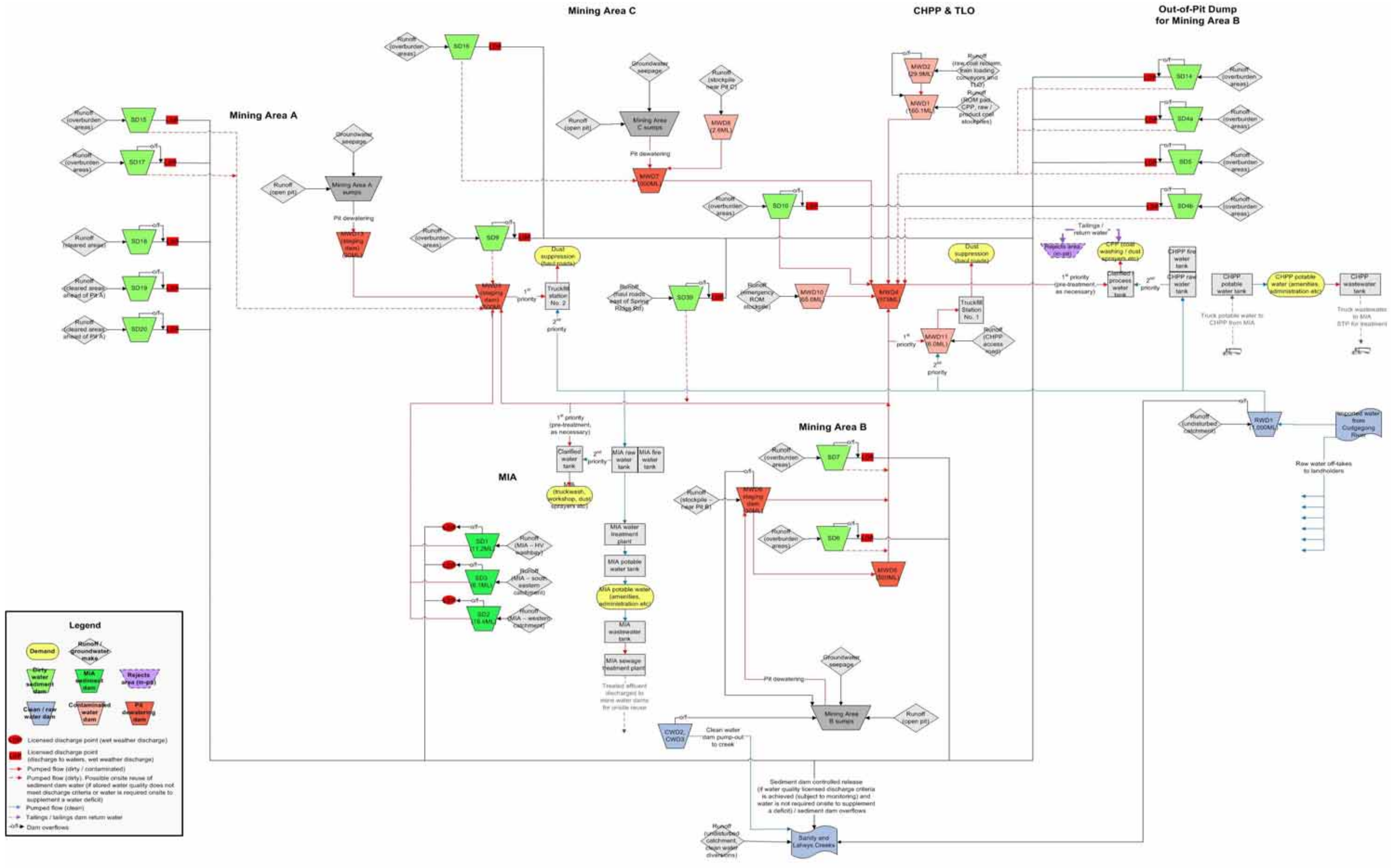

FIGURE 4.7
SCHEMATIC WATER MANAGEMENT SYSTEM CONCEPT FOR YEAR 4

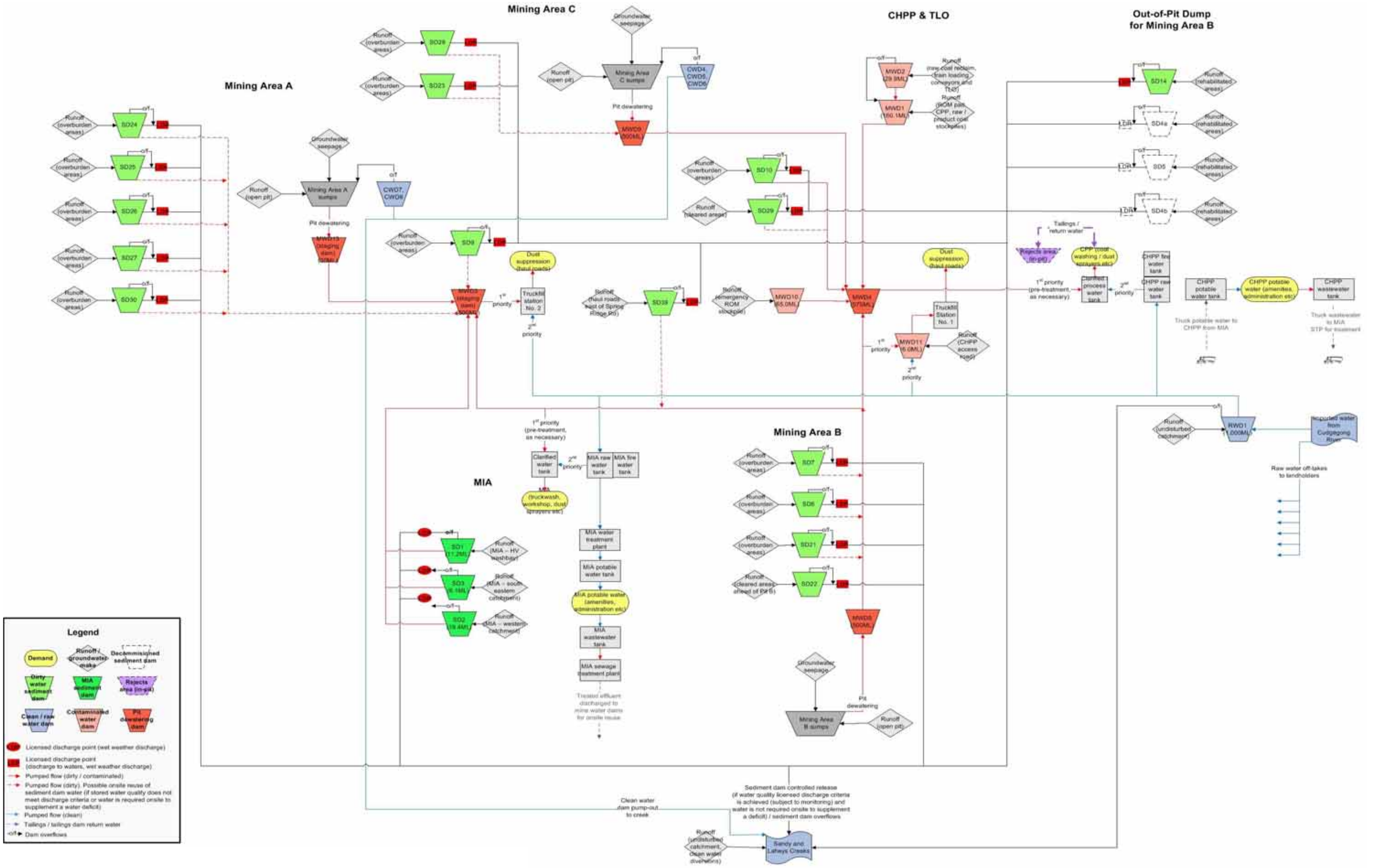

FIGURE 4.8
SCHEMATIC WATER MANAGEMENT SYSTEM CONCEPT FOR YEAR 12

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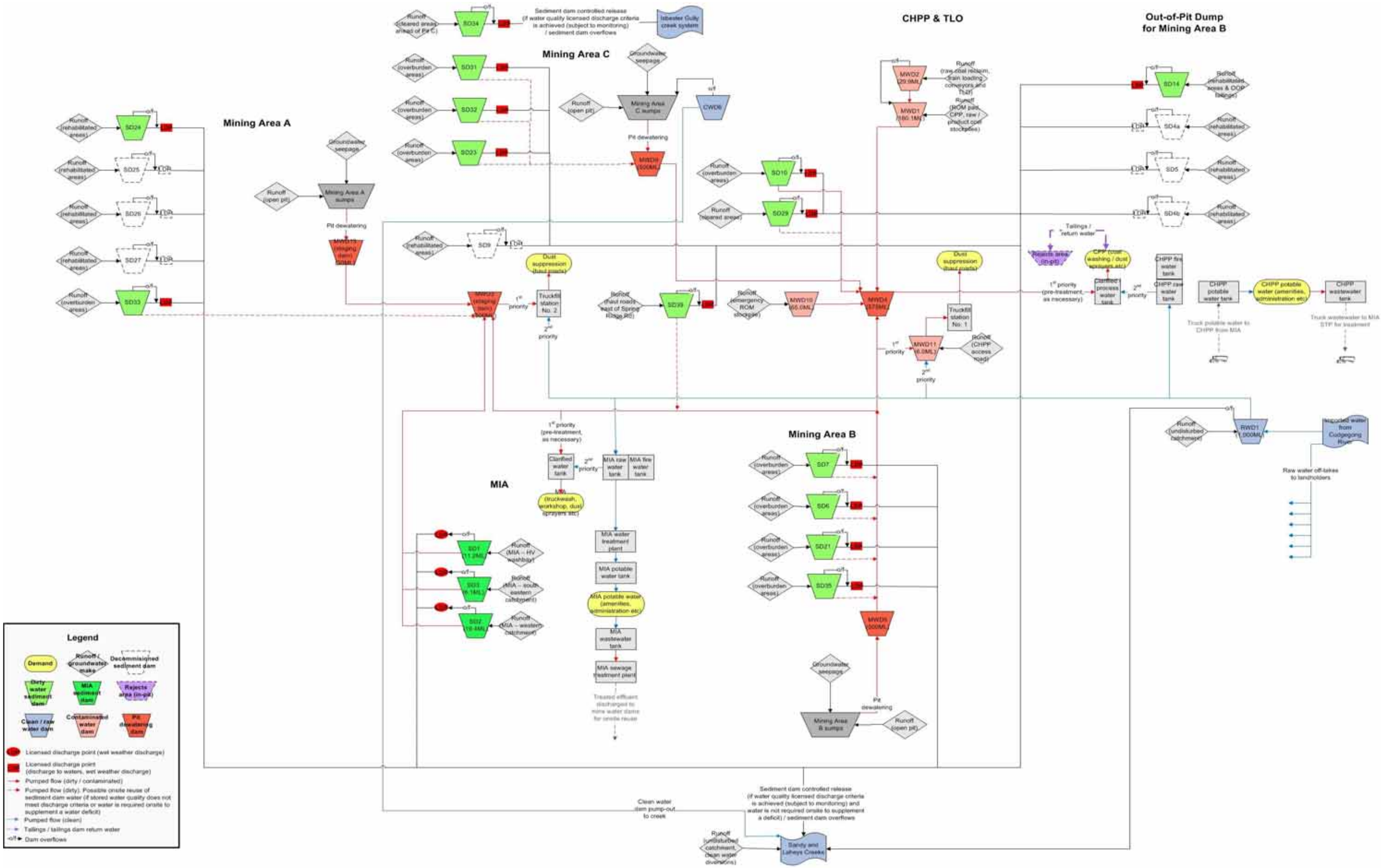


FIGURE 4.9
SCHEMATIC WATER MANAGEMENT SYSTEM CONCEPT FOR YEAR 16

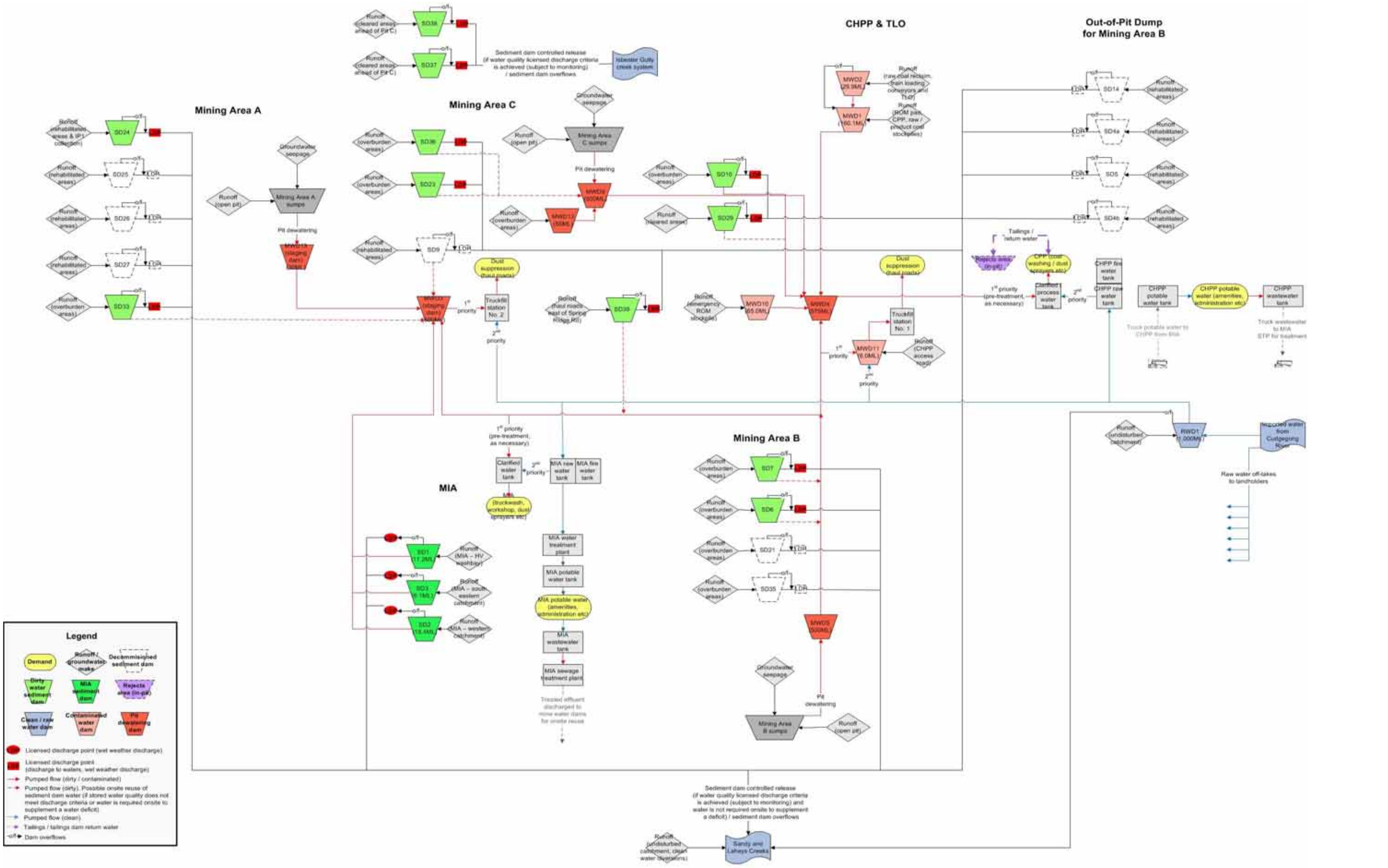


FIGURE 4.10
SCHEMATIC WATER MANAGEMENT SYSTEM CONCEPT FOR YEAR 20

5. Site water balance modelling

5.1 Modelling approach

Water balance modelling has been undertaken to: assess the performance of the Project's water management system; estimate annual runoff volumes: potential water deficits and surpluses over the life of the mine; and confirm the preliminary sizing of water management dams.

5.1.1 GoldSim model

A water balance model of the Project was developed using GoldSim software, a widely used platform for mine site water balance studies.

The GoldSim model was used to calculate the volume of water in storages at the end of each day by taking into account daily rainfall-runoff inflow, groundwater inflow, evaporation from the storage, water usage, pumping between storages in the form of a pumping policy, and storage overflow. The network diagrams provided in Figure 4.6 to

Figure 4.10 show the conceptual layout and interconnectivity of storages for the mine site.

The model logic for water transfers between storages and from their catchments to the discharge points are based on schematics of water management concepts, as presented in Figures 4-1 to 4-5.

Separate GoldSim models were developed for Year 1, 4, 12, 16 and 20 mine landforms to assess the site water balance over the life of the mine. Each model simulated 111 years of climate data (see Figure 3.2 and Figure 3.3) based on a daily time step. The climate data were statistically analysed to present potential water balances for 10th (dry), 50th (median) and 90th (wet) percentile years.

Put simply, the model took Year 1 of the mine and ran it though the first year of the meteorological data. It then ran Year 1 through the second year of meteorological data, carrying forward all water in dams stored at the end of the first year. This process was repeated for each of the 111 years of the climatic data set. This allowed an understanding of the impacts of a series of successive dry and wet years on the performance of the mine's water management system.

The above modelling sequence was subsequently repeated for Years 4, 12, 16 and 20, in each case carrying forward retained water from one year to the next.

5.1.2 Proposed operating scenario

The following two operating scenarios were considered:

- **Scenario A — release sedimentation dam water**. In this scenario, overburden runoff is displaced to the creek system in accordance with relevant licensing requirements. This is based on the assumption that the water quality of overburden runoff will be similar to pre-mining conditions (with the exception of elevated suspended solids) and that water quality criteria would be met following settling of suspended solids in sedimentation dams. Other water affected by mining is fully used in meeting the Project's water demands. Clean water is diverted from mining operations to the downstream creeks.

- **Scenario B — reuse sedimentation dam water**. In this scenario, water captured in sedimentation dams is pumped to the nearest mine water management dam if the volume stored in that dam is less than 25% capacity. Otherwise, the sedimentation dam water is displaced to the creek system in accordance with relevant licensing requirements. All other model operating rules are the same as for Scenario A.

Further analysis of the above two scenarios concluded that Scenario B would be the most appropriate scenario to be implemented for the Project. Scenario B has the advantage of reducing the volume of water from sedimentation dams released to downstream creeks and reducing the demand for raw water from the Macquarie and Cudgegong Regulated River Water Source.

On this basis, Scenario B has been adopted as the proposed operating scenario to be used in the water balance model and water management system conceptual design presented in this report.

5.2 Model assumptions

The water balance model has been developed and refined to a level of detail suitable for conceptual design and environmental impact assessment of water management infrastructure. Some assumptions and simplifications were incorporated into the model that may need to be expanded for other applications:

- Pump rating curves have not been discretely modelled, and therefore the model does not represent delays that could occur when transporting water around the site.
- Runoff parameters have been selected using data published for other similar projects with limited quantitative data to assess the runoff characteristics of disturbed catchments at the Project site.
- The tailings emplacement areas are included in the model, and the reuse of direct rainfall-runoff from the limited tailings emplacement area catchments and tailings bleed are considered in the water balance. Tailings emplacements have been designed with sufficient freeboard to avoid overflows.
- It has been assumed that pumping from the Macquarie and Cudgegong Regulated River Water Source occurs at a rate of 20 ML/d, as required to maintain a storage volume of at least 50% capacity in the raw water dam. When assessing the adequacy of existing water access licences (WALs) held by CHC, it has been assumed in the main modelling analysis that extraction of the full amount of high security access licences is possible from the Cudgegong River throughout the life of the Project. A Framework for Extraction Strategy Agreement is currently subject to formal agreement between CHC and State Water Corporation. It is to be established to allow CHC to extract river water in accordance with its Works Approval. To maintain environmental flows in the system, the Agreement does not permit extraction when flow is less than 25 ML/d at the downstream Yamble Bridge NOW gauging station. To account for this constraint a sensitivity analysis has been undertaken to determine the impact on the water supply to the mine. This is discussed further in Section 5.4.3.
- It has been assumed that WALs held by CHC in the Macquarie and Cudgegong Regulated River Water Source have a high reliability of supply. The system is managed under the Water Sharing Plan to ensure available water determinations for regulated river high security access licences are credited with 100% through a repeat of the worst period of low inflows to this water source. Sufficient volumes of water are set aside from assured inflows into this water source and reserves held in Windamere Dam to provide for this assumption of available water determinations. Therefore, CHC can expect to have 3,311 ML credited to their water allocation accounts and available for use in every water year.
- While the model assesses the performance of the system under historical extremes that may reasonably be expected to reoccur in the future, it does not specifically quantitatively incorporate the potential impact of future climate change on runoff, given the limited influence this could have during the life of the mine.
- Groundwater seepage rates should be considered provisional only. Additional groundwater modelling during detailed design, and any potential dewatering that may be required to minimise seepage to the pit, could alter extraction rates provided in Section 5.4.2.
- The existing Sandy Creek and Laheys Creek catchments comprise numerous small farm dams, which have not been included in the water balance model. The only exceptions are the existing ‘ Woolandra’ farm dams (Woolandra West Dam and Sausage Dam) located in the upper reaches of the Blackheath Creek catchment which have been included in the baseline model because of their large capacity and potential to impact on baseline flows in Laheys Creek. These farm dams, however, have not been considered in the operational or final landform water balance models nor included

in the proposed water management system for the Project, rather, it has been assumed that runoff from their catchments will be returned to Laheys Creek.

This report presents a conceptual water management system that will be refined and optimised as detailed design proceeds, and the quantity and quality characteristics of surface runoff and groundwater seepage are better understood.

Since completion of the water balance model, the CHPP layout has been modified to provide a more efficient working arrangement. The CHPP and associated infrastructure will remain outside of the nearby drainage line (Blackheath Creek) and will be above the 1 in 100 year flood level. The main EA shows the ROM pad on the south side of Blackheath Creek about 550 m from Laheys Creek (Figure 3.15 of the main EA). It is now proposed that the ROM pad will be north of Blackheaths Creek about 1,200 m from Laheys Creek. In this position, it will be in mining areas A and C reducing the haul distances and greenhouse gas emissions and it will be further from Laheys Creek, which allows associated sedimentation dams to be above the 1 in 100 year flood level. Out-of-pit waste rock emplacement AC-OOP will be built up around the west, north and east sides of the ROM pad.

The water balance model has not been updated to reflect the relocation of the ROM pad as it does not significantly change the outcome of the modelling and downstream impacts due to releases of water from the mine. The previous location of the ROM pad south of Blackheath Creek will still form part of an infrastructure area that will need to drain to the mine water dam MWD10 as already assumed. Runoff from the small catchment of the relocated ROM pad will also need to be captured and treated/reused locally, but this would have a negligible difference on the water balance of the mine as currently modelled.

5.3 Model data

5.3.1 Catchments

The change in the disturbed area and land use over the life of the Project is summarised in Table 5-1. Note that this table does not include undisturbed areas draining to the Project's water management system.

Table 5-1 Change in disturbed area over life of the Project

Land use	Area (ha)				
	Year 1	**Year 4**	**Year 12**	**Year 16**	**Year 20**
Active overburden emplacement	363	686	970	987	958
Active mine	50	292	537	471	444
Cleared area	256	135	126	143	64
Infrastructure/haul roads	173	232	268	309	274
Rehabilitated overburden	0	340	1,127	1,787	2,225
Tailings emplacement area	38	132	227	113	130
Topsoil stockpile	8	4	0	0	0
Total	**887**	**1,822**	**3,254**	**3,809**	**4,094**

Catchment boundaries for the water management system were delineated using conceptual mine plans and reasonable assumptions about the likely destination of runoff.
Catchment boundaries are shown on the conceptual water management system plans

provided in Figures 4-1 to 4-5 for Year 1, 4, 12, 16 and 20 mine landforms. A summary of catchment areas is provided in

Table 5-2.

Table 5-2 Catchment areas over life of Project

Name	Area (ha)				
	Yr 1	Yr 4	Yr 12	Yr 16	Yr 20
Surface water management system					
RWD	50.5	50.5	50.5	50.5	50.5
CD1	16.7	0.0	0.0	0.0	0.0
CD2	0.0	84.8	0.0	0.0	0.0
CD3	0.0	30.3	0.0	0.0	0.0
CD4	0.0	0.0	39.2	0.0	0.0
CD5	0.0	0.0	29.1	0.0	0.0
CD6	0.0	0.0	40.2	26.3	0.0
CD7	0.0	0.0	56.8	0.0	0.0
CD8	0.0	0.0	48.4	0.0	0.0
CD9	8.3	8.3	8.3	8.3	8.3
CD10	58.3	58.3	58.3	58.3	58.3
MD1 / MW2	76.4	76.4	76.4	76.4	76.4
MD3	0.0	0.0	0.0	0.0	0.0
MD4	0.0	0.0	0.0	0.0	0.0
MD5	0.0	0.0	0.0	0.0	0.0
MD6	1.9	1.9	0.0	0.0	0.0
MD7/MD9	0.0	0.0	0.0	0.0	0.0
MD8	0.0	1.0	0.0	0.0	0.0
MD10	25.5	25.5	25.5	25.5	25.5
MD11	0.0	0.0	0.0	0.0	0.0
MD12	0.0	0.0	0.0	0.0	54.9
Mining Area A	32.1	165.5	113.7	73.6	347.2
Mining Area B	20.2	236.6	485.3	713.2	697.1
Mining Area C	0.0	181.9	492.8	590.2	193.6
SD1	7.5	7.5	7.5	7.5	7.5
SD2	13.0	13.5	13.7	13.6	13.6
SD3	4.0	4.1	4.1	4.1	4.0
SD4a	56.5	56.5	0.0	0.0	0.0
SD4b	28.2	91.0	0.0	0.0	0.0
SD5	72.7	77.8	0.0	0.0	0.0
SD6	45.0	30.4	221.9	208.9	205.9
SD7	102.4	120.4	119.6	150.0	101.6
SD8	5.6	0.0	0.0	0.0	0.0
SD9	79.0	81.3	232.2	0.0	0.0
SD10	16.7	62.0	109.8	145.5	158.6
SD11	179.2	0.0	0.0	0.0	0.0
SD12	6.7	0.0	0.0	0.0	0.0
SD13	38.3	0.0	0.0	0.0	0.0
SD14	0.0	146.3	146.4	304.5	0.0
SD15	0.0	156.6	0.0	0.0	0.0
SD16	0.0	109.7	0.0	0.0	0.0
SD17	0.0	155.4	0.0	0.0	0.0
SD18	0.0	8.1	0.0	0.0	0.0
SD19	0.0	6.4	0.0	0.0	0.0
SD20	0.0	12.6	0.0	0.0	0.0

SD21	0.0	0.0	49.9	16.9	0.0
SD22	0.0	0.0	39.5	0.0	0.0
SD23	0.0	0.0	99.9	95.4	133.1
SD24	0.0	0.0	145.0	0.0	54.1
SD25	0.0	0.0	46.8	0.0	0.0
SD26	0.0	0.0	21.0	0.0	0.0
SD27	0.0	0.0	28.8	0.0	0.0
SD28	0.0	0.0	89.1	0.0	0.0
SD29	0.0	0.0	16.1	69.1	85.0
SD30	0.0	0.0	234.1	0.0	0.0
SD31	0.0	0.0	0.0	281.4	0.0
SD32	0.0	0.0	0.0	103.0	0.0
SD33	0.0	0.0	0.0	105.1	244.9
SD34	0.0	0.0	0.0	10.8	0.0
SD35	0.0	0.0	0.0	15.1	0.0
SD36	0.0	0.0	0.0	0.0	177.6
SD37	0.0	0.0	0.0	0.0	15.1
SD38	0.0	0.0	0.0	0.0	24.8
SD39	7.8	7.8	7.8	7.8	7.8
Tailings emplacement areas	158.5	158.5	252.8	113.0	130.0
Creek systems					
Laheys Creek	10254.4	9295.7	9096.0	8799.2	8787.2
Tributary 1	738.4	738.4	824.4	821.1	739.0
Tributary 2	278.4	278.4	278.4	278.4	278.4
Tributary 3	2454.2	2454.2	2454.2	2439.2	2389.5
Sandy Creek (upstream of conf)	13901.3	14264.5	13839.2	13814.6	13808.4
Sandy Creek (downstream conf)	2951.6	2430.4	1491.9	1420.2	1402.0
Sub-total	**31689**	**31689**	**31394**	**30847**	**30280**
Rehab to Laheys Creek	0.0	0.0	294.3	617.1	1040.5
Rehab to Sandy Creek US	0.0	0.0	0.0	0.0	11.2
Rehab to Sandy Creek DS	0.0	0.0	0.0	225.2	357.2
Total rehab to creek	0.0	0.0	294.3	842.3	1408.9
Total	**31689**	**31689**	**31689**	**31689**	**31689**

The catchment area draining to the water management system increases between Years 1 and 12. The catchment area draining to the system decreases after this time, as runoff from established rehabilitated areas are returned to the creek system.

5.3.2 Dam sizes

5.3.2.1 Sedimentation dams

Sedimentation dam capacities adopted in the water balance model are summarised in Table 5-3. Capacities are based on the design criteria outlined in Section 2.3.1. It may be possible to reduce the size of individual sedimentation dams by providing the required storage volume in multiple dams. This will be investigated during detailed design.

Note that SD1, SD2 and SD3 have adopted a runoff coefficient of 0.85 to account for the hardstand surfaces in the mine infrastructure areas, whereas the other sedimentation dams capturing runoff from overburden emplacements have adopted a runoff coefficient of 0.4.

Table 5-3 Sedimentation dam capacities

Dam ID	Description	Maximum catchment area (ha)	Capacity (ML)		
			Settling zone	Sediment zone	Total
SD1	Sedimentation dam capturing runoff from infrastructure area west of run-of-mine stockpile pad	7.5	9.3	1.9	11.2
SD2	Sedimentation dam capturing runoff from mine infrastructure area	13.0	16.2	3.2	19.4
SD3	Sedimentation dam capturing runoff from mine infrastructure area	4.1	5.1	1.0	6.1
SD4a	Sedimentation dam capturing runoff from overburden emplacement	56.5	14.3	7.1	21.4
SD4b	Sedimentation dam capturing runoff from overburden emplacement	91.0	23.2	11.6	34.7
SD5	Sedimentation dam capturing runoff from overburden emplacement	77.8	21.2	10.6	31.7
SD6	Sedimentation dam capturing runoff from overburden emplacement	248.1	62.8	31.4	94.2
SD7	Sedimentation dam capturing runoff from overburden emplacement	181.7	46.0	23.0	69.0
SD8	Sedimentation dam capturing runoff from topsoil stockpile	5.6	1.4	0.7	2.1
SD9	Sedimentation dam capturing runoff from overburden emplacement	266.1	67.3	33.7	101.0
SD10	Sedimentation dam capturing runoff from overburden emplacement	158.6	40.1	20.1	60.2
SD11	Sedimentation dam capturing runoff from overburden emplacement	179.2	45.3	22.7	68.0
SD12	Sedimentation dam capturing runoff from cleared area ahead of pit	6.7	1.7	0.8	2.5
SD13	Sedimentation dam capturing runoff from cleared area ahead of pit	38.3	9.7	4.8	14.5
SD14	Sedimentation dam capturing runoff from overburden emplacement	304.5	77.1	38.6	115.7
SD15	Sedimentation dam capturing runoff from overburden emplacement	156.6	39.6	19.8	59.4
SD16	Sedimentation dam capturing runoff from overburden emplacement	109.7	27.8	13.9	41.6
SD17	Sedimentation dam capturing runoff from overburden emplacement	155.4	39.3	19.7	59.0
SD18	Sedimentation dam capturing runoff from cleared area ahead of pit	8.1	2.1	1.0	3.1
SD19	Sedimentation dam capturing runoff from cleared area ahead of pit	6.4	1.6	0.8	2.4
SD20	Sedimentation dam capturing runoff from cleared area ahead of pit	12.6	3.2	1.6	4.8
SD21	Sedimentation dam capturing runoff from overburden emplacement	49.9	13.4	6.7	20.0
SD22	Sedimentation dam capturing runoff from cleared area ahead of pit	39.5	10.0	5.0	15.0
SD23	Sedimentation dam capturing runoff from overburden emplacement	133.1	33.7	16.8	50.5
SD24	Sedimentation dam capturing runoff from overburden emplacement	145.0	68.2	34.1	102.4
SD25	Sedimentation dam capturing runoff from overburden emplacement	248.0	67.5	33.7	101.2

Dam ID	Description	Maximum catchment area (ha)	Capacity (ML)		
			Settling zone	Sediment zone	Total
SD26	Sedimentation dam capturing runoff from overburden emplacement	21.0	12.8	6.4	19.2
SD27	Sedimentation dam capturing runoff from overburden emplacement	28.8	14.3	7.2	21.5
SD28	Sedimentation dam capturing runoff from overburden emplacement	89.1	22.5	11.3	33.8
SD29	Sedimentation dam capturing runoff from overburden emplacement	85.0	21.5	10.8	32.3
SD30	Sedimentation dam capturing runoff from overburden emplacement	234.2	61.5	30.8	92.3
SD31	Sedimentation dam capturing runoff from overburden emplacement	281.4	71.2	35.6	106.8
SD32	Sedimentation dam capturing runoff from overburden emplacement	103.0	26.1	13.0	39.1
SD33	Sedimentation dam capturing runoff from overburden emplacement	303.2	86.8	43.4	130.2
SD34	Sedimentation dam capturing runoff from cleared area ahead of pit	10.8	2.7	1.4	4.1
SD35	Sedimentation dam capturing runoff from overburden emplacement	15.1	3.8	1.9	5.7
SD36	Sedimentation dam capturing runoff from overburden emplacement	177.6	67.7	33.9	101.6
SD37	Sedimentation dam capturing runoff from cleared area ahead of pit	15.1	3.8	1.9	5.7
SD38	Sedimentation dam capturing runoff from cleared area ahead of pit	24.8	6.3	3.1	9.4
SD39	Sedimentation dam capturing runoff from cleared area ahead of pit	7.8	2.0	1.0	3.0

5.3.2.2 Mine water dams

Dam capacities adopted in the water balance model are summarised in Table 5-4. Capacities are based on the design criteria outlined in Section 2.3.2. Mine water dams were initially sized to capture runoff from the 100-year ARI 72-hour design storm event. The capacity of mine water dams was then confirmed by water balance modelling, with the dam being sized to achieve no discharge when operated as part of the overall site water management system under historical climate conditions. In most cases, this involved increasing the capacity above the 100-year ARI 72-hour storm event to cater for extended wet periods.

Note that no allowance has yet been made for sediment storage, but this will be considered during detailed design. For comparison purposes, estimated runoff volumes for the 100-year ARI 24-hour and 72-hour storm events are also provided in Table 5-4.

Table 5-4 Dam capacities

Dam ID	Description	Maximum catchment area (ha)	Volume (ML)		
			Adopted capacity (from water balance modelling)	100-year ARI 24-hour runoff volume	100-year ARI 72-hour runoff volume
MWD1	Water storage dams capturing infrastructure runoff from CHPP	64.7	160.1	86.4	110.1

Dam ID	Description	Maximum catchment area (ha)	Volume (ML)		
			Adopted capacity (from water balance modelling)	100-year ARI 24-hour runoff volume	100-year ARI 72-hour runoff volume
MWD2	Water storage dams capturing infrastructure runoff from CHPP	11.7	29.9	15.6	19.9
MWD3	Mine water dam receiving water pumped from Mining Area A	0.0	500	-	-
MWD4	Central mine water dam receiving surplus water pumped from other mine water dams	0.0	375	–	–
MWD5	Mine water dam receiving water pumped from Mining Area B	0.0	500	–	–
MWD6	Staging dam for dewatering from Mining Area B (also captures runoff from stockpile pad near Mining Area B)	1.9	30	2.5	3.2
MWD7	Mine water dam receiving water pumped from Mining Area C	0.0	500	–	–
MWD8	Infrastructure water storage dam capturing runoff from stockpile pad near Mining Area C	1.0	2.6	1.3	1.7
MWD9	Mine water dam receiving water pumped from Mining Area C (replaces MWD7)	0.0	500	–	–
MWD10	Water storage dams capturing infrastructure runoff from CHPP	25.5	65.0	34.1	43.4
MWD11	Mine water dam supplying truck fill	2.3	6.0	3.1	3.9
MWD12	Receives runoff from overburden and pumps to MWD9	54.9	50	73.3	93.4
MWD13	Staging dam for dewatering from Mining Area A	0.0	30	-	-

Note: Excludes sediment storage.

Mine water dams (MWD4, MWD5, MWD7 and MWD9) will have a ‘turkeys nest’ configuration and have minimal catchment area receiving mainly pumped inflows from the pit sumps. Mine water dams have therefore been sized based on the results of historical water balance modelling to achieve a reasonable level of pit dewatering when operated as part of the site water management system over the 111-year water balance simulation.

No limit has been applied in the water balance model on the volume of in-pit sump storage. As detailed in Section 4.4, CHC has flexibility to transfer pit water between open cuts without needing to release surpluses during extended periods of wet weather.

5.3.2.3 Clean water highwall dams

Table 5-5 summarises the clean water highwall dam capacities adopted in the water balance model. Capacities are based on the design criteria outlined in Section 2.3. There may be an opportunity to provide additional highwall dams during detailed design to further reduce the catchment draining to the pits.

Table 5-5 Clean water highwall dam capacities

Dam ID	Description	Maximum catchment area (ha)	Capacity (ML)
CWD1	Dam capturing clean water ahead of SD11 overburden catchment	16.7	10.5
CWD2	Dam capturing clean water ahead of mining area B	84.8	53.3
CWD3	Dam capturing clean water ahead of mining area B	30.3	19.1
CWD4	Dam capturing clean water ahead of mining area C	39.2	24.6
CWD5	Dam capturing clean water ahead of mining area B	29.1	18.3
CWD6	Dam capturing clean water ahead of mining area B	40.2	25.2
CWD7	Dam capturing clean water ahead of SD28 overburden catchment	56.8	35.7
CWD8	Dam capturing clean water ahead of SD30 overburden catchment	48.4	30.4
CWD9	Dam capturing clean water ahead of western out of pit tailings area	8.3	5.0
CWD10	Dam capturing clean water ahead of eastern out of pit tailings area	58.3	37.0

5.3.2.4 Tailings emplacement areas

The tailings emplacement capacities and catchment areas adopted in the water balance model are summarised in Table 5-6.

Table 5-6 Tailings emplacement area characteristics modelled in water balance

Year	Emplacement area status	Emplacement area capacity when empty (ML)	Catchment area (ha)
Year 1	Active		
	- Stage 1 out-of-pit East	- 1100 ML	82.2 ha (19.5ha tailings emplacement +62.7ha undisturbed/cleared area)
	- Stage 1 out-of-pit West	- 1100 ML	76.3 ha (18.7ha tailings emplacement + 57.6ha undisturbed/cleared area)
Year 4	Active		
	- Stage 2 out-of-pit East	- 7,400 ML	82.2 ha (61.2ha tailings emplacement +21.0ha undisturbed area)
	- Stage 2 out-of-pit West	- 6,600 ML	76.3 ha (71.2ha tailings emplacement + 5.1ha undisturbed area)
Year 12	Active		
	- In-pit 2	- 8,800 ML	32.7 ha
	- In-pit 3	- 13,500 ML	29.1 ha
	Drying		
	- Stage 2 out-of-pit East		82.2 ha (61.2ha tailings emplacement +21.0ha undisturbed/rehab area)
	- Stage 2 out-of-pit West		76.3 ha (71.2ha tailings emplacement + 5.1ha undisturbed/rehab area)
	- In-pit 1		32.5 ha
Year 16	Active		
	- In-pit 3	- 13,500 ML	29.1 ha
	- In-pit 4	- 8,200 ML	18.7 ha
	Drying		
	- Stage 2 out-of-pit East		82.2 ha (61.2ha tailings emplacement +21.0ha undisturbed/rehab area)
	- Stage 2 out-of-pit West		76.3 ha (71.2ha tailings emplacement + 5.1ha undisturbed/rehab area)
	- In-pit 1		32.5 ha
	- In-pit 2		32.7ha
Year 20	Active		
	- In-pit 5	- 8,500 ML	26.6 ha
	- In-pit 6	- 4,900 ML	22.9 ha
	Drying		
	- In-pit 2		32.7 ha
	- In-pit 3		29.1 ha
	- In-pit 4		18.7 ha
	Under rehabilitation		
	- In-pit 1		32.5 ha
	Rehabilitated		
	- Stage 2 out-of-pit East		0 ha
	- Stage 2 out-of-pit West		0 ha

Definitions:
Active: In receipt of tailings and rainfall-runoff on catchment collected within site water management system.
Drying: Not in receipt of tailings and rainfall-runoff on catchment collected within site water management system.
Undergoing rehabilitation: Rainfall-runoff on catchment collected within site water management system.
Rehabilitated: Rainfall-runoff on catchment directed to creek.

5.3.2.5 Stage-storage relationships

Stage-storage relationships for dams were included in the water balance model and were estimated based on assumed side slopes of 1:3 (V:H) and assumed depths of 2.5 m for sedimentation dams and 5.0 m for mine water dams. This assumption will be refined during detailed design once the final configuration of site dams is established.

The stage-storage relationship for the proposed 1,000 ML raw water dam is summarised in Table 5-7.

Table 5-7 Stage-storage relationship for proposed raw water dam

Volume (ML)	Surface area (ha)
0	0.0
50	2.6
100	4.0
150	5.1
200	6.0
250	7.0
300	7.8
350	8.6
400	9.5
450	10.2
500	10.9
600	12.2
700	13.5
800	14.7
900	15.9
1000	16.9

5.3.3 Pump rates

The following pump rates were adopted in the water balance model:

- pit sump to mine water dam MWD3, MWD5, MWD7/9 (via staging dams):
  - 8.6 ML/d (100 L/s) each if current sump volume between 0 ML and 100 ML
  - 13.0 ML/d (150 L/s) each if current sump volume between 100 ML and 200 ML
  - 21.6 ML/d (250 L/s) each if current sump volume greater than 200 ML
- mine water dam MWD1 to MWD4 — 20 ML/d
- mine water dam MWD10 to MWD4 — 10 ML/d
- mine water dams MWD3, MWD5 and MWD7/9 to MWD4 — 15 ML/d each
- low flow outlets of sedimentation dams to creek — sized to treat and empty ‘settling zone’ over five days (various sizes)
- sedimentation dams to mine water dams (for on-site reuse) — sized to empty ‘settling zone’ over five days (various sizes).

- tailings emplacement decant pond to MWD3 — 13.0 ML/d.

It has been assumed that pumping from the Macquarie and Cudgegong Regulated River Water Source occurs at a rate of 20 ML/d. Note that the river pump is capable of pumping 24 ML/d, however, this rate has not been modelled in the water balance as it does not allow for resting of the pumps and could not be maintained on a long term basis.

5.3.4 Operating rules

Operating rules would be subject to ongoing development and refinement. The following operating rules have been assumed in the water balance model:

- The pit sumps are dewatered to mine dewatering dams MWD3, MWD5 and MWD7/9 as follows:
  - Mining Area A dewatered to MWD3 (via staging dam MWD13).
  - Mining Area B dewatered to MWD5 (via staging dam MWD6).
  - Mining Area C dewatered to MWD7/9.
- Pumping from pit sumps stops if the corresponding mine dewatering dam capacity exceeds 90%. During extended wet periods, surplus mine water will be stored in the mine pits once dewatering dams have reached their capacity.
- MWD1 pumps to MWD4 when its capacity exceeds 0% in order to draw down this dam following a storm event. Pumping from MWD1 to MWD4 stops if MWD4 exceeds 92.5% capacity.
- MWD2 pumps to MWD1 when its capacity exceeds 0% in order to draw down this dam following a storm event. Overflows from MWD2 discharge to MWD1.
- MWD10 pumps to MWD4 when its capacity exceeds 0% in order to draw down this dam following a storm event. Pumping from MWD10 to MWD4 stops if MWD4 exceeds 92.5% capacity.
- MWD3, MWD5 and MWD7/9 all pump to MWD4. When MWD4 exceeds 87.5% capacity, pumping to MWD4 is only allowed from one dewatering dam at a time, and this dam is selected as the dam with the highest corresponding pit water volume (for example, if Pit B has the highest pit stored water volume on that day, then pumping from MWD5 to MWD4 would be allowed on that day to promote dewatering of this pit as a priority). When MWD4 is less than 50% capacity, pumping to MWD4 is allowed from more than one dewatering dam at the same time, if required. Note that the trigger level to stop pumping from MWD1, MWD2 and MWD10 to MWD4 is set higher than 87.5% in order to promote the reuse of water from dams MWD1, MWD2 and MWD10 as a priority over pit water.
- Pumping out from MIA sedimentation dams SD1, SD2 and SD3 occurs when the water level reaches the ‘settling zone’ in order to draw down these dams following a storm event. SD1, SD2 and SD3 pump to mine water dam MWD3. Pumping from SD1, SD2 and SD3 stops if MWD3 exceeds 92.5% capacity.
- Pumping out from overburden sedimentation dams occurs when the water level reaches the ‘settling zone’ in order to draw down the ‘settling zone’ following a storm event. Water is pumped to the nearest mine water dam (MWD3 / MWD4 / MWD5 / MWD7/9) if the volume stored in that mine water dam and MWD4 is less than 25% capacity (MWD4 must also be at less than 25% capacity in order to ensure that mine water is utilised as

a priority over overburden sedimentation dam water). Otherwise, clarified water is released into the creek system.

- The 'sediment zone' of sedimentation dams is 50% full of sediment throughout the simulation. The model assumes that water stored above the assumed sediment level in the 'sediment zone' evaporates over time, and is not released to the creek or reused onsite.
- Where selected overburden sedimentation dams (SD6, SD7 and SD10) have been provided with an additional 'reuse zone', water can be stored in the 'reuse zone' on a long term basis for reuse onsite as and when required. However, water cannot be stored in the 'settling zone' on a long term basis, as the 'settling zone' must be maintained in a drawn down state as much as practical to cater for subsequent design storm events. Following a rainfall event, overburden sedimentation dams should be drawn down within the nominated 5 day management period.
- Sedimentation dam overflows are included in the model. Overflows from sedimentation dams are typically displaced to the creek (although some overflow to other sedimentation dams, or are located at low points and do not overflow).
- A portion of the CHPP make-up water demand is always sourced from the raw water dam RWD1, as high-quality water is required. The remaining CHPP make-up water demand is sourced from the following dams (in order of priority):
  - MWD4.
  - RWD1.
- MIA demands are sourced from the following dams (in order of priority):
  - MWD3.
  - MWD5 (via MWD3).
  - RWD1.
- Haul road dust suppression demands are sourced from two truckfill stations. The truckfill stations source water from the following dams (in order of priority):
- Truckfill No. 1 (located near the CHPP).
  - MWD4 (via MWD11).
  - MWD5 (via MWD4).
  - RWD1.
- Truckfill No. 2 (located near the MIA).
  - MWD3.
  - MWD5 (via MWD3).
  - RWD1.
- Potable water demands are always sourced from RWD1, as high-quality water is required.

- RWD1 is maintained at 50% capacity (unless water is not available from the river to 'top-up' the raw water dam). When the raw water dam falls below 50% capacity, imported water is pumped into the dam from the Macquarie and Cudgegong Regulated River Water Source at a rate of 20 ML/d, up to a maximum of 3,311 ML/a. Note that for the 'base scenario' it was assumed that up to 3,311 ML/a would be available from the Cudgegong River and no allowance was made for possible pumping restrictions due to low flows in the Cudgegong River. A sensitivity analysis was then undertaken to assess the impact of possible river water restrictions.

- The tailings decant pond pumps to MWD3 when its capacity exceeds 0% in order to draw down this pond following a storm event. Pumping from the tailings decant pond to MWD3 stops when MWD3 exceeds 92.5% capacity. During extended wet periods, surplus mine water will be stored in the tailings decant pond once MWD3 exceeds 92.5% capacity. Note that the trigger level to stop pumping from the pit to MWD3 is set lower that 92.5% in order to promote the reuse of tailings decant water as a priority over pit water.

5.4 Water inputs

Water inputs for the Project comprise:

- surface water runoff
- groundwater seepage to the open pit
- imported water.

5.4.1 Surface water runoff

The AWBM rainfall-runoff model (using the Data Drill daily rainfall and evapotranspiration data) was incorporated into the GoldSim model to generate a daily time series of runoff from undisturbed, disturbed and rehabilitated catchments. The AWBM rainfall-runoff model and parameters are described in Section 3.5.1.

5.4.2 Groundwater seepage

Groundwater seepage rate estimates for the open pits are provided in the Project's groundwater assessment report (Parsons Brinckerhoff 2013), and are summarised in Table 5-8**Error! Reference source not found.** and Figure 5.1**Error! Reference source not found.**.

Table 5-8 Groundwater seepage rate estimates

Year	Seepage rate (ML/a)			
	Mining area A	Mining area B	Mining area C	Total
1	23	107	0	130
2	192	329	23	544
3	298	465	61	824
4	368	600	101	1,069
5	331	591	108	1,030
6	446	767	183	1,396
7	875	1,001	231	2,107

Year	Seepage rate (ML/a)			
	Mining area A	Mining area B	Mining area C	Total
8	733	1,325	381	2,439
9	912	1,067	357	2,336
10	884	1,202	369	2,455
11	950	1,196	369	2,515
12	453	1,444	550	2,447
13	253	1,361	530	2,144
14	592	1,614	596	2,802
15	646	1,517	527	2,690
16	637	1,237	529	2,403
17	801	821	403	2,025
18	1227	803	52	2,082
19	278	633	33	944
20	194	631	337	1,162
21	0	31	0	31
Total	**11,093**	**18,742**	**5,740**	**35,575**

Source: Parsons Brinckerhoff 2013

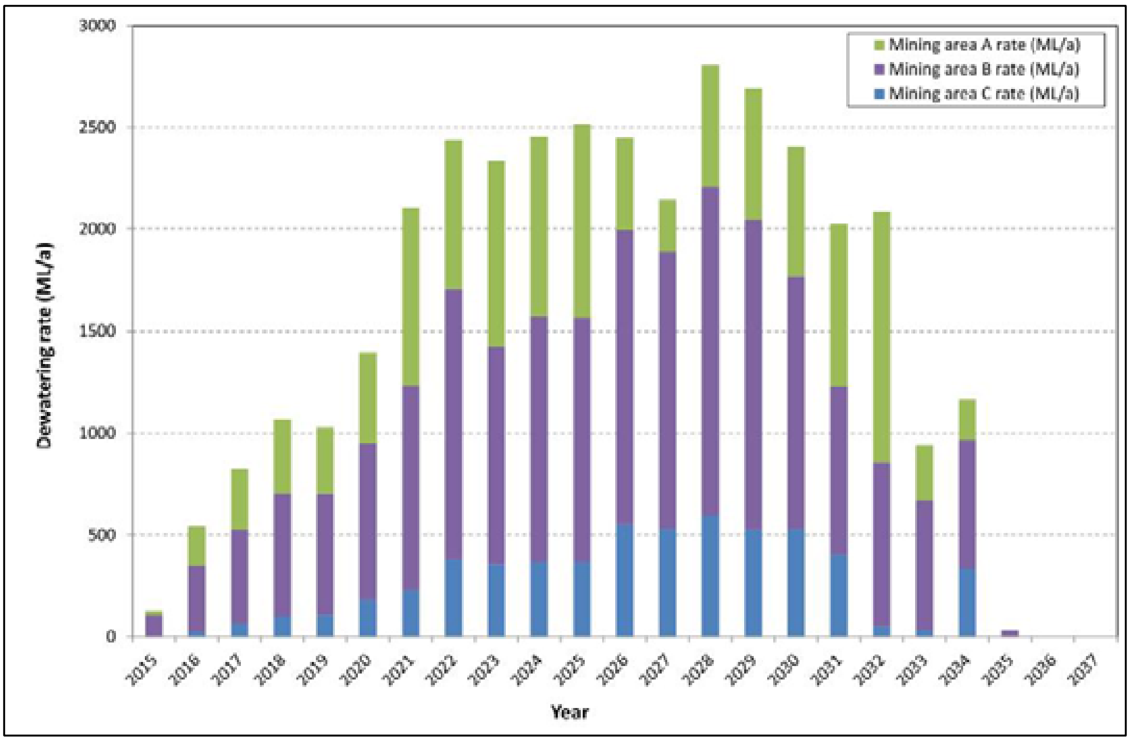


Figure 5.1 Plot of groundwater seepage estimates

From Table 5-8**Error! Reference source not found.** and Figure 5.1**Error! Reference source not found.** it can be seen that the peak groundwater seepage rate is expected to occur in Year 14 (year 2028) of the project, with a total seepage rate of 2,802 ML/a to mining areas A, B and C. Of the Year 1, 4, 12, 16 and 20 'snapshot' landforms modelled in the water balance, the peak groundwater seepage rate is 2,447 ML/a in Year 12, which is 355 ML/a less than the Year 15 peak of 2,802 ML/a.

5.4.3 Imported water

Raw water will be imported to the mine site to meet demands during a water deficit, and also to provide a high-quality water source (e.g. for potable applications and CHPP raw water applications). Imported water will be stored in the raw water dam. CHC currently holds 3,311 unit shares of high-security WALs from the Macquarie and Cudgegong Regulated River Water Source. The Cudgegong River Framework for Extraction Strategy Agreement between CHC and State Water allows CHC to extract river water in accordance with its Works Approval. The following conditions apply:

- Extraction not permitted when flow <25 ML/d at downstream Yamble Bridge NOW gauging station.
- Maximum pump rate 24 ML/d.
- Access not permitted to water comprising bulk transfer from Windamere Dam to Burrendong Dam.
- CHC will submit water orders to State Water Corporation as required by Water Supply Works Approval (which requires not less than 4 days notice).

The Cudgegong River at Yamble Bridge adjusted flow record has been provided by State Water, and is provided in the plot in Figure 5.2. It is understood that State Water has made the following adjustments to the flow record:

- Daily flow records include a 'surplus' component which is the result of incomplete extraction of water orders by irrigators sourced from Windamere Dam.
- Daily flow adjusted when appropriate to remove bulk transfer flow from Windamere Dam to Burrendong Dam.
- State Water deemed that pre-1995 flow record not representative of current regulated river management (i.e. catchment-wide irrigation developments in infancy, pre-dates presence of 'surplus' flows); as such the adjusted record is for period 1995 to 2011.

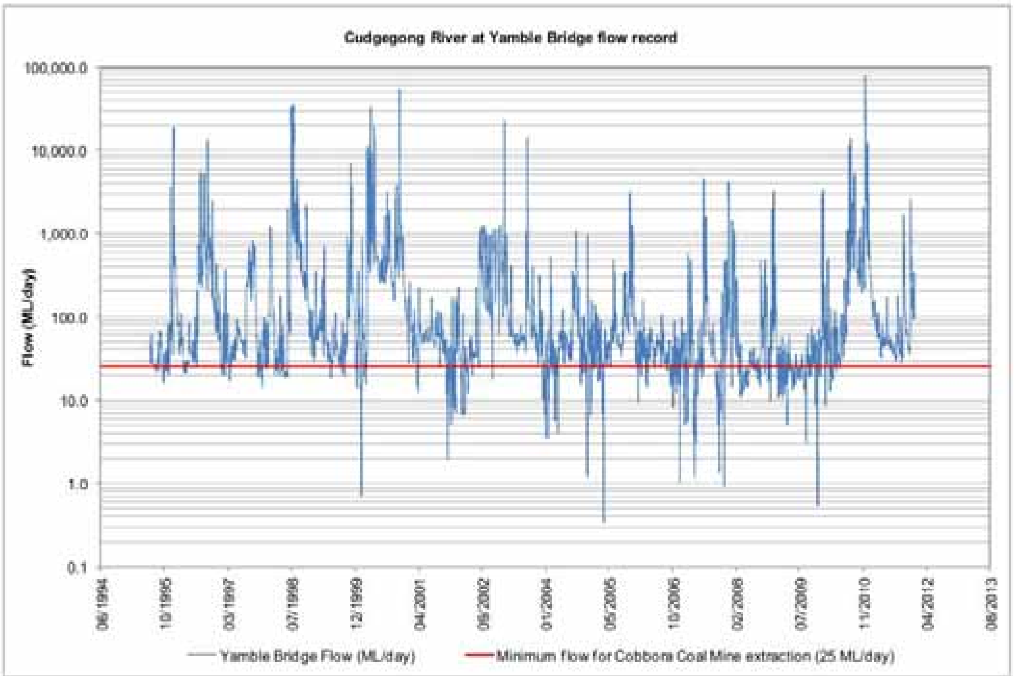


Figure 5.2 Cudgegong River at Yamble Bridge streamflow record

An analysis of the Cudgegong River at Yamble Bridge adjusted flow record from 1995 to 2011 is provided in Table 5-9. The analysis shows that, on average, 85% of days in the year have streamflows greater than or equal to 25 ML/d at Yamble Bridge. The worst case year on record, in terms of prolonged low flows, occurred in 2009 when there were only 186 days with streamflows greater than or equal to 25 ML/d at Yamble Bridge.

Table 5-9 Analysis of Cudgegong River at Yamble Bridge streamflows from 1995 to 2011

Year	**Number of days in month when streamflow ≥ 25ML/d**												**Total**
	Jul	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	
1995	30	17	27	17	18	30							**N/A**
1996	31	31	30	31	30	31	31	29	23	14	31	30	**342**
1997	31	31	30	30	17	23	21	23	18	30	31	30	**315**
1998	31	31	30	31	30	31	30	28	23	25	23	8	**321**
1999	31	25	27	31	30	27	31	28	31	30	29	30	**350**
2000	31	31	30	31	30	31	19	24	29	30	31	30	**347**
2001	31	31	30	31	25	10	31	28	31	19	31	30	**328**
2002	31	31	30	31	28	31	18	24	19	25	13	28	**309**
2003	31	31	30	31	30	23	31	28	31	30	31	30	**357**
2004	31	31	30	28	20	22	13	16	22	20	31	30	**294**
2005	31	31	30	31	30	31	24	28	24	15	31	30	**336**
2006	30	27	20	16	26	23	21	27	23	30	30	30	**303**
2007	31	31	14	8	18	31	18	20	22	5	26	27	**251**
2008	19	10	30	28	15	31	25	29	20	2	0	23	**232**
2009	10	3	26	28	18	26	23	10	10	21	0	11	**186**
2010	31	31	30	31	30	31	22	28	10	24	30	30	**328**
2011	31	31	30	31	30	23	31	28	31	30	31	30	**357**
Average	**29**	**27**	**28**	**27**	**25**	**27**	**24**	**25**	**23**	**22**	**25**	**27**	**310**
Average as % of days in month / year	**93%**	**86%**	**93%**	**88%**	**83%**	**86%**	**78%**	**88%**	**74%**	**73%**	**80%**	**89%**	**85%**

For the purposes of water balance modelling, the main modelling scenarios assumed that up to 3,311 ML/a would be available from the Cudgegong River, and no allowance was made for possible pumping restrictions on days when there is less than 25 ML/d flow in the Cudgegong River at Yamble Bridge. A sensitivity analysis was then undertaken to assess the impact of possible water restrictions on water security at the mine site.

5.5 Water demands

Demands for the Project comprise:

- CHPP make-up water
- mine infrastructure areas, such as workshop and vehicle washdown areas
- haul road dust suppression
- potable water
- miscellaneous uses, such as construction water.

5.5.1 Construction water

Potable water would also be required for the construction phase but this was not included in the water balance model. CHC estimated that a peak of 10 ML/a will be required for the general construction workforce (based on a conservative 100 L/d per person and 550-person workforce peak occurring in September to October 2014), and 36.5 ML/a will be required for the construction accommodation village (based on upper limit of 250 L/d per person and 400-bed village).

This is a combined peak potable water demand of 46.5 ML/a for the construction phase. Note that the demand for the construction accommodation village of 250 L/d per person has been allowed for as an upper limit, but is more likely to be in the order of 100 L/d per person.

Potable water would most likely be trucked to site from the Wellington or Warrumbungle Shire Council supply during the initial construction phase. It is proposed to use raw water pumped to site from the Macquarie and Cudgegong Regulated River Water Source for potable applications after the first six months of construction, assuming that it is treated to potable standard prior to use.

5.5.2 Coal-handling and preparation plant

Coal production rates and CHPP demand estimates have been provided by QCC Resources Pty. Limited based on experience with similar project designs and are summarised in Table 5-10.

The CHPP will utilise a mix of raw water and mine water. The breakdown of the minimum raw water and mine water component is provided in Table 5-10.

Table 5-10 CHPP make-up water demand estimates

Year	Product coal (Mt/a)	Raw water demand (minimum) (ML/a)	Mine water demand (ML/a)	Tailings return (%)	Tailings return (ML/a)	Mine water demand net of tailings return (ML/a)	Total CHPP make-up water demand (ML/a)
1	0.7	33	134	30	33	101	134
4	11.2	431	2188	30	527	1661	2092
12	12.0	462	2345	15	282	2062	2524
16	12.0	462	2345	15	282	2062	2524
20	12.0	462	2345	15	282	2062	2524

5.5.3 Mine infrastructure area

Water will be required in the mine infrastructure area for vehicle washing and workshop use, as well as dust suppression for the coal stockpiles and train loading facility.
Mine infrastructure area demands were provided by CHC and are summarised in Table 5-11.

Table 5-11 Mine infrastructure area demand estimates

Year	Mine infrastructure area demand (ML/a)
1	9
4	140
12	150
14	150
20	150

Water for use in the mine infrastructure area will be sourced from raw water.

5.5.4 Haul road dust suppression

Mine water will be used for dust suppression on haul roads. Dust-suppression demand estimates are based on a method provided by Environ Australia Pty. Limited and are summarised in Table 5-12.

Table 5-12 Haul road dust-suppression demand estimates

Year	Haul road dust suppression (ML/a)
1	376
4	968
12	1,651
16	1,603
20	1,371

Dust suppression water demand is based on dust emission control efficiency obtained by cyclic wet suppression. The efficiency afforded by the application of water, requires periodic reapplication to maintain the desired average efficiency (Cowherd et al., 1988). Guidance from reference literature was sought to establish the control efficiency of this measure. Control efficiencies given for dust suppression by watering were found to vary significantly, reflecting variations in the effectiveness of watering strategies given site-specific practices and climatic factors. The NPI EETM for Mining (2012), by example, defines control efficiencies for two levels of watering as follows:

- 50% dust control efficiency for level 1 watering (2 litres/m²/hour)
- 75% dust control efficiency for level 2 watering (>2 litres/m²/hour)

The derivation of these control efficiencies, and their applicable circumstances (frequency of traffic passes, climatic conditions etc.) are not documented in the manual.

The control efficiency likely to be achieved at the Project was estimated taking into account site specific conditions including the number of truck passes, rainfall and evaporation rates, and the frequency and intensity of watering. For this purpose a model was derived, based on the empirical model of Cowherd et al. (1988), for the estimation of the average control efficiency of watering.

$$c = 100 - \frac{0.8\, p\, d\, t}{i}$$

where,

c = average control efficiency (%)

d = average hourly daytime traffic rate (hr-1); estimated range of 68 trucks/hr (Year 1) to 90 trucks/hr (Year 16) on haul roads

i = application intensity (L/m^2), calculated range of 0.68 L/m^2 (Year 1) to 0.91 L/m^2 (Year 16)

t = time between applications (hr); one water cart pass per hour accords with the application intensity calculated (i.e. t = 1)

p = potential average hourly daytime moisture deficit (mm/hr), derived from the average evaporation rate minus the average rainfall rate to be 0.31 mm/hr

The potential average hourly moisture deficit was derived based on daily data sourced from the Data Drill database, developed by DERM. The long-term average evaporation rate is 1,735 mm/year, with the long-term average rainfall rate being 625 mm/year. A 10-hour "daytime" was assumed.

The annual average control efficiency was estimated to be 75%. For emission estimation purposes reference was therefore made to the control efficiency published in the NPI EETM for Mining (2012) given as 75%.

To account for dust emission control provided by rainfall, no application of water was assumed to occur on rain days defined by at least 1 mm of rainfall. The long-term average is 66 rain days.

Water for dust suppression on haul roads will be sourced from mine water as a priority (via truck fill stations). It is assumed that such water will be of suitable quality for this purpose.

5.5.5 Potable water

Potable water will be required in the administration and amenities buildings during mine operation. CHC has estimated that up to 100 L/d per person on shift would be required. A summary of the potable water demand estimates for the mine operation is provided in Table 5-13.

Table 5-13 Potable water demand estimates

Year	Potable water demand (ML/a)
1	5
4	10
12	15
16	15
20	10

The source of potable water will be determined during detailed design. Given the small potable water demand relative to the total site demand during the operational phase, it is not expected that potable water will have a significant impact on the site water balance.

Wastewater from on-site facilities such as workshops, process and administration buildings will be managed by an on-site sewage treatment system. Treated effluent will be returned to the mine water system for onsite reuse but has not been specifically included in the water balance model, as volumes are not expected to be significant when compared to other inputs to the system.

5.5.6 Demand summary

A summary of water demands for the Project is provided in Table 5-14. Of the 'snapshot' years assessed, the peak demand is estimated to be 4,340 ML/a occurring in Year 12 of the Project.

Table 5-14 Water demand summary

Year	Product coal (Mt/a)	CHPP make-up water (ML/a)	Mine infrastructure area demand (ML/a)	Haul road dust suppression (ML/a)	Potable water demand (ML/a)	Total site demand (ML/a)
1	0.7	134	9	376	5	524
4	11.2	2092	140	968	10	3,210
12	12.0	2524	150	1,651	15	4,340
16	12.0	2524	150	1,603	15	4,292
20	12.0	2524	150	1,371	10	4,055

5.6 Other losses

5.6.1 Evaporation

Evaporation estimates for open water bodies were based on Data Drill sourced daily Morton's Lake evaporation data. The Data Drill calculates Morton's Lake evaporation using Morton's formula for shallow lakes (Morton 1983). Evaporative surface area for dams has been determined based on the stage-storage relationships provided in Section 5.3.2.

5.6.2 Seepage from dams

Some water will be lost from dams as a result of seepage through the foundation. Site dams should have low seepage losses and, depending on the subsoils, an engineered liner may be required.

Water balance modelling has assumed seepage losses to be negligible. This assumption is in line with the Manual for Assessing Hazard Categories and Hydraulic Performance of Dams (DERM, 2012) that requires that the assessment of system performance risks to the environment (particularly overflows from dams and adequacy of storage capacity), must assume that there are no seepage losses from dams. This assumption is intended to be conservative from the perspective of containment performance but may not be conservative for other outcomes of operational simulation modelling (such as water supply reliability).

6. Results of water balance assessments

This section summarises key results related to the site water balance. The results discussed here were obtained from the water balancing modelling as discussed in Section 1.

6.1 Modelling results for proposed operating scenario

The site water balances for the proposed operating scenario were obtained by assuming that runoff captured in sedimentation dams will be reused on-site to assist in meeting water demands for the Project. Water from sedimentation dams is assumed to be pumped to mine water dams for reuse on-site when these dams have sufficient spare capacity to accept additional flows.

Capacity is assumed sufficient if the stored volume is less than 25% capacity, to maintain spare capacity for pit dewatering in the event of extended wet periods. If mine water dams do not have spare capacity, then sedimentation dam water would be released to the creek system, after water quality criteria are met.

Annual summaries of water balances for the 10th (calendar year 1967), the 50th (calendar year 1906) and the 90th (calendar year 1990) percentile rainfall years were summarised from the daily water balance results. These are presented in Table 6-1 to Table 6-3. These results are representative of typical dry, average and wet years. Note that the apparent imbalance in the results tables is a result of on-site storage.

The results of the water balances are discussed in Sections 6.1.1 to 0.

Table 6-1 Annual site water balance — 10th percentile dry year

	Units	Pre- mining	Year 1	Year 4	Year 12	Year 16	Year 20	Post-mining
Catchment breakdown								
Water management system (WMS)								
▪ Raw water dam	Ha	0	51	51	51	51	51	0
▪ Clean water/highwall dams	Ha	0	83	182	280	93	67	0
▪ Sedimentation dams	Ha	0	662	1,148	1,633	1,539	1,234	0
▪ Mine water dams and pits	Ha	0	156	689	1,194	1,479	1,395	384
▪ Refuse ponds	Ha	0	158	158	253	113	130	0
Existing west 'Woolandra' farm dams	Ha	1,034	0	0	0	0	0	–
Undisturbed	Ha	30,655	30,578	29,462	27,984	27,573	27,405	27,555
Established rehabilitation returned directly to creek	Ha	–	0	0	294	842	1,409	3,750
Total study catchment	Ha	***31,689***	***31,689***	***31,689***	***31,689***	***31,689***	***31,689***	***31,689***
Proportion of study catchment in WMS	%	-	3.5%	7.0%	10.8%	10.3%	9.1%	1.2%
Inflows								
Runoff								
Water management system								
▪ Raw water dam	ML/a	–	1	1	1	1	1	–
▪ Sedimentation dams	ML/a	–	139	242	288	295	233	–
▪ Mine water dams and pits	ML/a	–	195	506	862	725	722	–
▪ Clean water/highwall dams	ML/a	–	2	3	5	2	1	–
Total WMS runoff	ML/a	**–**	***337***	***753***	***1,157***	***1,023***	***957***	**–**
Undisturbed and established rehabilitation returned directly to creek	ML/a	575	576	555	532	537	547	642
Existing west 'Woolandra' farm dams	ML/a	19	-	-	-	-	-	–
Groundwater seepage into pit	ML/a	–	131	1,069	2,446	2,403	1,163	–
Imported river water	ML/a	–	120	1,820	2,600	2,520	3,240	–
Sedimentation dam water reused on-site	ML/a	–	26	119	144	161	125	–
Outflows								
WMS dam evaporation (net of direct rain)	ML/a	–	313	480	492	474	317	–
CHPP make-up demand	ML/a	–	134	2,092	2,524	2,524	2,524	–
Haul road dust-suppression demand	ML/a	–	376	968	1,651	1,603	1,371	–
Mine infrastructure area demand	ML/a	–	9	140	150	150	150	–
Potable water demand	ML/a	–	5	10	15	15	10	–
Total WMS demand	ML/a	**–**	***524***	***3,210***	***4,340***	***4,292***	***4,055***	**–**
Sedimentation dam overflows to creek	ML/a	–	0	0	0	0	0	–
Sedimentation dam controlled releases to creek	ML/a	–	41	1	0	1	0	–
Clean water/highwall dam controlled releases to creek	ML/a	–	1	3	5	1	0	–
Raw water dam overflows to creek	ML/a	–	0	0	0	0	0	–
Existing west 'Woolandra' farm dams overflows	ML/a	0	–	–	–	–	–	–
Total flow at study catchment outlet	ML/a	**575**	**618**	**559**	**538**	**540**	**548**	**642**

Notes: Excludes runoff to refuse disposal ponds catchments

Table 6-2 Annual site water balance — 50th percentile median year

	Units	Pre-mining	Year 1	Year 4	Year 12	Year 16	Year 20	Post-mining
Catchment breakdown								
Water management system (WMS)								
▪ Raw water dam	Ha	0	51	51	51	51	51	0
▪ Clean water/highwall dams	Ha	0	83	182	280	93	67	0
▪ Sedimentation dams	Ha	0	662	1,148	1,633	1,539	1,234	0
▪ Mine water dams and pits	Ha	0	156	689	1,194	1,479	1,395	384
▪ Refuse ponds	Ha	0	158	158	253	113	130	0
Existing west 'Woolandra' farm dams	Ha	1,034	0	0	0	0	0	0
Undisturbed	Ha	30,655	30,578	29,462	27,984	27,573	27,405	27,555
Established rehabilitation returned directly to creek	Ha	0	0	0	294	842	1,409	3,750
Total study catchment	Ha	***31,689***	***31,689***	***31,689***	***31,689***	***31,689***	***31,689***	***31,689***
Proportion of study catchment in WMS	%	–	3.5%	7.0%	10.8%	10.3%	9.1%	1.2%
Inflows								
Runoff								
Water management system								
▪ Raw water dam	ML/a	–	3	3	3	3	3	–
▪ Sedimentation dams	ML/a	–	279	505	611	616	491	–
▪ Mine water dams and pits	ML/a	–	348	917	1,579	1,356	1,355	–
▪ Clean water/highwall dams	ML/a	–	6	11	17	6	4	–
Total WMS runoff	ML/a	**–**	***636***	***1,436***	***2,210***	***1,980***	***1,853***	**–**
Undisturbed and established rehabilitation returned directly to creek	ML/a	1,852	1,855	1,787	1,707	1,713	1,732	1,933
Existing west 'Woolandra' farm dams	ML/a	63						–
Groundwater seepage into pit	ML/a	–	131	1,069	2,446	2,403	1,163	–
Imported river water	ML/a	–	160	1,300	1,840	1,780	2,540	–
Sedimentation dam water reused on-site	ML/a	–	96	153	137	150	179	–
Outflows								
WMS dam evaporation (net of direct rain)	ML/a	–	85	221	230	215	110	–
CHPP make-up demand	ML/a	–	134	2,092	2,524	2,524	2,524	–
Haul road dust-suppression demand	ML/a	–	376	968	1,651	1,603	1,371	–
Mine infrastructure area demand	ML/a	–	9	140	150	150	150	–
Potable water demand	ML/a	–	5	10	15	15	10	–
Total WMS demand	ML/a	**–**	***524***	***3,210***	***4,340***	***4,292***	***4,055***	**–**
Sedimentation dam overflows to creek	ML/a	–	0	0	0	0	0	–
Sedimentation dam controlled releases to creek	ML/a	–	103	217	318	325	194	–
Clean water/highwall dam controlled releases to creek	ML/a	–	2	10	17	3	0	–
Raw water dam overflows to creek	ML/a	–	0	0	0	0	0	–
Existing west 'Woolandra' farm dams overflows	ML/a	0	0	0	0	0	0	–
Total flow at study catchment outlet	ML/a	**1,852**	**1,960**	**2,014**	**2,046**	**2,043**	**1,930**	**1,933**

Notes: Excludes runoff to refuse disposal ponds catchments

Table 6-3 Annual site water balance — 90th percentile wet year

	Units	Pre- mining	Year 1	Year 4	Year 12	Year 16	Year 20	Post-mining
Catchment breakdown								
Water management system (WMS)								
▪ Raw water dam	Ha	0	51	51	51	51	51	0
▪ Clean water/highwall dams	Ha	0	83	182	280	93	67	0
▪ Sedimentation dams	Ha	0	662	1,148	1,633	1,539	1,234	0
▪ Mine water dams and pits	Ha	0	156	689	1,194	1,479	1,395	384
▪ Refuse ponds	Ha	0	158	158	253	113	130	0
Existing west 'Woolandra' farm dams	Ha	1,034	0	0	0	0	0	–
Undisturbed	Ha	30,655	30,578	29,462	27,984	27,573	27,405	27,555
Established rehabilitation returned directly to creek	Ha	–	0	0	294	842	1,409	3,750
Total study catchment	Ha	***31,689***	***31,689***	***31,689***	***31,689***	***31,689***	***31,689***	***31,689***
Proportion of study catchment in WMS	%	–	3.5%	7.0%	10.8%	10.3%	9.1%	1.2%
Inflows								
Runoff								
Water management system								
▪ Raw water dam	ML/a	–	44	44	44	44	44	–
▪ Sedimentation dams	ML/a	–	1,114	2,220	3,013	2,912	2,313	–
▪ Mine water dams and pits	ML/a	–	784	2,245	4,056	3,929	3,954	–
▪ Clean water/highwall dams	ML/a	–	71	152	234	78	57	–
Total WMS runoff	ML/a	**–**	***2,015***	***4,662***	***7,347***	***6,964***	***6,368***	**–**
Undisturbed and established rehabilitation returned directly to creek	ML/a	26,088	26,123	25,168	24,197	24,617	25,230	28,830
Existing west 'Woolandra' farm dams	ML/a	881						–
Groundwater seepage into pit	ML/a	–	131	1,069	2,446	2,403	1,163	–
Imported river water	ML/a	–	0	360	380	400	400	–
Sedimentation dam water reused on-site	ML/a	–	66	103	103	103	134	–
Outflows								
WMS dam evaporation (net of direct rain)	ML/a	–	259	184	475	459	219	–
CHPP make-up demand	ML/a	–	134	2,092	2,524	2,524	2,524	–
Haul road dust-suppression demand	ML/a	–	376	968	1,651	1,603	1,371	–
Mine infrastructure area demand	ML/a	–	9	140	150	150	150	–
Potable water demand	ML/a	–	5	10	15	15	10	–
Total WMS demand	ML/a	**–**	***524***	***3,210***	***4,340***	***4,292***	***4,055***	**–**
Sedimentation dam overflows to creek	ML/a	–	225	371	489	565	369	–
Sedimentation dam controlled releases to creek	ML/a	–	870	1706	2377	2208	1775	–
Clean water/highwall dam controlled releases to creek	ML/a	–	16	104	191	24	0	–
Raw water dam overflows to creek	ML/a	–	0	0	0	0	0	–
Existing west 'Woolandra' farm dams overflows	ML/a	819						–
Total flow at study catchment outlet	ML/a	**26,906**	**27,241**	**27,355**	**27,301**	**27,462**	**27,439**	**28,830**

Notes: Excludes runoff to refuse disposal ponds catchments

6.1.1 Adequacy of CHC's existing water entitlements

The volume of imported water required depends on the rainfall that occurs during the mining period. The estimated imported water requirements for typical dry (10th percentile), median (50th percentile) and wet (90th percentile) rainfall years are summarised in Table 6-4.

Table 6-4 Summary of imported water requirement for dry, median and wet years

Year	Total site demand (ML/a)	Groundwater seepage (ML/a)	Imported water for a dry year (ML/a)	Imported water for a median year (ML/a)	Imported water for a wet year (ML/a)
1	524	131	120	160	0
4	3,210	1,069	1,820	1,300	360
12	4,340	2,446	2,600	1,840	380
16	4,292	2,403	2,520	1,780	400
20	4,055	1,163	3,240	2,540	400

The peak annual requirement for imported water is expected to occur in Year 20 when the groundwater seepage to the pits remains relatively high at 1,163 ML/a and the site demand remains close to the peak at 4,055 ML/a.

The Project is estimated to require 3,240 ML/a of imported water under a typical 10th percentile dry year (i.e. 1967) and 400 ML/a under a typical 90th percentile wet year (i.e. 1990) during Year 20. The estimated requirement for a dry year in Years 1, 4, 12, 16 and 20 can be met by the current entitlement of 3,311 ML/a from the Macquarie and Cudgegong Regulated River Water Source, assuming 100% allocation.

Figure 6-1 presents a sequence of the simulated annual requirement for imported water for Year 20. This figure shows that the requirement for imported water peaks at 3,240 ML/a, and does not exceed the entitlement of 3,311 ML/a.

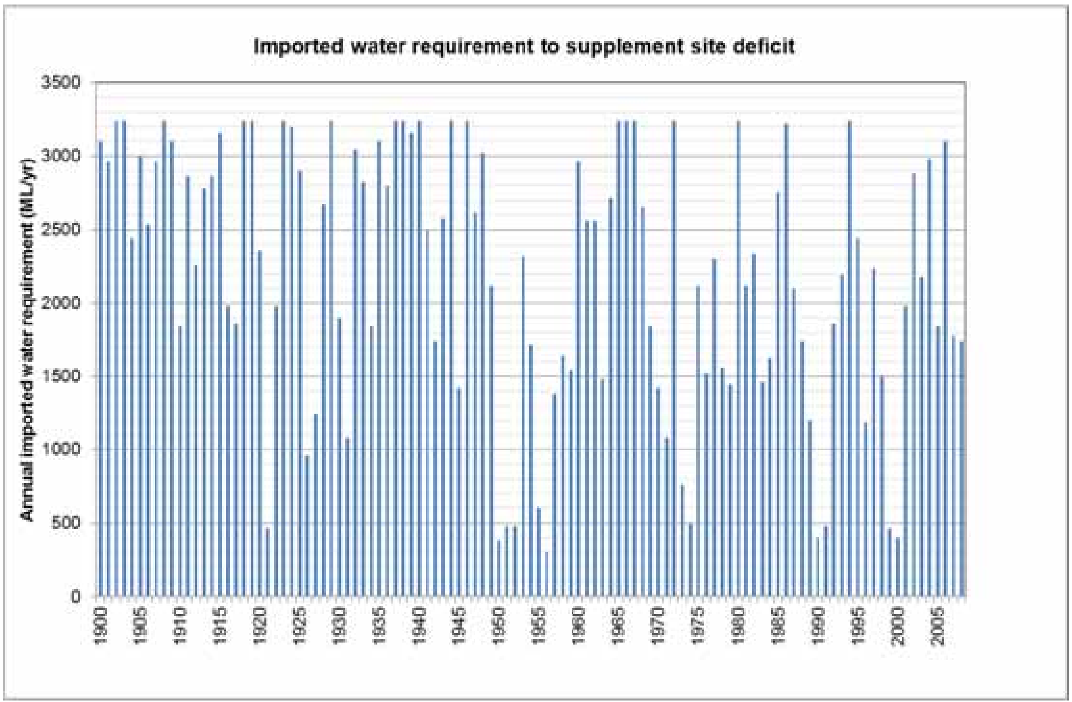


Figure 6.1 Annual imported water requirement over the 111-year water balance simulation for Year 20

The main modelling analysis assumed 100% allocation of high security water entitlements. However, in reality, pumping from the Cudgegong River may be restricted because of low flows in the river and a sensitivity analysis has been undertaken to assess the impact of possible river water restrictions on water security at the mine site.

The Cudgegong River Framework for Extraction Strategy Agreement currently subject to formal agreement between CHC and State Water Corporation allows CHC to extract river water in accordance with its Works Approval. The Agreement does not allow extraction when flow is less than 25 ML/d at downstream Yamble Bridge NOW gauging station.

An analysis of the Cudgegong River at Yamble Bridge adjusted flow record from 1995 to 2011 is provided in Section 5.4.3. The analysis indicated that on average 85% of days in a year have streamflows greater than or equal to 25 ML/d at Yamble Bridge. The worst case year in terms of low river flows, occurred in 2009 when only 51% of days had streamflows greater than or equal to 25 ML/d at Yamble Bridge.

In the absence of a historical streamflow record for the Cudgegong River at Yamble Bridge extending from 1889 to 2012, the worst case year of 2009 of the available record from 1995 to 2011 was adopted for the sensitivity analysis. The 2009 streamflow record was repeated to develop a 111 year 'worst case' synthetic streamflow sequence at Yamble Bridge. The water balance model was then simulated assuming that pumping from the Cudgegong River could only occur on days when streamflow at Yamble Bridge exceeded 25 ML/d. In accordance with the Framework for Extraction Strategy Agreement, a maximum pumping rate of 24 ML/d was used. All other modelling assumptions remain the same for the purpose of this sensitivity analysis.

The sensitivity analysis involved running the synthetic streamflow sequence for mining year 20 when mine water demands are highest to determine the risk of a water deficit occurring at the peak demand year. The results of the sensitivity test as simulated daily time series of water pumped from the river and water deficit for year 20 are provided in Figure 6.2.

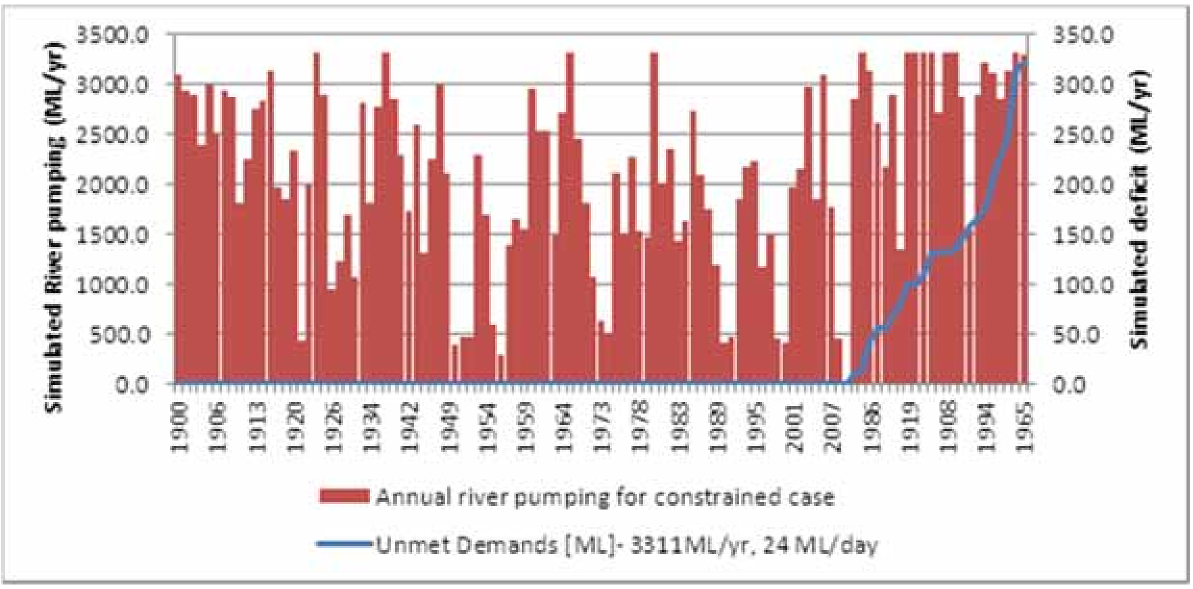


Figure 6.2 Results of sensitivity analysis of river water restrictions for peak demand year 20

The results show that there is a probability of a water deficit of 21% for the peak demand year 20, i.e. a deficit is predicted for a total of 24 years out of the 111 year sequence. The deficit ranges from 11ML to 334ML and exceeds 300ML in only 3 years. The peak deficit is 334ML, which equates to only 13% of the peak CHPP demand at year 20. It is noted that the unused Aquifer Access Licence entitlement at year 20 is in excess of 1.5GL, which is far greater than the maximum streamflow deficit.

It should also be noted that the assessed probability of a water deficit is based on a highly conservative sensitivity test that assumes the peak demand year will coincide with extremely dry conditions equivalent to the lowest flow on record in the Cudgegong River under regulated conditions.

Options available to CHC to manage any potential water deficits are discussed in Section 7.1.

6.1.2 Dam performance

Mine water dams

Mine water dams have been sized to achieve no discharge when operated as part of the site water management system under historical climate conditions, as determined through water balance modelling.

Pumping to mine water dams from the pit will cease when a maximum operating level is achieved (refer to Section 5.3.4 for assumed operating rules). This will maintain adequate freeboard in these dams, so that small runoff events from the local catchment will not cause the dams to overflow following extended periods of pit dewatering.

The water balance results show that dams MD1, MD2, MD3 and MD8 and mine water dams MD4, MD5, MD7, MD9, MD10, MD11 and MD12 do not overflow during the 111-year water balance simulation. Mine water dam MD6, which captures runoff from the stockpile adjacent to mining area B, overflows to mining area B during extended wet periods, but does not overflow to the creek system. As such, no offsite releases of mine water, process water or infrastructure water is expected during the operating life of the Project. This water will be reused on-site in the mining process. Storage will be provided in-pit during extended wet periods, until mine water dams have capacity to receive pit water.

Although mine water dams are not expected to overflow under even extreme weather conditions (with the exception of MD6 that overflows to mining area B), dam safety requirements dictate that spillways must be provided. Spillways from MD1, MD2, MD3, MD4 and MD5 will be directed to Laheys Creek. Spillways from MD7, MD8 and MD9 will directed to sedimentation dams located at the low points of the overburden emplacement areas.

Sedimentation dams

The large infrastructure sedimentation dams SD1, SD2 and SD3 have been designed to capture runoff from the MIA for the 20 year ARI 72 hour storm. They have been further tested in the water balance model and resized as required to ensure that they do not overflow for the modelled climate conditions and impact on the surface water environment.

The other overburden sedimentation dams have been sized to contain the 95th percentile 5-day duration storm event; however, they do overflow for conditions that exceed this event within the climate sequence modelled. Figures 6.3 to 6.6 provide sample output from the water balance model for sedimentation dams SD10 and SD31 that demonstrate the storage and overflow characteristics of the overburden sedimentation dams.

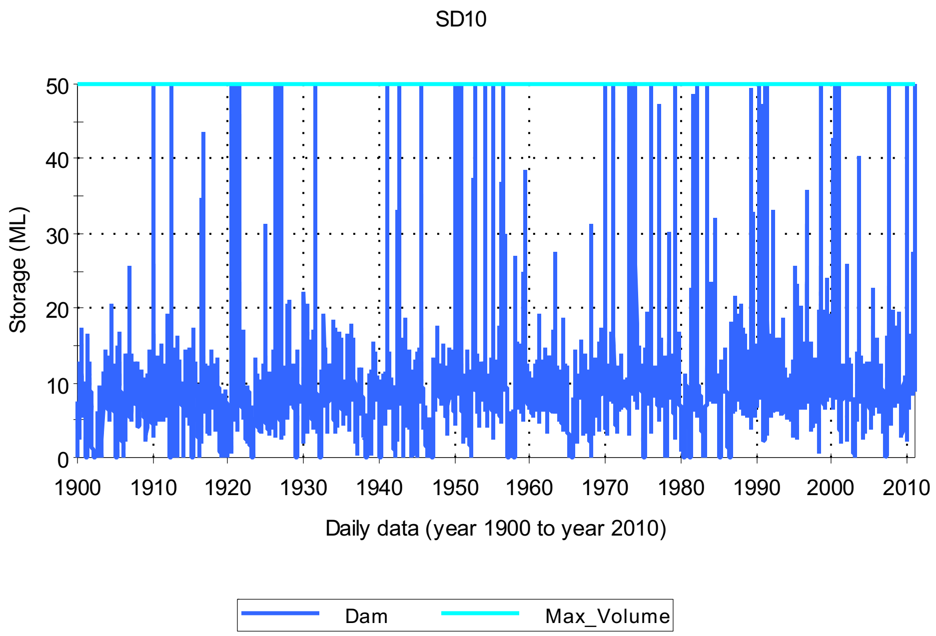


Figure 6.3 Simulated dam storage and overflows for SD10 for mining year 16

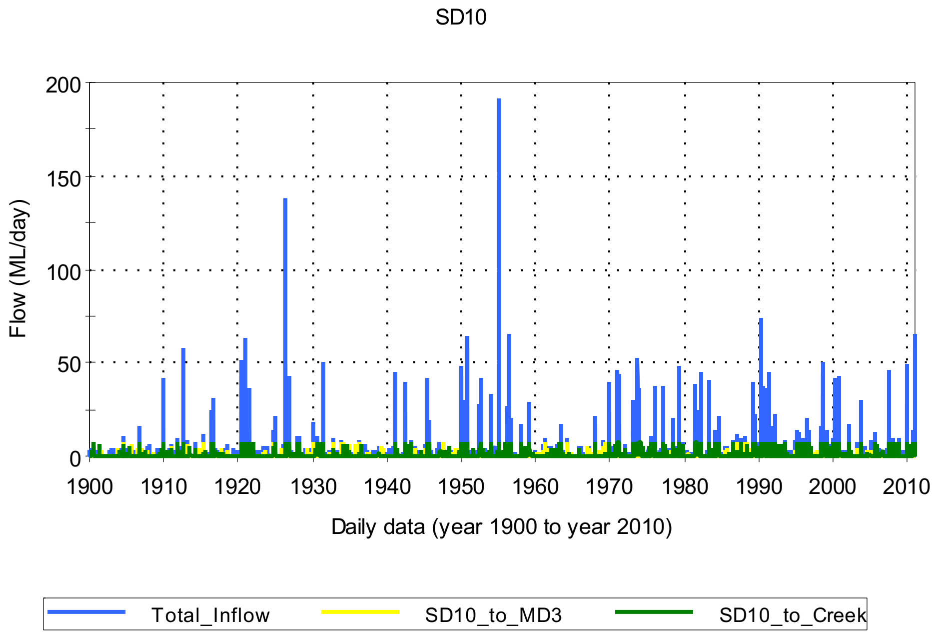


Figure 6.4 Simulated total inflow to SD10, pumping to MD3 and overflows to the creek from SD10 for mining year 16

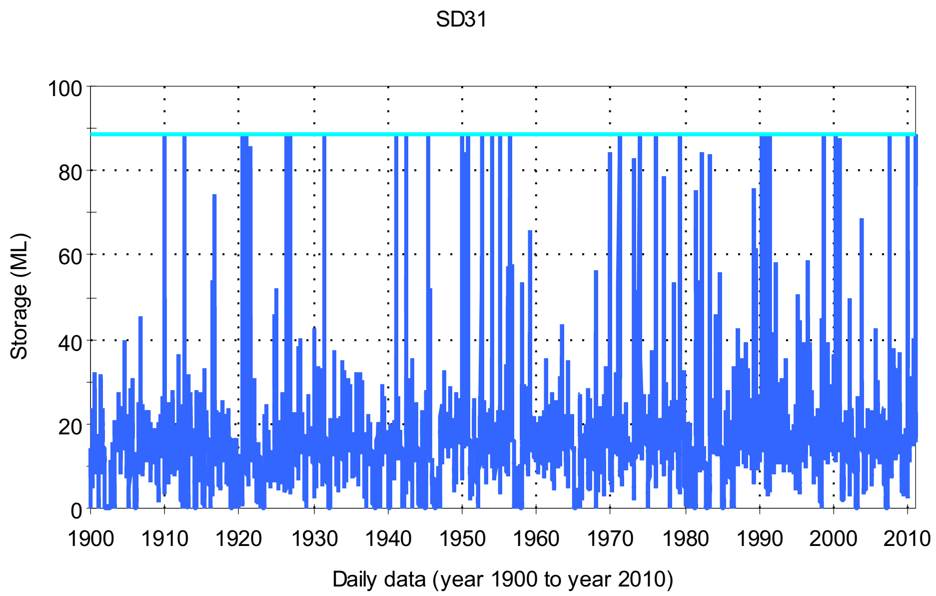


Figure 6.5 Simulated dam storage and overflows for SD 31 for mining year 16

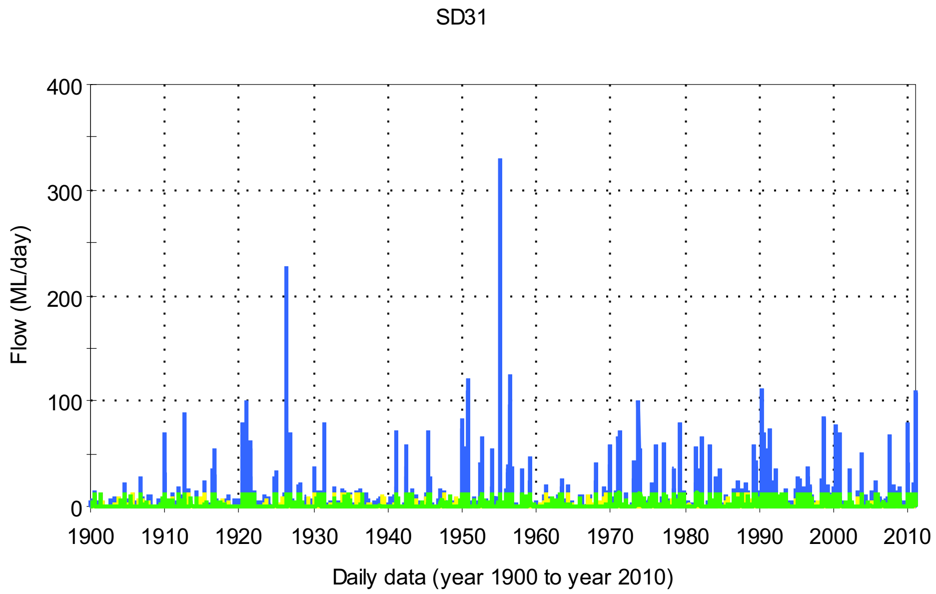


Figure 6.6 Simulated total inflow to SD31, pumping to MD3 and overflows to the creek from SD31 for mining year 16

6.1.3 Frequency of in-pit flooding

Pit sumps will be able to store relatively small volumes of pit water without interruption to mining activities. However, during extended wet periods, with standard capacity dewatering systems, relatively large volumes of water will accumulate in-pit and may interrupt mining activities. The maximum in-pit storage volumes over the 111-year water balance simulation are provided in Table 6-5.

Table 6-5 Maximum in pit storage volumes

Year	Maximum stored volume (ML/a)		
	Mining area A	Mining area B	Mining area C
1	176	166	0
4	805	894	401
12	1023	1671	1678
16	718	2345	2162
20	1414	1973	662

Predicted stored volumes per mining area over the 111-year water balance simulation are provided in Figures 6-7 to 6-9 for Year 16, when the mining area catchment is greatest and the estimated maximum volume stored in the combined pits is greatest. Predicted frequencies of in-pit flooding per mining area over the 111-year water balance simulation for Year 16 are illustrated by plots provided in Figures 6-10 to 6-12.

The mine water dam sizes have been chosen to provide a reasonable level of pit availability over the 111-year water balance simulation. The results show that although the pit dewatering system should be able to maintain dry pits, during extended wet periods mining may be interrupted by in-pit flooding.

Large volumes of water are only stored in-pit infrequently. The water balance analyses indicate that the probability of the pit water volume exceeding the peak combined volume of 5,225 ML in mining areas A, B and C sumps in year 16 is less than 10%. Should that occur, CHC would nominate one or more open cuts to receive water from the other excavations. The nominated pits would be selected to minimise overall disruption to the output of the mine.

CHC has mapped the worst case flood extent within each mining area as shown in Figures 6-13 and 6-14. These show that under the worst case scenario mining operations can continue, albeit with access via a single ramp during peak flooding.

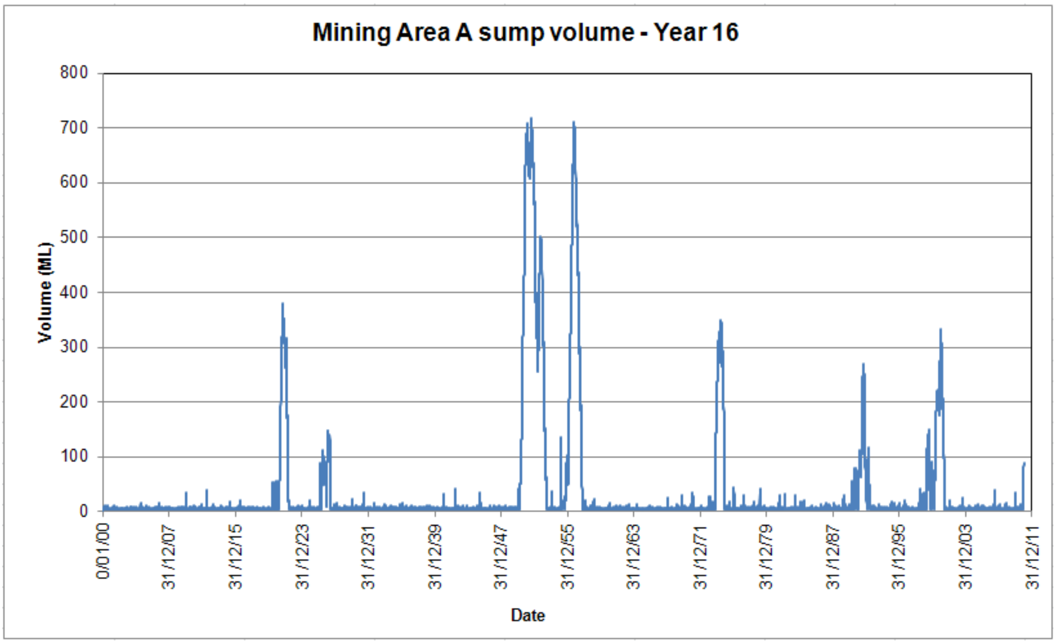


Figure 6.7 Stored volume in mining area A over the 111-year water balance simulation for Year 16

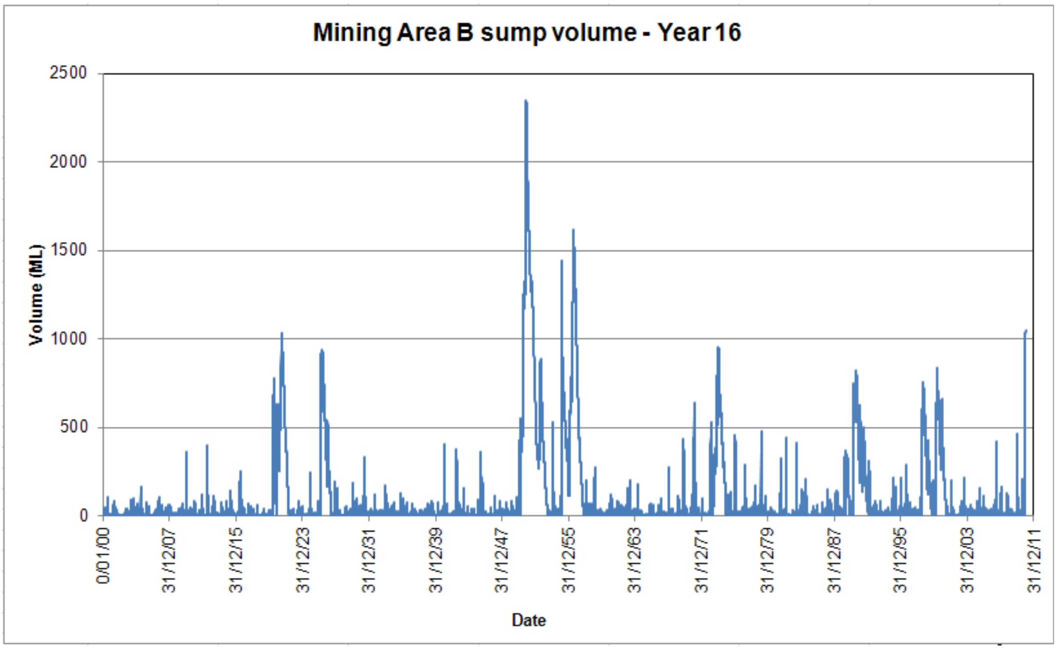


Figure 6.8 Stored volume in mining area B over the 111-year water balance simulation for Year 16

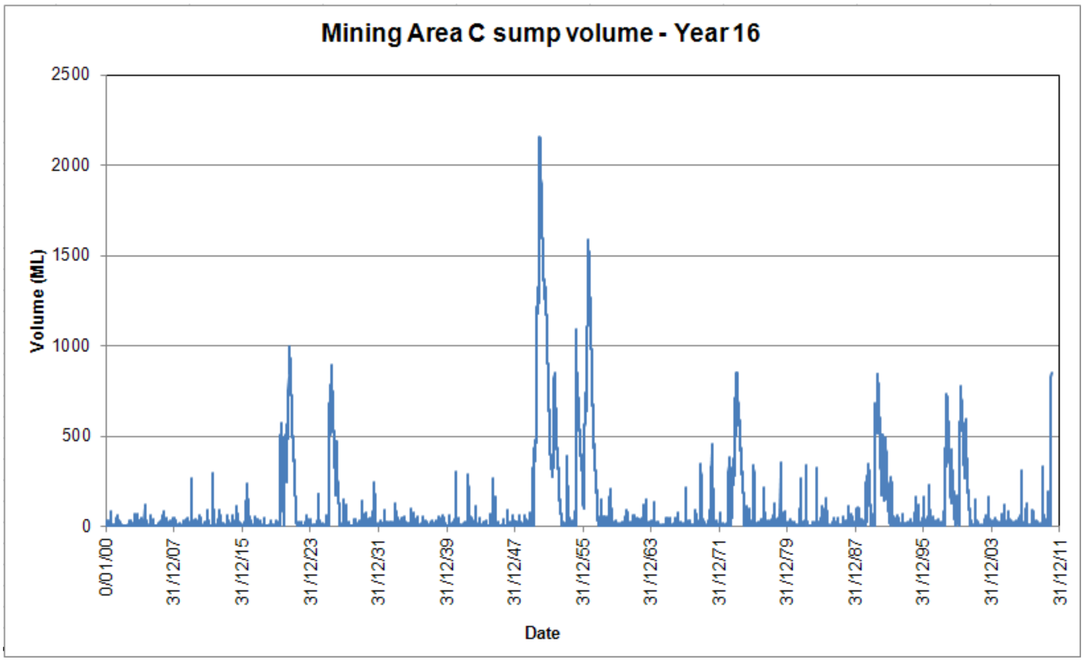


Figure 6.9 **Stored volume in mining area C over the 111-year water balance simulation for Year 16**

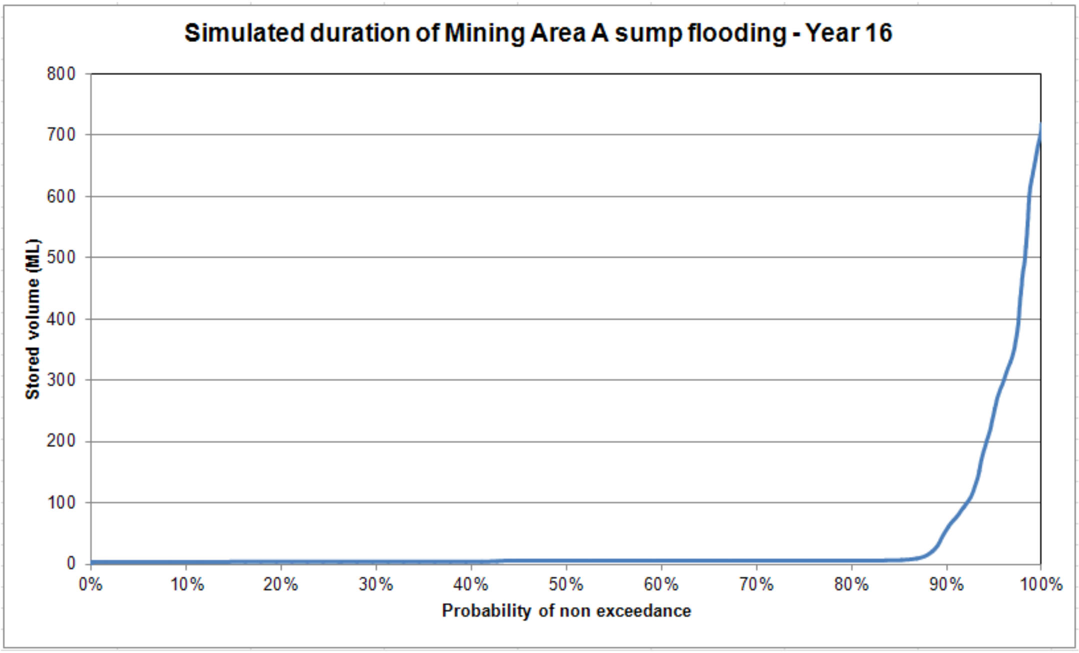


Figure 6.10 **Frequency of in-pit flooding for mining area A over the 111-year water balance simulation for Year 16**

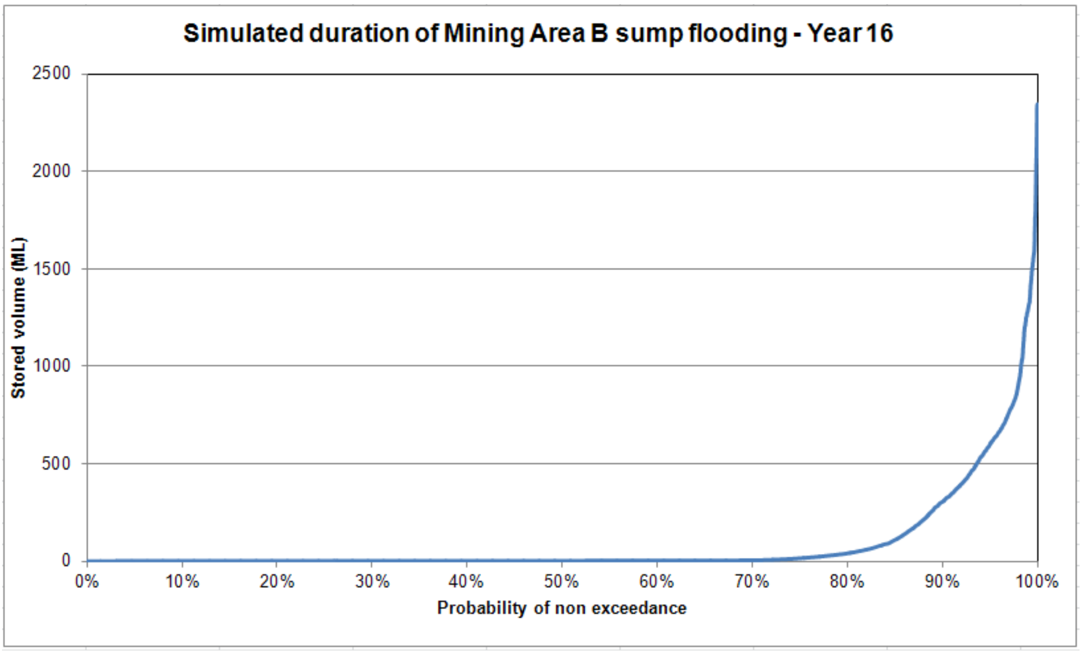


Figure 6.11 Frequency of in-pit flooding for mining area B over the 111-year water balance simulation for Year 16

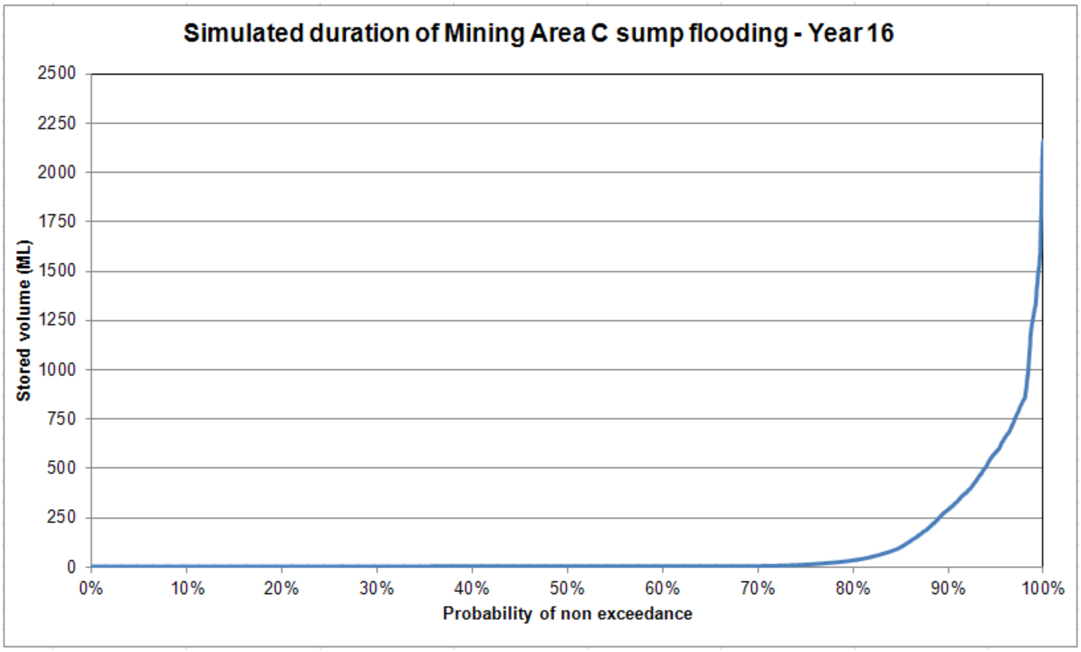
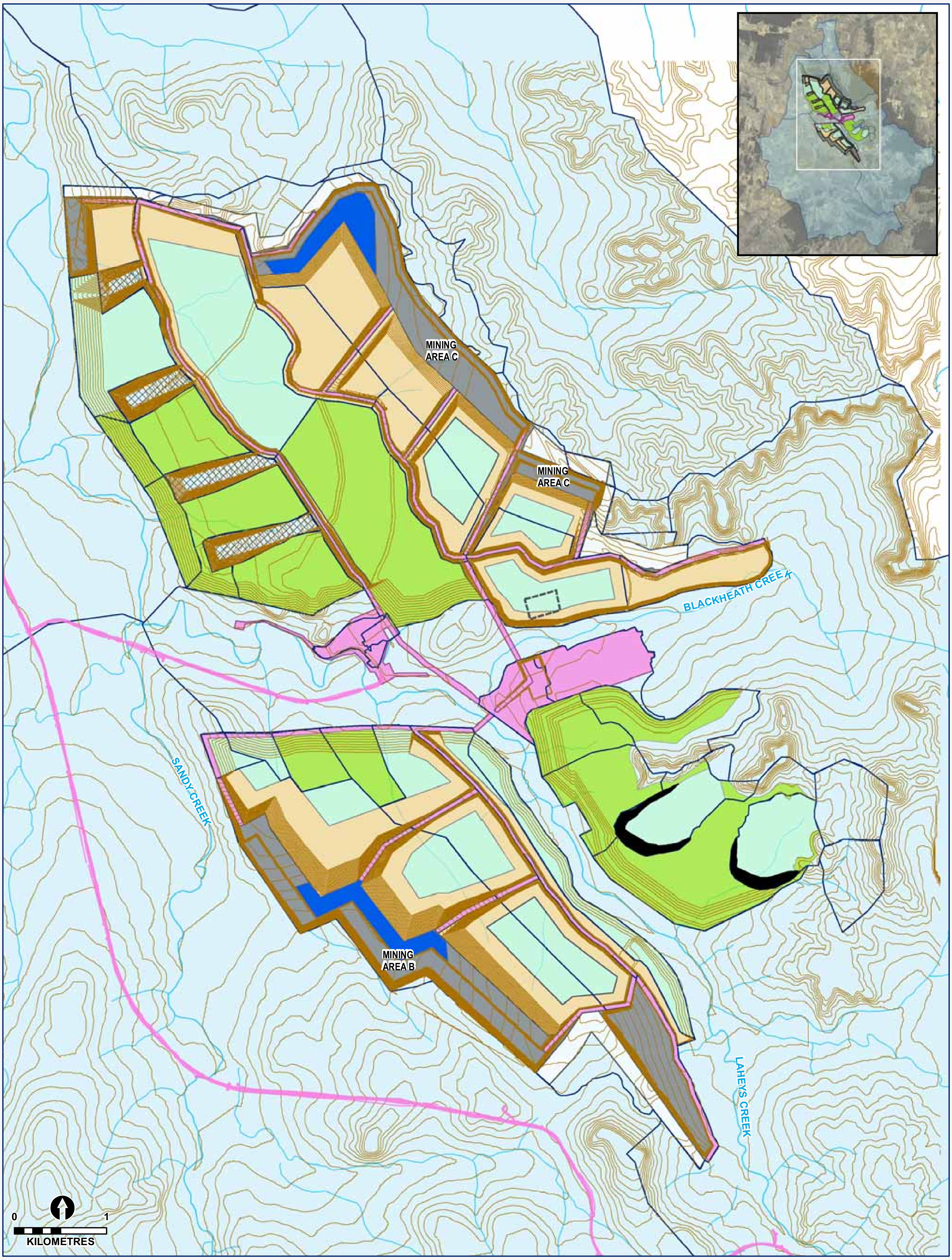


Figure 6.12 Frequency of in-pit flooding for mining area C over the 111-year water balance simulation for Year 16



- Year 16 mine elevation model 2m contours
- Year 16 catchment boundary
- Dam walls
- Year 16 in-pit flood extents

Year 16 mine landuse

- Relocated ROM pad
- Active emplacement
- Active mine
- Cleared area
- Haul road / infrastructure
- Rehabilitated emplacement area
- Established rehabilitation area
- Tailings emplacement area
- Topsoil stockpile

Figure 6.13 Worse case pit flooding for mining areas B and C (Year 16)

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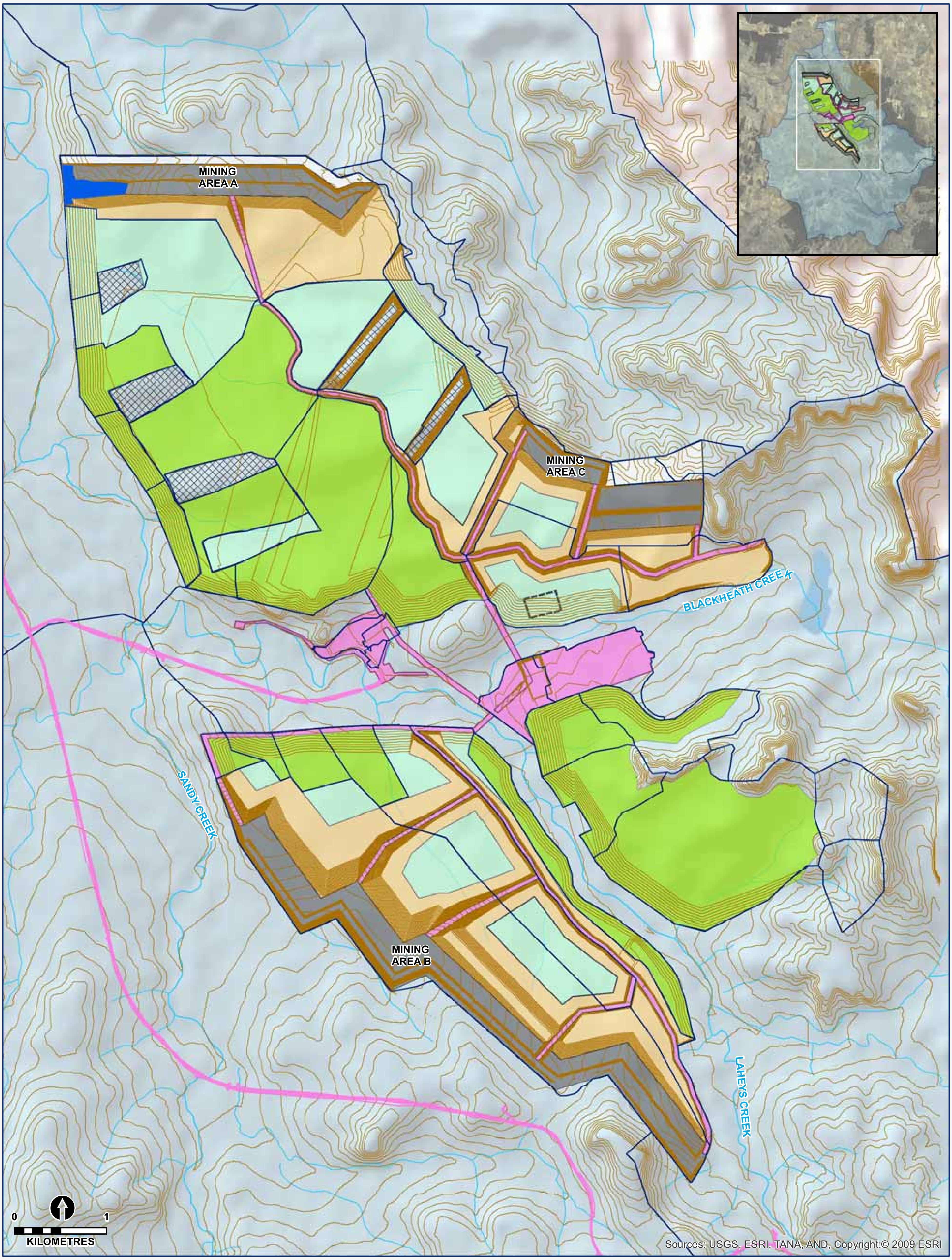

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- Year 20 mine elevation model 2m contours
- Year 20 catchment boundary
- Year 20 in-pit flood extents

Year 20 mine landuse

- Relocated ROM pad
- Active emplacement
- Active mine
- Cleared area
- Haul road / infrastructure
- Rehabilitated emplacement area
- Established rehabilitation area
- Tailings emplacement area

Figure 6.14 Worst case pit flooding for mining area A (Year 20)

6.1.4 Mining impacts on surface water flow regime

The water balance analyses indicate that the surface water flow regime downstream of the site will generally decrease for dry years. This may be attributed to the capture and on-site reuse of runoff from disturbed areas. The water balance analyses indicate that there will be an increase in downstream flows for median and wet years. This may be attributed to additional runoff from clean water catchments (e.g. clean water dam releases and diversions) and the mine water management dams not having spare capacity to store sedimentation dam water for reuse during median and wet years, so treated sedimentation dam water would be displaced into receiving watercourses.

Table 6-6 summarises estimated annual runoff volumes to the creek system for a 50th percentile median rainfall year. The median runoff volume is predicted to increase by between 4% and 10% during operations. Table 6-7 summarises the expected change in annual creek flows downstream of the site during mining compared to pre-mining conditions for dry, median and wet years.

Table 6-6 Median annual flow in Sandy Creek, Flyblowers Creek, Isbester Gully and Unnamed Tributary 1

	Units	Pre-mining	Year 1	Year 4	Year 12	Year 16	Year 20	Post-mining
Natural catchment								
Undisturbed catchment runoff and established rehabilitation runoff returned directly to creek	ML/a	1,852	**1,855**	**1,787**	**1,707**	**1,713**	**1,732**	**1,933**
Existing west 'Woolandra' farm dams catchment runoff	ML/a	62	–	–	–	–	–	–
Existing west 'Woolandra' farm dams overflows	ML/a	0	–	–	–	–	–	–
Release from water management system to creek								
Sedimentation dam overflows	ML/a	–	**0**	**0**	**0**	**0**	**0**	–
Sedimentation dam controlled release	ML/a	–	**103**	**217**	**318**	**325**	**194**	–
Clean water/highwall dam controlled release	ML/a	–	**2**	**10**	**17**	**3**	**0**	–
Raw water dam overflows	ML/a	–	**0**	**0**	**0**	**0**	**0**	–
Total flow at study catchment outlet	ML/a	**1,852**	1,960	2,014	2,046	2,043	1,930	**1,933**
Percentage change from pre-mining			6%	9%	10%	10%	4%	4%

Table 6-7 Summary of expected changes to pre-mining creek flows during mining

Year	Net change from pre-mining flow (%)		
	10th percentile (dry year)	50th percentile (median year)	90th percentile (wet year)
1	+7	+6	+4
4	-3	+9	+5
12	-6	+10	+5
16	-6	+10	+5
20	-5	+4	+5

7. Potential impacts and mitigation measures

7.1 Impact of imported water supply on mining

The water balance for the Project indicates that an external water source is required for the Project. Reuse of sedimentation dam water, mine water runoff and groundwater seepage generated within the site are not adequate to meet the water demand under median climatic conditions throughout the life of the Project.

The peak annual demand for imported water is expected to occur in Year 20. The simulated annual requirement for imported water for Year 20 is provided in the plot in Figure 7.1 based on the 111-year water balance simulation. The annual rainfall depth is also provided on the plot in Figure 7.1.

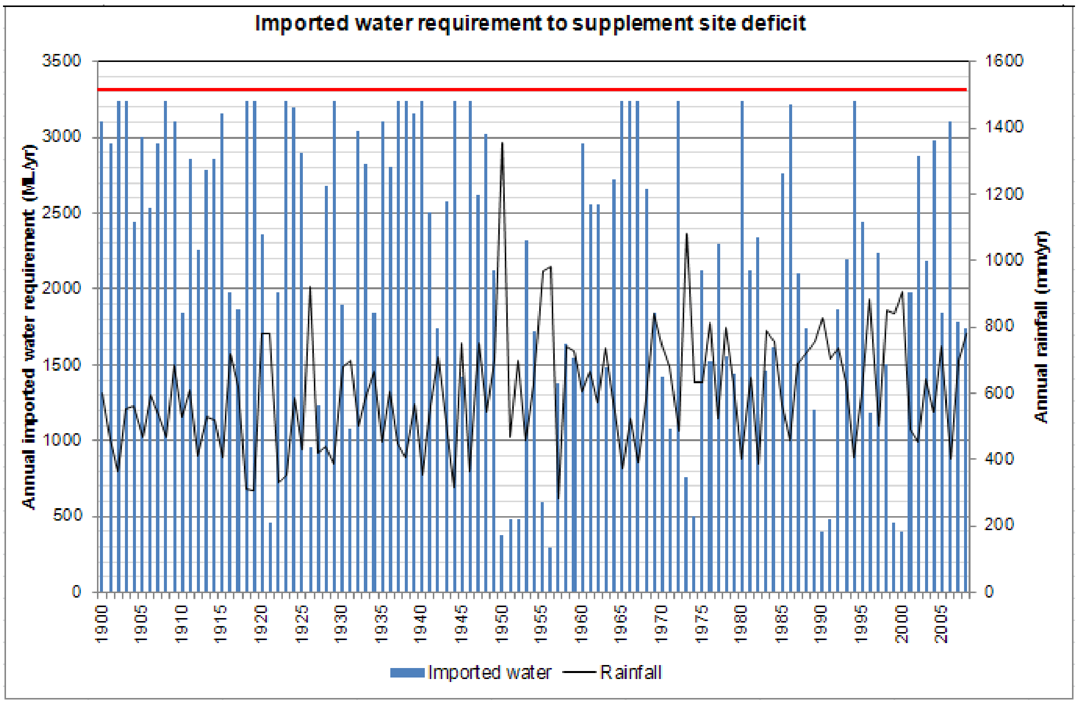


Figure 7.1 Comparison of annual imported water requirements over the 111-year water balance simulation for Year 20

It is estimated that 3,240 ML/a of imported water would be required for a 10th percentile dry rainfall year in Year 20. Figure 7.1 shows that for very dry years, the requirement for imported water approaches, but does not exceed, the 3,311 ML/a maximum entitlement in Year 20 for 19 out of the 111 years modelled.

If very dry climatic conditions (i.e. drier than a 5th percentile rainfall year) are experienced in Year 20, appropriate mitigation measures will be required to prevent a water shortage. Mitigation measures may also be required if very dry climatic conditions are experienced in Years 18 to 19, as groundwater seepage to the pits reduces significantly in Year 19 due to the ongoing infilling of voids. The reduction to seepage will conversely increase unused aquifer access licence entitlement, which will permit mitigation of water demand by licensed groundwater extraction.

Table 7-1 Imported water requirement for a 10th percentile dry year

Year	Total site demand (ML/a)	Imported water (ML/a)	Groundwater seepage (ML/a)	Total WMS runoff (ML/a)	Sedimentation dam water reused on-site (ML/a)	System releases (overflows and releases) (ML/a)
1	524	120	131	337	26	42
4	3,210	1,820	1069	753	119	4
12	4,340	2,600	2446	1,157	144	5
16	4,292	2,520	2403	1,023	161	2
20	4,055	3,240	1163	957	125	0

As discussed in Section 5.4.3, the Macquarie and Cudgegong Regulated River Water Source is a reliable source of water and the full high security WAL entitlement of 3,311 ML/a has been assumed to be available to the Project when required.

The Framework for Extraction Strategy Agreement with State Water Corporation will set constraints on the volume and timing of extraction which could affect the above assumption. CHC is working with State Water Corporation to ensure as much of its licence entitlement as possible can be extracted during period of excess flows in the river, as it will assist State Water Corporation to more efficiently manage water resources in the Macquarie and Cudgegong Rivers.

Should water shortages occur during worst-case climatic conditions, CHC has a number of alternative options available to ensure the mine continues to operate. These are summarised as follows, in decreasing order of priority:

- Implementation of additional operational efficiencies and water conservation measures to reduce mine water demand. These include minimising make-up water demands by implementing further water conservation measures in the CHPP process water system and reducing water demands for dust suppression with environmentally safe additives that help to bind dust particles to reduce airborne dust potential.
- Employing unused aquifer access licence entitlement to withdraw groundwater to supplement water supply.
- Purchasing of ‘General’ WALs from the open water market, if they are available, to meet the additional demand, and selling them back to the market when no longer required.
- Reducing coal production rates to match available water supplies. This is a worst-case scenario and assumes that the above options are either not available or not effective at providing additional water or reducing water demand.

7.2 Wet weather impacts on mining

There is a probability of around 10% in the mid to later years of mining (Years 12 to 16) that in-pit sumps will store significant volumes of water. A compartmentalised area would be required in-pit to store this water during wet years. Alternatively, the Project would need to provide an emergency storage for such events.

During extended wet periods, the rate of pit dewatering could exceed the rate at which water is reused on-site and dewatering ceases because mine water dams are full. During these periods, additional in-pit water storage will be provided in inactive mining areas. This would allow for the continuation of dewatering activities in active mining areas during wet periods, thus minimising significant disruptions to mining operations. Appropriate locations for these types of in-pit storages will be identified during detailed design.

7.3 Impacts of mining on the surface water regime

The water balance analyses indicate that the surface water flow regime downstream of the Project will generally decrease for dry years. This may be attributed to the capture and on-site reuse of runoff from disturbed areas. The water balance analyses indicate that there is an increase in downstream flows for median and wet years. This may be attributed to releases from clean water catchments (diversions and clean water dam releases) and the water management dams not having spare capacity to store sedimentation dam water for reuse during median and wet years.

At the completion of mining, flows are expected to increase for dry, median and wet years. This increase may be attributed to a net increase in catchment area draining to the creek because of the decommissioning of the west 'Woolandra' farm dams.

8. Post mining

8.1 Final landform

The final rehabilitated Year 21 landform is shown in Figure 8-1. The design of the final landform backfills the voids in former mining areas A and C through considerable investment in spoil rehandling and rehabilitation. The former mining areas A and C will be free draining to the creek systems.

Figure 8-1 shows indicative flow paths for runoff from areas A and C. At area A the final landform topography drops approximately 40m over approximately 3.4km where the runoff enters the floodplain of lower Sandy Creek. This topography provides a gradient of around 1 in 100, which will allow for controlled non-erosive runoff velocities.

At area C the final landform topography drops approximately 16m over approximately 1km where the runoff enters the floodplain of Blackheath Creek. This topography provides a gradient of around 1 in 50, which is relatively steep, although is similar to the natural gradient of the nearby upper reaches of Blackheath Creek. Further consideration will need to be given at the design stage to ensuring a stable landform along the drainage path from area C, and measures such as rock armouring, vegetation tolerant of high flow velocities, provision of a shallower gradient meandering outlet channel or a combination of measures may need to be adopted.

The final landform will comprise a single final void lake corresponding to former mining area B. Backfilling the entire mining area B is not economically feasible; however, almost half the area of the void at the end of mining will be backfilled. A cross section through the fnal void lake is provided in Figure 8-2, which is repeated from the Groundwater Assessment report (PB, 2013).

Rainfall falling on the final void and groundwater seepage will slowly fill the void to form a lake in mining area B. The water level in the void will be influenced by the balance between inflows from rainfall and groundwater, and outflows due to evaporation losses and seepage into the groundwater system. The salinity level in the void will be influenced by the salinity concentrations of rainfall falling directly on the void, groundwater inflow, surface water runoff from rehabilitated overburden emplacement areas, and the evaporative concentration of these inflows.

This chapter assesses the potential impacts of the final void lake in mining area B on the surface water environment. Potential groundwater impacts are discussed in the groundwater assessment report (Parsons Brinckerhoff 2013).

8.1.1 Catchment areas

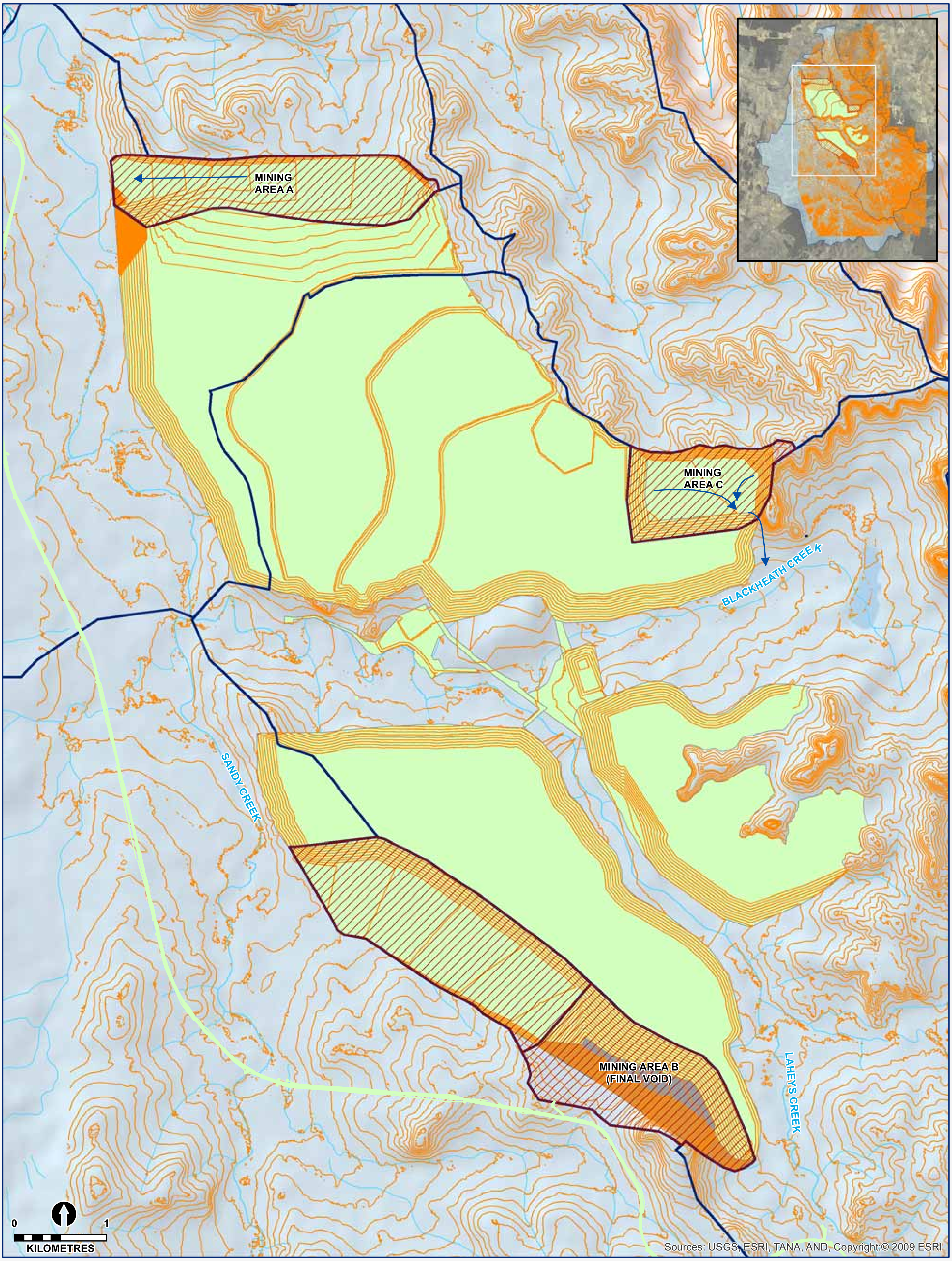
The catchment areas for the final landform are provided in Table 8-1. The catchment boundaries are shown in Figure 8-1.

Table 8-1 Final landform catchment areas

Sub-catchment	Area (ha)
Final void	
Total area draining into void B	242
Creek system	
Sandy Creek (upstream of Laheys Creek confluence)	14,063
Sandy Creek (downstream of Laheys Creek confluence)	2,231
Laheys Creek	11,665
Flyblowers Creek	680
Isbester Gully	2,387
Unnamed Tributary 1	278
Subtotal	31,304
Total	**31,688**

Table 8-1 shows that the catchment area draining to the final void B is 242 ha. The total catchment area of Sandy Creek at its confluence with the Talbragar River increases from 27,183 ha for the existing situation to 27,959 ha for the final landform (an increase of 2.9%). Although a catchment of 242 ha continues to drain to final void B, runoff from the 1,034 ha west 'Woolandra' farm dam catchments will be returned directly to the creek for the life of the Project and after mining, which would offset this reduction.

The catchment area for Unnamed Tributary 1 of the Talbragar River remains unchanged. The catchment area for Flyblowers Creek, however, reduces from 738 ha to 680 ha (a reduction of -7.9%), and the catchment area for Isbester Gully reduces from 2,454 ha to 2,387 ha (a reduction of -2.7%). Refer to Section 3.1 Table 3-1 for existing catchment areas.



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Year 21 mine elevation model 2m contours

Year 21 surface water drainage

Final landform catchments

Final void subcatchment boundary

Catchment boundary

Year 21 landuse

Final pit void

Rehabilitation area

Figure 8.1 Final landform

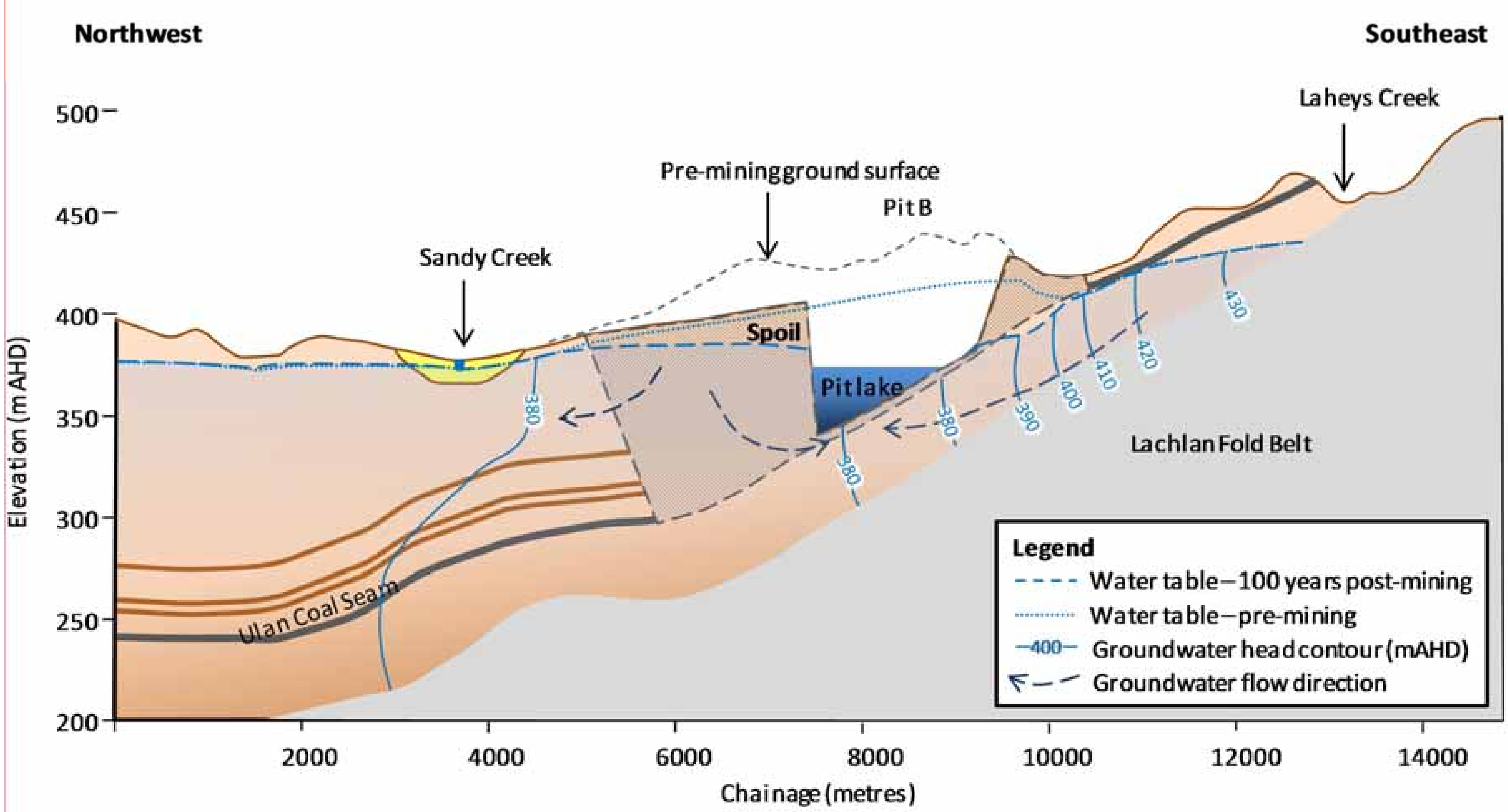

Figure 8.2 **Typical section for final void B**

8.2 Final void B water and salt balance

This section outlines the key results of the final void water and salt balance for mining area B. The following revisions have been made to the final void water and salt balance modelling undertaken for the original Surface Water Assessment:

- The original modelling of the final void for the Surface Water Assessment utilised a groundwater inflow / outflow relationship derived based on groundwater modelling for a previous version of the landform. The modelling was revised based on groundwater modelling of the current landform undertaken as part of the groundwater assessment report (Parsons Brinkerhoff, 2013).

- To further account for uncertainty in the groundwater inflow estimates related to the permeability and storage characteristics of the spoil material and potential heterogeneity in Ulan Coal Seams, the estimated groundwater inflows were varied stochastically using a normal distribution from 85% to 115% of the revised estimates.

- The original modelling of the final void for the Surface Water Assessment utilised a single 1000 year long rainfall sequence developed by repeating the 122 year Data Drill sourced historical rainfall record. This modelling approach provided results that were based on only one realisation of the historical data. The revised modelling utilises stochastically generated rainfall sequences. The stochastic data provides alternative realisations that are equally likely to occur as the historical data, and provides an indication of the uncertainty in the model results associated with climate variability.

- To facilitate the stochastic modelling approach, a GoldSim model of the final void was developed. A daily timestep was adopted in the model.

8.2.1 Modelling approach

Water balance modelling has been undertaken to predict the long-term behaviour of final void B following the completion of mining.

The final void water balance model was developed using GoldSim software. The model was used to calculate the volume of water in the final void at the end of each day taking into account rainfall-runoff inflow, groundwater inflows/outflows and evaporation. The model was also used to calculate the salinity concentration in the final void at the end of each day. Instantaneous mixing of the various inflow types was assumed in the model, and no allowance was made for the stratification of the final void.

The final void water balance model was simulated at a daily time step for a period of 1,000 years. The model was simulated 100 times using 100 replicates (or sequences) of stochastic rainfall data.

8.2.2 Model data

8.2.2.1 Rainfall data

Stochastic rainfall data was generated using the Stochastic Climate Library (SCL) model developed by the CRC for Catchment Hydrology. ‘Stochastic climate data are random numbers that are modified so that they have the same characteristics (in terms of mean, variance, skew, long-term persistency etc) as the historical data from which they are based.

Each stochastic replicate (sequence) is different and has different characteristics compared to the historical data, but the average of each characteristic from all the stochastic replicates is the same as the historical data' (CRC for Catchment Hydrology, 2007).

A set of 100 stochastic replicates was generated at a daily timescale from the Data Drill sourced 122 year historical record from 1889 to 2011. Each of the 100 stochastic replicates had a length of 122 years. As it was necessary to simulate the final void model for a period of 1000 years to establish equilibrium conditions, each of the 100 stochastic replicates was extended to a length of 1000 years by repeating the 122 year stochastic replicates.

The SCL model displays the mean and percentiles of various statistics of the generated data and the corresponding values in the historical data. The SCL model was generally found to satisfactorily reproduce the statistics in the historical data in the stochastically generated data. The annual mean, standard deviation, skewness and lag one autocorrelation were all within the default tolerances adopted in the SCL model.

8.2.2.2 Evaporation data

Evaporation estimates were based on average evaporation data for each month of the year calculated from the Data Drill sourced 122 year historical record from 1889 to 2011 (refer to Section 3.2 for Data Drill). Average evaporation data for each month of the year is provided in Table 8-2. Note that the potential evapotranspiration data was utilised within the AWBM rainfall-runoff model, whereas the lake evaporation data was utilised when calculating evaporative losses from the final void lake.

Table 8-2 Average daily evaporation (based on Data Drill from 1889 to 2011)

Month	Lake evaporation (mm/d) [1]	Potential evapotranspiration (mm/d) [2]
January	6.5	6.0
February	5.8	5.4
March	4.6	4.3
April	3.1	3.0
May	1.9	1.9
June	1.3	1.4
July	1.4	1.5
August	2.2	2.0
September	3.3	2.9
October	4.6	4.1
November	5.7	5.2
December	6.4	5.9

[1] Lake evaporation calculated using the Morton formula for shallow lakes (source: Morton 1983).

[2] Potential evapotranspiration calculated using the Penman-Monteith formula (source: Food and Agriculture Organization of the United Nations 1998).

8.2.2.3 Void stage-area-volume relationship

The stage-area-volume relationship for final void B is provided inFigure 8.3. The assumed base of the final void is 342 m AHD and the assumed top of the void is approximately 407 m AHD.

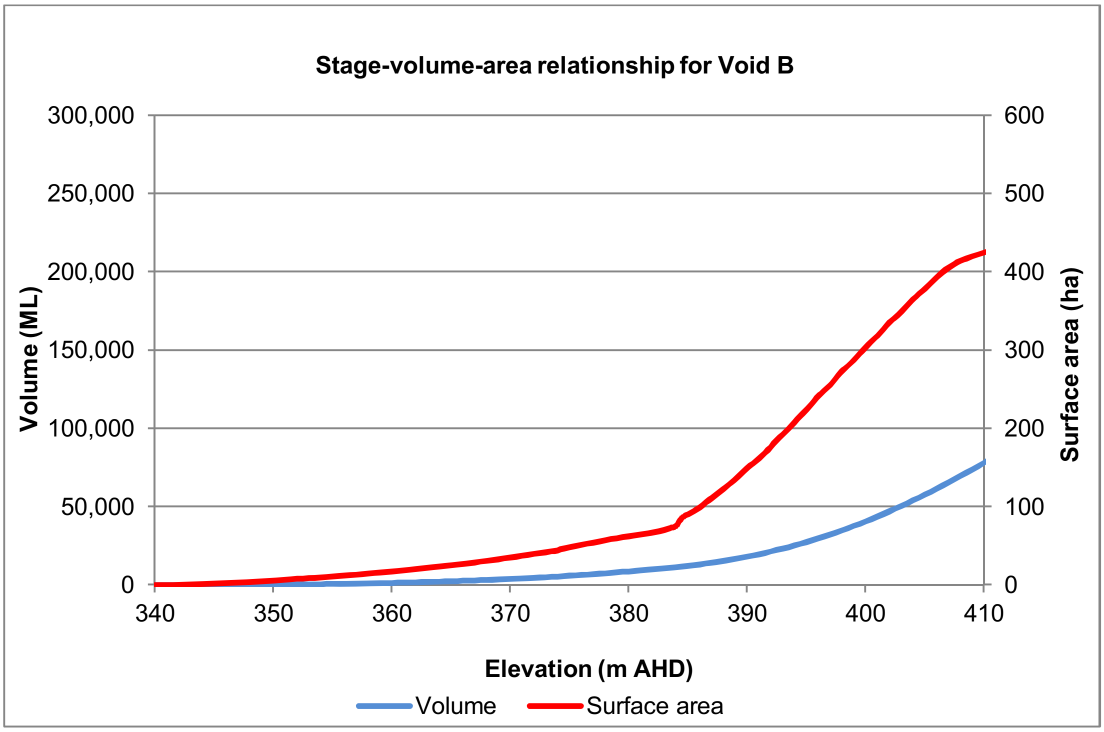

Figure 8.3 **Stage-area-volume relationship for final void B**

8.2.2.4 Salinity concentrations

The following average salinity values were assumed for the various inflows:

- **Direct rainfall falling on final voids** — assumed salinity value of 10 mg/L.
- **Groundwater inflow** — assumed salinity value of 2,806 µS/cm (1,796 mg/L) for void B based on baseline groundwater monitoring undertaken as part of the Project's groundwater assessment report (Parsons Brinckerhoff 2013). The value for void B is based on the average value for porous rock (Triassic) of 3,993 µS/cm and the average value for porous rock (Permian) of 1,618 µS/cm, as both of these units are likely to be present in the pit.
- **Surface water runoff from rehabilitated overburden emplacement areas, void walls and natural catchment** — assumed salinity value of 238 µS/cm (152 mg/L) based on the median salinity of overburden leachate obtained from the geochemical assessment (GeoTerra 2012). Note that it was not considered appropriate to adopt the salinity values obtained from baseline water quality monitoring in Sandy Creek and Laheys Creek, as salinity values in the creeks are likely to be elevated because of groundwater baseflow contributions and evaporative concentration effects in semi-permanent pools during dry periods.

A conversion factor of 0.64 was assumed between µS/cm and mg/L (or ppm). This is in line with unit conversion information published on the Office of Environment and Heritage website. Note that this is an approximate conversion, rather than an absolute conversion, and is based on the fact that water chemistry is sodium chloride and carbonate dominated.

8.2.3 Water inputs

8.2.3.1 Surface water runoff

The AWBM rainfall-runoff model was incorporated into the GoldSim model to generate a daily time series of runoff from the final void catchment based on the 100 replicates of stochastic rainfall data. Evaporation estimates for use in the AWBM model were based on potential evapotranspiration averages for each month of the year calculated from the Data Drill sourced historical record from 1889 to 2011. Average potential evapotranspiration data is provided in Section 8.2.2.2 Table 8-2.

The AWBM rainfall-runoff model and parameters are described in Section 3.5.1. The 'pit' AWBM parameters were adopted for the final void. The 'established rehabilitation' AWBM parameters were adopted for rehabilitated overburden areas. The 'undisturbed' AWBM parameters were adopted for areas that had not been disturbed by mining operations.

8.2.3.2 Direct rainfall

Direct rainfall volumes for the final void were calculated based on the stage-area-volume relationship provided above in Section 8.2.2.3.

8.2.3.3 Groundwater inflows

The numerical groundwater model for the Project was used to estimate groundwater inflow and outflow rates to/from void B. The adopted groundwater inflow versus lake water elevation relationship is provided in Figure 8-4. Note that there is no groundwater outflows for lake water elevations below 380 m AHD. This is because the creek bed level is approximately 380 m AHD – refer to the groundwater assessment report (Parsons Brinckerhoff 2012).

The simulated daily timeseries of groundwater inflows is provided in Figure 8-5.

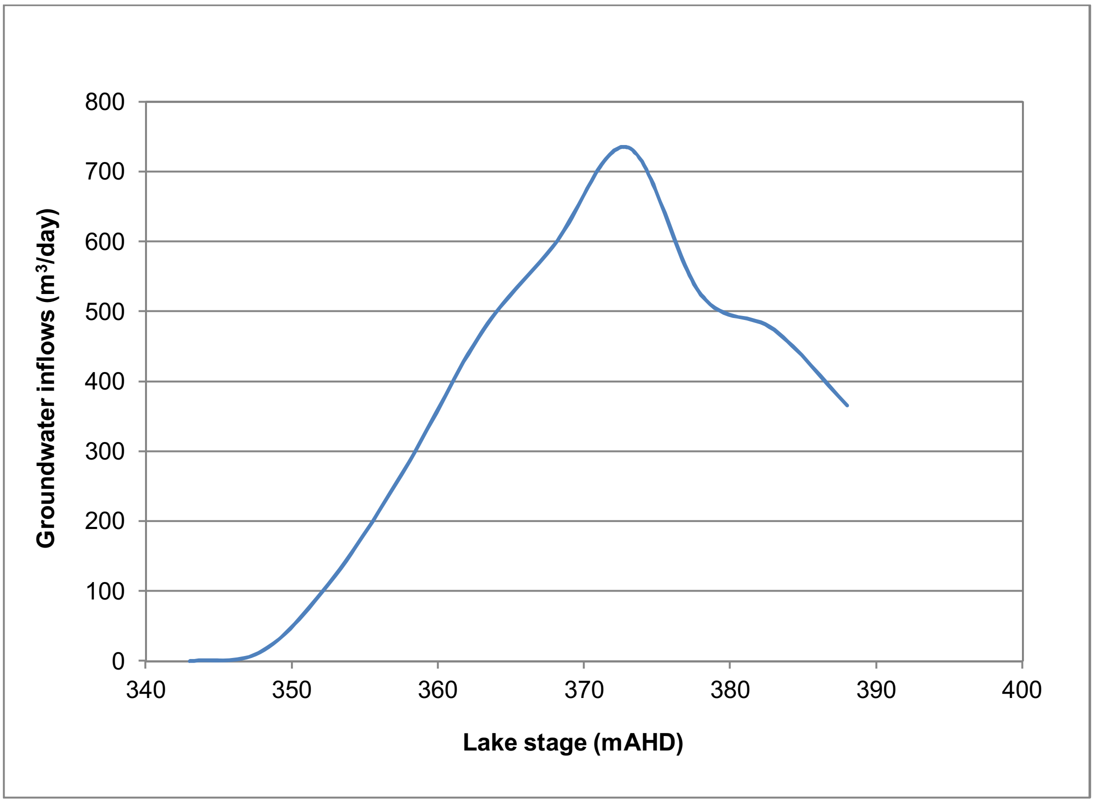

Figure 8.4 Groundwater inflow relationship

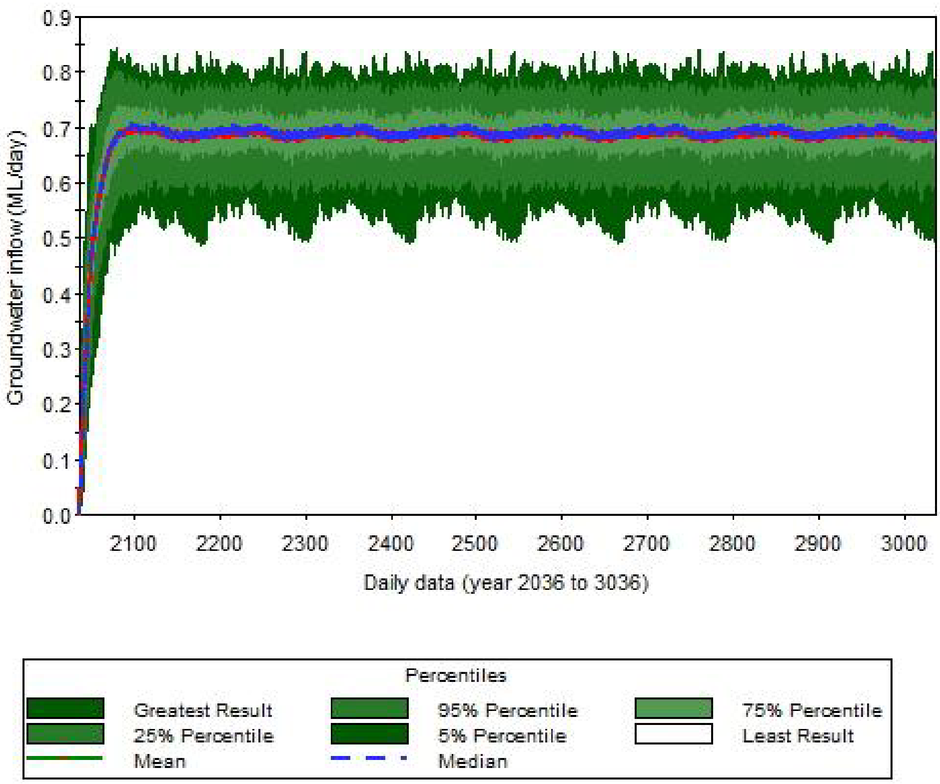

Figure 8.5 Final void B groundwater inflow estimates over 1000 year water balance simulation

8.2.4 Water outputs

8.2.4.1 Evaporation

Evaporation estimates for the final void lake were based on lake evaporation averages for each month of the year calculated from the Data Drill sourced historical record from 1889 to 2011. Average lake evaporation data is provided in Section 8.2.2.2 Table 8-2. The evaporative surface area of the void lake was determined based on the stage-area-volume relationship provided above in Section 8.2.2.3.

The simulated daily time series of final void lake evaporation is provided in Figure 8.6. Daily evaporation from the void ranged from approximately 0.4 to 3.2 ML/d.

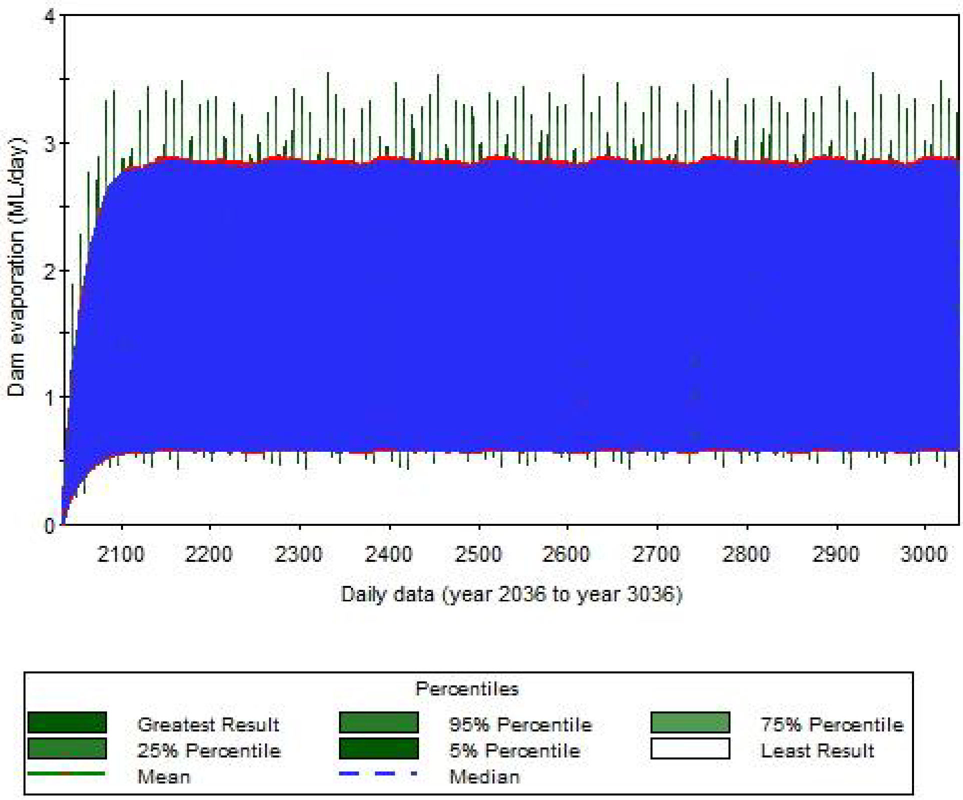

Figure 8.6 Final void B lake evaporation estimates over 1000 year water balance simulation

8.2.4.2 Groundwater outflows

Another potential output from the void could have been groundwater outflows that might have occurred concurrently to the groundwater inflows. However, as discussed above in 8.2.3.3, groundwater modelling indicated that there would be no outflows below lake stage elevations of 380 m AHD.

8.2.5 Model results

Plots of the simulated daily timeseries of final void water volume and level are provided in Figure 8-7 and Figure 8-8 respectively. The results indicate that water levels in final void B will initially rise steeply, but will reach an average equilibrium level by around Year 2100 (approximately 65 years after the completion of mining). The 'least result', median and 'greatest result' simulated equilibrium water levels were approximately 372.2, 374.2 and 378 m AHD respectively for the maximum level of the lake. The 'greatest result' equilibrium water level, which represents the greatest result of the 100 climate replicates, is 29 m below the top-of-void level of 407 m AHD. The void is therefore not expected to overtop. The 'greatest result' equilibrium water level is 2 m below the creek level of 380 m AHD. The void is therefore expected to be a net groundwater sink. No groundwater outflow will occur from the void lake towards the creek.

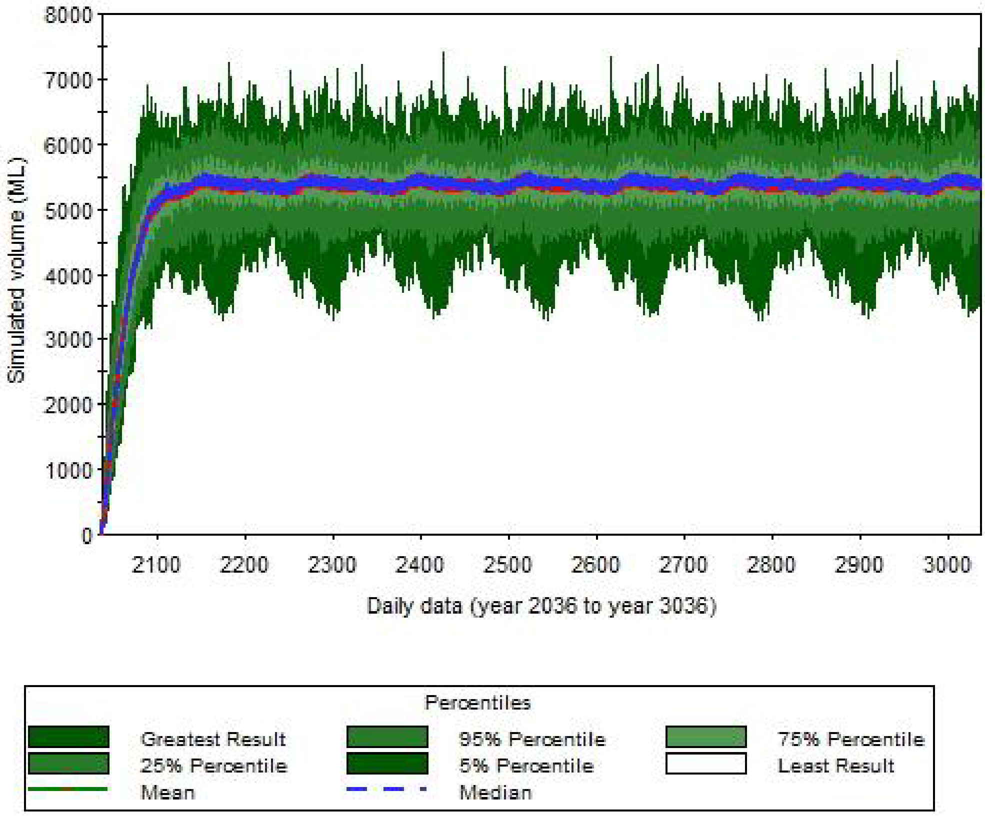

Figure 8.7 Final void B stored volume estimates over 1000 year water balance simulation

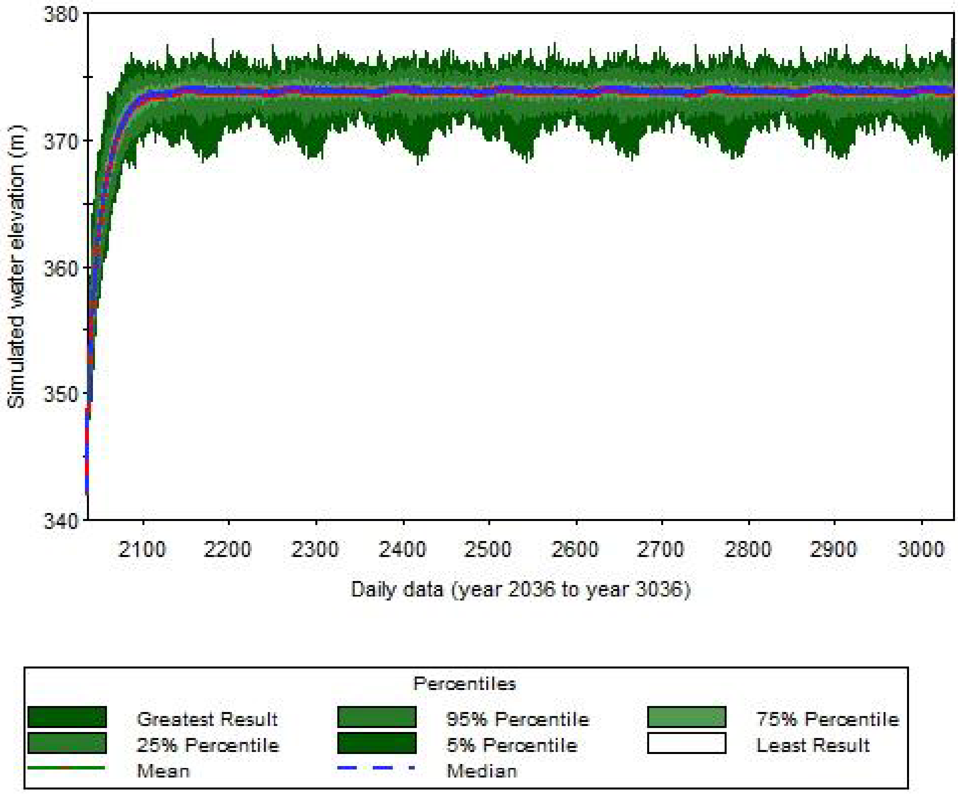

Figure 8.8 Final void B water level estimates over 1000 year water balance simulation

The simulated daily time series of final void salinity is provided in Figure 8-9 based on the 1,000-year water balance simulation. The results indicate that after 1000 years, the salinity levels in the void lake will not stabilise and will continue to rise due to evaporation. The 'least result', median and 'greatest result' simulated peak salinity concentrations were approximately 72,700, 89,000 and 114,300 mg/L respectively.

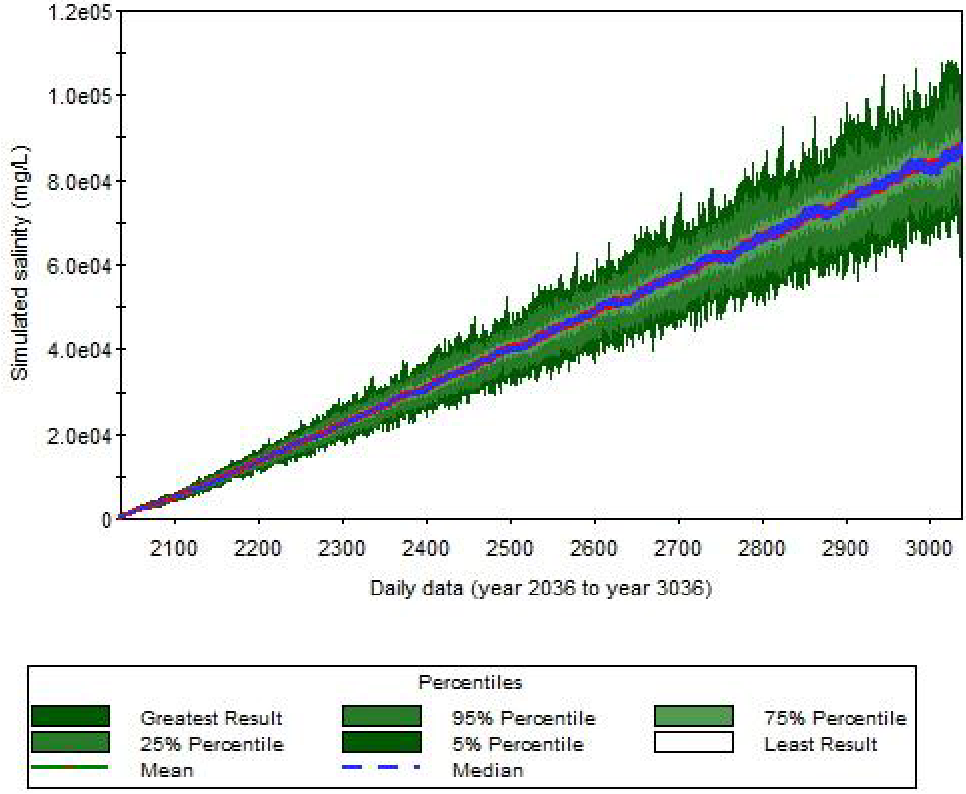

Figure 8.9 Final void B salinity estimates over 1000 year water balance simulation

It should be noted that the above results do not take into account detailed processes such as lake stratification and mixing.

Table 8.3 provides a summary of the statistics output from the simulations.

Table 8-3 Summary statistics from the last 100 years out of 100 sets of 1000-year simulations

Statistics	Lake level (m AHD)		Water volume (ML)		GW Inflow (ML/d)		Lake evaporation (ML/d)		Salinity (mg/L)	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
Mean	373.42	374.12	5,212	5,526	0.67	0.70	0.57	2.88	76,946	88,256
Standard .Deviation.	0.85	1.42	364	627	0.05	0.06	0.03	0.26	5,705	8,588
Least Result	367.94	372.15	3,191	4,674	0.48	0.57	0.41	2.56	58,549	72,667
5th percentile	370.83	372.56	4,167	4,841	0.56	0.61	0.49	2.62	64,405	77,549
25th percentile	372.64	372.56	4,871	4,841	0.63	0.61	0.54	2.62	71,875	77,549
Median	373.45	374.22	5,214	5,555	0.67	0.71	0.57	2.88	76,963	88,981
75th percentile	373.88	374.22	5,399	5,555	0.70	0.71	0.58	2.88	80,851	88,981
95th percentile	374.74	376.34	5,796	6,593	0.75	0.80	0.62	3.32	85,253	104,210
Greatest Result	375.34	377.99	6,086	7,494	0.78	0.84	0.65	3.63	92,296	114,260

8.2.6 Potential impacts

The final void water balance indicates that the water level will initially rise steeply, but reach an average equilibrium over time. The void lake will not overtop and will act as a groundwater sink, and is therefore not expected to impact nearby surface water bodies because it would be effectively isolated from the surrounding environment.

The salinity in the lake will increase over time. No groundwater will flow from this lake towards the creek, which offers benefit in terms of not increasing salinity in Sandy Creek. However, the void may potentially develop into a stratified lake which may lead to anoxic conditions. Water quality issues related to a confined stratified hypersaline lake have not been assessed at this stage as this would be appropriate as part of a detailed mine closure plan.

The final landform has been designed to eliminate two voids and minimise the scale of the third mining void. An unavoidable impact will be the formation of an isolated saline lake that cannot be eliminated at an economically viable cost. As no saline groundwater will migrate from the lake, the final void impacts are considered acceptable for the local surface water environment.

8.2.7 Management measures

The final landform has been designed to minimise the formation of void lakes at the mining areas. Due to economic constraints on the Project a void lake is unavoidable in the south eastern end of mining area B. This void will not overtop and will act as a net groundwater sink. Given that the final landform has been designed to minimise void lake formation as far as economically practical, further mitigation measures are not considered necessary.

9. Conclusions

The water management system presented in this report has been designed to segregate different water types and provide operational flexibility to ensure the mine can operate over the proposed 21-year mine life under a range of climatic conditions.

Clean water from undisturbed catchments will be diverted around the mine to Sandy Creek and Laheys Creek as much as practical. This will assist to maintain flows in the creek system. Runoff from disturbed areas, such as overburden emplacement areas, will be directed to a number of sedimentation dams strategically placed throughout the mine to allow settling. Captured water will be either reused on-site to supplement a site water deficit or released to Sandy Creek and Laheys Creek when water quality criteria have been met. Sedimentation dams will overflow to the creek system when the design storm criteria is exceeded.

Mine runoff and groundwater seepage captured in-pit will be pumped to mine water dams. Runoff from the CHPP, coal stockpiles and rail loading facilities will be captured in infrastructure storage dams. This water potentially contains suspended solids, salts and other materials. These classes of water will be used to meet site demands as a priority over raw water and will not be released to the creek system under any circumstances.

The water balance for the Project has been analysed to predict annual runoff volumes, identify likely water deficits and surpluses, and quantify possible storage overflows. GoldSim software was used to develop a water balance model that simulated expected operations at various mine stages (Years 1, 4, 12, 16 and 20), using historical daily rainfall and evaporation data.

It is predicted that the Project will increase surface runoff compared to the pre-mining conditions. However, this increase will be offset by the capture, reuse and evaporation of runoff in the site water management system. The water balance modelling predicted that the surface water flow regime downstream of the site will generally slightly decrease for dry years, but generally increase for median and wet years. The increase in median wet years may be attributed to clean water releases / diversions and water captured in sedimentation dams being returned to the creeks in wet conditions when it is not required by the mine. The median flow downstream of the site is predicted to increase by between 4% and 10% during operations.

The water balance modelling suggests that site demands cannot be met by harvesting on-site water. Water deficits would occur throughout the life of the mine under median climatic conditions, and imported water will be required to make up this deficit. Imported water requirements peak in Year 20 when production rates and demands are high but groundwater seepage has dropped off. CHC's current water entitlements of 3,311 ML/a from the Macquarie and Cudgegong Regulated River Water Source are adequate to meet requirements for a 10th percentile dry rainfall year throughout the life of the mine, assuming a full allocation of high security entitlements.

In Year 20 there is a low probability that current entitlements may not be adequate to meet demands for a very dry year (i.e. drier than those experienced in the last 111 years). If very dry climatic conditions are experienced in the final years of mining, greater water economies or alternate water supply sources would be required to maintain mining operations at full production. A conservative sensitivity test of lowest recorded river flows coinciding with the peak demand year 20 was undertaken which concluded that there is a 21% probability of a water deficit at year 20 under these conditions. However, the peak predicted deficit of 334ML is only 13% of the estimated CHPP demand for year 20.

The final landform has been designed to minimise the formation of void lakes at the mining areas. Due to economic constraints on the Project a salt lake is unavoidable in the south eastern end of mining area B. This void will not overtop and will act as a groundwater sink and therefore will not impact on the surface water environment. Given that the final landform has been designed to minimise void lake formation as far as economically practical, further mitigation measures are not considered necessary.

10. References

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Appendix F

Cobbora Holding Company water entitlements

Cobbora Holding Company water entitlements

Table F-1 Cobbora Holding Company Entitlements – Water Management Act 2000 Approvals

No	WMA Approval Number	Kind of Approval	Status	Water Source	Work Type	Description	No. of Works	Location (Lot/DP)	Reference Number	WAL Number
1	80CA715225	Water Supply Works And Water Use	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 6, DP 754317	80AL715224	29138
2	80CA715385	Water Supply Works And Water Use	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 68, DP 750751	80AL715384	28898
3	80WA702573	Joint Water Supply Works	Current	Macquarie And Cudgegong Regulated Rivers Water Source	Diversion Works - Pumps	Pumping Plant	1	Lot 3, DP 854204	80AL702570	10438
4	80WA704498	Water Supply Works	Current	Macquarie And Cudgegong Regulated Rivers Water Source	Diversion Works - Pumps	380mm Axial Flow Pump	2	Lot 38, DP 750780	80AL705452	27785
5	80WA706213	Basic Rights	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Extraction Works GW	Bore	1	Lot 101, DP 754305	N/A	N/A
6	80WA706331	Basic Rights	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 33, DP 754305	N/A	N/A
7	80WA706332	Basic Rights	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 31, DP 754305	N/A	N/A
8	80WA706347	Basic Rights	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 90, DP 754301	N/A	N/A
9	80WA706945	Basic Rights	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 14, DP 249194	N/A	N/A
10	80WA707153	Basic Rights	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 29, DP 754305	N/A	N/A
11	80WA707171	Basic Rights	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Extraction Works Gw	Artesian Bore	1	Lot 6, DP 754302	N/A	N/A
12	80WA707186	Basic Rights	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 29, DP 754305	N/A	N/A
13	80WA707249	Basic Rights	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 3, DP 802679	N/A	N/A
14	80WA707250	Basic Rights	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 100, DP 754301	N/A	N/A
15	80WA708894	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 18, DP 754317	N/A	N/A
16	80WA710715	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore		Lot 2, DP 253275	N/A	N/A
17	80WA711150	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore		Lot 22, DP 613344	N/A	N/A
18	80WA711946	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 4, DP 253275	N/A	N/A
19	80WA712591	Water Supply Works/Bore	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 14, DP 248947	N/A	N/A
20	80WA712863	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 4, DP 253275	N/A	N/A
21	80WA713096	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 37, DP 754334	N/A	N/A
22	80WA713130	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 22, DP 613344	N/A	N/A
23	80WA713174	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 25, DP 754334	N/A	N/A
24	80WA713205	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 22, DP 613344	N/A	N/A
25	80WA713436	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 2, DP 839623	N/A	N/A
26	80WA713709	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Well	1	Lot 2, DP 1041071	N/A	N/A
27	80WA714127	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 21, DP 754317	N/A	N/A
28	80WA714326	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 21, DP 754289	N/A	N/A
29	80WA714328	Basic Rights	Current	Lachlan Fold Belt MDB Groundwater Source	Extraction Works Gw	Bore	1	Lot 21, DP 754289	N/A	N/A

Table F-2 Cobbora Holding Company Entitlements – Water Management Act 2000 Access Licenses

No	WMA WAL number	Category [Subcategory]	Dept. Reference No.	Status	Water Source	Tenure Type	Water Sharing Plan [Zone]	Share Components (units or ML)	Extraction Times or Rates	Nominated Work Approval(s)
30	28898	Aquifer	80AL715384	Current	Lachlan Fold Belt MDB Groundwater Source	Continuing	NSW Murray Darling Basin Fractured Rock Groundwater Sources	82	Subject to conditions water may be taken at any time or rate	80CA715385
31	29138	Aquifer	80AL715224	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Continuing	NSW Murray Darling Basin Fractured Rock Groundwater Sources	188	Subject to conditions water may be taken at any time or rate	80CA715225
32	29462	Aquifer	80AL707460	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Continuing	NSW Murray Darling Basin Fractured Rock Groundwater Sources	350	Subject to conditions water may be taken at any time or rate	80CA707461
33	29554	Aquifer	90AL822441	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Continuing	NSW Murray Darling Basin Fractured Rock Groundwater Sources	486	Subject to conditions water may be taken at any time or rate	90CA822442
34	29478*	Aquifer	80AL707494	Current	Gunnedah - Oxley Basin MDB Groundwater Source	Purchase in progress	NSW Murray Darling Basin Fractured Rock Groundwater Sources	150	Subject to conditions water may be taken at any time or rate	80CA707495
35	10438	Regulated River [High Security]	80AL702570	Current	Macquarie And Cudgegong Regulated Rivers Water Source	Continuing	Macquarie And Cudgegong Regulated Rivers Water Source [That part of the Water Source Upstream of the Upper Limit of Lake Burrendong]	1,000	Subject to conditions water may be taken at any time or rate	80WA702573
36	27785	Regulated River [High Security]	80AL705452	Current	Macquarie And Cudgegong Regulated Rivers Water Source	Continuing	Macquarie And Cudgegong Regulated Rivers Water Source [That part of the Water Source Upstream of the Upper Limit of Lake Burrendong]	2,311	Subject to conditions water may be taken at any time or rate	80WA704498
37	34440	Unregulated River	80AL718191	Current	Lower Talbragar River Water Source	Continuing	Macquarie Bogan Unregulated and Alluvial Water Sources	1,737	Subject to conditions water may be taken at any time or rate	80CA718192
38	34444	Unregulated River	80AL718189	Current	Lower Talbragar River Water Source	Continuing	Macquarie Bogan Unregulated and Alluvial Water Sources	43	Subject to conditions water may be taken at any time or rate	80CA718190