

Wandoan Coal Project

Addendum to water supply and water management technical report

September, 2009

Wandoan Joint Venture



Parsons Brinckerhoff Australia Pty Limited ABN 80 078 004 798

Level 4, Northbank Plaza

69 Ann Street

Brisbane QLD 4000

GPO Box 2907

Brisbane QLD 4001

Australia

Telephone +61 7 3854 6200

Facsimile +61 7 3854 6500

Email brisbane@pb.com.au

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Author: Michael Batchelor

Signed:

Reviewer:

Signed:

Approved by:

Signed:

Date: September 2009

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Executive summary

The Addendum to the Water Supply and Management technical report results from a number of refinements made to the EIS assessment studies in response to submissions on the EIS, and to account for modifications and refinements to the Project since publication of the EIS.

Existing environment

Streamflow and flooding

Since completion of the EIS, NRW (now DERM) completed a site inspection of the Wandoan Coal Project Area, and advised which drainage features have been determined as watercourses under the *Water Act 2000*. The list of structures requiring *Water Act 2000* approval has been updated following this assessment.

Additional ground survey data has been incorporated into the hydraulic flood models, and the flood inundation assessment over the eastern MLA areas has been extended upstream and downstream. Historical flood observations have been collated from local landholders and compared to modelled flood levels in these events. The results show that within the limitations of the available data, the flood models are a reasonable representation of observed flood conditions in these events.

Fluvial morphology

For the Supplementary EIS, no further studies were conducted for fluvial morphology. All reference regarding fluvial morphology should be directed to the EIS Volume 1, Chapter 11 Water Supply and Management and associated technical reports.

Water quality

Water quality assessment – the proposed network of operational monitoring points and discharge points has been revised to reflect changes to the Project. Additional background water quality monitoring was made available, and this has been included in the assessment of existing ambient water quality.

Water use

In addition to the licensed water users identified in the EIS, it is acknowledged that several other landholders take groundwater and surface water supplies for stock and domestic supplies (for which a licence is not required). Further details of the surface water impact assessment have been presented to describe the potential impact on seasonal variation and low flows in watercourses. Low flows are of particular interest to stock and domestic users as they can be important for filling waterholes during dry periods.

Potential impacts and mitigation measures

Site water management system

A number of changes have been made to the number, size and location of water management dams to cater for changes made to the mine layout. Under the revised mine layout and schedule, twelve potential discharge points from the MLA areas are nominated.

The historical simulation water balance assessment has been updated for the revised water management system layout, and additional details have been provided on downstream impacts.

The conclusions of the study are unchanged, but the following revised details and additional findings are drawn from the model results:

- the total flow from sediment dams make up a relatively small proportion of flows near the MLA boundary. By Year 30 of the Project, they will make up 4.1% of mean annual flows in the creeks immediately downstream near the MLA boundary. At Duck Creek, where the proportion of flow is greatest, overflows will make up 33% of total mean annual catchment flow near the MLA boundary. At 9 km downstream, at the Horse Creek/Duck Creek confluence, the sediment dam overflows would make up only 0.4% of the mean annual Horse Creek flow
- if extreme wet weather occurs in later years of the mine development (when the area of disturbance and open pits is largest), large water storage volumes and/or reuse will be critical to managing the risk of interruptions to mining. Significant water storage is likely to be available in disused pits in the vicinity of the Mud Creek and Summer Hill pits for this purpose, alternatively, a high capacity water pipeline could be provided to rapidly deliver water from the pits for use in the CPP
- during historical wet periods, up to 1,440 ML of runoff could be expected to accumulate in the larger operational mine pits. However, for 90% of the modelled climate record, surface water accumulation in the operational pits would be minimal. Operation of the proposed pit/process WMS would see the pits dewatered to in-pit storage levels that would see mining interruptions kept to an acceptable frequency
- the adopted environmental dam sizes have been chosen to provide a reasonable level of pit availability without spills from the site under modelled historical climatic conditions. The volume adopted to meet these operational criteria is in excess of the 10% Annual Exceedance Probability (AEP) Design Storage Allowance (DSA) volume recommended in the DME Technical Guidelines (1995) (and in most cases the 1% AEP DSA – the requirement for a high hazard dam)
- during wet periods similar to the wettest on record, up to 18,000 ML of water storage in addition to the proposed environmental dams could be required to manage pit/process water from throughout the site without significant off-site discharges (for the Year 30 pit layout). If the quality of overburden runoff is such that water captured in sediment dams is unsuitable for release except in larger flows, this volume requirement increases to 33,000 ML. A number of potential storage locations are likely to be available to temporarily store this excess water, in inactive mine pits
- the mine and associated WMS is expected to reduce mean annual flows immediately downstream of the MLA areas by approximately 3.2% by Year 30 of the Project. The greatest impact is on the Mud Creek and Spring Creek catchments, where mean annual flows will be reduced by 13.4% and 13.8% respectively. Further downstream on Juandah Creek, just upstream of the Horse Creek/Juandah Creek confluence where the nearest surface water entitlement (a harvesting entitlement) is held, the total impact is reduced to a decrease of 2.3% of mean annual flow by Year 30 of the Project
- the modelling indicates the Project will cause almost no change to the seasonal variation of average median flow in the receiving waters. A very small change in some low flow spell statistics is expected. The number of flow spells for Spring Creek and Mud Creek will decrease slightly and their mean duration will increase slightly as a result of the Project. The largest impact is on Spring Creek where there is a reduction in the number of flow spells of 16% and a corresponding increase in low flow spell duration of 19%. At Mud Creek, the change in these statistics is a reduction of 2% and an increase of

2% for the frequency and duration respectively. The impacts on flows in other creeks other creeks is minimal.

The proposed mine WMS design will be refined and optimised during detailed design and the ongoing operations of the Project.

Water quality monitoring

Due to modifications and refinements of pit layouts and scheduling, additional water quality monitoring points are recommended. Water quality monitoring parameters and trigger values have also been reviewed following submissions on the EIS and modifications and refinements of pit layouts and scheduling.

Flooding and diversions

The former NRW (now DERM) has confirmed that during licensing, proposed stream diversions will be assessed against the NRW Central West Regional Office guideline document 'Watercourse Diversions – Central Queensland Mining Industry'. During the detailed design and licensing process, the diversion layout will be refined, and the design flow conditions compared to those occurring locally, with a view to ensuring the diverted section of creek maintains similar geomorphology to other streams in the vicinity.

The impact of the proposed diversion designs on flood levels was investigated using the updated flood models. The results of the modelling confirmed that at the Woleebee Creek diversion, peak flood flows could potentially increase due to a loss of flood storage, and consequently, downstream peak flood levels in Juandah Creek could increase by up to 300 mm during the 1% AEP design flood. Mitigation measures to reduce the potential impact of the operation on off-site flood levels have also been investigated, and it has been demonstrated that effective mitigation measures can be incorporated into the creek diversion design of Woleebee Creek.

At other locations, the modelling of the diversion design concept showed limited upstream increases in flood levels. The impact on neighbouring infrastructure will be reassessed in detail as part of the Water Licensing process.

Potable water supply and sewage

The projected potable and sewage water demands have been reviewed in consultation with Western Downs Regional Council (WDRC). The peak operational and construction demands are essentially unchanged from those quoted in the EIS.

The conceptual potable water supply and wastewater disposal system conceptual designs have been developed further in consultation with WDRC to ensure that existing residual capacity in these systems is maintained.

More detailed consideration has been given to the irrigation of treated effluent from the upgraded Wandoan wastewater treatment plant to reduce discharges to nearby Juandah Creek. Modelling of the water and nutrient balance for the proposed effluent irrigation sites has been undertaken using the MEDLI software. The results demonstrate that high sustainable irrigation use rates are achievable if sufficient wet weather effluent storage is provided.

Details of the effluent irrigation can be found in the Wandoan effluent reuse study, undertaken using MEDLI modelling, included as Attachment D.

Construction raw water supply

Further investigations of water supplies available on the MLA areas suggest it is likely that a large proportion of the construction water demand could be satisfied from existing on-site surface water dams. Water from this and other on-site sources will be used as a priority. However, as surface water supplies are rainfall dependent, to ensure that construction is not interrupted by the unavailability of water, a bore will be constructed into the Precipice Sandstone aquifer of the Great Artesian Basin (GAB) on the eastern edge of the MLA 50230. This bore will only be used as a last resort, if other supplies are not available. Ownership of the bore will be transferred to WDRC to provide additional potable water supply capacity, once the operational raw water supply pipeline is available.

Operational raw water supply

The Project water demand estimates have been refined in light of changes to the production schedule and unit water consumption. Water captured in the site water management system will be prioritised for haul road watering. The net result is that while total site water requirements have increased slightly, the overall imported raw water demand is unchanged.

If CSM water is selected as the raw water source, small quantities will need to be treated to allow its use for some purposes, such as vehicle wash down and fire fighting. A small reverse osmosis plant will be provided for this purpose.

1. Introduction

1.1 Background

1.2 Scope

This report is an Addendum to the overall water supply and management technical report contributing to the Supplementary Environmental Impact Statement (EIS) for the Wandoan Coal Project (Project). This report provides the additional background details addressing those components of the EIS Terms of Reference relating to water supply, and management of surface water on the MLA areas.

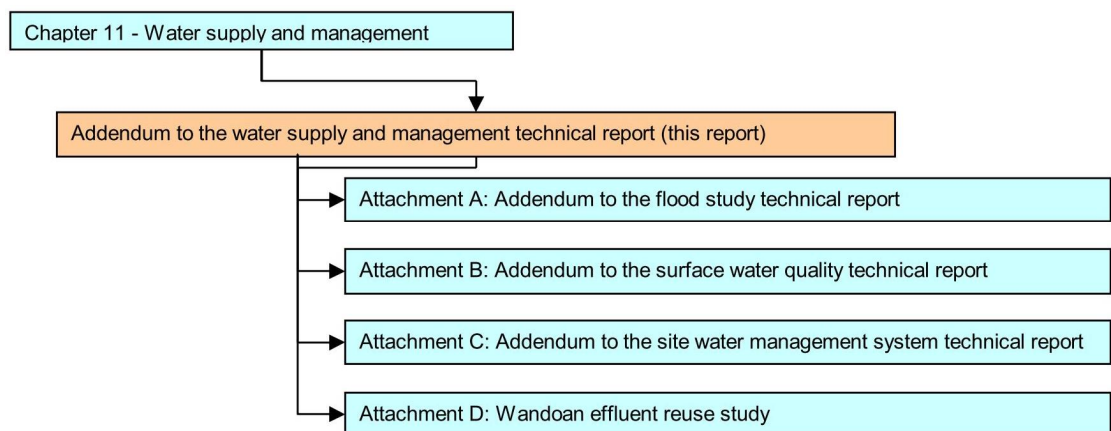


Figure 1-1: Relationship between various water technical reports

This addendum report addresses issues raised in submissions on the EIS, and revises the assessment where the Project description has been modified since EIS publication. Chapter 6 Project Operations of the Supplementary EIS outlines the modifications and refinements to the Project, with the points below outlines how the Project refinements have influenced surface water issues.

Water management

- Water quality assessment has been revised with additional background water quality monitoring made available, and included in the assessment of existing ambient water quality. The proposed network of operational monitoring points and discharge points has also been revised to reflect changes to the proposed pit layouts.
- Conceptual water management system design has incorporated a number of changes including the number, size and location of water management dams to cater for changes made to the mine layout.
- Historical simulation water balance assessment has been updated for the revised water management system layout, and additional details have been provided on downstream impacts. An indicative assessment of site water quality has also been prepared for comparison with the proposed licensed receiving water quality limits and water quality objectives.

- Flood impact assessment has incorporated additional ground survey data into the hydraulic flood models, and the flood inundation assessment over the eastern portion of the site has been extended upstream and downstream. Historical flood observations have been collated from local landholders and compared to modelled flood levels in these events to validate the model. Mitigation measures to reduce the potential impact of the operation on off-site flood levels have also been investigated.

Water supply

- The Project water demand estimates have been refined in light of changes to the production schedule and unit water consumption.
- The potable water supply and wastewater disposal system conceptual designs have been further refined to improve their performance.

2. Relevant legislation and guidelines

In undertaking these assessments, the key relevant Acts are the *Water Act 2000* (Water Act) and the *Environmental Protection Act 1994* (EP Act). Other applicable legislation includes:

- Water Regulation 2002
- Environmental Protection Policy (Water) 1997
- Water Resource (Great Artesian Basin) Plan 2006
- Great Artesian Basin Resource Operations Plan 2006
- Water Resource (Fitzroy Basin) Plan 1999
- Fitzroy Basin Resource Operations Plan 2006
- *Water Supply (Safety and Reliability) Act 2008*.

2.1 Water Act 2000

2.1.1 Structures requiring approval under the Water Act

Construction of a number of structures will necessitate disturbance to the bed and banks of declared watercourses on the MLA areas, and consequently licensing under the Water Act. Details of these structures will be finalised close to their respective construction dates and submitted to DERM (formerly NRW) with an application for a Riverine Protection Permit and/or Water Licence application. The currently expected licensed works and their approximate construction dates are listed below and shown in the revised Figure 3-5. The list has been revised now that NRW has confirmed which streams have been determined watercourses under the Water Act.

Table 2-1: Works potentially requiring approval

	Purpose	Watercourse	Approximate year of construction
1	Rail crossing	Frank Creek	-1
2	Haul road crossing	Frank Creek	3
3	Conveyor, access and dragline walk road crossing	Woleebee Creek	4
4	Haul road crossing	Woleebee Creek	9
5	Conveyor, access and dragline walk road crossing	Mud Creek	9
6	Haul road crossing	Spring Creek	10
7	Stream diversion A at Turkey Hill Pit	Spring Creek/Unnamed Creek	10
8	Stream diversion B at Summer Hill Pit	Mount Organ Creek/Unnamed Creek	18
9	Stream diversion C at Mud Creek Pit	Mud Creek/Unnamed Creek	18

	Purpose	Watercourse	Approximate year of construction
10	Stream diversion F at Woleebee Creek Pit	Woleebee Creek/Wandoan Creek/Blackant Creek	9
11	Stream diversion G at Leichardt Pit	Frank Creek	15

The EIS identifies a number of proposed levees for flood protection. The authorisation of levee banks on mining tenements falls under the jurisdiction of the Environmental Protection Act. However where they form plugs for the existing watercourses, some levees may be incorporated into the licensing of the watercourse diversions, and would be assessed under the *Water Act 2000* and the *Integrated Planning Act 1997*, in negotiation with DERM.

2.1.2 Referable dams

Since July 2008, referable dams have been legislated under the *Water Supply (Safety and Reliability) Act 2008*. The exact number and design details of referable dams will not be finalised until the detailed design stage and during operations of the Project. DERM will be consulted once designs are finalised. A population at risk assessment will be carried out for each dam which is more than 8 m high and has:

- a storage capacity of more than 500 ML
- a storage capacity of more than 250 ML and a catchment area more than three times the maximum surface area of the dam at full supply level.

The final configuration of the site dams will be established during later design stages and during mine operations, and will depend on the availability of construction materials and the relative costs of excavation and embankment construction. Under the currently proposed mine site Water Management System (WMS), several of the proposed environmental dams meet the storage and catchment criteria that define when a failure impact assessment is required. The number and characteristics of the currently proposed dams are described in detail in the Addendum to the Site WMS Technical Report in Attachment C.

There are also several sediment dams and proposed flood levees potentially greater than 8 m high, which will have the potential to store large volumes of water, and thus meet the criteria for undertaking a failure impact assessment. The levees are described in more detail in the Addendum to Flood Study Technical Report in Attachment A.

The floodplain areas downstream of these structures are sparsely populated and it is unlikely any would be deemed category two dams. A detailed assessment will be carried out following detailed design of any dam and levee structures which meet the relevant assessment criteria.

2.1.3 Water Resource (Fitzroy Basin) Plan 1999 (amended August 2005)

The Water Resource (Fitzroy Basin) Plan 1999 is in the process of review and a replacement Water Resource Plan is anticipated to be finalised by September 2010.

2.1.4 Fitzroy Basin Resource Operations Plan (ROP)

Environmental management on mining tenements

The EIS noted a provision in the Fitzroy Basin Resource Operations Plan (ROP) for licences to be granted on mining tenements to contain runoff or seepage which would otherwise flow from the site, to allow for suitable storage and treatment. The EIS does not reference any proposed take of water from a watercourse that would be required to remove mine site seepage or runoff that is unavoidably discharged or proposed to be discharged into a watercourse. It is unlikely that this provision (section 6.1.3 of the ROP) would be triggered for a new mine development and the DERM (formerly NRW) recommends the removal of the reference to the grant of these licences from the EIS.

The current exemption in the ROP for the take of overland flow for activities authorised under a mining tenement applies to this Project. However the Water Resource (Fitzroy Basin) Plan 1999 is currently under review. This review may result in possible amendments to the overland flow provisions during the life of the mine development. The WJV will consult with DERM prior to taking overland flow to ensure appropriate approvals/exemptions are in place.

2.1.5 Water resource (Great Artesian Basin) Plan 2006

2.1.6 Great artesian basin resource operations plan

2.1.7 Existing land and water management plans

2.1.8 Guideline for watercourse diversions

DERM (formerly NRW) has confirmed that during licensing, watercourse diversions will be assessed against the NRW Central West Regional Office guideline document 'Watercourse Diversions – Central Queensland Mining Industry'. During the detailed design and licensing process, each watercourse diversion layout will be refined, and the design flow conditions compared to those occurring locally, with a view to ensuring the diverted section of creek maintains similar geomorphology to other streams in the vicinity.

2.2 Environmental Protection Act, Environmental Protection Regulations, and Environmental Protection Policy Water

The Environmental Protection Act and Regulations govern dams that contain hazardous waste, as defined under the Act and Regulations. For dams containing hazardous waste, the Mandatory Reporting Level (MRL) is defined as the available storage volume below the spillway crest, equivalent to the lower of the AEP (design risk) 72 hour storm or the AEP wave allowance, at which DERM (formerly EPA) must be advised.

In estimating the MRL, it is appropriate to adopt consistent design volumetric runoff coefficients to reflect the likelihood of wet antecedent rainfall conditions for all design event durations. As a result, the 72 hour duration design storm results in a larger volume of runoff than for shorter duration storms, as the total rainfall depth is always larger. Details of the design rainfalls and MRLs are provided in the Addendum to the Site WMS Technical Report in Attachment C.

3. Existing environment

3.1 Climate

3.1.1 Rainfall

3.1.2 Evaporation

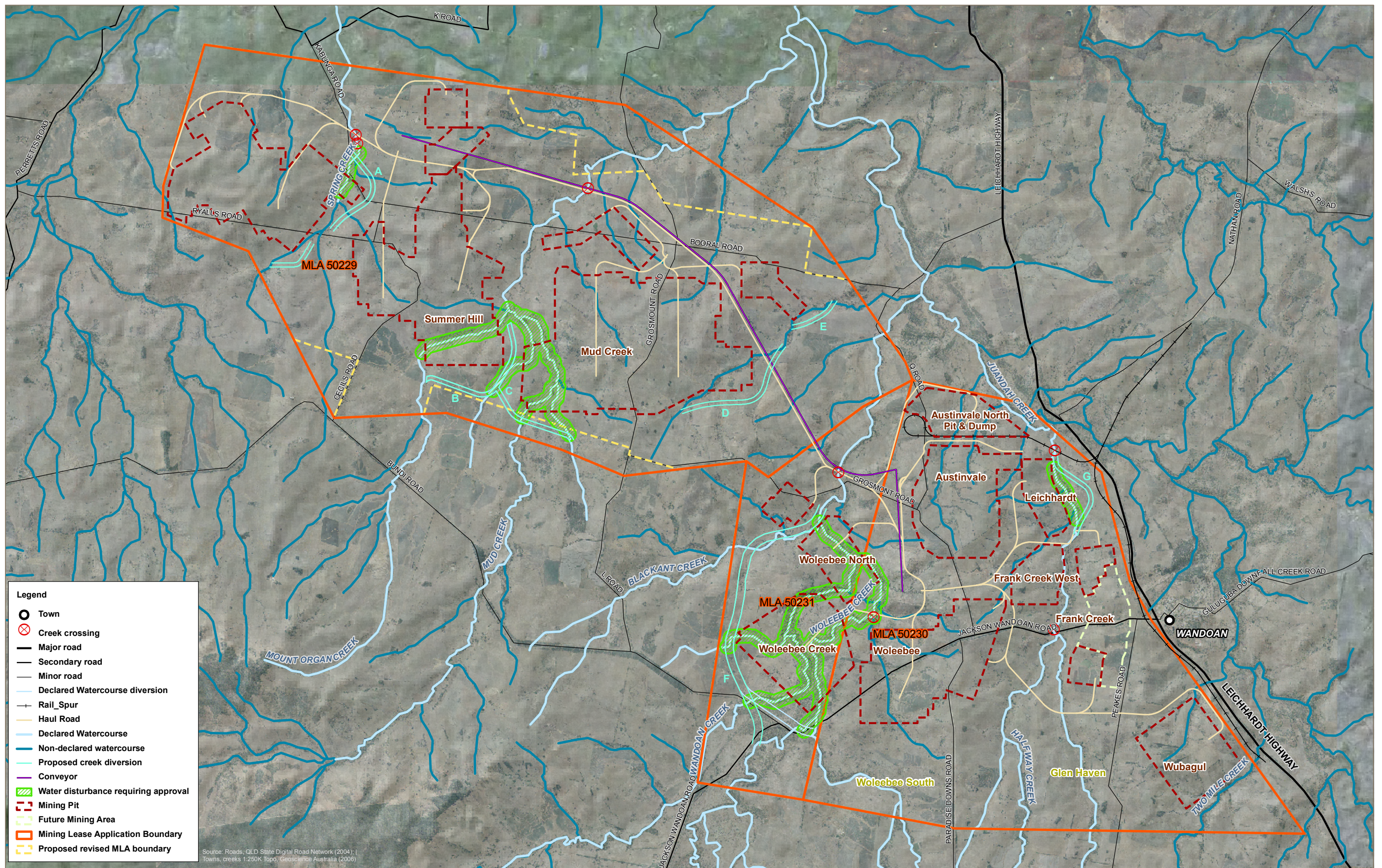
3.2 Existing catchment characteristics

3.2.1 Local catchments

Since publication of the EIS, NRW completed a site inspection of the Wandoan Coal Project MLA areas, and advised which drainage features have been determined as watercourses under the *Water Act 2000*. The revision to Table 3-1 below lists these watercourses, which are also highlighted in the revision to Figure 3-5.

Table 3-1: Streams and Declared Watercourses crossing the MLA areas

Stream	Flows to	Downstream discharge point	Watercourse (Y/N)	Catchment area (ha)	Length (km)
Duck Creek	Horse Creek	Y	N	2,200	10
Un-named Creek	Horse Creek	Y	N	2,300	23
Un-named Creek	Horse Creek	Y	N	2,270	23
Spring Creek	Horse Creek	Y	Y	6,730	67
Mount Organ Creek	Juandah Creek	N	Y	11,370	114
Mud Creek	Juandah Creek	Y	Y	17,530	175
Un-named Creek	Juandah Creek	Y	N	2,410	24
Un-named Creek	Juandah Creek	Y	N	1,040	10
Blackant Creek	Wandoan Creek	N	Y	3,740	37
Wandoan Creek	Woleebee Creek	N	Y	11,640	116
Woleebee Creek	Juandah Creek	Y	Y	75,330	753
One-Arm Man Creek	Woleebee Creek	N	Y	2,260	23
Halfway Creek	Frank Creek	N	N	2,290	23
Frank Creek	Juandah Creek	Y	Y	9,120	91
Two Mile Creek	Juandah Creek	Y	N	2,450	7



0 2 4 6 8 10
Kilometres



Figure 3-1
Watercourses on the Wandoan MLAs

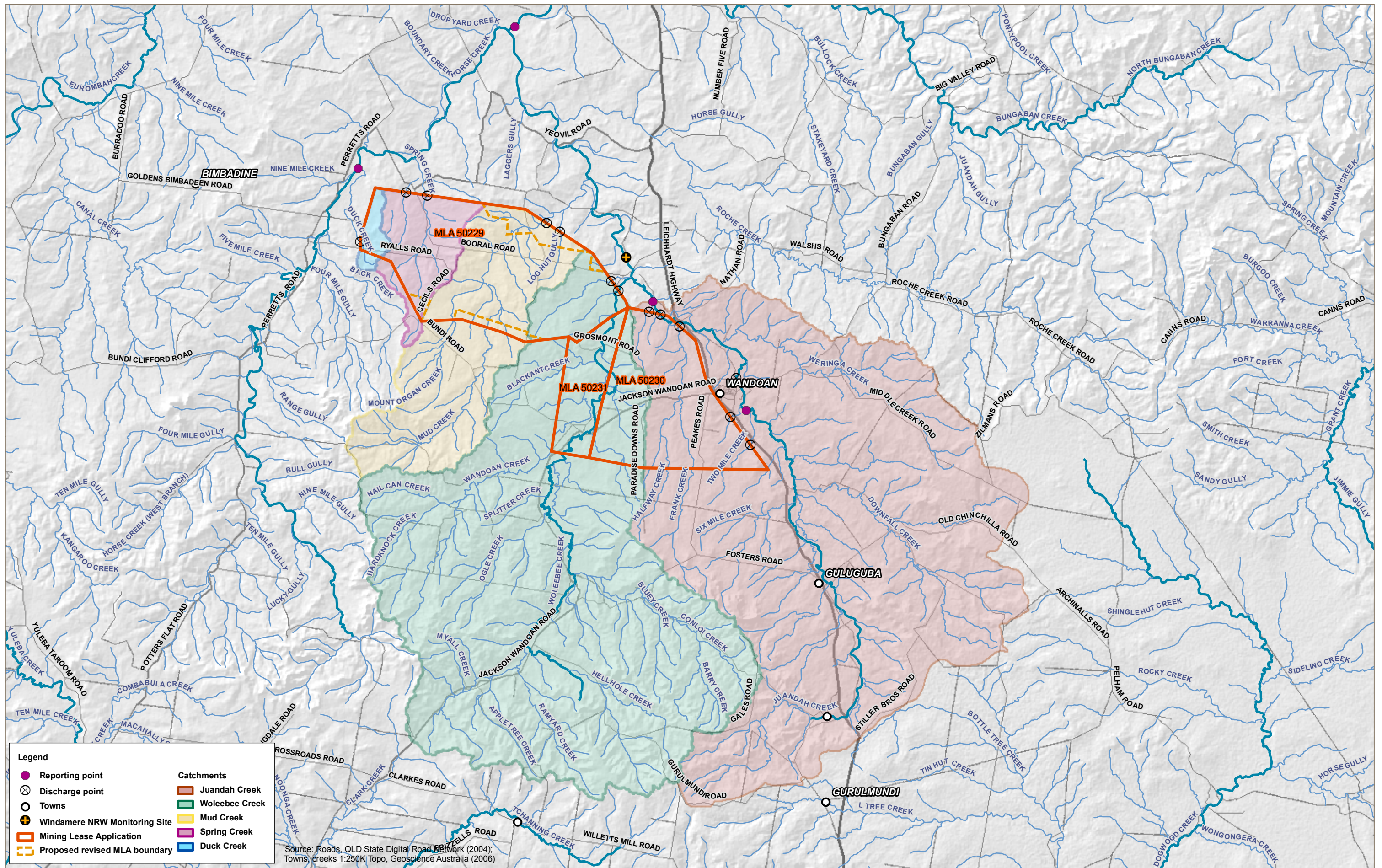


Figure 3-2
Major drainage systems crossing the MLAs

3.2.2 Regional catchments

3.3 Streamflow

Based on the results of historical simulation runoff and water balance modelling, details of seasonal distribution of low flows are provided in the Addendum to the Site WMS Technical Report given in Attachment C. Other low flow descriptors such as the mean duration and frequency of low flow spells are also provided for comparison with post-mine flow conditions.

The results of this analysis indicate there is almost no change to seasonal flow variation and there is a very small change to some of the low flow spell statistics. The number of flow spells for Spring Creek and Mud Creek have decreased slightly and the mean duration has increased.

3.4 Flooding

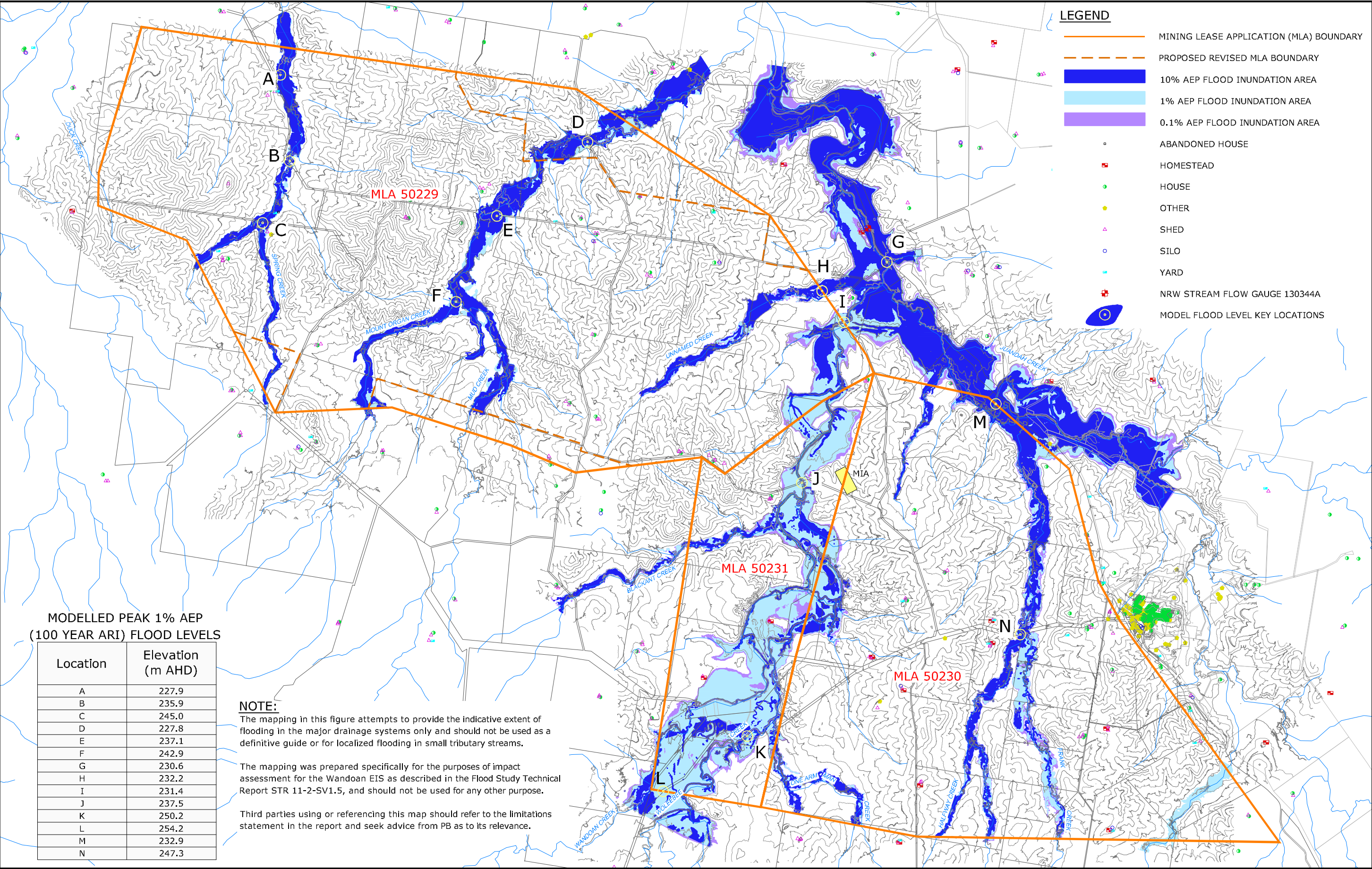
An addendum flood study report has been prepared to expand upon the hydraulic modelling that was performed for the EIS. The hydraulic modelling comprised a MIKE11 model for the creek systems in the eastern MLA areas and separate HECRAS models were constructed for the other creeks. In the revised study, the MIKE11 model was refined and expanded, and a new HECRAS model was constructed for Two Mile Creek in the south-eastern corner of the MLA areas. No changes have been made to the assessments prepared for Mud Creek, Mount Organ Creek or Spring Creek. The results of the modelling are shown in the updated Figure 3-16.

Additional flood level observations have also been obtained from local landholders regarding flooding in the 1983 and 1991 flood events. High water marks observed in these events have been compared to hydraulic modelling results for model verification.

Insufficient continuous rainfall data was available for the 1983 event to allow this event to be modelled, but the observed extents of inundation lie within the range of the 2% AEP and 1% AEP events, as would be expected based on the measured peak flow rate at the DERM (formerly NRW) streamflow gauge of Windamere.

Limited rainfall data was available across the eastern Juandah Creek catchment during 1991 flood event, but this was extended to allow the flood event to be modelled. The MIKE11 model results indicate reasonable matches to the observed extents of flooding for this event.

Full details of the updates to the flood studies are provided in the Addendum to flood study technical report in Attachment A.



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3.5 Stream morphology

3.6 Surface water quality

Additional baseline water quality data has been made available for assessment by the WJV:

- daily log data recorded at generally four hourly intervals and/or during a change in (stream) water level; measuring temperature, conductivity, pH, DO and turbidity parameters, recorded from March 2008 to March 2009. This data builds on the previous data presented in the EIS technical report that assessed water quality data from March 2008 to mid September 2008
- event data sampling undertaken by Ecowise Environmental supplied by the WJV.

These datasets were used to update the description of the background surface water quality. The following changes to the observations reported in the EIS are noted:

- elevated heavy metal levels particularly iron, copper and zinc, with exceedances for chromium, copper, lead, nickel and zinc at some sites compared to ANZECC guidelines
- elevated Chlorophyll a at some sites.

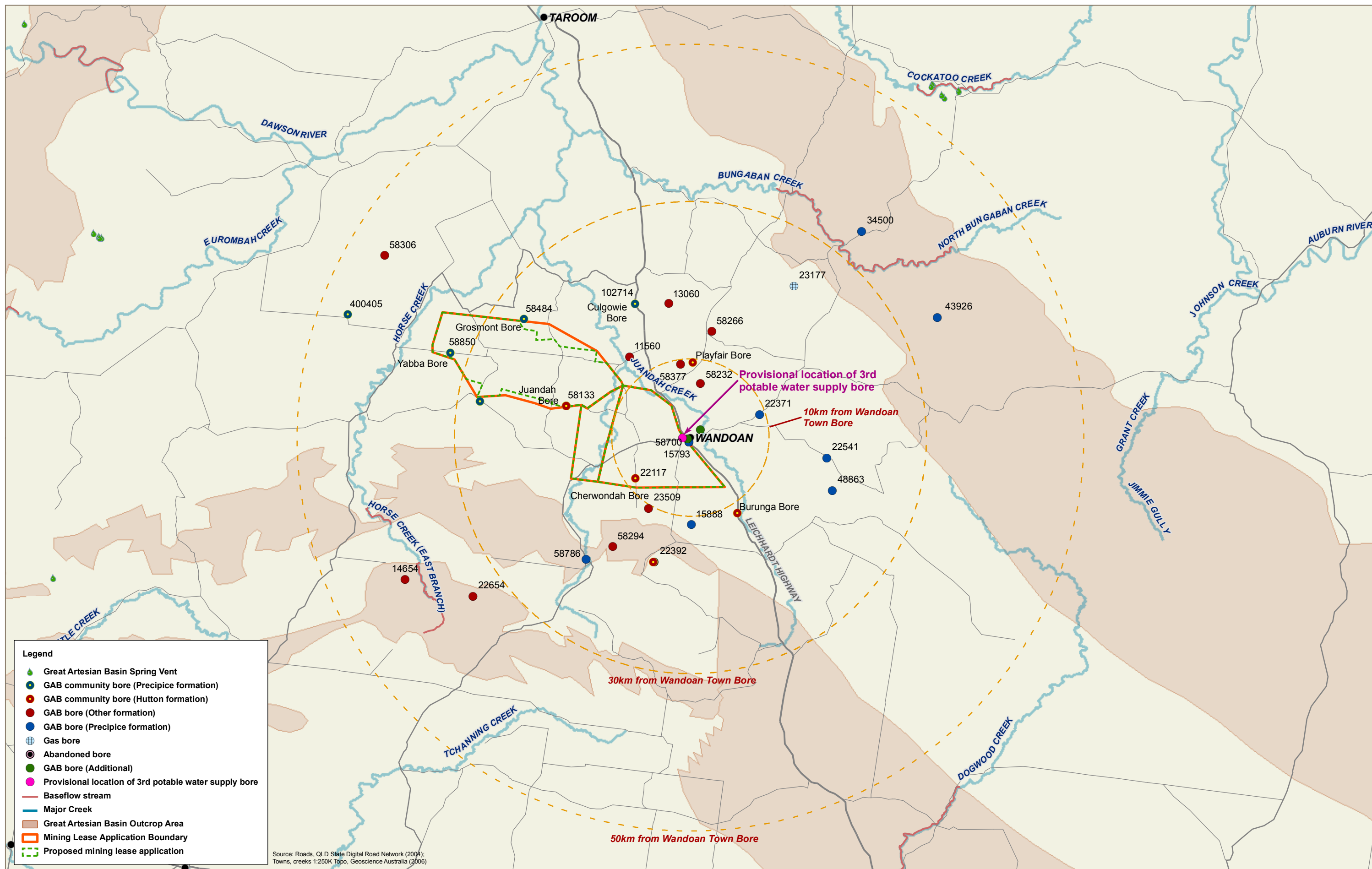
3.7 Groundwater quality

3.8 Surface water use in the catchment

Due to the intermittent nature of streamflow in the area (and the availability of groundwater from the GAB aquifers, there is only limited use of surface water in the immediate vicinity of the Project Area. While a small number of licensed surface water users have been identified in the downstream reaches of watercourses crossing the MLA areas, a number of adjacent landholders are reliant on surface water flows for stock and domestic purposes.

3.9 Groundwater use in the area

Some additional bores into the aquifers of the GAB have been identified since the publication of the EIS. Details of these bores have been added to Figure 3-18. For further information refer to Chapter 10 Groundwater of the Supplementary EIS.



0 5 10 15 20 25
Kilometres



Figure 3-4
Selected GAB bores, springs and baseflow streams near Wandoan

4. Water supplies

4.1 Potable water supplies

The proposed strategy for delivering potable water from the upgraded Wandoan Town Water Treatment Plant (WTP) has been developed further since publication of the EIS. The forecast demands and proposed infrastructure upgrades have been revised following further negotiation with the Western Downs Regional Council (WDRC), as described in the following sections.

4.1.1 Potable water demand

Unit water demands have been agreed with WDRC. While the number and distribution of Project personnel to be accommodated has been refined with recent changes to the manning schedule, the total potable water demands remain essentially unchanged from the original EIS estimate.

The bulk of mine staff will be housed in the accommodation facilities to be constructed at a location remote from Wandoan township. The on-site population of the accommodation facilities will vary over time, and will peak during the second year of construction. During the operational phase, the on-site mine staff population will reduce.

All potable water infrastructure will be upgraded to cater for the maximum expected demand during the construction phase. During the operational phase, this will result in reserve capacity for WDRC to supply other potential customers.

4.1.2 Existing potable water supply and treatment system

4.1.3 Proposed potable water infrastructure upgrades at Wandoan water treatment plant

4.1.4 Infrastructure required

WDRC will own and operate all off-site infrastructure, including upgrades to the WTP, WTP pump station and the potable water transmission pipeline to the mining lease boundary. A valve chamber with isolation valve will be provided at the boundary to the mining lease demarcating the WJV/WDRC interface point. A bulk meter will be installed on the WTP pump station discharge pipework to facilitate measurement and billing of water consumed on the mine site.

The contractor responsible for the accommodation facilities construction will connect internal reticulation to the bulk supply infrastructure from a point near the access road.

New off-site infrastructure will be financed by the WJV, and comprise the following:

- replacement borehole pumps in both Bore 1 and Bore 2 to cater for the design flow. This provides for a level of redundancy in the supply of raw water to Wandoan Town, with the bores acting in a duty and standby capacity respectively

- a new 1,150 m deep, steel lined bore into the Precipice Sandstone aquifer of the Great Artesian Basin (GAB) equipped with a new bore pump. This bore will be constructed by the WJV and used initially for construction water supplies if required as a last resort when there is a lack of available surface water, then handed over to WDRC after the construction period for incorporation into the potable water supply system
- upgrades of the existing WTP by construction of a new treatment chain comprising aeration, flocculation, lamella clarification and filtration. The process will be as per the existing treatment process for water from Bore 1. The upgrade will include the installation of equipment to facilitate the dosing of potable water with fluoride in line with the Queensland Government's policy on fluoridation. The upgraded treatment works will be capable of a throughput of 26 L/s. The block of land adjacent to the existing WTP may need to be purchased to accommodate the envisaged expansion
- upgrading of the existing electrical infrastructure at the WTP to cater for the additional demands of the new treatment chain and transmission pump station, and provision of radio communication between the WTP and accommodation facilities' tanks to facilitate pump control
- a new 450 kL reinforced concrete ground reservoir for bulk storage, sized to store the water demand from the mine for a 24 hour period
- a new dedicated potable water transmission pump station comprising two water pumps (one duty and one standby) with a duty point of 12 L/s at 40 m and a motor control centre, contained in a galvanised pump shed matching the existing pump
- installation of an in-line electromagnetic flow meter on the discharge pipework within the pump station in order to record volumes of water supplied to the WJV to facilitate accurate billing by WDRC
- a new 320 m long 150 mm nominal diameter uPVC bulk potable water transmission pipeline from the WTP pump station to the mining lease boundary. The pipeline alignment will run within the Jackson Wandoan Road reserve to the mining lease boundary, including a crossing of the Leichhardt Highway considering any safety and traffic issues associated with open cut trenches in roads
- an isolation valve chamber constructed at the mining lease boundary to provide separation between WJV and WDRC infrastructure
- on exiting the mining lease boundary, pipelines will be located within road reserves. The proposed pipe-crossing of the Leichhardt Highway will be auger or thrust bored to obviate traffic and safety issues associated with open trench excavation in roads
- an on-site storage capacity of 3.5 days potable water demand will be provided, comprising one day at the WTP and a further 2.5 days at the point of demand on the MLA areas.

The WJV will be responsible for the delivery of potable water from the mining lease boundary to the accommodation facilities and MIA, including:

- a new 10,130 m long 150 mm nominal diameter uPVC bulk potable water transmission pipeline from the mining lease boundary to bulk water storage tanks located between the security building and accommodation facilities. The pipeline alignment will run parallel to the mining lease boundary to the mine access road, and then parallel to the access road to the accommodation facilities

- 3 x 380 kL galvanised steel potable/fire water storage tanks to serve the accommodation facilities. The tanks have been sized to provide 2.5 days storage of potable water in addition to four hours of fire flow at the accommodation facilities
- a pump shed sized to house accommodation facilities reticulation and fire pumps as well as transmission pumps for supplying the MIA storage tanks
- a bank of three variable speed reticulation pumps to supply the accommodation facilities reticulation, an electric fire pump, a diesel fire pump, a booster assembly and all associated pipework and valves sized to supply the probable simultaneous flow rate (PSFR) to the accommodation facilities
- a new 4,420 m long 80 mm nominal diameter uPVC PN12 bulk potable water transmission pipeline from the accommodation facilities pump station to the MIA potable water storage tanks
- a 100 kL galvanised steel potable water storage tank to serve the MIA and CHPP areas. The tank have been sized to provide 2.5 days storage of potable water
- a bank of two variable speed pumps to supply potable water demand at the MIA, CHPP and train loadout.

4.2 Sewage treatment and disposal

The proposed strategy for transferring all sewage from the source to the upgraded Wandoan Town Wastewater Treatment Plant (WWTP) has been further refined. The proposed upgrades have been revised following further negotiation with WDRC, as described in the following sections.

4.2.1 Infrastructure required

Wandoan Town is serviced by a waterborne sewerage reticulation network draining to an existing WWTP. A dedicated system is proposed from the mine site direct to the WWTP, that is, the proposed strategy does not entail using any part of the existing network to deliver mine sewage to the WWTP. Any new accommodation developed within Wandoan will be connected to the existing or expanded sewage reticulation within the township.

The upgraded WWTP has been designed to treat sewage to a Class B recycled water standard in accordance with the Queensland Water Recycling Guidelines (2005). During dry weather, effluent will be disposed by way of irrigation locally and during wet weather, effluent will be stored in additional storage capacity to be constructed as part of the proposed upgrade. Excess wet weather flows will be discharged to Juandah Creek, as is currently allowed by the existing discharge licence.

All infrastructure beyond the MLA area boundary be owned and maintained by WDRC. A valve chamber with isolation valve will be provided at the boundary to the mining lease demarcating the interface point.

A sewerage manhole tie-in point will be provided at the accommodation facilities sewage pumping station. The contractor responsible for the accommodation facilities construction will connect internal accommodation facilities reticulation to this tie in point

All sewage generated at the MIA, CHPP and accommodation facilities will be transferred back to the Wandoan Town WWTP by way of a series of pump stations and rising mains. A

120 kL capacity reinforced concrete storage tank containing an in-line macerater and aeration pump will be provided to store peak flows adjacent to the main transfer pump station. This pump station will deliver flows to the mining lease boundary via a 8,870 m long 150 mm diameter mPVC rising main.

New infrastructure outside of the MLA area boundary and accommodation facilities will comprise the following:

- a mine-dedicated 1,530 m long, 150 mm diameter mPVC rising main from the mining lease boundary adjacent to the existing Wandoan airstrip. The pipeline alignment will run within an existing road reserve from the mining lease boundary, including a crossing of the Leichhardt Highway. The pipeline will be laid in a sleeve under the road, constructed by way of auger/thrust boring, to obviate traffic and safety issues associated with open cut excavation in roads
- a break-pressure manhole at a high point east of the Wandoan Golf Course boundary and a new mine dedicated 810 m long, 225 mm diameter uPVC gravity pipeline from this point to the WWTP, including a crossing of Golden Street
- an isolation valve chamber constructed at the mining lease boundary to provide separation between the WJV and WDRC infrastructure.

Design guidelines and methodology

The following design criteria have been utilised for sizing of the sewage transfer infrastructure:

- Department of Natural Resources and Mines – Planning Guidelines for Water Supply and Sewerage
- average daily waste water generation has been based on 55 litres per capita per day water use at the MIA and CHPP, and 240 (litres per capita per day) at the accommodation facilities with 90% of this being returned to the sewerage system as wastewater
- pumps have been sized based on their ability to pump twice the peak wet weather flow individually. This is more conservative than the Queensland guidelines; however, it results in a minimum sump size for a given inflow, resulting in lower capital construction costs for a marginal additional pump cost
- all rising mains have been designed to have a minimum velocity of 1 m/s to facilitate self cleansing conditions
- all of the pump stations will be located out of the 100 year flood inundation levels, and proximate to access roads and power supply
- all pump stations will be submersible below ground installations, with an elevated motor control centre in a weather proof kiosk. All pumps will be controlled on a simple level transducer that will switch pumps off and on
- all sewage pumping stations will be connected into the emergency power system to achieve compliance with DERM's (formerly EPA's) zero discharge to creek requirement
- the main transfer pump station located at the accommodation facilities has been designed with a storage capability to allow for pumping back to the WWTP at lower than peak wet weather flows, reducing the required rising main diameter and head at which pumps have to deliver. The storage facility will be a below ground covered reinforced

concrete tank fitted with a macerator to reduce chances of pump and pipeline blockages and an aerator pump to aerate sewage whilst in storage to avoid odour problems. This facility is designed to store peak flows and will fill and drain over the period of the day.

Sewage treatment demand

- Based on historic treatment data, the average sewage generated per capita from Wandoan town is 280 L per capita per day, which has been used for flow generation calculations.
- The potable water consumption for the mine site was set as 240 L per capita per day. It has been assumed that 90% of this water will be wastewater and will be returned to the WWTP for treatment. This is a conservative approach which is justified by the largely domestic use envisaged on the mine site.
- Based on the design criteria provided above and population figures utilised for the potable water projections, a maximum average daily dry weather flow (ADWF) of approximately 510 kL has been adopted.
- Peak wet weather flows have been calculated based on a multiplier of 7 x ADWF for the town (based on historical observations at the WWTP) and 1.2 for the mine infrastructure, based on an infiltration of 20% for stormwater.
- A contingency of 15% has been applied to volumetric flows to cover unforeseen growth in population served by the plant.

Pump stations and pipework have been sized for peak staffing levels during the construction phase and operation phases. Downward fluctuations in these numbers will result in spare capacity in the system.

4.2.2 Upgrading of Wandoan WWTP

Adopted process overview

The following process is proposed for the Wandoan WWTP upgrade:

- bar screens (existing) and pumping tank (converted from Imhoff tank) for town wastewater
- bar screens and degritter for mine wastewater
- 2 x buffer tanks
- 2 x Intermittently decanted aeration lagoon (IDAL)/Intermittently decanted extended aeration lagoon (IDEAL) units
- 2.6 x 2,250 m² facultative lagoons (existing)
- effluent overflow on land or via gulley to Juandah Creek (existing)
- 2 x storage lagoons for reuse after chlorination (existing)
- aerobic digester
- expanded sludge drying bed with disposal to landfill.

IDAL/IDEAL process

The intermittently decanted (extended) aeration lagoon process has been successfully operated within Australia, used mainly in cases of wastewater treatment in small townships. The process is similar to a sequential batch reactor (SBR), but works with a continuous inflow and intermittently decanted effluent.

The four hour cycle for this system will go through the following process, with continuous fill:

- two hour aeration
- one hour settling
- one hour decant.

The cycle times could be modified to suit the influent characteristics. Extending or decreasing the cycle times based on the influent characteristics or for process improvements is also possible.

Part of one of the existing ponds will be converted to an intermittently decanted extended aeration lagoon (25% by volume conversion). To prevent short-circuiting, baffles will be installed to ensure that the influent will undergo treatment prior to decanting. This will allow a separate area in the IDAL/IDEAL unit for anoxic/anaerobic reactions. The reactor will also be configured to a width-to-length ratio between 3.5 to 6, for added protection against short-circuiting within the reactor.

Sludge from the aerated lagoon will be aerobically digested in a digester tank and transferred to sludge drying beds by way of pumps for drying. Dried sludge will be disposed to the local landfill as is the current practice

The process design criteria summarised in Table 4-1 have been utilised for the preliminary design of the WWTP upgrade.

Table 4-1: Design criteria for preliminary design of the WWTP upgrade

Process unit	Design criteria
Inlet works (from mine catchment)	18 L/s maximum (from the mine site pump station)
Buffer tanks	2 units To handle total peak wet weather flow of 1600 kL (PWWF) Detention time of 0.75 days at PWWF Internal mixers of 5 W/m ²
Submersible pumps (in balancing tanks)	Capacity = 5 x ADWF (planning guidelines) Capacity = 2.5 x ADWF for each pump
IDAL/IDEAL reactor	2 units Capacity = 3 x ADWF (Planning Guidelines) Q _{in} = 510 kL/d (i.e. ADWF) Width to length ratio = 3.5 to 6 VER = 0.3 SRT = 20 days Internal mixers of 5 W/m ² (non-aerated)

Process unit	Design criteria
	Decanter maximum depth = 0.8 m BOD_{in} = 220 mg/L CBOD estimates = 0.6 kg O₂/BOD (carbon to cell conversions) CBOD estimates = up to 1.5 kg O₂/BOD (endogenous respiration and extended aeration) MLSS (fully mixed reactor) = 2,500 mg/L MLSS (settled sludge) = 4,000 mg/L (85% settled of bottom water level)
Aerobic digester	WAS pumping = ~5% of influent 1 x WAS pump 20% of total TSS represents removal of VSS SRT = 20 days 1 tank O₂ requirements for complete oxidation of BOD contained in primary sludge varies from ~1.6 to 1.9 kg/kg destroyed Oxygen residual should be maintained at 1 mg/L or above under all operating conditions
Sludge drying beds	Paved drying beds % of dry solids after decant = 8% % of dry solids required for disposal = 30% Free water pan evaporation rate = 195 cm/yr Total rainfall = 0.67 m/yr

The following new infrastructure or infrastructure upgrades will be required in order for WDRC to treat and dispose of the projected flows from the proposed mine and associated growth in the town:

- the construction of a new inlet channel and screens for a dedicated supply pipeline from the mine
- the construction of a flow splitter tank and pipework to two buffer tanks
- the construction of 2 x 600 kL interconnected buried reinforced concrete buffer tanks at the inlet to the works containing submersible sewage pumps
- the conversion of a portion of one of the existing ponds to create a two cell continuously fed, intermittently decanted IDAL/IDEAL process plant described above
- the construction of a reinforced concrete sludge digester with sludge pump to the drying beds
- the construction of new drying beds
- provision of mechanical aerators in the converted pond and sludge digester
- the upgrade of the current electricity supply to the WWTP from a single phase to a three phase power supply to suit the demands of the proposed upgrade.

Effluent discharge

The Wandoan WWTP currently treats an average daily flow of 104 kL. Approximately 52 kL of the treated effluent is discharged to Juandah Creek and the balance is pumped to an earth dam at the Council owned showground by way of a 2,300 m long, 75 mm diameter rising main, from where it is used to irrigate the showground fields. Infrastructure exists to pump treated effluent to an earth dam on the adjacent Wandoan Golf Course to use for irrigation; however this is not being used at present. The current discharge licence permits the use of treated effluent for irrigation purposes.

The existing environmental discharge licence criteria for the current Wandoan WWTP are shown in Table 4-2. The water quality release limits appear to conform to Class C recycled water standards, as per the Queensland Water Recycling Guidelines. WDRC personnel report that the existing WWTP generally appears to meet the licence requirements.

Table 4-2: Environmental discharge licence criteria – Wandoan WWTP

Parameter	Unit	Release limit	Limit type
Licensed effluent release volume	kL/day	220	maximum
Five day BOD	mg/L	20	80 th percentile
Suspended solids	mg/L	30	80 th percentile
pH		6.0–8.5	range
E. coli	organisms/100 mL	<1,000	maximum
Total dissolved solids		1,000	maximum
Dissolved oxygen	mg/L	2	minimum

Waste streams from the upgraded plant will include:

- sludge – approximately 33.9 wet tonnes of sludge will be produced annually. The sludge will be pumped from the aerobic digester onto the sludge beds for drying. The dried sludge will be removed to the local landfill site as is the current practice
- treated effluent – treated effluent will be chlorinated and pumped to the showgrounds and golf course for irrigation. The effluent will meet the quality specifications of Class B recycled water.

It is proposed to use the treated effluent to irrigate the Showground and Golf course — a total area of 28.9 ha. Effluent treated to this standard may only be irrigated under controlled conditions. For the golf course, this effectively means during the night time. During wet weather, additional effluent storage will be required to prevent excessive discharge to waterways.

A nutrient and water balance has been carried out using the MEDLI software model to confirm the effluent irrigation would be sustainable at rates approaching 90% reuse. If a 20 ML wet weather effluent storage pond is provided, the following rates can be achieved as shown in Table 4-3.

Table 4-3: Sustainable effluent irrigation rates based on MEDLI modelling

Scenario	Flow (kL/d)	Proposed irrigation area (ha)	Inflows to pond (ML/yr)	Irrigation rate (ML/ha/yr)	Effluent reuse (%)	Overflow (ML/yr)	Average daily overflow (kL/d)
Construction	520	Showground (15.9 ha)	203	6.3	50.7	95	261
	520	Showground and Golf course (28.9 ha)	203	5.6	82.2	34	92
Operation	322	Showground (15.9 ha)	125	6.3	83.0	19	52
	322	Showground and Golf course (28.9 ha)	125	4.0	94.4	5	14

When irrigating on the showground and golf course, it is possible to achieve greater than 90% reuse during operational phases.

In order to achieve the above reuse rates and to comply with normal DERM license conditions and Queensland Water Recycling Guidelines (2005), the following infrastructure and procedural commitments are required from Western Downs Regional Council and the golf course owners:

- controlled access to irrigation areas
- monitoring of shallow groundwater quality in the proposed irrigation areas
- monitoring of soil conditions in the irrigation areas
- development of a Recycled Water Management Plan
- a total of 20 ML of effluent storage to manage wet weather flows
- adequate irrigation equipment to achieve the required irrigation rate on dry days.

4.3 Construction raw water

4.3.1 Construction raw water demand

The total construction water demand of 700 ML is unchanged.

4.3.2 Potential construction water sources – assessment of alternatives

Surface water

Further investigations of water supplies available on the MLA areas and accommodation facilities suggest it is likely that a large proportion of the construction water demand could be satisfied by existing surface water dams. Water from these and other sources within the

MLA areas and adjacent accommodation facilities will be used as a priority. If the period leading up to and during the construction period is relatively wet, it is possible that the entire construction water demand could be drawn from surface water supplies. However, as this supply is rainfall dependent, it is possible that the available yields are well below the site demands.

A new bore into the precipice sandstone aquifer of the GAB

While the alternative construction water supplies described will be used preferentially, the most reliable source is the Precipice Sandstone aquifer of the GAB. As a result, the WJV proposes to establish a new bore into the Precipice Sandstone aquifer at a site adjacent to the Jackson Wandoan Road within the MLA area, as shown in Figure 3-18. If insufficient supplies are available from on site sources, this bore will be used to as a last resort to draw construction water supplies under a temporary water permit issued by DERM. When the operational raw water pipeline has been commissioned, the construction water bore will be handed over to the Western Downs Regional Council as a potable water supply bore to be used in conjunction with its other two bores.

4.4 Operations raw water

4.4.1 Operations raw water demand

While there have been some changes to the production schedule and CHPP unit water use since the publication of the EIS, the total adopted operations raw water demand is essentially unchanged.

The major demand for operations raw water is the CHPP. The estimated unit raw water requirement for the CHPP has increased to approximately 290 L/t ROM. This is the net make-up demand after allowing for the return of recycled decant water from the tailings system. When possible, haul road watering demands will be supplied by the site water management system. Historical simulation of the system suggests with the exception of very dry periods, sufficient yields should be available if the water management system is designed and operated appropriately.

4.4.2 Potential raw water sources

The WJV is no longer pursuing the raw water supply option of coal seam gas associated water sourced from Spring Gully (owned by Origin) and Fairview (owned by Santos) to the west of the MLA areas, as described in the EIS Volume 3 Western Coal Seam Methane (CSM) Water Supply Pipeline.

Raw water will not be obtained from the Precipice Sandstone or Hutton Sandstone aquifers of the GAB for operational purposes.

4.4.3 Raw water quality

The current proposal for the Coal Seam Gas raw water supply option is to deliver raw water to the Project for use in the CPP at 4,000 mg/L TDS. This water is unlikely to be suitable for washing vehicles, fire water or for general wash purposes around the MIA and CPP. A reverse osmosis or similar appropriate treatment facility will treat the raw water if required to reduce salinity prior to use on the site for the purposes mentioned above. The treatment

facility will be located near the CPP, and reject water will be disposed of in the tailings storage facilities.

The raw water supply option from Glebe Weir Raising will not require any water to be treated by reverse osmosis.

4.4.4 Raw water storage

Sufficient raw water storage will be provided adjacent to the CPP to deliver the water demand if the supply pipeline is out of service. The raw water storage dam is currently expected to store approximately 400 ML (or approximately 14 days supply), though this volume will be confirmed during later design phases to provide acceptable system reliability.

5. Potential impacts

This section assesses the potential impacts of the Project on water resource environmental values (EVs). Objectives and practical mitigation measures for protecting water resource EVs are set out in Chapter 6 below.

With the exception of some minor numerical differences, the potential impacts on water resources of the Project area are essentially unchanged from those identified in the EIS. However, further details are provided in the sections below.

5.1 Potential impacts on surface water quality

Surface water runoff quality is most likely to be impacted by the quality of runoff from overburden dumps prior to full rehabilitation. The characteristics of the overburden and soils are described in Chapter 9 of the Supplementary EIS and are briefly summarised below.

- the geochemical testing yielded electrical conductivity (EC) values ranging between 280 $\mu\text{S}/\text{cm}$ and 1,080 $\mu\text{S}/\text{cm}$. Runoff from overburden dumps could therefore potentially have moderately elevated salinity, but with some dilution, such levels would not constitute high levels compared to the receiving waters. At Juandah Creek at the DERM stream gauging station at Windamere, the long-term EC range is 108 $\mu\text{S}/\text{cm}$ to 865 $\mu\text{S}/\text{cm}$ with a median of 315 $\mu\text{S}/\text{cm}$. Recent event monitoring for the Project at Juandah Creek (downstream) site finds EC to range between 110 $\mu\text{S}/\text{cm}$ to 282 $\mu\text{S}/\text{cm}$ with a median of 117 $\mu\text{S}/\text{cm}$, however this is based on three samples. Recent daily monitoring for the Project at the Juandah Creek (downstream) site finds EC to range between 1 $\mu\text{S}/\text{cm}$ to 839 $\mu\text{S}/\text{cm}$ with a median of 362 $\mu\text{S}/\text{cm}$, which is slightly above ANZECC guidelines of 30 to 350 $\mu\text{S}/\text{cm}$.

Additionally, the conservative solute modelling capabilities of the IQQM have been used to estimate a possible range of salinities in the environmental dams over the Project life, with further discussion given in the Addendum to the Site WMS Technical Report in Attachment C. The results show:

- TDS levels would only meet sub-lethal or toxic levels when stored water volumes drop to very low levels due to evaporation and subsequent TDS concentration
- following large rainfall events, when the risk of discharge is highest, the salinity concentrations drop significantly. The maximum salinity level, when water levels are less than 100 ML from the spillway crest, are typically slightly above drinking water standard and the stored water would therefore be considered uncontaminated or low-toxicity waste and suitable for reuse on the site.

5.2 Potential impacts on surface water flows

The Project could potentially impact downstream water flows, specifically in the waters of the nearby streams shown in Figure 3-6, Horse Creek, Spring Creek, Mud Creek, Woleebee Creek and Juandah Creek, the Dawson River, and the Fitzroy River:

- the potential impacts on downstream surface water flows (other than flood flows) are described in detail in the Addendum to the Site WMS Technical Report in Attachment C. While the local impacts on mean annual streamflow in the smaller

tributaries crossing the MLA areas could be significant, the modelled impact on flows further downstream, is relatively small due to the contribution of the larger tributaries diverted across the MLA areas

- the mine and associated WMS is expected to reduce mean annual flows immediately downstream of the MLA areas by approximately 3.2% by Year 30 of the Project. The greatest impact is on the Mud Creek and Spring Creek catchments, where mean annual flows will be reduced by 13.4% and 13.8% respectively. Further downstream on Juandah Creek, just upstream of the Horse Creek/Juandah Creek confluence where the nearest surface water entitlement (a harvesting entitlement) is held, the total impact is reduced to a decrease of 2.3% of mean annual flow by Year 30 of the Project
- the nearest licensed water user downstream of the MLA areas is a water harvester just upstream of the confluence of Horse Creek and Juandah Creek. The mean annual flow at this location is estimated to be 71,450 ML/a. The impact of the Project on mean annual flow at this location is estimated to be 1,767 ML/a, or 2.5% of existing mean annual flow. The impact of the Project on the ability to take the larger flows usually accessed under a water harvesting licence, and consequently the available yield, is therefore likely to be relatively small
- an irrigator is also licensed to take supplies from Juandah Creek just upstream of the Dawson River confluence. At this location, the estimated mean annual flow is 134,000 ML/a, and the reduction in mean annual flow makes up approximately 1.3% of the total. The impact on the available yield at this location is therefore likely to be small
- other downstream water users are located closer to Taroom, downstream of the Dawson River confluence, and further downstream at Glebe Weir. The mean annual flow in the Dawson River downstream of the Juandah Creek confluence is very large (362,000 ML/a) compared to the flow at Wandoan (<0.5%), and the effect of the Project's mine site activities on water yield is likely to be negligible
- low flow analysis shows minimal impact on average median monthly flows, but a small impact on number of flow spells for some of the creeks crossing the MLA areas. The largest impact is on Spring Creek where there is a reduction in the number of flow spells of 16% and a corresponding increase in spell duration of 19%. This reduces to 2% for both statistics for Mud Creek, and for the remaining creeks is much less again.

5.3 Potential impacts on flooding

Works, including levees and creek diversions are proposed to manage flood flows around the active mine areas. These in turn could potentially cause:

1. Increased upstream flood levels due to construction of flood levees and creek diversions
2. Increased downstream flood flows and flood levels due to loss of flood storage by construction of flood levees.

The impacts of the Project on flooding are described in detail in the Addendum to the Flood Study Technical Report included in Attachment A.

5.4 Potential impacts on aquatic habitat

Refer to the Addendum to the Aquatic Ecology technical report, STR 17B-1-SV1.5 for the potential impacts on aquatic habitat associated with the MLA areas.

5.5 Potential impacts on the GAB precipice sandstone aquifer and associated springs and baseflow watercourses

As the total construction water demand estimates are unchanged, and the proposed construction water bore will be drawing from the same vicinity as the existing Wandoan town bores, the preliminary drawdown impact assessment remains unchanged from the EIS. The impact on the nearest Spring Vents and Baseflow streams would be reduced slightly due to the greater distance to the new bore (compared to if supplies were drawn from the Wandoan town bores, as originally proposed). Thirteen bores shown in Figure 3-18 have been identified as penetrating the Precipice Sandstone aquifer within 35 km of Wandoan. Of these, only six (excluding the Wandoan Town bores) are likely to be water bores that continue to tap the Precipice Sandstone: all are subartesian. The nearest other bores are approximately 10 km from the existing Wandoan Town Bores. Based on the preliminary drawdown impact assessment presented in the EIS, the maximum additional drawdown at these bores at the end of the construction period is likely to be less than 4 m, and will be less than 0.5 m within five years of construction ceasing.

6. Mitigation measures

6.1 Surface water quality

The key water quality mitigation measure is the site water management system (WMS). The system will evolve over the Project life, to the Year 30 layout shown in Figure 6-1. Details are provided in the Site WMS Technical Report included in Attachment C. In summary:

- CSM by-product water may be used as the water source for dust suppression on haul roads. Concerns were raised submissions on the EIS regarding the long-term impact of soil salinity and sodium adsorption ratio (SAR) in relation to haul road dust suppression with CSM water.

Measures to limit the potential for impacts from the use of CSM water for dust suppression include:

- using water captured in the site water management system in preference to CSM water when the WMS water is available
- providing small catch drains to intercept runoff from haul road table drains during low flows. These catch drains will help limit the extent of salt discharge during small rainfall events
- the WJV has commissioned an experimental program to assess potential accumulation of salt in the haul road surface, the potential for precipitated salts to be dissolved and mobilised by rainfall, in relation to rainfall intensity. The results of this program will guide future management if salt accumulation is found to be an issue on haul roads
- conducting periodic monitoring of salinity and sodicity during the operation of haul roads, including the road drains and adjacent land. If monitoring suggests a build-up of high levels of salinity or SAR, suitable management measures will be investigated and implemented
- a large portion of the haul roads requiring dust suppression are associated with pit ramps. These ramps will be removed and buried as a component of decommissioning and rehabilitating mine pits
- conducting measurements of soil salinity and sodicity prior to the decommissioning of a haul road, including the road surface, drains, sediment ponds and adjacent land. Where required, material with high salinity or SAR will be excavated and appropriately disposed of, or otherwise remediated, during haul road rehabilitation.

6.1.1 Water quality monitoring

Changes to the locations of the upstream Frank Creek and the upstream Juandah Creek monitoring points to cater for the addition of Wubagul Pit have been nominated. These new monitoring sites should be commissioned as soon as possible to provide baseline water quality prior to mine operations.

A new monitoring point is also proposed at the wastewater treatment plant discharge point. This monitoring point will be part of licence compliance system for the wastewater treatment plant.

A submission on the EIS identified the need to include monitoring and analysis of selenium in both water and sediment samples an important water quality issue identified in the 'Report to Queensland Premier Review of the Fitzroy River Water Quality Issues'. Selenium has been added to the list of recommended parameters for water quality monitoring for the Project.

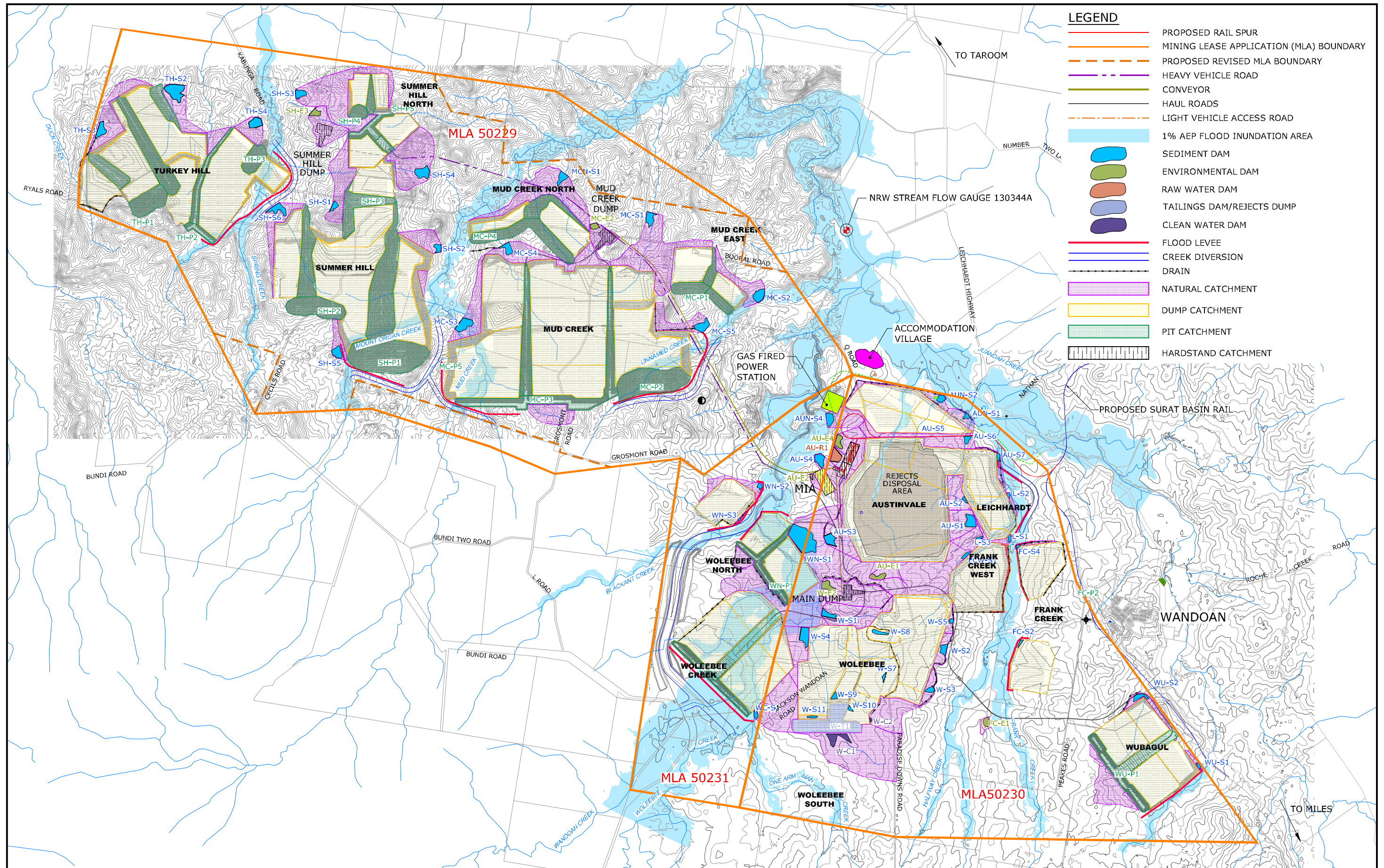
A site assessment of each daily monitoring site will be undertaken to identify a stream water level associated with extreme low flow conditions, and this will be reflected in the water quality monitoring procedure.

Water quality monitoring of the upstream and downstream receiving waters and the WMS will continue throughout the Project. The proposed future water quality monitoring program is discussed in detail in the Addendum to the Surface Water Quality Technical Report given in Attachment B.

6.1.2 Raw water storage

If coal seam methane water is adopted as the raw water supply source, it is possible the raw water dam will need to be lined to protect nearby groundwater and soils. Consideration will be given to clay-lining the dam if suitable materials can be obtained locally. Alternatively, a polyethylene liner may be used to provide an impermeable barrier to leakage. Sufficient monitoring bores will also be installed to detect any leakage before it contaminates downstream water resources. Details of the proposed groundwater monitoring network are provided in the Supplementary EIS Volume 1, Chapter 10 Groundwater.

6.1.3 At source erosion and sediment controls



Plotted By: duffyt Plot Date: 03/09/09 - 1104 Cad File: J:\A442-ENG\PROJ\213306K_Wandoan_Supplie\09_CADD\DRAWINGS\Figures_TechReport\Figure 6-1.dwg

6.2 Surface water flow

A watercourse diversion strategy will be developed for the life of the Project. The strategy will ensure that sufficient time will be allowed for the statutory licensing and approval process prior to construction and vegetation establishment.

Since publication of the EIS, DERM (formerly NRW) has confirmed that the 'Watercourse Diversions – Central Queensland Mining Industry' regional guideline will be applied in the consideration of applications for water licences and development permits for the watercourse diversions within the Project area.

Information outlined to be considered under the guideline will be required as part of the process for these Authorisations. Local geomorphological characteristics of the watercourses will be taken into account in the design of these structures. The design and construction will be to a standard that allows relinquishment of the licence prior to end of mine life. Specific criteria to be applied in accordance with the guidelines include:

- the channel capacity must be at least equivalent to the capacity of the channel existing in that vicinity
- the length of the channel must be nearly equivalent to the length of the channel it replaces
- the channel must exhibit features similar to the natural existing watercourse such as meanders, terraces, benches, etc
- the design must consider the capacity of the floodplain to deal with out of channel flows
- the design must consider potential impacts on the adjoining reaches of the watercourse
- a channel geometry design should be developed that considers the upper limits for the hydraulic characteristics shown in Table 6.1 below.

Table 6-1: Streams and declared watercourses crossing MLA areas

Scenario	Stream power (W/m ²)	Velocity (m/s)	Shear stress (N/m ²)
Two year ARI (no vegetation)	<35	<1.0	<40
Two year ARI (vegetated)	<60	<1.5	<40
50 year ARI	<220	<2.5	<80

The final number and location of diversions required to facilitate mining will be confirmed and finalised as the mine plan and schedule are refined during mine operations. During the licensing process, the diversion layout will be refined and subject to detailed design.

6.3 Flooding

The impact of conceptual designs of the proposed creek diversions on flood levels was investigated using flood models. The results of the modelling show that at the Woleebee Creek diversion, peak flood flows may increase slightly due to a loss of flood storage, and consequently, downstream peak flood levels in Juandah Creek could increase by up to

300 mm during a 1% AEP flood. At other locations, the modelling of the diversion design concept showed limited upstream increases in flood levels.

The former NRW (now DERM) has indicated that as part of the diversion design licensing process, it would expect to see increased flood levels confined to on lease areas. The diversion concepts will be revised during later stages of design and during Project operations, as the mine pit layout and schedule progress. The WJV will wherever possible instigate mitigation measures to confine flood afflux to the MLA areas; however, this may not always be possible without sterilising large coal reserves. If this is the case, any potentially affected properties and infrastructure will be identified, and the owners notified during the licensing process.

Options for mitigating the downstream impacts of the Woleebee Creek diversion have been investigated for this Supplementary EIS. The details of these refinements and the impact of the proposed creek diversions and levees on upstream and downstream flood conditions are described in detail in the Addendum to the Flood Study Technical Report in Attachment A.

6.4 Aquatic habitat

Measures for mitigating impacts on aquatic habitat are essentially unchanged to that described in the EIS. Further details are provided in Chapter 17B Aquatic Ecology, and the associated addendum to the technical report.

Impacts of stream diversions on fish passage will be mitigated by sizing channels to avoid design velocities in excess of 1 m/s as far as practical, as most fish are unlikely to be able to migrate upstream when velocities exceed 1 m/s.

Where possible, the diversion channels will be designed such that natural instream flow velocities are not exceeded, with preliminary modelling described in the Addendum to the Flood Study Technical Report in Attachment A indicating that this will generally be achievable. Preliminary modelling indicates that design velocities of less than 1 m/s can be achieved during instream flow events (such as the one year ARI flow) in most locations; the exceptions are where existing instream velocities exceed this limit. During larger flow events (up to 50 year ARI flow) floodplain flow velocities less than 1 m/s are also likely to be achievable at all locations. Consideration of both ecological and hydraulic flow objectives will be incorporated into the detailed design phase for each stream diversion.

6.5 Water supply impacts on the GAB

The results of the previous analysis indicate the impacts of drawing construction water from the Precipice Sandstone aquifer are relatively small and temporary. While significant material impacts are therefore not anticipated, existing Precipice Sandstone bores will be monitored to gauge the impact of the proposal where possible. If impacts on nearby bores attributable to the Project are greater than anticipated, the WJV will work to make good any loss of water supply. Details of the proposed groundwater monitoring network are provided in Supplementary EIS Volume 1, Chapter 10 Groundwater, Section 10.8.

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Attachment A

Addendum to flood study technical
report

Wandoan Coal Project Flood study technical report Addendum 1

August 2009

Wandoan Joint Venture



Parsons Brinckerhoff Australia Pty Limited ABN 80 078 004 798

Level 4, Northbank Plaza

69 Ann Street

Brisbane QLD 4000

GPO Box 2907

Brisbane QLD 4001

Australia

Telephone +61 7 3854 6200

Facsimile +61 7 3854 6500

Email brisbane@pb.com.au

NCSI Certified Quality System ISO 9001

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Author: Martin Jacobs – Senior Water Engineer

Signed:

Reviewer: Sharon Fong – Acting Team Manager Surface Water, Eric Lam – Principal Hydrologist

Signed:

Approved by: Helen D'Arcy – Senior Environmental Engineer

Signed:

Date: August 2009

Distribution: Wandoan Joint Venture

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Attachment C	Streamflow gauge 130344A
Attachment D	MIKE11 model details
Attachment E	MIKE21 model build details
Attachment F	Plans and maps
Attachment G	MIKE11 model validation

Glossary of terms

Adopted Middle Thread Distance (AMDT)	The distance from the mouth of the watercourse or the confluence of the watercourse with the main watercourse measured along the middle of the watercourse.
Afflux	Rise in flood level caused by a hydraulic structure.
AMG	Australian Map Grid — Cartesian co-ordinate system derived from a Universal Transverse Mercator projection of latitudes and longitudes on the Australian Geodetic Datum (AGD) (now superseded).
AGD84	The coordinate reference system used in Australia prior to the introduction of GDA94.
Annual Exceedence Probability (AEP)	The probability that a given rainfall total accumulated over a given duration will be exceeded in any one year (see footnote).
Attenuation	The reduction of flood peaks due to storage effects.
Australian Height Datum (AHD)	The datum used for determining elevations in Australia which uses a national network of bench marks and tide gauges, and has set mean sea level as zero elevation. Elevations in metres above Australian Height Datum are annotated with the suffix m AHD (see below).
Average Recurrence Interval (ARI)	The average, or expected, value of the periods between exceedences of a given rainfall or a stream flow over a given duration (see footnote).
BoM	Bureau of Meteorology.
Catchment	The area of land which collects and transfers rainwater into a waterway.
Confluence	Area where two or more waterways come together to form one waterway.
Critical storm duration	The critical storm duration is the duration of rainfall that will result in the highest peak flood levels at a particular location.
Discharge	The rate of flow measured in volume per unit time (such as ML/day).
Downstream	In the direction of flow of a stream or river i.e. away from the source.
DTM	Digital Terrain Model.
EIS	Environmental Impact Statement.
EP Act	Queensland Environment Protection Act 1994.
Erosion	The process by which soil and rocks are loosened, worn away and removed from parts of the Earth's surface. Includes removal of debris supplied to the streams by slope wash, mass movement, and gullies.
Flood plain	That portion of a river valley that is covered during periods of high flood water.
Flow	See discharge.
Frequency	A measure of the number of occurrences per unit of time.
GDA94	Geocentric Datum of Australia. The coordinate reference system currently used in Australia to define co-ordinate systems.
Groundwater	Water found underground in porous rock or soil strata.
Headwaters	Upstream section of a river before it is joined by main tributaries. Typically smaller in width and flow than the main section of the river.
HEC-RAS	A computer program that models water flow hydraulics of rivers and channels.
Hydraulic	Mechanical properties of liquids.
Hydraulic analysis	Refers to the assessment of flood levels, flows and velocities in waterways, creeks and rivers.
Hydrograph	A record of the discharge of a creek, stream or river over time.
Hydrology	The study of the occurrence, distribution, and chemistry of all waters of the earth.

Hydrologic analysis	Refers to the estimation of flows that enter waterways, creeks and rivers.
IFD	Intensity Frequency Duration.
Impervious Surfaces	Artificial structures such as pavements and building roofs, which replace naturally pervious soil.
Left/Right Bank	Defined for a watercourse with the observer facing downstream.
Log Pearson Type III flood frequency curve	A method described in Australian Rainfall and Runoff to relate flood peaks to annual exceedence probability.
m AHD	Metres (above the) Australian Height Datum. Refers to the number of meters above Australia's theoretical reference surface, approximately equivalent to the height above sea level.
MDL	Mineral Development Licence.
MGA	Map Grid of Australia – current Cartesian co-ordinate system for use in Australia derived from a Universal Transverse Mercator projection of latitudes and longitudes on the Geocentric Datum of Australia (GDA).
MIKE11	A one dimensional computer program that performs a hydraulic analysis of rivers, channels and water bodies.
MIKE21	A two dimensional computer program that performs a hydraulic analysis of rivers, channels and water bodies.
MIKE Flood	A computer program that combines the MIKE11 and MIKE21 programs.
MLA	Mining Lease Application.
NRW	Queensland Department of Natural Resources and Water.
PB	Parsons Brinckerhoff Australia.
Peak discharge	The maximum discharge or flow during a flood.
Photogrammetry	Remote sensing technology used to determine geometric properties about objects from photographic images.
Pluviograph	A rain gauge which automatically records, usually in graph form, the cumulative amount of rainfall with reference to time.
Rainfall Intensity	Depth of rainfall per unit time.
Rational Method	A procedure for determining peak discharge, which corresponds to a critical storm duration and specified catchment characteristics.
Reach	Portion of a stream channel between two specified points.
Recharge	The process involving the infiltration of water from the surface to groundwater.
Runoff	The portion of rainfall which becomes surface flow.
SRTM	Shuttle Radar Topographic Mission.
Temporal	Relating to time as distinguished from space.
Topography	Concerned with local detail in general, including relief and vegetative and human-made characteristics.
Tributary	A stream or river that does not reach the sea but joins another major river (parent river), swelling its discharge. Sometimes described in terms of "left bank" or "right bank", referring to the bank of the parent river that the tributary connects to.
Upstream	In the opposite direction of the flow of a stream or river, i.e. towards the source.
Weir	A small overflow type dam in a stream or river, generally used to raise the water level or divert its flow.
XP-RAFTS	A computer program that models urban and rural stormwater drainage by analysing rainfall and runoff in any land use area.

Probabilities, ARI and AEP

See http://www.bom.gov.au/hydro/has/ari_aep.shtml. The Average Recurrence Interval (ARI) and Annual Exceedence Probability (AEP) are both a measure of the rarity of an event. With ARI expressed in years, the relationship is:

$$AEP = 1 - \exp(-1/ARI)$$

This results in the following conversions.

ARI (years)	Percent annual exceedence probability	Fraction annual exceedence probability
1	63.2% AEP	0.632AEP
2	39.3% AEP	0.393AEP
5	18.1% AEP	0.181AEP
10	9.5% AEP	0.095AEP
20	4.9%AEP	0.049AEP
50	2%AEP	0.02AEP
100	1% AEP	0.01AEP
1000	0.1% AEP	0.001AEP

Executive summary

This report is an addendum to the Wandoan Coal Project Flood Study Technical Report that was prepared by Parsons Brinckerhoff Australia in November 2008 (reference 2133006C-RTP025-F:mj), referred to hereafter as the Flood Study Report, November 2008. This addendum describes further investigations that were undertaken by Parsons Brinckerhoff Australia, which are based on the same models that were used in the Flood Study Report, November 2008.

Chapter and section headers in this report mirror the chapter and section headers in the Flood Study Report, November 2008 and where the elements of the previous work are not revised, a reference to the Flood Study Report, November 2008 is used (the text is not reproduced in this addendum report).

The scope remains as originally proposed:

- to quantify storm flows by hydrological modelling and calibration to available field data
- to determine existing flow patterns, extents of flooding and flood levels by hydraulic modelling
- to determine potential impacts on flooding upstream and downstream of the Project site by hydraulic modelling of changes to conditions within the Project site
- to recommend appropriate mitigation measures for potential impacts and to determine appropriate flood protection to the various infrastructure elements of the Project.

As noted in the Flood Study Report, November 2008, the quantum of flows in the various creeks was estimated with hydrological models by using the program XP-RAFTS. This model was calibrated to flow observations at the Windamere streamflow gauge on Juandah Creek. The Rational Method was used for some of the smaller streams. Flow patterns, extents of flooding and flood levels were determined with hydraulic models by using the programs MIKE11 and HECRAS. A range of average recurrence intervals (ARIs) and storm durations were analysed.

The current addendum study expands upon the hydraulic modelling that was performed for the previous report. The hydraulic modelling comprised a MIKE11 model for the creek system in the eastern half of the MLA and separate HECRAS models were constructed for the other creeks. In the current study, the MIKE11 model was refined and expanded, and a new HECRAS model was constructed for the Two Mile Creek in the south-eastern corner of the MLA. No changes have been made to the assessment prepared for Mud Creek, Mount Organ Creek or Spring Creek.

The conclusions to this addendum study align broadly with the Flood Study Report, November 2008. A number of creeks are affected by the proposed mining works, but there are no appreciable impacts upstream. Flood levels downstream, in Juandah Creek, are increased; the main reason being the reduction in floodplain storage in Woleebee Creek. A potential mitigation strategy has been identified to restore flood levels in Juandah Creek, which comprises the construction of a partial floodplain embankment within the MLA at the upstream end of the Woleebee Creek diversion. Further work is needed for the engineering design of the creek diversions, particularly with respect to stream power and the transitions to connecting creeks.

It is recommended that additional rain and stream flow gauges be provided to supplement existing on-site measurements; the hydrological and hydraulic models are developed further for engineering design; and further hydraulic analysis includes levee breach assessment, if required under dam legislation.

1. Introduction

1.1 Background

See the Flood Study Report, November 2008

2. Methodology of assessment

2.1 Scope of works

See the Flood Study Report, November 2008.

2.2 Assessment approach

See the Flood Study Report, November 2008.

2.3 Available data

See the Flood Study Report, November 2008.

2.3.1 Topography and aerial photography

See the Flood Study Report, November 2008.

Additional photogrammetric aerial survey was acquired for the following areas:

- Juandah Creek to the north of the Windamere Stream Gauge
- Woleebee Creek and One Arm Man Creek at the southern MLA boundary
- the area around Wubagul Pit in the south-east corner of the MLA.

The digital terrain model was also refined in the eastern portion of flood model, with the use of updated survey information.

2.3.2 Design rainfall intensity data

See the Flood Study Report, November 2008.

2.3.3 Observed pluviograph records

See the Flood Study Report, November 2008.

2.3.4 Observed flood records – streamflow gauge

See the Flood Study Report, November 2008.

2.3.5 Observed flood records – anecdotal evidence

Observations were acquired from local residents during a site visit in May 2009, see Appendix G.

3. Existing environment

3.1 Flood flow regime description

See the Flood Study Report, November 2008.

3.2 Modelling of existing environment

See the Flood Study Report, November 2008.

The existing creeks are modelled using MIKE11 and HECRAS.

The MIKE11 model covers the creek system in the eastern half of the MLA, which includes Wandoan Creek, Woleebee Creek, Halfway Creek, Frank Creek, Blackant Creek and part of Juandah Creek. The current model expands on the model that was used in the Previous Report and it includes some refinements to better represent the overland flow paths and islands that form during flood conditions. The MIKE11 model was then modified to represent the developed case, as described in Section 4 below.

Separate HECRAS models were constructed for the various creeks not covered by the MIKE11 model. As part of the current study, a new HECRAS model was constructed for Two Mile Creek in the south-eastern corner of the MLA. This model was used to estimate the extents of inundation in the existing case only.

As in the case of the modelling for the Flood Study Report, November 2008, it is acknowledged that the flood modelling will need to be further developed and refined for detailed design. The current modelling is considered to be appropriate at this stage for the identification of impacts within the MLA and in surrounding areas. Though every care has been exercised in the construction of the flood models to ensure that they adequately represent the behaviour of the creek system, it should be recognized that the model results may change according to the development of the detailed design of the levees, creek diversions and structures.

Examples of the strategies employed in the construction of the flood models include:

- cross sections were not permitted to overlap, especially at bends, to avoid duplication of flood plain storage. The exceptions to this rule were the junctions to some branches in which it was important to maintain conveyance
- break-out flow paths were identified by iteration. Where earlier flood mapping identified a break-out flow path, a new branch was constructed to explicitly model the flow path. Cross sections for the previous branches and the new branch were truncated as required to avoid overlapping and duplication of flood storage
- the Sands Dam on unnamed creek was included as a weir structure
- road crossings have been modelled in coarse detail only (major road embankments are included, but details of road decks, piers etc are not). It should be noted that the impacts of proposed structures within the MLA are not expected to extend beyond the MLA.

Further checks were undertaken on the hydrological model, as described in Section 5.1.5 below. It was considered that the further calibration was not warranted at this stage because of the quality of the available data and the principal purpose of model outputs, which is the identification of impacts. As noted in the Flood Study Report, November 2008, further calibration and refinement of the hydrological model may be required for the detailed design.

- Figure 3-1: Typical Juandah Creek cross section CH 14097 looking downstream – See the Flood Study Report, November 2008**
- Figure 3-2: Typical Woleebee Creek cross section CH 16403 looking downstream – See the Flood Study Report, November 2008**
- Figure 3-3: Typical Frank Creek cross section CH 11513 looking downstream – See the Flood Study Report, November 2008**
- Figure 3-4: Typical Spring Creek cross section CH 4572 looking downstream – See the Flood Study Report, November 2008**
- Figure 3-5: Typical Spring Creek Tributary cross section CH 999 looking downstream – See the Flood Study Report, November 2008**
- Figure 3-6: Typical Mount Organ Creek cross section CH 4317 looking downstream – See the Flood Study Report, November 2008**
- Figure 3-7: Typical Mud Creek cross section CH 14798 looking downstream – See the Flood Study Report, November 2008**
- Figure 3-8: Typical Mud Creek cross section CH 20422 looking downstream – See the Flood Study Report, November 2008**
- Figure 3-9: Typical Unnamed Creek cross section CH 11264 looking downstream – See the Flood Study Report, November 2008**
- Figure 3-10: Juandah Creek longitudinal profile downstream – See the Flood Study Report, November 2008**
- Figure 3-11: Woleebee Creek longitudinal profile downstream – See the Flood Study Report, November 2008**
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- Figure 3-15: Spring Creek Tributary longitudinal profile downstream – See the Flood Study Report, November 2008**
- Figure 3-16: Mount Organ Creek longitudinal profile downstream – See the Flood Study Report, November 2008**

Figure 3-17: Mud Creek longitudinal profile downstream – See the Flood Study Report, November 2008

Figure 3-18: Unnamed Creek longitudinal profile downstream – See the Flood Study Report, November 2008

3.3 Two Mile Creek

As noted above, Two Mile Creek was modelled with a new HECRAS model to determine extents of inundation within the MLA. A typical cross section and a longitudinal profile are shown in Figures 3-20 and 3-21. The extents of inundation are also shown in Figure 5.4.

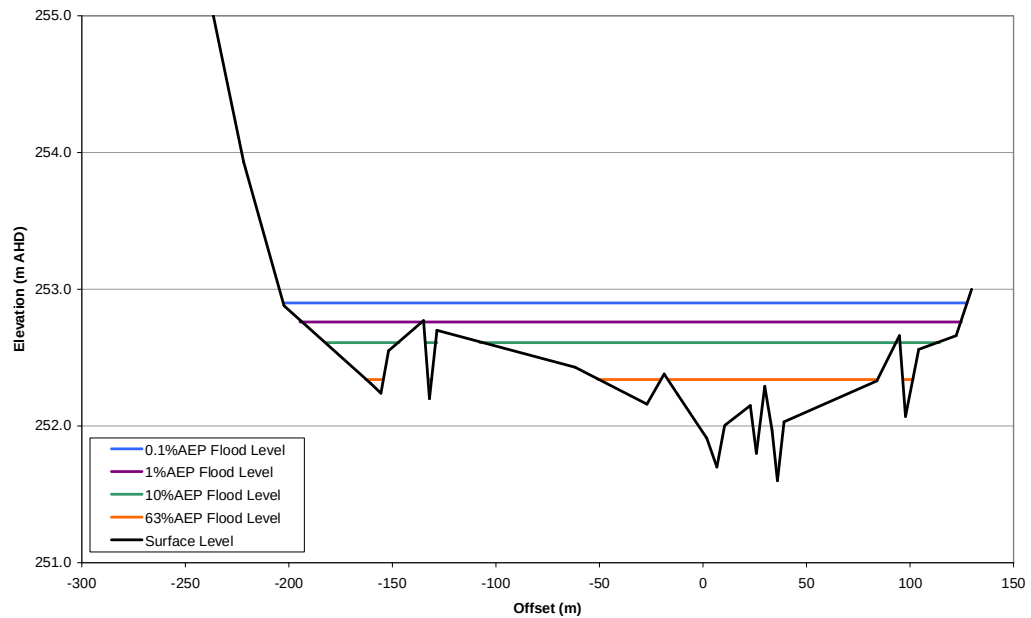


Figure 3-20: Typical Two Mile Creek cross section CH 157.92 looking downstream

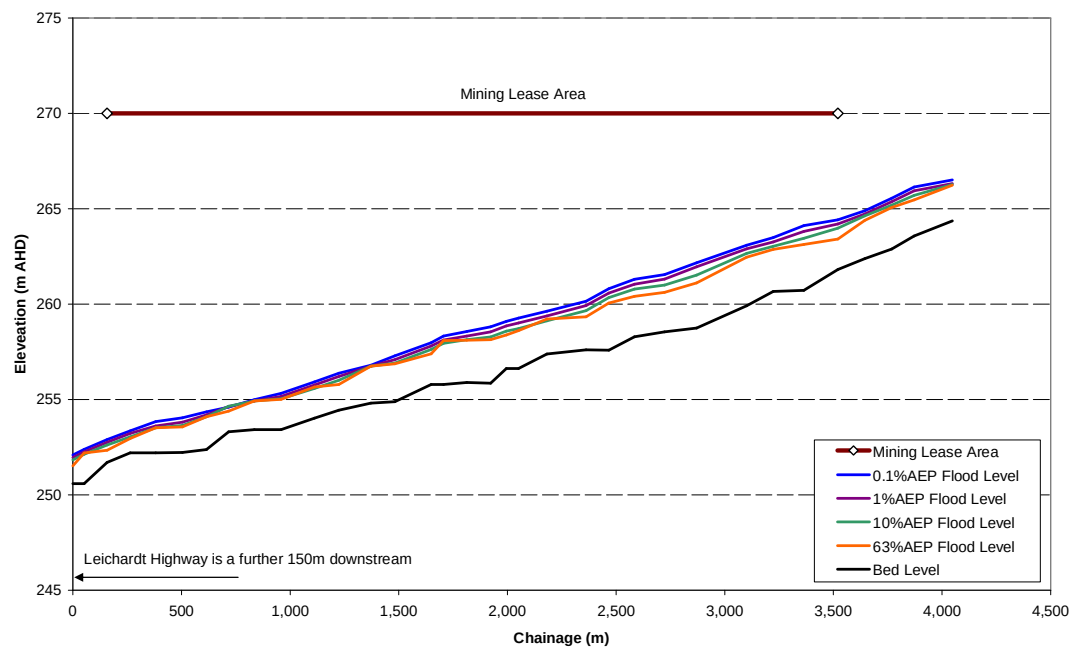


Figure 3-21: Two Mile Creek longitudinal profile

4. Description of proposed development

4.1 Proposed levees

See the Flood Study Report, November 2008.

As in the Flood Study Report, November 2008, levees were included in the model on the left and right banks of Frank Creek. Details are included in Table D-5 in Appendix D.

4.2 Proposed creek diversions

The following creek diversions were reviewed as part of this Addendum;

- the diversion which conveys flows from Woleebee, Wandoan and Blackant Creek along the western boundary of MLA 50231
- the diversion which conveys flows from Frank Creek along the north-east corner of Leichhardt Pit.

These creek diversions were based on the diversions in the Flood Study Report, November 2008, but with further development of invert levels and diversion channel cross sections. It is acknowledged that further optimisation is needed to design the diversions, but the current arrangement is considered to be appropriate for the purposes of assessing impacts at the MLA site boundaries.

Criteria for the design of creek diversions are give in *Watercourse Diversions – Central Queensland Mining Industry* (NRW, 2008), which places considerable emphasis on the requirement for diversions to mimic natural watercourses in the vicinity. This document also sets limiting values on flow characteristics, as shown in Table 4-1.

Table 4-1: Limiting flow characteristics

Scenario	Stream power (Watts/m ²)	Velocity (m/s)	Shear stress (N/m ²)
39% AEP (two year ARI), no vegetation	<35	<1.0	<40
39% AEP (two year ARI), vegetated	<60	<1.5	<40
2% AEP (50 year ARI)	<220	<2.5	<80

4.2.1 Woleebee Creek diversion

The cross section of the proposed Woleebee Creek Diversion is compared with a typical cross section of Woleebee Creek in Figure 4-1 and a longitudinal profile is illustrated in Figure 4-2. This cross section is located at the downstream end of the diversion channel. As indicated in Figure 4-1, the proposed cross section retains a well defined low flow channel, which will follow a sinuous course within the broader diversion channel. The proposed diversion channel is much narrower than the existing flood plain.

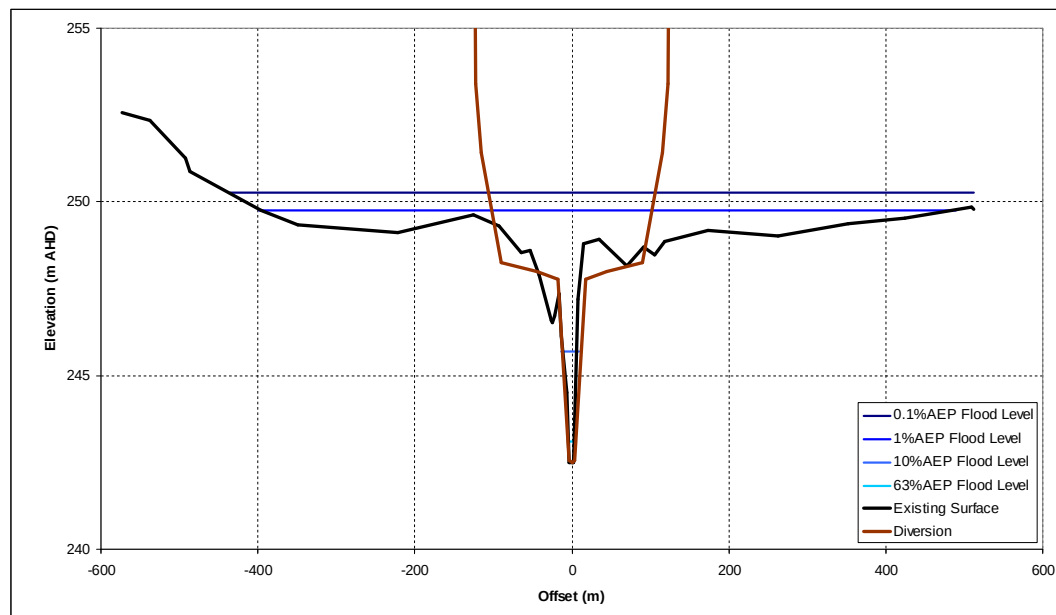


Figure 4-1: Cross section at Wandoan 4085.85 (confluence of Wandoan Creek and Diversion Channel) compared to cross section of proposed diversion channel

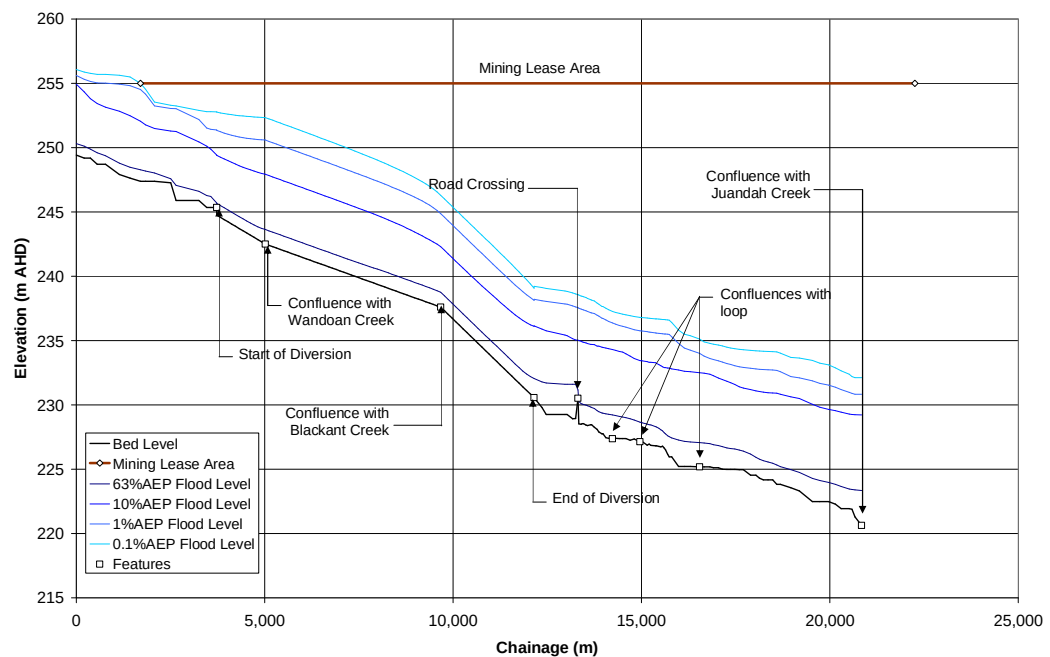


Figure 4-2: Longitudinal profile of proposed diversion channel on Woleebee Creek

The longitudinal profile of the Woleebee Creek diversion has been determined to provide a fit between the proposed diversion channel section and the creeks where they join the diversion channel. These creeks comprise One Arm Man Creek, Woleebee Creek, Wandoan Creek and Blackant Creek.

The estimated maximum design flows in the diversion channel are summarised in Table 4-2. The resulting flow characteristics are summarised in Tables 4-3 and 4-4.

Table 4-2: Estimated maximum flows in diversion channel (from DIV_WOLEEBEE 8424.25)

AEP	ARI	Event duration	Flow (m ³ /s)
63% AEP	1	36 hours	10
10% AEP	10	30 hours	170
1% AEP	100	36 hours	814
0.1% AEP	1000	30 hours	1669

Table 4-3: Calculated flow characteristics – existing Woleebbee Creek (from WOLEEBEE 15442.83)

Scenario	Flow	Stream power	Velocity (m/s)		Shear stress
	(m ³ /s)	(Watts/m ²)	Central channel only	Total section	(N/m ²)
63% AEP (one year ARI)	8	2	0.4	0.4	5
39% AEP (two year ARI) interpolated	78	9	0.7	0.7	13
10% AEP (10 year ARI)	148	12	0.8	0.7	16
2% AEP (50 year ARI) interpolated	453	14	0.8	0.5	17
1% AEP (100 year ARI)	759	16	0.9	0.6	18

Table 4-4: Calculated flow characteristics – proposed diversion channel Woleebbee Creek (from DIV_WOLEEBEE 8424.25)

Scenario	Flow	Stream power	Velocity (m/s)		Shear stress
	(m ³ /s)	(Watts/m ²)	Central channel only	Total section	(N/m ²)
63% AEP (one year ARI)	10	20	0.8	0.8	25
39% AEP (two year ARI) interpolated	90	91	1.5	1.5	61
10% AEP (10 year ARI)	170	138	1.8	1.8	78
2%AEP (50 year ARI) interpolated	492	275	2.3	1.6	118
1% AEP (100 year ARI)	814	358	2.6	1.8	138

The results shown in Table 4-4 indicate that stream power and shear stress in this reach exceed the maximum values allowed for in *Watercourse Diversions – Central Queensland Mining Industry* (NRW, 2008). Stream power, shear stress and velocity also exceed the values for the existing creek. These increases are attributed to the narrowing of the flood plan (Figure 4-1) and the increase in channel gradient (see Figure 4-2). Some of this increase may be reduced through further development of the design of the diversion

channel by changing the channel dimensions and vertical profile, but it is expected that the full restoration of existing channel characteristics will not be possible.

Table 4-5: Comparison of flow path lengths – Woleebee Creek diversion channel from chainage 3726.05 to 15442.86

Scenario	Low flow channel	Flood plain
Existing Woleebee Creek	11.717 m	8.950 m
Proposed diversion channel	>8.424 m, subject to detailed design – depends upon sinuosity of channel in flood plain	8.424 m

4.2.2 Frank Creek diversion

The cross section of the proposed Frank Creek Diversion is compared with a typical cross section of Frank Creek in Figure 4-7 and a longitudinal profile is illustrated in Figure 4-8. This cross section is located at the downstream end of the diversion channel. As indicated in Figure 4-7, the proposed cross section retains a well defined low flow channel, which will follow a sinuous course within the broader diversion channel. The proposed diversion channel is much narrower than the existing flood plain.

The estimated maximum design flows in the diversion channel are summarised in Table 4-6. The resulting flow characteristics are summarised in Tables 4-7 and 4-8.

Table 4-6: Estimated maximum flows in diversion channel (from DIV_FRANK 1599.00)

AEP	ARI	Event duration	Flow (m ³ /s)
63% AEP	1	36 hours	3
10% AEP	10	30 hours	78
1% AEP	100	36 hours	187
0.1% AEP	1000	30 hours	323

Table 4-7: Calculated flow characteristics – existing Frank Creek (from FRANK 12509.19)

Scenario	Flow	Stream power	Velocity (m/s)		Shear stress
	(m ³ /s)	(Watts/m ²)	Central channel only	Total section	(N/m ²)
63% AEP (one year ARI)	3	8	0.6	0.6	13
39% AEP (two year ARI) interpolated	40	28	1.0	0.7	28
10% AEP (10 year ARI)	77	21	0.9	0.5	24
2% AEP (50 year ARI) interpolated	131	21	0.9	0.5	24
1% AEP (100 year ARI)	185	25	0.9	0.6	26

Table 4-8: Calculated flow characteristics – proposed diversion channel Frank Creek (from DIV_FRANK 1559.00)

Scenario	Flow	Stream power	Velocity (m/s)		Shear stress
	(m ³ /s)	(Watts/m ²)	Central channel only	Total section	(N/m ²)
63% AEP (one year ARI)	3	2	0.4	0.4	5
39% AEP (two year ARI) interpolated	41	9	0.7	0.3	13
10% AEP (10 year ARI)	78	12	0.8	0.4	16
2%AEP (50 year ARI) interpolated	133	16	0.9	0.5	19
1% AEP (100 year ARI)	187	20	1.0	0.6	21

The results in Table 4-8 fall well within the guideline values. Table 4-8 also indicates that stream power and shear stresses may be reduced in the proposed diversion channel. Further development of the design is needed to align stream power in the proposed channel to existing stream power; if the stream power is too low, then the proposed channel may experience excessive sedimentation.

Table 4-9: Comparison of flow path lengths – Frank Creek diversion channel from chainage 10150.17 to 13120.60

Scenario	Low flow channel	Flood plain
Existing Woleebee Creek	2,970 m	2,650 m
Proposed diversion channel	>3,115 m, subject to detailed design – depends upon sinuosity of channel in flood plain	3,115 m

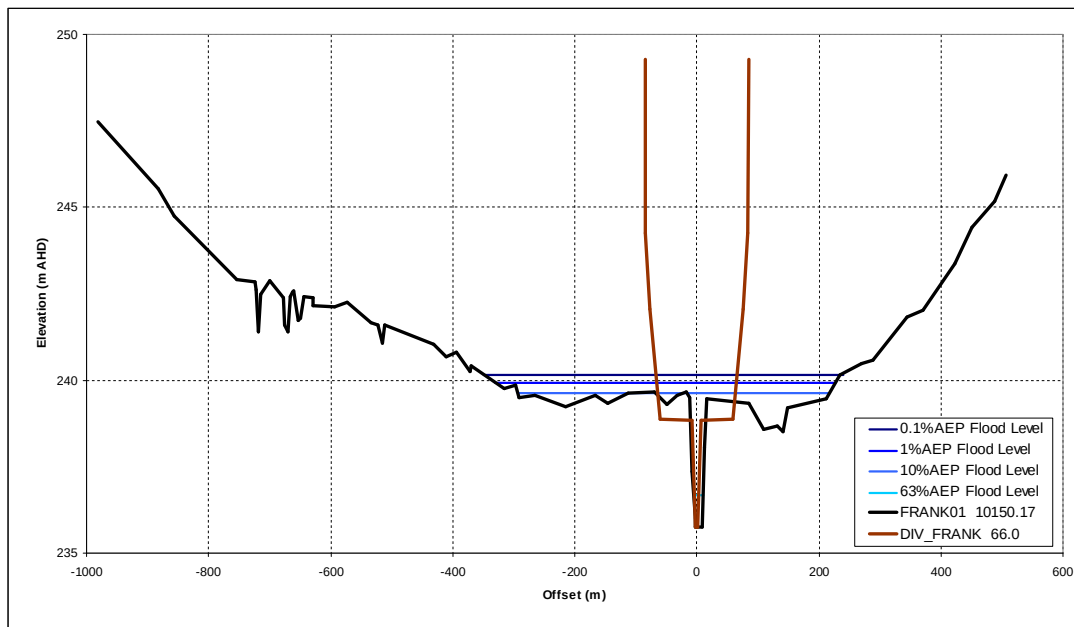


Figure 4-3: Cross section at Frank Ch 10150.17 (upstream end of diversion channel) compared to cross section of proposed diversion channel

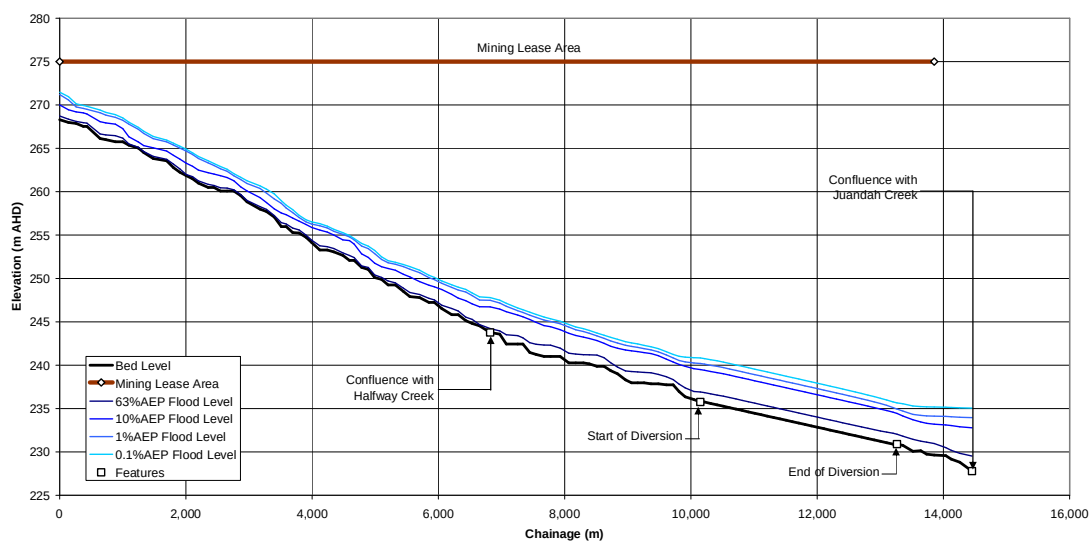


Figure 4-4: Longitudinal profile of proposed diversion channel on Frank Creek

4.2.3 Spring Creek and Spring Creek Tributary diversions

See the Flood Study Report, November 2008.

4.2.4 Unnamed Tributary diversion

See the Flood Study Report, November 2008.

4.2.5 Mud Creek/Mount Organ Creek diversion

See the Flood Study Report, November 2008.

5. Modelling

5.1 Hydrological modelling

See the Flood Study Report, November 2008.

5.1.1 Model catchment area

See the Flood Study Report, November 2008.

5.1.2 Runoff routing model

See the Flood Study Report, November 2008.

5.1.3 XP-RAFTS model calibration

See the Flood Study Report, November 2008.

5.1.4 Influence of GAB intake areas

See the Flood Study Report, November 2008.

5.1.5 Model calibration

See the Flood Study Report, November 2008.

Additional information has been added regarding flooding in the 1983 and 1991 flood events. Flood level observations during these events were collated by Xstrata staff as described in section 5.4.6.

Full calibration of the flood model to historical events is problematic due to the spatial and temporal paucity of rainfall data over the catchment. Typical flood peaks persist for durations of less than one day, therefore continuous (sub-daily) rainfall data is required to properly represent flood flows in historical events. These sub-daily rainfall data are only available at a few locations and they are not consistently available for all historic events.

Continuous rainfall data (collected at sub-daily intervals) is available at only one pluviograph station in the catchment – located near the southern Juandah Creek catchment boundary. This pluviograph station was not operational during the 1983 event, and no others are in the nearby vicinity outside the catchment. As a result, no attempt was made to calibrate the models to the 1983 observations.

Good continuous data was collected for the 1991 flood event, and this was supplemented with daily rainfall data over the western portions of the catchment to give the full rainfall depth and distribution. However, for the catchment of Juandah Creek to the east of Wandoan, no daily rainfall data was available. In an attempt to reproduce the flood hydrograph observed at the Windamere streamflow gauge, the spatial distribution of rainfall was extrapolated to the west as shown in the replacement Figure 5 in Attachment F. This distribution gave a reasonable match to the observed flood peak, but results in an overestimate of the total runoff volume observed in this event. The model probably

overestimates the flow in Juandah Creek, but given that four rainfall gauges were in operation across the western catchment, rainfall estimates for Woleebee Creek and its tributaries should be reasonable. In the absence of more definitive data, this distribution was adopted for use in validation against observed flood levels.

5.2 Comparison of XP-RAFTS and flood frequency analysis

See the Flood Study Report, November 2008.

Figure 5-1: Annual series flood frequency analysis – Windamere stream flow gauge
See the Flood Study Report, November 2008

5.3 Adopted design conditions for impact assessment

5.3.1 Design flood hydrographs - XP-RAFTS

See the Flood Study Report, November 2008.

5.3.2 Design peak flows — Rational Method

See the Flood Study Report, November 2008.

5.4 Hydraulic modelling

The current study expands upon the modelling performed for the Flood Study Report, November 2008. Modelling comprises a MIKE11 model that covers the eastern half of the MLA and separate HECRAS models for the other creeks. The MIKE11 model was refined and expanded, and a new HECRAS model was constructed for the Two Mile Creek in the south-eastern corner of the MLA. The MIKE11 model for the existing case was then modified to represent the developed case, with its creek diversions.

5.4.1 Boundary conditions

MIKE11

Tailwater levels in the MIKE11 model were revised to accommodate the additional reach of Juandah Creek downstream, of the Windamere Stream Gauge. The strategy used in the Flood Study Report, November 2008 was used, which was to estimate a rating curve at the downstream end of Juandah Creek, based on the properties of the last cross section and the assumption of normal flow. As the stream gauge at Windamere is located an appreciable distance upstream, the potential effect of the assumptions used in the determination of the rating curve are diminished in the model's calculations of conditions at the stream gauge, and the gauged data can be used to verify the results.

HECRAS

An additional HECRAS model was constructed for Two Mile Creek following the procedures described in the Previous Report.

Flows into the HECRAS model for Two Mile Creek were estimated by the rational method in accordance with the Queensland Urban Drainage Manual. This model extended beyond the MLA boundary to the Leichardt Highway road crossing, but as there were no reliable survey data of the crossing structure, normal depth conditions were assumed as a boundary condition at this location.

5.4.2 Bed roughness

See the Flood Study Report, November 2008.

5.4.3 Existing infrastructure

See the Flood Study Report, November 2008.

5.4.4 Model limitations

See the Flood Study Report, November 2008.

5.4.5 Comparison of hydrographs

See the Flood Study Report, November 2008.

Figure 5-2: Hydrographs at Juandah creek stream flow gauge – 63%AEP 36 hour storm – see the flood study report, November 2008.

Figure 5-3: Hydrographs at Juandah creek stream flow gauge – 1%AEP 36 hour storm – see the flood study report, November 2008.

5.4.6 Model validation

Anecdotal flood inundation information was collated from nearby landholders by Xstrata personnel so that the model results could be further validated against local observations.

Observations were available from the two largest floods in the Windamere stream gauge record (1983 and 1991). Pluviograph data from these events were scarce, however, for the 1991 event; sufficient data were available to reasonably reproduce the flow hydrograph at the Windamere streamflow gauge. Catchment flows from this event were input to the hydraulic model to generate the extent of inundation expected in the 1991 event. The modelled extents compare well with the observations, with the model overestimating flood levels slightly in Juandah Creek.

The results and observations are described in more detail Appendix G.

5.5 Sensitivity analysis

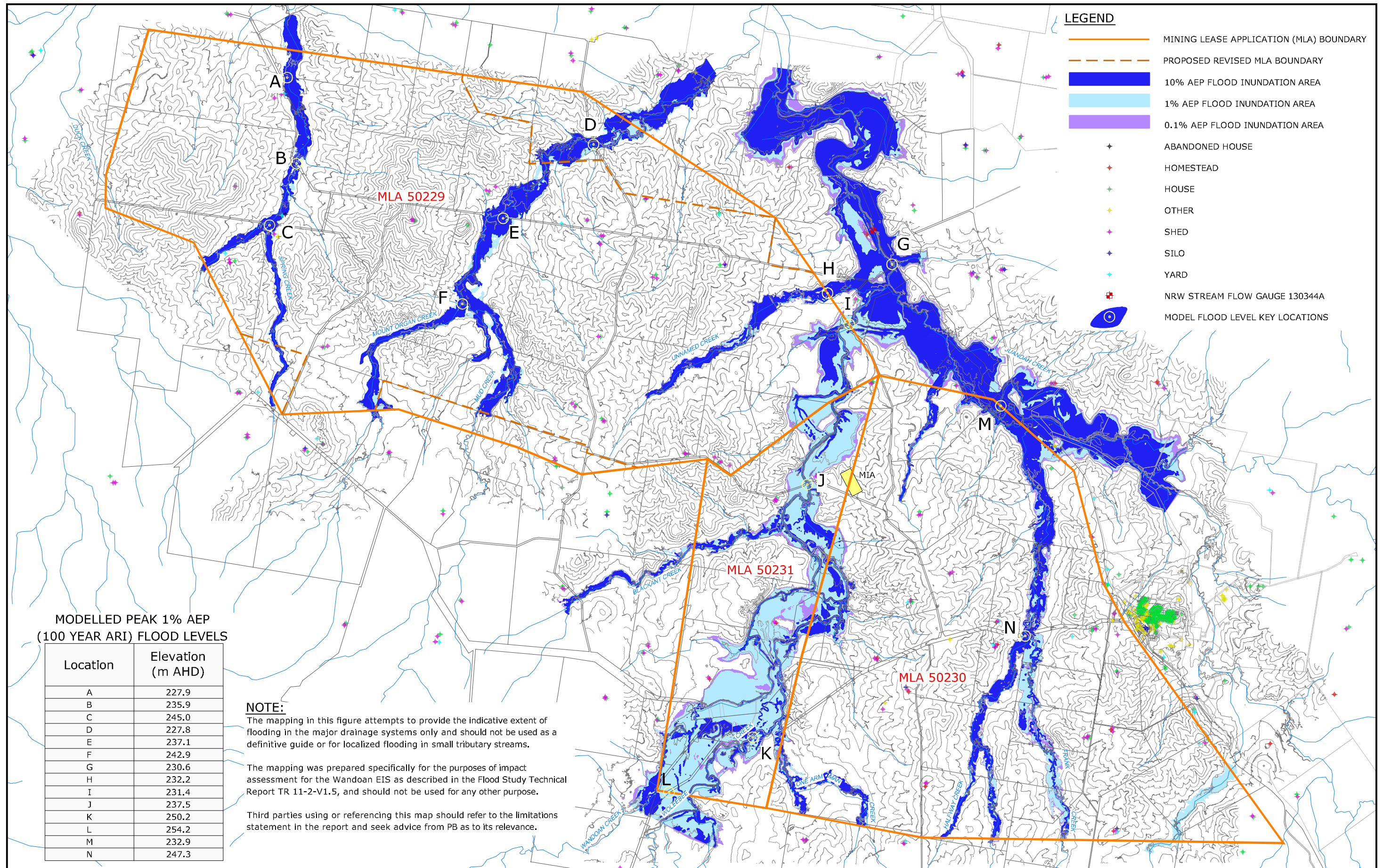
5.5.1 Sensitivity to Juandah Creek rating curve

See the Flood Study Report, November 2008.

The model's sensitivity to the Juandah Creek rating curve has been reduced in this Addendum by extending the model boundary downstream of the stream gauge location.

5.6 Flood inundation plans

The current flood model covers more creeks than the previous report. The MIKE11 model yields additional data for the tributary between Woleebee Creek and Frank Creek (Branch07) and the overland flow path near the Windamere Stream gauge (Branch09). The HECRAS model yields data for Two Mile Creek. The flood inundation plans have been revised accordingly, as shown in Figure 5-4. It is important to interpret this drawing with care, as the inundation lines are intended to show the extent of flooding in the major creeks that have been modelled only, and not the minor tributary gullies.



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6. Potential impacts and mitigation measures

Potential residual flood impacts from the proposed flood mitigation works are illustrated as changes in maximum flood levels as shown in Figures 6-1 and 6-7, changes to flows as shown in Figures 6-8 to 6-9, and in the afflux map in Figure 6-10. The afflux map also shows the infrastructure that could be affected.

6.1 Potential impacts to creeks

Figures 6-1 to 6-7 show the potential afflux in various creeks due to the proposed mine works with no provision for mitigation measures. The afflux is the increase in maximum flood level.

Juandah Creek – Figure 6-1

Maximum flood levels downstream of the MLA are increased by up to approximately 0.3m (1% AEP). The increased flood levels continue downstream at least as far as the limit of the current model, which is approximately 8.4 km downstream of the stream gauge at Windamere.

The major cause of the increase in flood levels in Juandah Creek is the increased inflows from Woleebee Creek and the decreased time to peak flows in Woleebee Creek. The changes to flows in Woleebee Creek and Juandah Creek are illustrated in the hydrographs in Figures 6-8 and 6-9.

Flows are increased in Woleebee Creek and the time to peak flow is reduced because of the reduction in flood storage in the flood plains. This reduction in storage results from the construction of the diversion channels. Essentially, in large floods, water breaks out of the creek channel and occupies a volume of flood storage in the flood plain. The filling and emptying of the storage in the flood plain reduces the flow discharged by the creek and increases the time for peak flow to be reached. The removal of this flood storage increases peak flows and increases the time for peak flow to be reached, which is demonstrated in the unmitigated case. Peak flows may be returned to existing values when the flood storage is restored by the potential mitigation works, as described Section 6.6.

Maximum flood levels at the Leichardt Highway are increased by less than 0.01m

Blackant Creek – Figure 6-2

In Blackant Creek, at the MLA boundary, the current model results in a large reduction in 63%AEP and 10%AEP flood levels and a large increase in 1%AEP and 0.1%AEP flood levels. This results from the current model including an abrupt transition into the diversion channel. The modelled transition does not allow for a smoothing of the channel or overbank areas at the confluence of Blackant Creek and the diversion channels, such that it includes some steps, which will not appear in the final form of the transition. Further refinement of the design of the junction between Blackant Creek and the diversion channel is expected to mitigate these impacts by providing a smoother transition. For example, the invert level of the diversion channel may be revised to provide a better match to existing 1%AEP and 0.1%AEP flood levels in Blackant Creek, whilst a drop structure may be incorporated to accommodate the abrupt fall in 63%AEP and 10%AEP flood levels. These refinements are necessary to reduce the potential for scour and flooding in the adjacent property.

Wandoan Creek – Figure 6-3

The current model demonstrates that there is no adverse impact on flood levels at the MLA boundary. Flood levels are increased just upstream of the diversion, which are caused by the transition to the diversion channel. As in the case of Blackant Creek, further refinement of the design of the diversion channel at the junction to Wandoan Creek is expected to mitigate these impacts within the MLA.

Woleebee Creek – Figure 6-4

The current model demonstrates that there is no adverse impact on flood levels at the upstream MLA boundary. Flood levels are generally increased just upstream of the diversion and in the reach between the diversion and the confluence with Juandah Creek. The 10%AEP event appears to reverse the trend; however the changes in this event also reflect a change in critical duration from 72 hours in the existing case to 30 hours in the ultimate case. The change in critical duration reflects the reduction in flood storage in Woleebee Creek due to the construction of the diversion channel. Further refinement of the design of the diversion channel is expected to mitigate the impacts due to abrupt changes at the ends of the diversion channel. However, there remains the issue of lost flood storage within the Woleebee Creek flood plain, which results in the increased flows discharged to Juandah Creek.

One Arm Man Creek – Figure 6-5

The current model demonstrates that there is no adverse impact on flood levels at the MLA boundary. There are increased on-site flood levels at the upstream end of the diversion channel which are caused by the transition to the diversion channel. Further refinement of the design of the diversion channel is expected to mitigate these impacts within the site.

Halfway Creek – Figure 6-6

The current model demonstrates that there is no adverse impact on flood levels at the MLA boundary. Impacts in Halfway Creek upstream of the confluence with Frank Creek are almost imperceptible. Given that the flood levels in Halfway Creek are not changed at the confluence with its tributary, Halfway02, flood levels upstream are not expected to change.

Frank Creek – Figure 6-7

The current model demonstrates that there is no adverse impact on flood levels at the MLA boundary. Flood levels within the MLA boundary are changed, as expected, because of the provision of flood levees and the diversion channel. The levees increase flood levels within the MLA boundary by up to approximately 0.08 m, but the greatest increase within the MLA boundary occurs at the upstream transition to the diversion channel.

Summary

In summary, the results of the modelling demonstrate the following:

- In general, there are no adverse changes to flood levels at the upstream boundaries of the MLA that are due to the proposed mine works. The model shows that at Blackant Creek there is significant drawdown for frequent events and a significant increase in flood levels for extreme events. This is due to the abrupt transition in the current model that will be mitigated by further detailed design. Elsewhere, along the southern MLA boundary for example, the model indicates no appreciable change to flood levels in the various creeks at the MLA boundary
- The proposed creek diversions provide less flood storage than the existing creeks. In particular the diversion on Woleebee Creek removes a significant volume of flood storage from the system. This result in higher flows being discharged to Juandah Creek, which increases the 1% AEP maximum flood levels in Juandah Creek by up to 0.3 m. The increased flood level in Juandah Creek extends at least as far as the limit of the model, which is located approximately 8.4 km downstream of the stream gauge at Windamere.

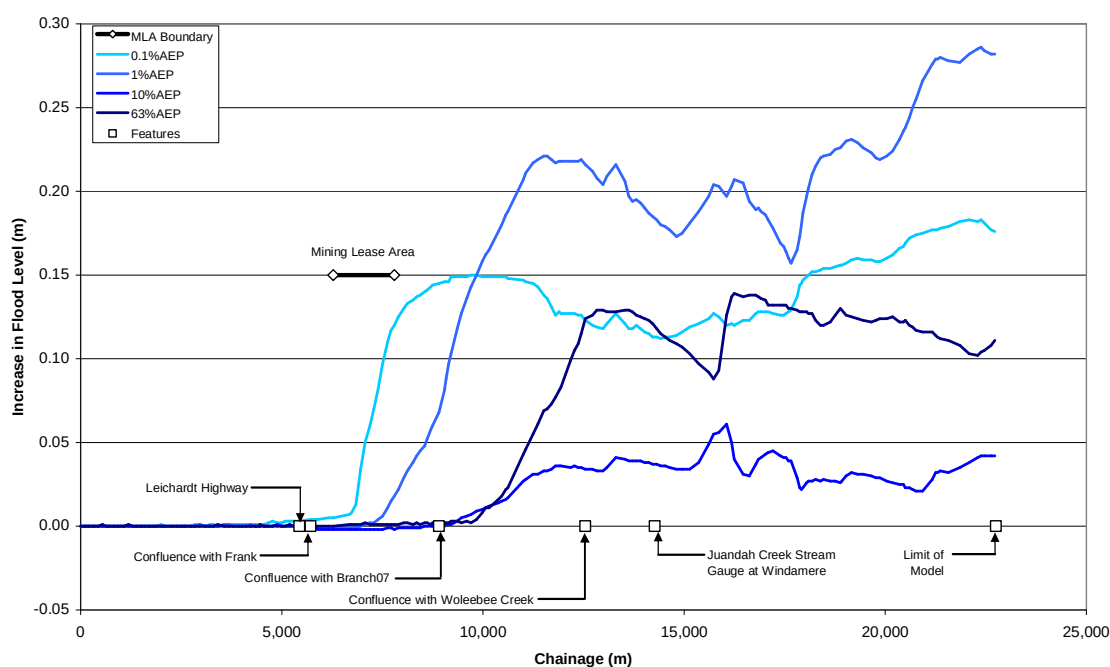


Figure 6-1: Changes to maximum flood levels on Juandah Creek

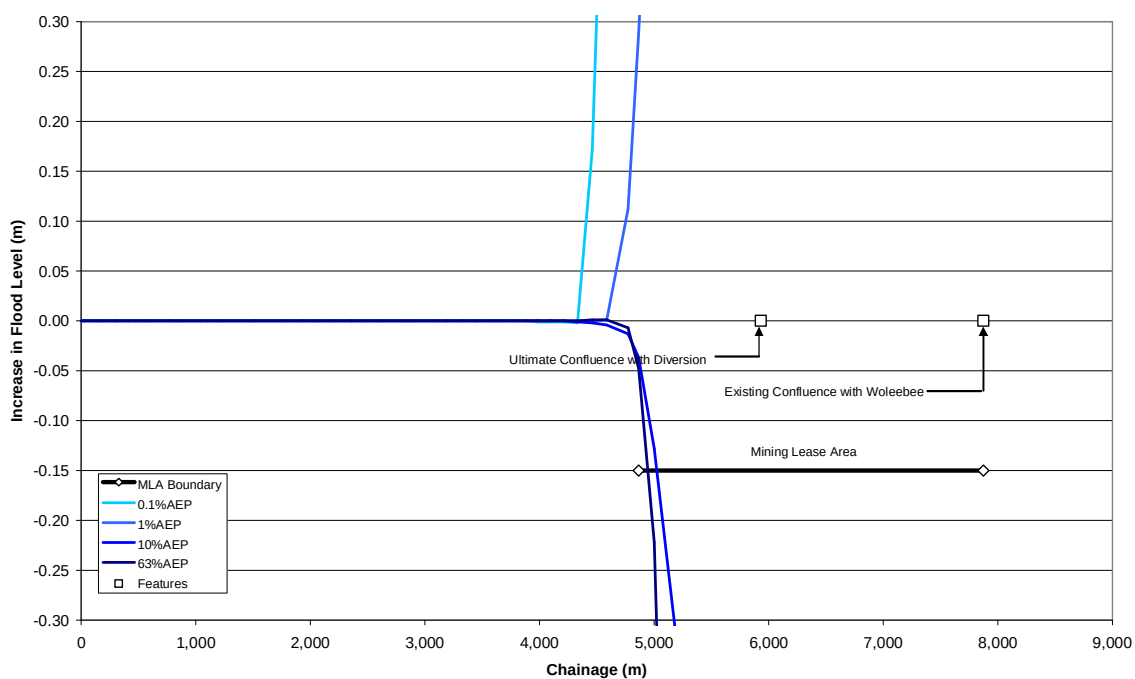


Figure 6-2: Changes to maximum flood levels on Blackant Creek

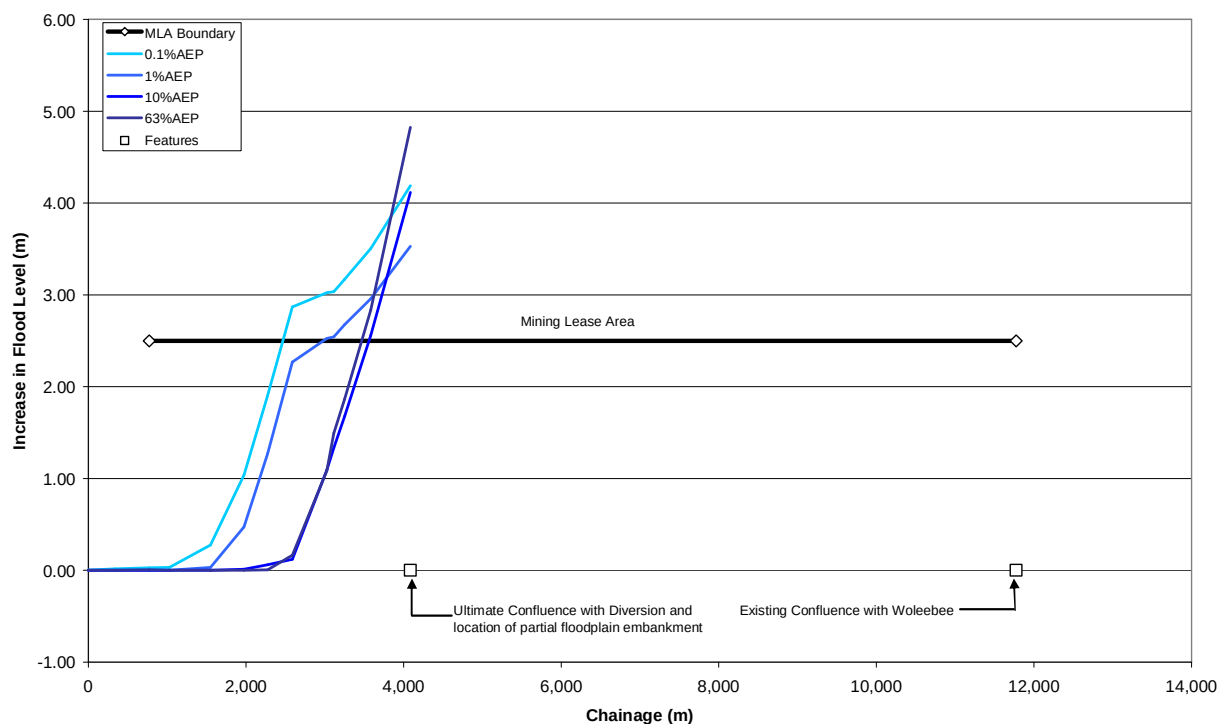


Figure 6-3: Changes to maximum flood levels on Wandoan Creek

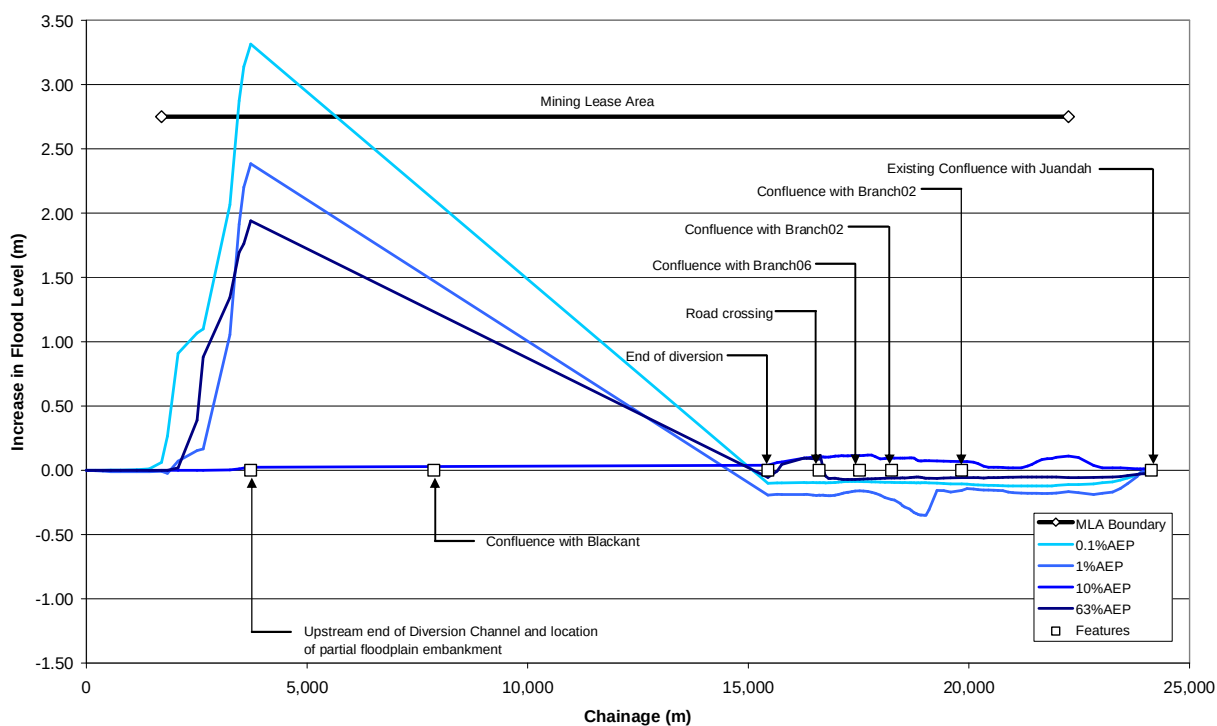


Figure 6-4: Changes to maximum flood levels on Woleebee Creek

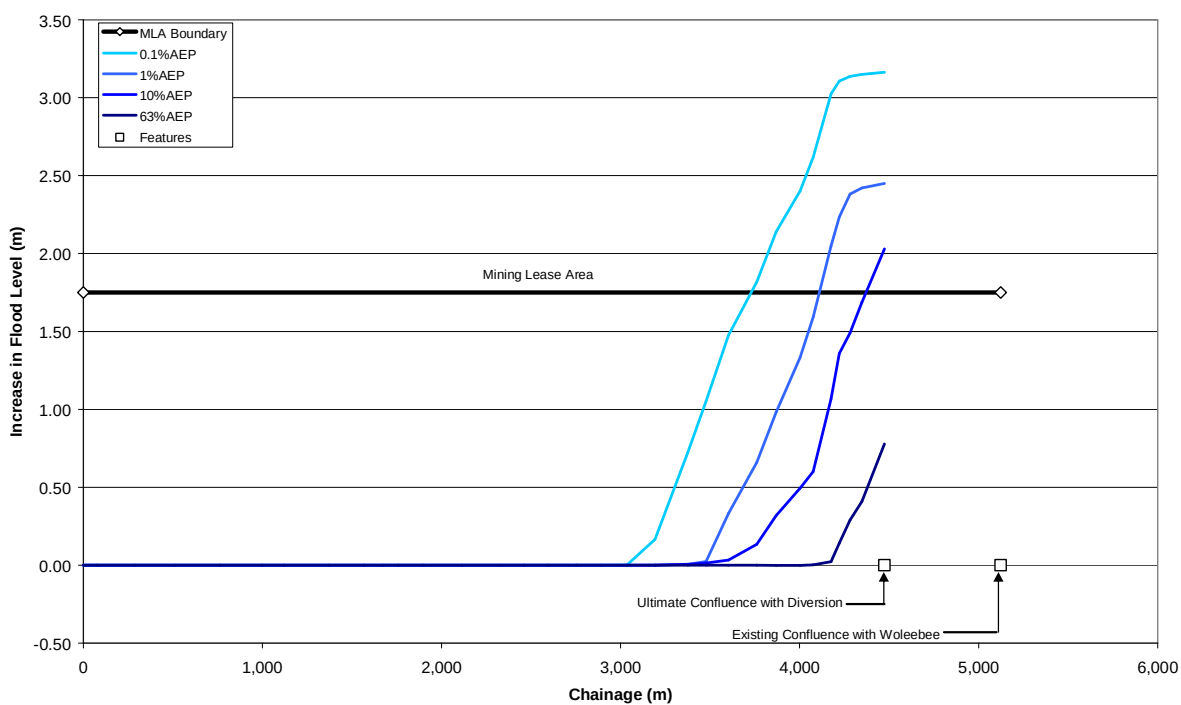


Figure 6-5: Changes to maximum flood levels on One Arm Man Creek

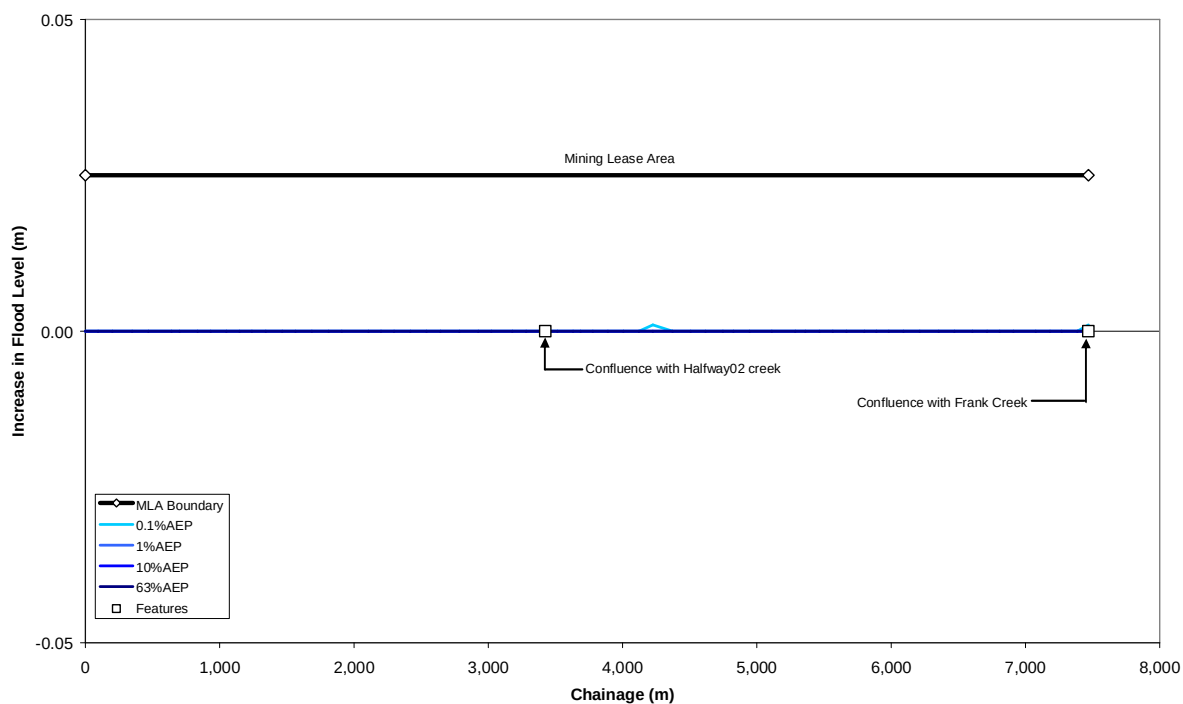


Figure 6-6: Changes to maximum flood levels on Halfway Creek

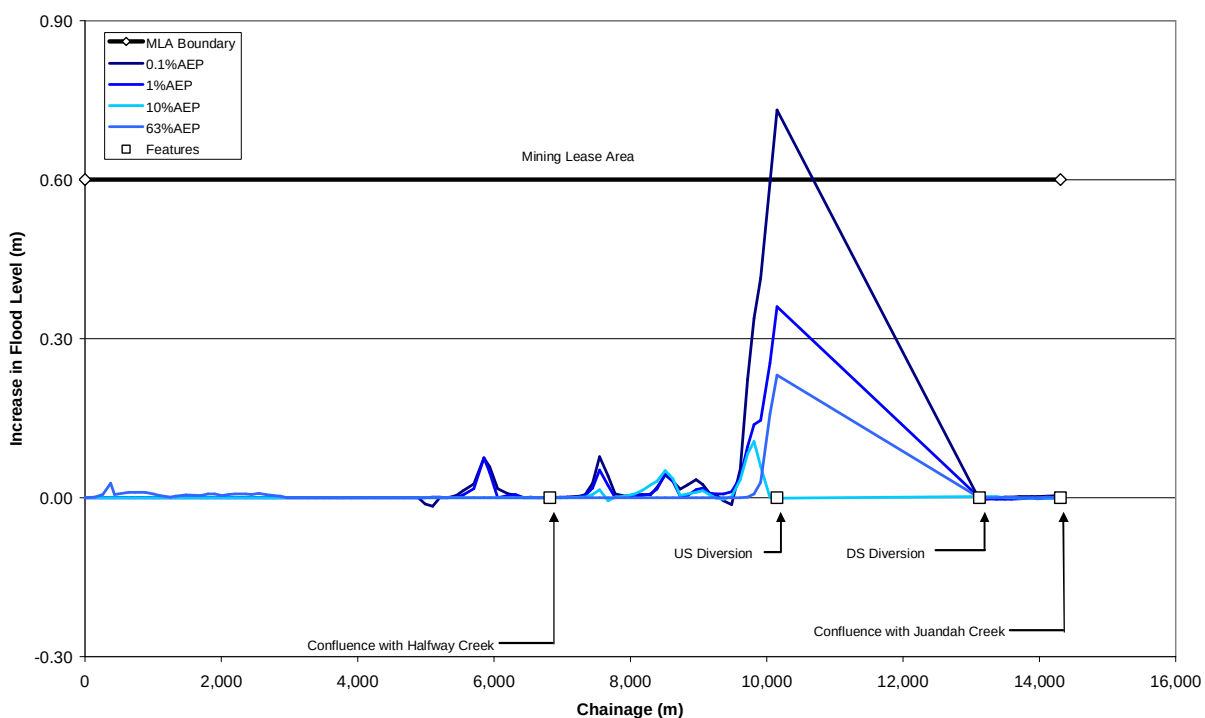


Figure 6-7: Changes to maximum flood levels on Frank Creek

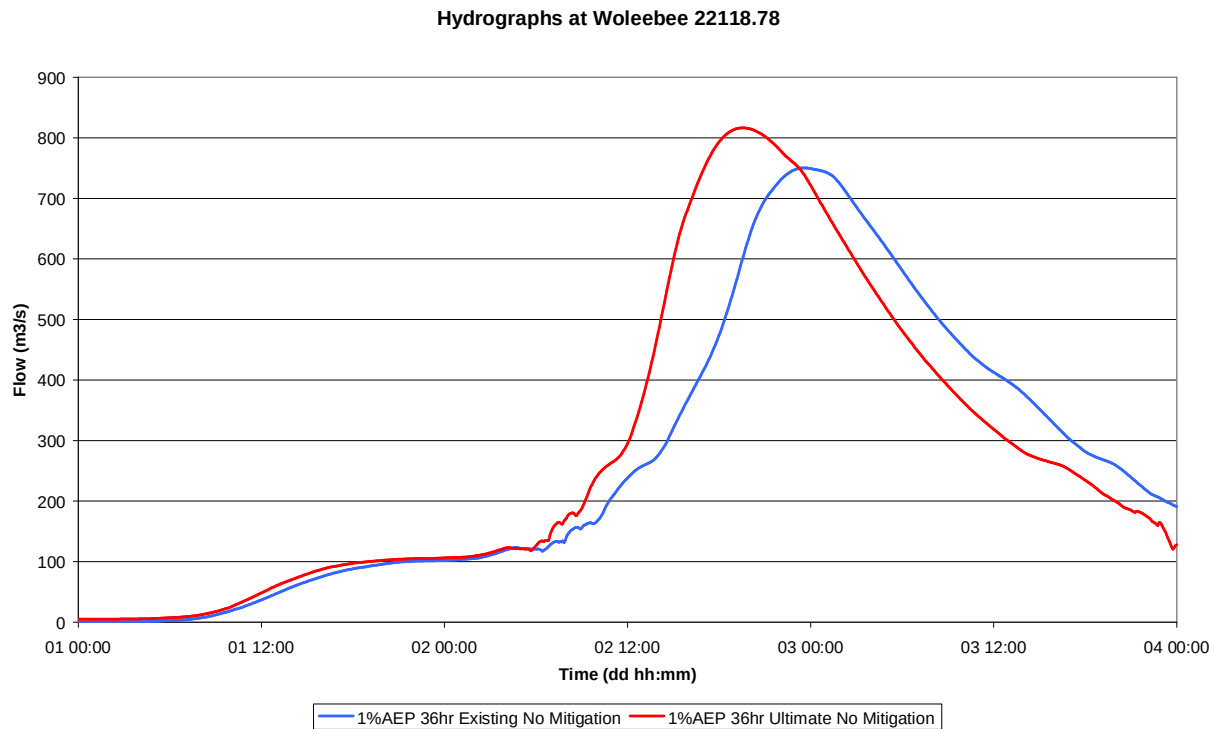


Figure 6-8: Changes to flows in Woleebee Creek upstream of confluence with Juandah Creek

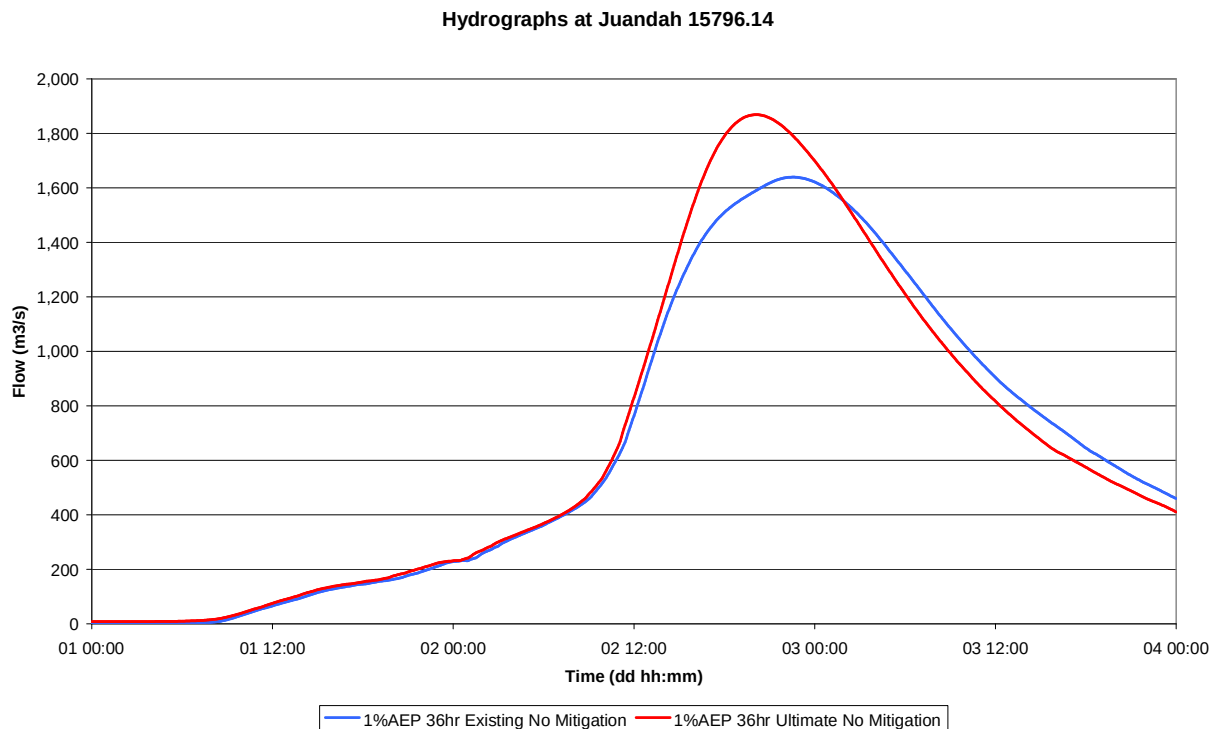


Figure 6-9: Changes to flows in Juandah Creek downstream of confluence with Woleebee Creek

6.2 Potential impacts to farm dams

See the Flood Study Report, November 2008.

6.3 Potential impacts to roads

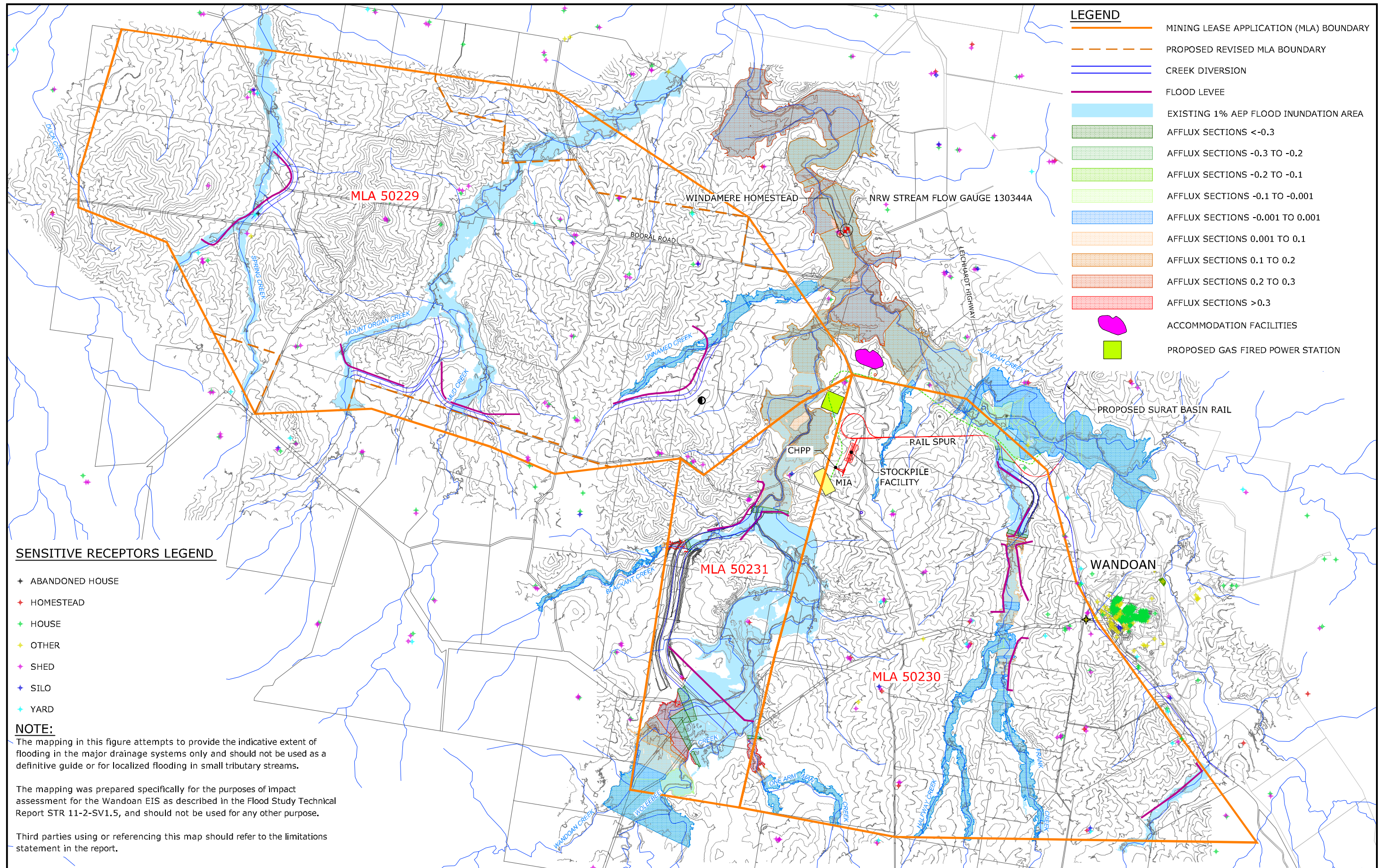
See the Flood Study Report, November 2008.

6.4 Potential impacts to dwellings

See the Flood Study Report, November 2008.

6.5 Mitigation measures for site infrastructure

See the Flood Study Report, November 2008.



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6.6 Mitigation of afflux in Juandah Creek

As noted above, the major cause of the increased flood levels in Juandah Creek is the removal of overbank flood storage from Woleebee Creek by the construction of the diversion channel. The following discussion addresses the physical feasibility of mitigating these impacts by flood mitigation works.

6.6.1 Potential Flood Mitigation Works – Partial floodplain embankment on Woleebee Creek Diversion Channel

Introduction

There is a considerable area of flood plain between the Wandoan Creek diversion channel and the upstream boundary of the Mining Lease Area (MLA). A partial floodplain embankment may be constructed at the upstream end of the Woleebee Creek diversion channel that can partially constrict flows entering the diversion. The embankment will not extend all the way across the channel like a weir, but it includes a narrow slot so that low flows can pass unimpeded. The partial floodplain embankment will force major floods to pond upstream, thus replacing much of the flood storage lost to the diversion channel. The structure dimensions were optimised to maximise flood storage within the MLA, whilst minimising the increase in flood levels at the MLA boundary.

Weir location, dimensions and constraints

The partial floodplain embankment is located at the confluence between Woleebee Creek and the diversion channel. The bunds and diversion channels constructed across the flood plain direct all flows to the partial floodplain embankment.

The behaviour of the partial floodplain embankment was assessed using the MIKE11 model with all the design ARIs and durations that were used in the impacts analysis. This ensured that the structure was tested under a wide range of flood conditions.

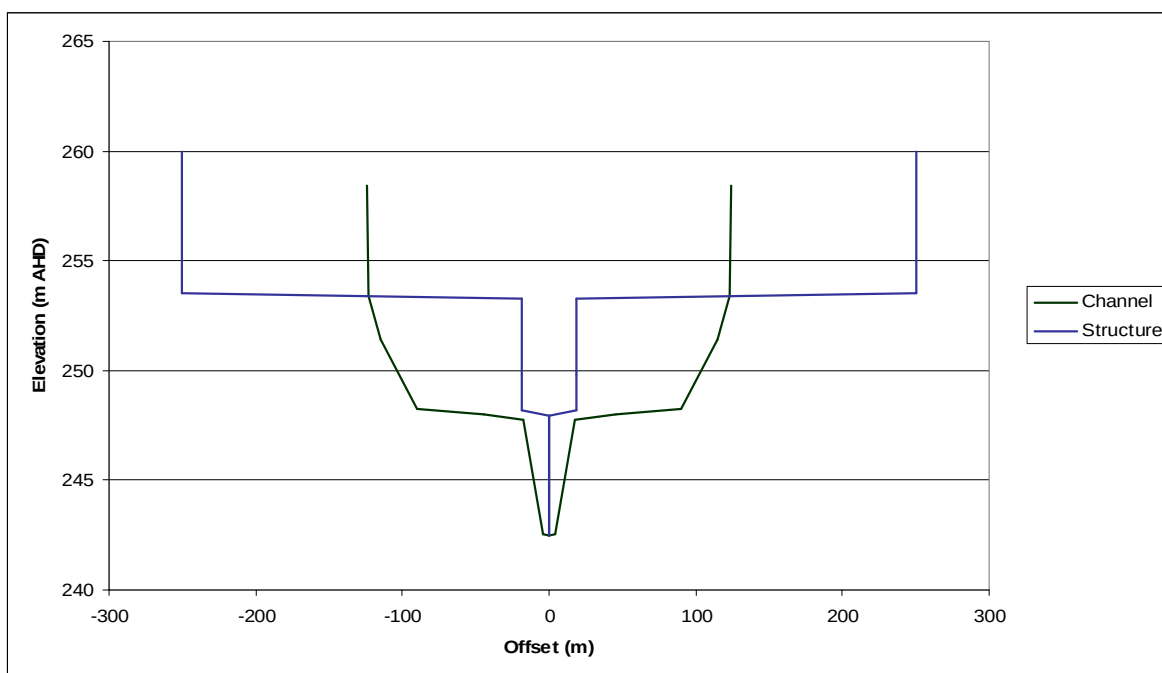
The dimensions of the structure were determined by trial and error until the afflux in Juandah Creek was reduced to a minimum. Maximum flood levels at the MLA boundary were then compared with existing conditions to assess the potential impacts on the neighbouring property to the south. The preliminary structure dimensions are summarised in Tables 6-1 and 6-2, and Figure 6-11.

Table 6-1: Preliminary structure location

Location	Div_Woleebee 1300
Easting	783,495
Northing	7,105,030

Table 6-2: Preliminary structure dimensions

Offset (m)	Elevation (m AHD)
-250.0	260.000
-187.5	253.300
-18.5	253.300
-18.5	250.000
-18.5	249.800
-18.5	247.970
-0.1	247.970
-0.1	242.548
0.00	242.497
0.01	242.497
0.1	242.547
0.1	247.970
18.5	247.970
18.5	249.800
18.5	250.000
18.5	253.300
187.5	253.300
250.0	260.000


Figure 6-11: Elevation of partial floodplain embankment structure on Woleebee Creek diversion channel

Results

Table 6-3: Summary of results at partial floodplain embankment

Location	Ref	AEP and critical duration			
		63% AEP	10% AEP	1%AEP	0.1%AEP
		36hr	72hr	48hr	30hr
Upstream of proposed structure (m AHD)	DIV_WOLEEBEE 1289.22	247.94	248.81	253.13	254.41
Opening velocity (m/s)	DIV_WOLEEBEE 1300.00 Velocity	5.5	5.5	5.5	5.5
Downstream of proposed structure (m AHD)	DIV_WOLEEBEE 1388.31	243.26	247.73	250.03	252.10

Table 6-4: Summary of results at selected locations

Location	MIKE11 Reference	Max Flood Level		
		Existing (m AHD)	Mitigated (m AHD)	Afflux (m)
63% AEP				
Southern MLA Boundary on One Arm Man Creek	ONEARMMAN01 0.00	264.66	264.66	0.00
Southern MLA Boundary on Wandoan Creek	WANDOAN01 770.98	251.73	251.74	0.01
Southern MLA Boundary on Woleebee Creek	WOLEEBEE01 1703.95	248.28	248.28	0.00
Upstream of proposed weir	DIV_WOLEEBEE 1289.22		247.94	
Juandah Creek downstream	JUANDAH01 22750.62	213.66	213.64	-0.02
10% AEP				
Southern MLA Boundary on One Arm Man Creek	ONEARMMAN01 0.00	266.12	266.12	0.00
Southern MLA Boundary on Wandoan Creek	WANDOAN01 770.98	253.71	253.71	0.00
Southern MLA Boundary on Woleebee Creek	WOLEEBEE01 1703.95	252.15	252.15	0.00
Upstream of proposed weir	DIV_WOLEEBEE 1289.22		249.81	
Juandah Creek downstream	JUANDAH01 22750.62	219.03	219.05	0.02
1% AEP				
Southern MLA Boundary on One Arm Man Creek	ONEARMMAN01 0.00	266.30	266.30	0.00
Southern MLA Boundary on Wandoan Creek	WANDOAN01 770.98	254.72	254.72	0.00
Southern MLA Boundary on Woleebee Creek	WOLEEBEE01 1703.95	254.51	254.50	-0.01
Upstream of proposed weir	DIV_WOLEEBEE 1289.22		253.13	
Juandah Creek downstream	JUANDAH01 22750.62	220.87	220.87	0.00
0.1%AEP				
Southern MLA Boundary on One Arm Man Creek	ONEARMMAN01 0.00	266.39	266.39	0.00
Southern MLA Boundary on Wandoan Creek	WANDOAN01 770.98	255.47	255.49	0.03
Southern MLA Boundary on Woleebee Creek	WOLEEBEE01 1703.95	254.99	255.06	0.06
Upstream of proposed weir	DIV_WOLEEBEE 1289.22		254.41	
Juandah Creek downstream	JUANDAH01 22750.62	222.77	222.76	-0.02

Plots of affluxes are shown in Figures 6-12 to 6-15.

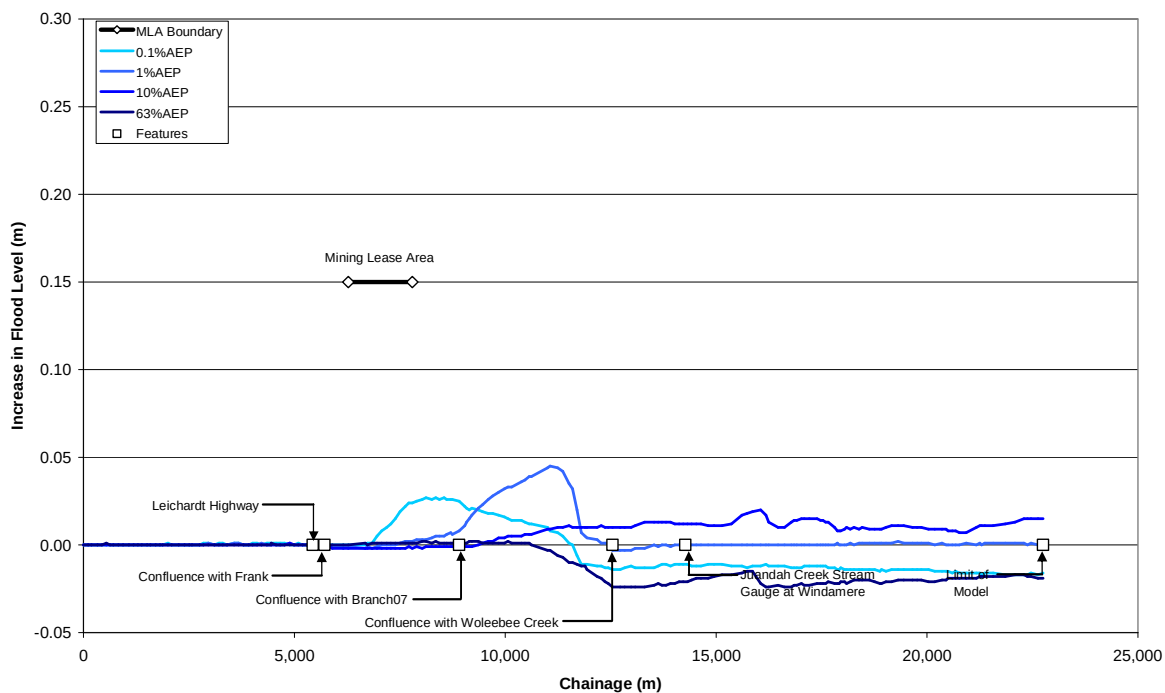


Figure 6-12: Changes to maximum flood levels on Juandah Creek following mitigation with partial floodplain embankment on Woleebee Creek diversion channel

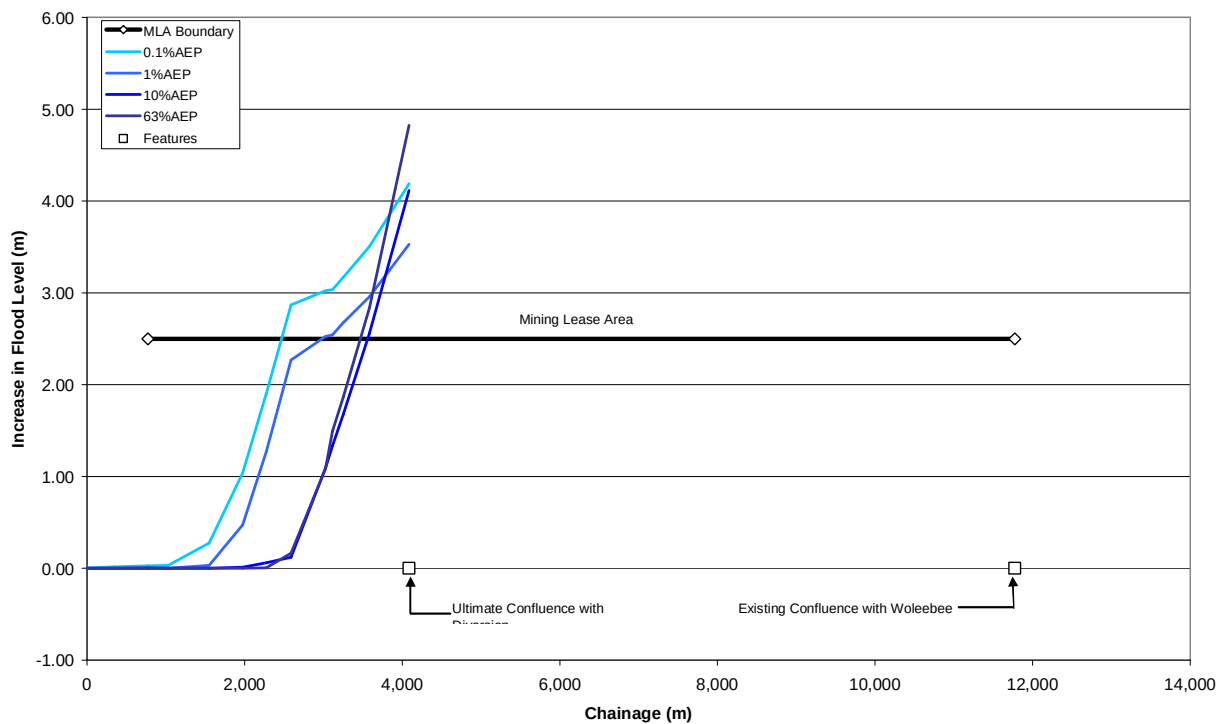


Figure 6-13: Changes to maximum flood levels on Wandoan Creek following mitigation with partial floodplain embankment on Woleebee Creek diversion channel

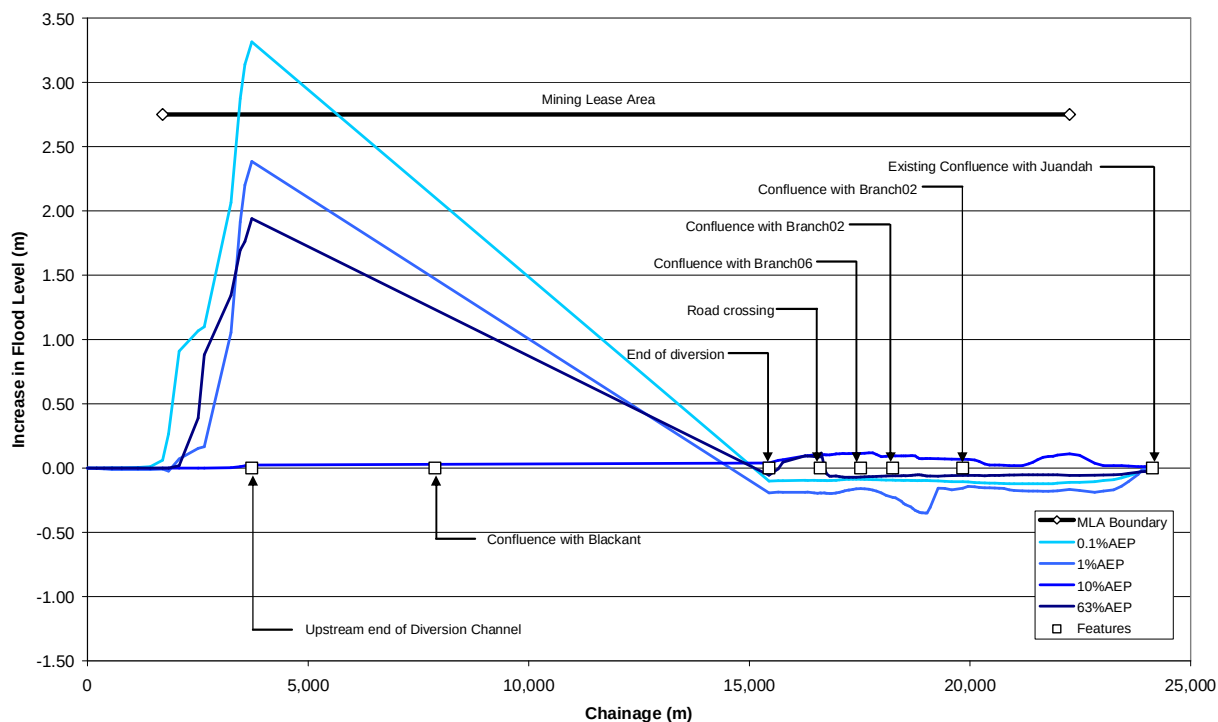


Figure 6-14: Changest to maximum flood levels on Woleebee Creek following mitigation with partial floodplain embankment on Woleebee Creek diversion channel

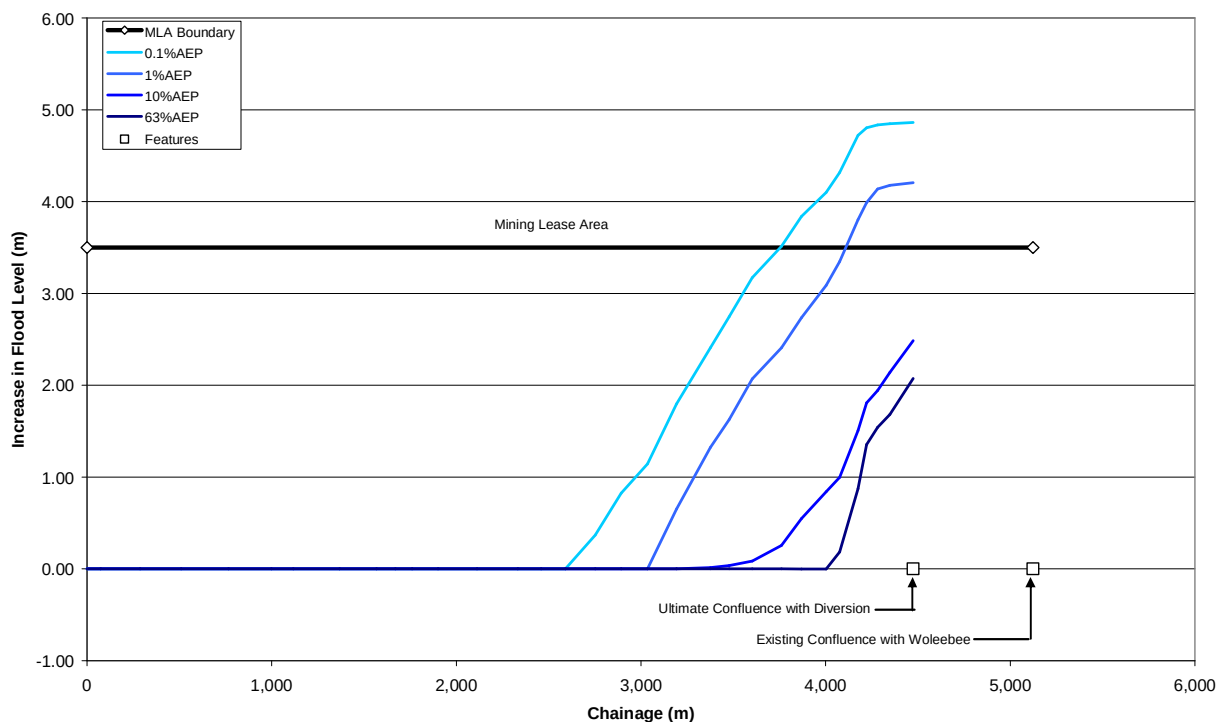


Figure 6-15: Changes to maximum flood levels on One Arm Man Creek following mitigation with partial floodplain embankment on Woleebee Creek diversion channel

Discussion

The results of the hydraulic modelling indicate that the construction of a partial flood plain embankment on the Woleebee Creek diversion channel can significantly reduce the potential increase in flood levels on Juandah Creek. The modelling also indicates that this can be done without significantly increasing flood levels at the southern MLA boundary.

The results indicate that the proposed structure does not reduce the increase in flood levels to zero at all locations. In particular, there is a residual increase in flood levels between the confluences with Frank Creek and Woleebee Creek. Increased flood levels in this area are attributed to the loss of flood storage in Frank Creek due to the diversion channel and levees there. Obviously, the structure on Woleebee Creek will not affect flows discharged from Frank Creek.

There are other residual increases in flood levels in Juandah Creek and at the Southern MLA boundary of up to approximately 0.06 m (60 mm), which were found to be extremely difficult to remove entirely by adjusting the weir dimensions only. As noted below, the increased flow in Frank Creek (arising from the diversion and levees) is expected to contribute to a small afflux in Juandah Creek upstream of the confluence with Woleebee Creek, in which case a structure on Woleebee creek would have no effect.

At this point, it is worth noting how the hydraulic model addresses the dynamic relationship between the weir dimensions and the flood storage upstream. The current model allows for no change to ground surface levels upstream of the structure, which is reflected by applying no change to the cross sections in the model. However, some cross sections have been removed in the mitigated model to allow for the inclusion of the diversion channel, and the cross sections for the diversions have been added. In the existing case, the cross sections were extracted such that they would fairly represent flood storage in the overbank areas; essentially, the cross sections for the existing case covered the area of the flood plain. The removal and addition of cross sections for the diversion channels has necessarily changed this coverage such that some parts of the flood plain may not be adequately represented in the mitigated case. Alternatively, the overlapping of some cross sections in the mitigated case can over-estimate the volume of flood storage in particular areas of the model, though every effort has been taken to minimise this. The model for the mitigated case may therefore not accurately represent available storage upstream of the weir, which can affect the calculation of flood levels, and hence affluxes. Given that the flood plain upstream of the weir is essentially a two-dimensional flow area, it is recommended that any further analysis of the weir is undertaken with a two dimensional model.

Should the residual increases in flood levels prove impossible to remove by the weir alone, further supplementary strategies can be explored, including;

- the excavation of material from the flood plain upstream of the weir in order to increase the flood storage available
- the construction of a similar structure on Frank Creek.

The attraction of the structure on Frank Creek is that it provides an opportunity to mitigate the small increases in flood levels upstream of the confluence with Woleebee Creek.

These supplementary strategies have not been explored to date.

7. Conclusions and recommendations

The following conclusions are drawn from the flood study:

- several watercourses are affected by the proposed mining works. Creek diversions are required to permit access to the mining pits and to mitigate potential impacts on water quality due to the contact of flood waters with pits or other disturbed areas
- the flood modelling indicates that the mining works will not cause appreciable impacts to flood levels at the upstream MLA boundaries. The model indicates significant drawdown in Blackant creek, but this can be mitigated by developing the detailed design of the transition to the diversion
- flood levels at the Leichardt Highway are affected by less than 0.01 m
- the current model includes a preliminary design of the diversion channels. The abrupt transitions to the channels cause significant changes to flood levels within the MLA, which can be mitigated by developing the detailed design further. The channel design also needs to be developed to provide adequate flood conveyance, to mitigate scour and to mimic, as far as practicable, existing natural conditions
- the flood modelling indicates that without mitigation, there is an appreciable increase in 1%AEP flood levels (up to approximately 0.3 m) in Juandah Creek downstream of the MLA and in Woleebee Creek at the northern boundary of the MLA. This is principally due to the loss of flood storage on the Woleebee Creek flood plain due to the construction of the Woleebee Creek diversion
- much of the potential impact on Juandah creek can be mitigated by the construction of a partial floodplain embankment on the Woleebee Creek Diversion Channel without significantly increasing flood levels at the Southern MLA boundary. Further analysis of this option might decrease impacts to zero, or supplementary works can be employed, such as the removal of material in the flood plain upstream of the channel or the construction of a similar structure on Frank Creek
- the Windamere homestead is the only dwelling identified by the flood modelling as being affected by increased flood levels in the vicinity of the site. The flood modelling indicates that the 1%AEP flood level would increase by approximately 0.26 m (with no mitigation), but it would not rise enough to reach the dwelling itself. Mitigation works can be implemented that can reduce flood levels to existing levels
- the flood modelling has indicated the extents of flooding around the MLA areas, which may be used to locate infrastructure with appropriate levels of flood immunity
- the current modelling outcomes have exposed some uncertainties which will be investigated before proceeding with detailed design (feasibility stage). These include:
 - the assumptions surrounding the estimation of spatial distribution of rainfall in the observed historic events and the design events
 - the approximations of changes to rainfall losses to areas in which the GAB intake beds predominate
 - the spatial distribution of creek roughness values, as represented by the Manning's 'n' values in the hydraulic models.

It is acknowledged that the concept design of the diversion channels requires further development. Based on the model outcomes to date, it is expected that the Frank Creek diversion can be feasibly constructed to the criteria specified in the NRW watercourse diversion guidelines, but the diversion on Woleebee Creek is expected to be problematic. It should be noted that future revisions to the design may change the estimated affluxes upstream of the MLA. The consequences of these impacts on nearby infrastructure will be investigated in more detail during later stages of design.

The following recommendations are made for further impact assessment:

- given the scarcity of usable rainfall and stream flow data, it is recommended that a hydrological monitoring program is instituted in which a number of rain gauges and flow monitors are installed around the MLA. The objective is to collect further data against which the hydrological and hydraulic models can be calibrated. In particular, the influence of the GAB intake beds needs to be assessed further
- the hydrological and hydraulic models should be developed further to provide an appropriate level of detail for the waterworks licence approval process and the design of the proposed structures
- further hydraulic analyses should also include breach assessments for the levees if required under the referable dam legislation.

References

See the Flood Study Report, November 2008.

Attachment A

IFD Data

See the Flood Study Report, November 2008

Attachment B

XP-RAFTS model details

See the Flood Study Report, November 2008

Attachment C

Streamflow gauge 130344A

See amended analysis in Appendix G

Attachment D

MIKE11 model details

Table D-1 – MIKE11 Boundaries

Existing				Ultimate			
Type	Reference	Branch	Chainage	Type	Reference	Branch	Chainage
Q-h	s=0.0012	Juandah01	22725.08	Q-h	s=0.0012	Juandah01	22725.08
Inflow	B04Local	Juandah01	19288.80	Inflow	B04Local	Juandah01	19288.80
Inflow	D01ATotal	Branch01	0.00	Inflow	D01ATotal	Branch01	0.00
Inflow	D01BLocal	Branch01	512.56	Inflow	D01BLocal	Branch01	512.56
Inflow	D01CLocal	Branch01	4538.44	Inflow	D01CLocal	Branch01	4538.44
Inflow	D01DLocal	Woleebee01	16604.50	Inflow	D01DLocal	Woleebee02	16604.50
Inflow	D01ETotal	Branch06	1387.51	Inflow	D01ETotal	Branch06	1387.51
Inflow	D01FLocal	Woleebee01	18493.15	Inflow	D01FLocal	Woleebee02	18493.15
Inflow	D01GLocal	Woleebee01	21881.42	Inflow	D01GLocal	Woleebee02	21881.42
Inflow	D02ALocal	Blackant01	5392.30	Inflow	D02ALocal	Blackant01	5392.30
Inflow	D02BLocal	Blackant01	7238.81	Inflow	D02BLocal	Div_Woleebee	7433.06
Inflow	D02CTotal	Woleebee01	16604.50	Inflow	D02CTotal	Woleebee02	16604.50
Inflow	D03ALocal	Branch03	694.40	Inflow	D03ALocal	Div_Woleebee	4361.14
Inflow	D03BTTotal	Woleebee01	9271.75	Inflow	D03BTTotal	Div_Woleebee	1289.22
Inflow	D03CTotal	Woleebee01	11058.50	Inflow	D03CTotal	Div_OneArmMan	0.00
Inflow	D03DTTotal	Woleebee01	11276.55	Inflow	D03DTTotal	Div_Woleebee	4361.14
Inflow	D03ETotal	Woleebee01	12108.43	Inflow	D03ETotal	Div_Woleebee	4361.14
Inflow	D04ALocal	Wandoan01	1028.94	Inflow	D04ALocal	Wandoan01	1028.94
Inflow	D04BLocal	Wandoan01	3026.91	Inflow	D04BLocal	Wandoan01	3026.91
Inflow	D04CLocal	Wandoan01	7894.38	Inflow	D04CLocal	Div_OneArmMan	0.00
Inflow	D04DLocal	Wandoan01	10040.45	Inflow	D04DLocal	Div_Woleebee	4361.14
Inflow	D05ATotal	Woleebee01	6212.14	Inflow	D05ATotal	Div_OneArmMan	0.00
Inflow	D05BTTotal	Woleebee01	0.00	Inflow	D05BTTotal	Woleebee01	0.00
Inflow	D11ATotal	Blackant01	0.00	Inflow	D11ATotal	Blackant01	0.00
Inflow	D11BLocal	Blackant01	4331.94	Inflow	D11BLocal	Blackant01	4331.94
Inflow	D12ALocal	OneArmMan01	3035.07	Inflow	D12ALocal	OneArmMan01	3035.07
Inflow	D12BTTotal	OneArmMan02	0.00	Inflow	D12BTTotal	OneArmMan02	0.00
Inflow	D12CLocal	OneArmMan01	74.44	Inflow	D12CLocal	OneArmMan01	74.44
Inflow	D13Total	OneArmMan01	0.00	Inflow	D13Total	OneArmMan01	0.00
Inflow	D14Total	Wandoan01	0.00	Inflow	D14Total	Wandoan01	0.00
Inflow	E01ATotal	Juandah01	13304.56	Inflow	E01ATotal	Juandah01	13304.56
Inflow	E01BTTotal	Juandah01	8034.07	Inflow	E01BTTotal	Juandah01	8034.07
Inflow	E01CTotal	Branch07	0.00	Inflow	E01CTotal	Branch07	0.00
Inflow	E02ATotal	Juandah01	2399.03	Inflow	E02ATotal	Juandah01	2399.03
Inflow	E02BTTotal	Juandah01	3209.81	Inflow	E02BTTotal	Juandah01	3209.81
Inflow	E02CTotal	Juandah05	1190.26	Inflow	E02CTotal	Juandah05	1190.26
Inflow	E02DTTotal	Juandah05	3510.16	Inflow	E02DTTotal	Juandah05	3510.16
Inflow	E02ETotal	Juandah01	6732.69	Inflow	E02ETotal	Juandah01	6732.69
Inflow	E02FTotal	Frank01	10842.48	Inflow	E02FTotal	Div_Frank	407.00
Inflow	E02GTotal	Frank01	8297.73	Inflow	E02GTotal	Frank01	8297.73
Inflow	E02HTotal	Frank01	8962.78	Inflow	E02HTotal	Frank01	8962.78
Inflow	E02ITotal	Frank01	10742.10	Inflow	E02ITotal	Div_Frank	407.00
Inflow	E02JTotal	Juandah01	3677.71	Inflow	E02JTotal	Juandah01	3677.71
Inflow	E02KTotal	Frank01	6054.39	Inflow	E02KTotal	Frank01	6054.39
Inflow	E03Total	Juandah01	0.00	Inflow	E03Total	Juandah01	0.00
Inflow	E09Total	Juandah01	437.88	Inflow	E09Total	Juandah01	437.88
Inflow	E12ALocal	Frank01	4591.52	Inflow	E12ALocal	Frank01	4591.52
Inflow	E12BLocal	Frank01	3174.17	Inflow	E12BLocal	Frank01	3174.17
Inflow	E12CLocal	Frank01	432.07	Inflow	E12CLocal	Frank01	432.07
Inflow	E13Total	Frank01	0.00	Inflow	E13Total	Frank01	0.00
Inflow	E14ALocal	Halfway01	4747.41	Inflow	E14ALocal	Halfway01	4747.41
Inflow	E14BTTotal	Halfway02	0.00	Inflow	E14BTTotal	Halfway02	0.00
Inflow	E14CLocal	Halfway01	702.11	Inflow	E14CLocal	Halfway01	702.11
Inflow	E15Total	Halfway01	0.00	Inflow	E15Total	Halfway01	0.00

Table D-2 – Typical MIKE11 SIM11 File – Existing

```
//Created      : 2009-06-5 11:10:31
//DLL id       : C:\PROGRA~1\COMMON~1\DHI\MikeZero\pfs2004.dll
//PFS version  : Nov 27 2007 20:39:43

[Run11]
  format_version = 107, 'MIKEZero, 2008'
  Comment = ''
  [Models]
    hd = true, false
    ad = false
    st = false
    wq = false, 1, 0
    rr = false
    ff = false
    da = false
    ice = false
    SimMode = 0
    QSS = 0
  EndSect //Models

  [Input]
    nwk = |..\Input\Existing.nwk11|
    xs = |..\Input\Wandoan.xns11|
    bnd = |..\Input\Ex_0001yr_1800min.bnd11|
    rr = ||
    hd = |..\Input\Existing.hd11|
    ad = ||
    wq = ||
    st = ||
    ff = ||
    rhd = ||
    rrr = ||
    da = ||
    ice = ||
  EndSect //Input

  [Simulation]
    [Simulation_Period]
      start = 2000, 1, 1, 0, 0, 0
      end = 2000, 1, 4, 11, 40, 0
      TimeStepType = 0
      timestep = 5
      timestepunit = 3
      dtFileName = ||
      dtItemName = ''
      dtItemNo = 0
      ddtMin = 1
      ddtMax = 30
      idtMinMaxUnit = 2
      ddtChangeRatio = 1.3
      bDelB_BFlag = true
      dDelB_BVal = 0.01
      dDelB_BLim = 0.01
      bDelQFlag = false
      dDelQVal = 1
      bDelQ_QFlag = true
      dDelQ_QVal = 0.01
      dDelQ_QLim = 0.01
      bDelhFlag = false
      dDelhVal = 0.01
      bDelh_hFlag = true
      dDelh_hVal = 0.01
```

```

    dDelh_hLim = 0.01
    bCourantFlagHD = false
    dCourantValHD = 10
    bCourantFlagAD = true
    dCourantValAD = 1
    ST_timestep_multiplier = 1
    RR_timestep_multiplier = 1
EndSect //Simulation_Period

[Initial_Conditions]
    hd = 2, |..\res11\Ex_Hotstart.res11|, false, 2000, 1, 1, 0, 0, 20
    ad = 0, ||, false, 1990, 1, 1, 12, 0, 0
    st = 0, ||, false, 1990, 1, 1, 12, 0, 0
    rr = 0, ||, false, 1990, 1, 1, 12, 0, 0
EndSect //Initial_Conditions

EndSect //Simulation

[Results]
    hd = |..\res11\Existing_0001yr_1800min.res11|, '', 5, 3
    ad = ||, '', 1, 0
    st = ||, '', 1, 0
    rr = ||, '', 1, 0
EndSect //Results

EndSect //Run11

[SYSTEM]
    ResultRootFolder = |..\Result|
    UseCustomResultFolder = false
    CustomResultFolder = ||
EndSect //SYSTEM

```

Table D-3 – Typical MIKE11 SIM11 File – Proposed

```

//Created      : 2009-06-12 15:8:34
//DLL id       : C:\PROGRA~1\COMMON~1\DHI\MikeZero\pfs2004.dll
//PFS version  : Nov 27 2007 20:39:43

[Run11]
    format_version = 107, 'MIKEZero, 2008'
    Comment = ''
    [Models]
        hd = true, false
        ad = false
        st = false
        wq = false, 1, 0
        rr = false
        ff = false
        da = false
        ice = false
        SimMode = 0
        QSS = 0
    EndSect //Models

    [Input]
        nwk = |..\Input\Ultimate.nwk11|
        xs = |..\Input\Wandoan.xns11|
        bnd = |..\Input\Ult_0001yr_1800min.bnd11|
        rr = ||
        hd = |..\Input\Existing.hd11|
        ad = ||
        wq = ||

```

```

st = ||
ff = ||
rhd = ||
rrr = ||
da = ||
ice = ||
EndSect //Input

[Simulation]
[Simulation_Period]
start = 2000, 1, 1, 0, 0, 0
end = 2000, 1, 4, 0, 0, 0
TimeStepType = 0
timestep = 20
timestepunit = 3
dtFileName = ||
dtItemName = ''
dtItemNo = 0
ddtMin = 1
ddtMax = 30
idtMinMaxUnit = 2
ddtChangeRatio = 1.3
bDelB_BFlag = true
dDelB_BVal = 0.01
dDelB_BLim = 0.01
bDelQFlag = false
dDelQVal = 1
bDelQ_QFlag = true
dDelQ_QVal = 0.01
dDelQ_QLim = 0.01
bDelhFlag = false
dDelhVal = 0.01
bDelh_hFlag = true
dDelh_hVal = 0.01
dDelh_hLim = 0.01
bCourantFlagHD = false
dCourantValHD = 10
bCourantFlagAD = true
dCourantValAD = 1
ST_timestep_multiplier = 1
RR_timestep_multiplier = 1
EndSect //Simulation_Period

[Initial_Conditions]
0 hd = 2, |..\res11\Ultimate_hotstart.res11|, false, 2000, 1, 1, 0, 0,

ad = 0, ||, false, 1990, 1, 1, 12, 0, 0
st = 0, ||, false, 1990, 1, 1, 12, 0, 0
rr = 0, ||, false, 1990, 1, 1, 12, 0, 0
EndSect //Initial_Conditions

EndSect //Simulation

[Results]
hd = |..\res11\Ultimate_0001yr_1800min.res11|, '', 5, 3
ad = ||, '', 1, 0
st = ||, '', 1, 0
rr = ||, '', 1, 0
EndSect //Results

EndSect //Run11

[SYSTEM]
ResultRootFolder = |..\Result|

```

```
UseCustomResultFolder = false
CustomResultFolder = ||
EndSect //SYSTEM
```

Table D-4 – Branch Connections

Existing			US Connection		DS Connection	
Branch	US Chainage	DS Chainage	Branch	Chainage	Branch	Chainage
Juandah01	0.00	22750.62	0	0.00	0	0.00
Branch01	0.00	8883.81	0	0.00	Woleebee01	23960.39
Branch02	0.00	2954.21	Woleebee01	18250.01	Woleebee01	19834.39
Branch03	0.00	1081.30	Wandoan01	10564.35	Woleebee01	11476.94
Blackant01	0.00	7873.96	0	0.00	Woleebee01	15002.51
Branch04	0.00	2401.06	Woleebee01	5521.97	Wandoan01	5980.40
Branch05	0.00	2567.11	Woleebee01	552.94	Wandoan01	3115.06
Branch06	0.00	1491.04	Woleebee01	17521.43	branch02	1263.65
Branch07	0.00	4415.85	0	0.00	Juandah01	8906.02
Branch08	0.00	2733.47	Woleebee01	22257.75	Juandah01	11892.22
Frank01	0.00	14309.35	0	0.00	Juandah01	5714.53
Halfway01	0.00	7469.74	0	0.00	Frank01	6821.30
Halfway02	0.00	3248.82	0	0.00	Halfway01	3424.43
Juandah03	0.00	1102.78	Juandah01	4453.48	Frank01	13892.44
Juandah04	0.00	547.42	Juandah01	4993.20	Juandah03	904.80
Juandah05	0.00	4730.38	Juandah01	4665.68	Juandah01	6961.23
Juandah06	0.00	2979.27	Juandah01	13715.49	Juandah01	15730.86
Juandah07	0.00	1421.05	Juandah01	4119.40	Frank01	13213.23
OneArmMan01	0.00	5123.64	0	0.00	Woleebee01	6111.85
OneArmMan02	0.00	1111.75	0	0.00	OneArmMan01	3191.87
Wandoan01	0.00	11773.65	0	0.00	Woleebee01	13762.42
Woleebee01	0.00	24133.28	0	0.00	Juandah01	12542.17

Ultimate			US Connection		DS Connection	
Branch	US Chainage	DS Chainage	Branch	Chainage	Branch	Chainage
Juandah01	0.00	22750.62	0	0.00	0	0.00
Branch01	0.00	8883.81	0	0.00	Woleebee02	23960.39
Branch02	0.00	2954.21	Woleebee02	18250.01	Woleebee02	19834.39
Blackant01	0.00	5392.30	0	0.00	Div_Woleebee	5946.65
Branch05	0.00	2567.11	Woleebee01	552.94	Wandoan01	3115.06
Branch06	0.00	1491.04	Woleebee02	17521.43	branch02	1263.65
Branch07	0.00	4415.85	0	0.00	Juandah01	8906.02
Branch08	0.00	2733.47	Woleebee02	22257.75	Juandah01	11892.22
Frank01	0.00	10150.17	0	0.00	Div_Frank	0.00
Halfway01	0.00	7469.74	0	0.00	Frank01	6821.30
Halfway02	0.00	3248.82	0	0.00	Halfway01	3424.43
Juandah03	0.00	1102.78	Juandah01	4453.48	Frank02	13892.44
Juandah04	0.00	547.42	Juandah01	4993.20	Juandah03	904.80
Juandah06	0.00	2979.27	Juandah01	13715.49	Juandah01	15730.86
Juandah07	0.00	1421.05	Juandah01	4119.40	Frank02	13213.23
OneArmMan01	0.00	4474.36	0	0.00	Div_OneArmM	0.00
OneArmMan02	0.00	1111.75	0	0.00	OneArmMan01	3191.87
Wandoan01	0.00	4085.85	0	0.00	Div_Woleebee	1289.22
Woleebee01	0.00	3726.05	0	0.00	0	0.00
Branch09	0.00	969.36	Branch01	8064.85	Juandah06	762.77
Juandah05	0.00	4730.38	Juandah01	4665.68	Juandah01	6961.23
Div_OneArmMan	0.00	1832.64	0	0.00	Woleebee01	3726.05
Div_Woleebee	0.00	8424.50	Woleebee01	3726.05	Woleebee02	15442.86
Woleebee02	15442.86	24132.98	0	0.00	Juandah01	12542.17
Div_Frank	0.00	3115.85	0	0.00	Frank02	13120.63
Frank02	13120.63	14309.41	0	0.00	Juandah01	5714.53

Table D-5 – Levee Locations

Creek	Chainage	Coords at CL		Levee			
				Left Bank		Right Bank	
				X	Z	X	Z
FRANK01	4889.88	792994.16	7105354.40				
FRANK01	4990.19	792985.77	7105454.34			75.181	253.553
FRANK01	5098.83	792930.21	7105547.70			137.175	252.355
FRANK01	5202.99	792966.98	7105645.16			108.026	252.007
FRANK01	5318.58	792892.08	7105733.20			189.756	252.873
FRANK01	5444.61	792934.09	7105852.03			155.874	252.145
FRANK01	5547.70	792891.13	7105945.74			206.417	251.646
FRANK01	5702.71	793005.51	7106050.35			99.094	250.153
FRANK01	5850.41	793094.11	7106168.52			19.168	250.314
FRANK01	5937.03	793061.51	7106248.78			80.695	249.308
FRANK01	6054.39	793112.16	7106354.65			69.695	249.376
FRANK01	6211.31	792996.74	7106460.96			222.395	248.892
FRANK01	6313.16	793035.78	7106555.03			220.997	248.156
FRANK01	6434.32	793103.73	7106655.35			189.770	249.323
FRANK01	6535.62	793118.41	7106755.57			212.887	249.125
FRANK01	6650.68	793064.25	7106857.09			349.194	249.684
FRANK01	6821.30	792916.68	7106942.75				
FRANK01	6971.70	792833.21	7107068.22				
FRANK01	7076.61	792808.98	7107169.98				
FRANK01	7257.19	792994.44	7107265.08				
FRANK01	7339.17	792960.18	7107351.55				
FRANK01	7446.17	793021.01	7107439.58				
FRANK01	7547.06	793041.03	7107538.46				
FRANK01	7673.70	793139.89	7107617.60	-182.537	245.046		
FRANK01	7776.80	793133.86	7107720.52	-182.939	244.637		
FRANK01	7928.85	793255.22	7107812.12	-306.993	244.458		
FRANK01	8065.05	793169.38	7107917.87	-228.105	244.299		
FRANK01	8176.21	793219.16	7108017.26	-283.757	243.960		
FRANK01	8297.73	793186.01	7108134.18	-253.120	243.903	386.528	246.484
FRANK01	8389.53	793149.00	7108218.18	-194.189	244.474	374.122	246.463
FRANK01	8511.10	793088.81	7108323.81	-107.319	242.454	383.813	244.791
FRANK01	8623.89	793023.37	7108415.67	-23.643	241.690	414.792	244.935
FRANK01	8730.16	793042.35	7108520.23	-19.214	242.215	386.098	244.315
FRANK01	8838.39	793086.10	7108619.23	-40.867	242.445	331.411	243.688
FRANK01	8962.78	793128.49	7108736.18	-57.611	242.146	276.282	242.140
FRANK01	9060.16	793164.31	7108826.73	-73.295	242.160	229.565	241.732
FRANK01	9157.90	793168.36	7108924.38	-55.414	242.348	216.163	240.978
FRANK01	9258.42	793156.22	7109024.16	-55.997	242.655	214.695	241.675
FRANK01	9367.42	793129.63	7109129.88	-46.263	240.791	227.451	241.072
FRANK01	9484.99	793057.64	7109222.83	-22.087	245.999	288.849	241.494
FRANK01	9612.16	793122.59	7109332.17			211.574	242.002
FRANK01	9719.07	793180.44	7109422.05	-154.438	241.731	136.142	241.293
FRANK01	9814.55	793175.10	7109517.38	-335.500	246.386	125.391	240.033
FRANK01	9910.66	793202.09	7109609.62			284.741	239.009
FRANK01	10047.40	793313.17	7109689.36			363.069	241.497
FRANK01	10150.17	793370.23	7109774.84				
FRANK01		793415.50	7109863.52				

Attachment E

MIKE21 model build details

See the Flood Study Report, November 2008

Attachment F

Plans and maps

Attachment G

MIKE11 model validation

Introduction

This Appendix attempts to reconcile observed flood information with model outcomes. The section 'Available Anecdotal Evidence' includes the information gathered from residents and the following sections compare this evidence with model outcomes in terms of estimated extents of inundation and maximum flood levels.

Available Anecdotal Evidence

Anecdotal evidence of flooding was gathered by Sandy Hannibal from residents in May 2009. Most of the evidence related to the 1983 flood, with some relating to the 1991 flood. The text of the summary is reproduced as follows:

Flood observations for Woleebee and Juandah creeks.

Rick Mathews and I visited landowners along the Woleebee and also Juandah ck to obtain recollections of flooding and also to record flood heights.

Daryl Waugh provided flood heights along the creek both for 1983 and 1991

1991 not the edge of the water but a fence post as in historic photo (Reference – 1991 Observation O2) 787903 7114006N 230 m elevation the extent on the western side of the creek was 787045 7113999N 230 m

1983 eastern side 787829 7114824N 226 m

1983 and 1991 approx 787770 7115612N 230 m

Daryl commented that the closer Woleebee ck got to Juandah the more it would back up and the longer it stayed up for example near his house it would go down with in 48 hours but near Juandah it would be 5 or 6 days.

Brian and Maree Westman 1983 flood levels below Beechley on the Grosmont road western side of creek 786200 7111063N 238 m eastern side extending to 786796 7111339N 238 m another point on the eastern side of the creek the water extended out to 786816 7110339. Brian and Maree also agreed that the river would drop quickly as early as 24 hours.

Ian Baker owned Cambray in 1983 he recalled that when the flood came they had up to 200 mm of rain at the head of the creek near the ranges and that locally around 150 mm had fallen and that the flood waters started to recede overnight and then regained there banks with in 3 to 4 days there was approximately 60 acres around the Cambray homestead that did not go under the water reached up to 785827 7107280N 240 m southern side of house and Northern side 785817 7107545N 240 m

Brian and Marty Sands recollections are of Juandah creek mostly however say that by comparison Woleebee is a very fast running creek much faster than Juandah so that during a flood you would consider swimming Juandah but definitely not Woleebee.

During the 1983 floods the water went over the house dam and flood waters extended to 791008 7113942N 236 m and also covered a strainer post located at 791021 7114100N 236 m

Ron Fitzgerald recalls that during the 1991 flood Woleebee ck was full as were all the creeks around however had not burst its bank until 4 inches fell in the Conroy creek catchment over night.

One arm man ck would go over the bridge however not spread widely or very high."

The following photographs were also provided:



Reference – 1991 Observation 02

Attached text reads 'Photo's taken of flood February 1991 Woleebee Ck Wandoan on 'Wodonga' the Waugh family property photo's and reference points provided by Daryl Waugh'. Also 'Looking North west down Woleebee creek, strainer post at left with stay showing is reference point 787903E 7114006N 230 m'.



Reference – 1991 Observation 02

Attached text reads ‘This Photo looking up Woleebee creek where lane way crosses’.

The above observations are summarised in Tables G1 and G2 below.

Table G1 – Summary of Observations of 1983 Flood

Ref	Observation	Coordinates	
		Easting	Northing
1983 Observation 01	Extent of flooding - east bank	787770	7115612
1983 Observation 02	Extent of flooding - east bank	787829	7114824
1983 Observation 03	Extent of flooding - east bank	786796	7111339
1983 Observation 04	Extent of flooding - west bank	786200	7111063
1983 Observation 05	Extent of flooding - east bank	786816	7110339
1983 Observation 06	Extent of flooding - north side of house	785817	7107545
1983 Observation 07	Extent of flooding - south side of house	785827	7107280
1983 Observation 08	Flooded fence post - post covered	791021	7114100
1983 Observation 09	Extent of flooding - south bank	791008	7113942

Table G2 – Summary of Observations of 1991 Flood

Ref	Observation	Coordinates	
		Easting	Northing
1991 Observation 01	Extent of flooding - east bank	787770	7115612
1991 Observation 02	Extent of flooding - west bank	787045	7113999
1991 Observation 03	Flooded fence post (see photo)	787903	7114006

Validation events

1991 validation method

The 1991 flood was modelled in XP-RAFTS, so this was used as the primary event for the purposes of model validation.

The validation method comprised the derivation of a 'best fit' to the available data. This included:

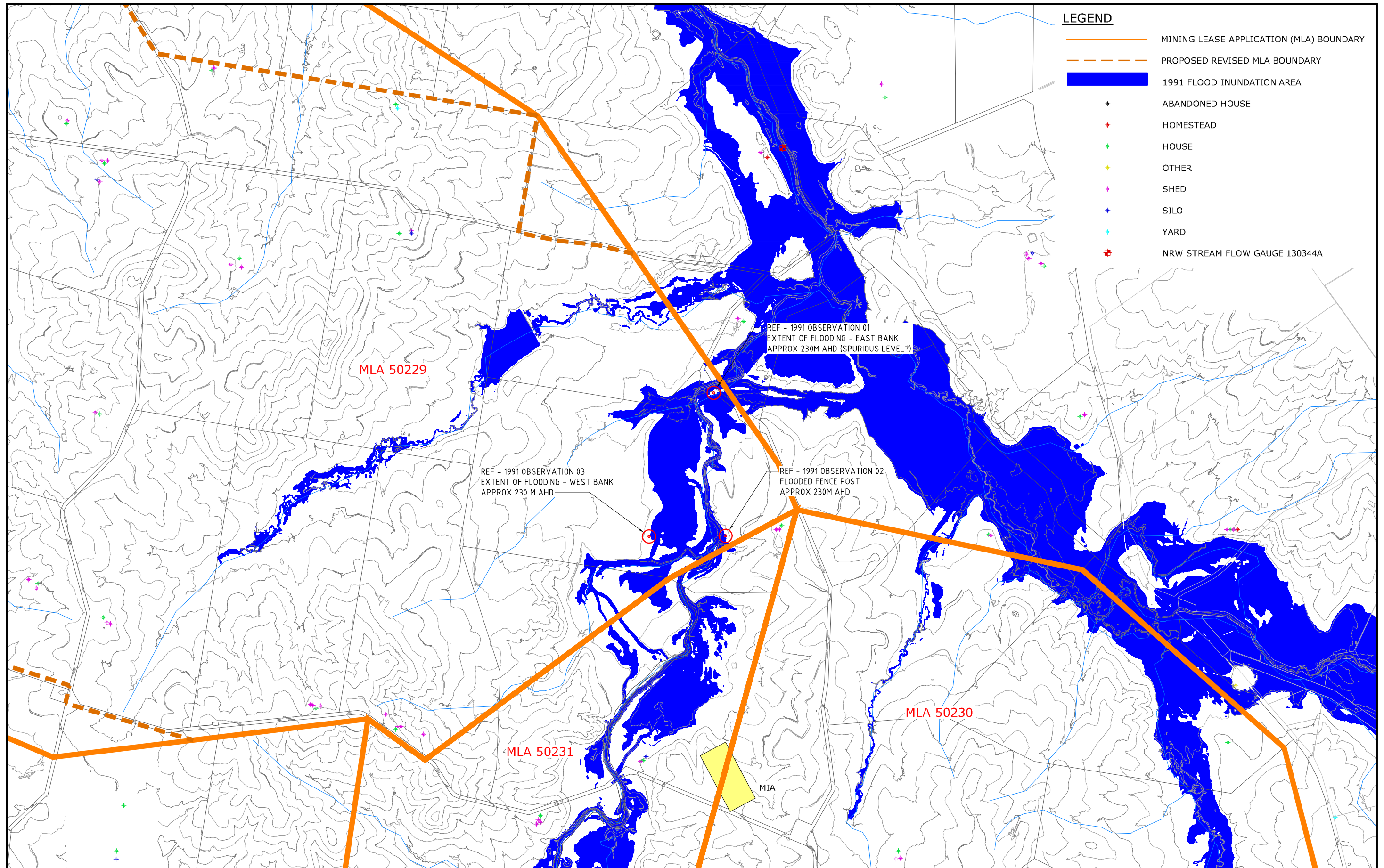
- comparison of computed flood extents with the observed flood extents
- comparison of computed flood levels with the observed levels
- comparison of the computed hydrograph with the recorded hydrograph at Windamere
- comparison of computed flow characteristics with qualitative statements in observations (e.g. velocities).

The 'best fit' was obtained by adjusting creek roughness values in the MIKE11 model for the existing case.

1991 Validation – Global Manning's $n = 0.06$

Results – extents of inundation

The modelled extents of inundation are shown in Figure G-1. This figure shows that there is good agreement between the observed extents of flooding and the model results.



Plotted By: hamilm Plot Date: 07/08/09 - 10:26 Cad File: J:\A442-ENG\PROJ\2133006K_Wandoan_Supple\09_CADD\DRAWINGS\Figures_TechReport\Figure G-1.dwg



Figure G-1
Extent of 1991 Flooding

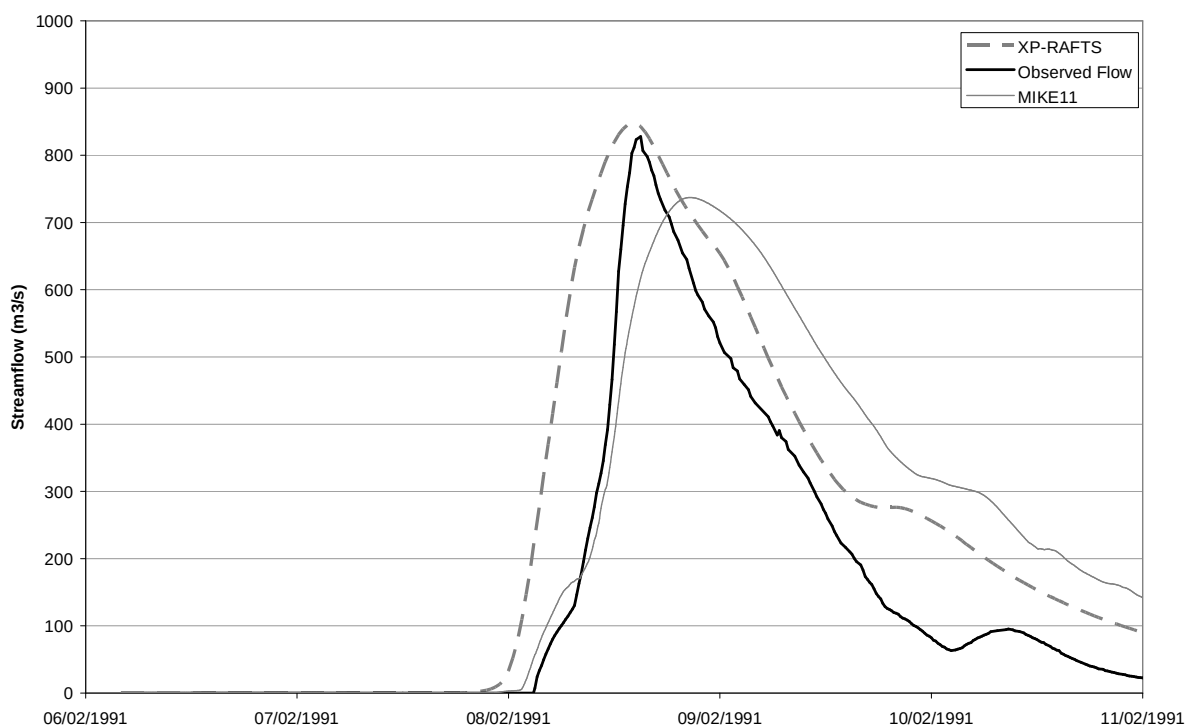
Results – flood levels

The computed flood levels, as indicated below, were reasonably close to the observed levels.

Ref	Observation	Coordinates		Elevation (m AHD)	
		Easting	Northing	From DTM	MIKE11
1991 Observation 01	Extent of flooding - east bank	787770	7115612	231.6	231.2
1991 Observation 02	Extent of flooding - west bank	787045	7113999	232.8	232.6
1991 Observation 03	Flooded fence post (see photo)	787903	7114006	233.1	232.8
Windamere Gauge	Maximum flood level	788544	7118363	227.6	228.1

Results – Windamere hydrograph

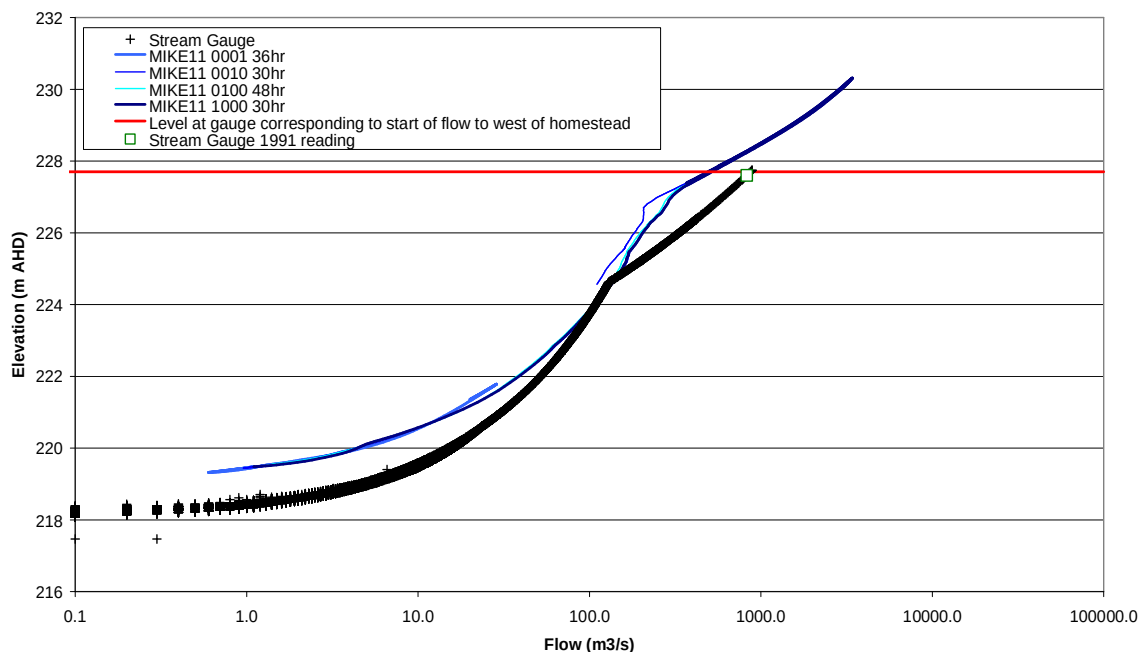
The computed hydrograph was considerably attenuated from the recorded hydrograph, as illustrated below. The peak flow was lower and the time to peak was extended.



Hydrographs at Windamere Stream Gauge for MIKE11 Calibration

The MIKE11 results show a good match for the rising limb of the hydrograph, but the MIKE11 model attenuates peak flows from 828 m³/s to 721 m³/s. The MIKE11 model also increases peak flood levels from 227.6 m AHD to 228.1 m AHD. The following plot compares the rating curves at the gauge.

Rating Curves at Windamere



The above plot shows a poor fit with the rating curve. In particular, the commencement of out-of-bank flow for the model is at about 226.5 m AHD, whereas the rating curve shows it at about 224.5 m AHD. Further work may be required to reconcile the rating curve with the model results, which may require further development of the hydrological model. PB has discussed the accuracy of the overbank part of the rating curve with the owners of the Stream Gauge (NRM). NRM considers that the overbank part of the rating curve is not as accurate as an estimate that is based on hydraulic analysis.

It is also noted that the rainfall distribution in the XP-RAFTS model is based on widely spaced rain gauge data, which may adversely affect the reliability of the distribution of flows into the MIKE11 model.

1991 Validation – Sensitivity tests on global Manning's n

Further trials were attempted that varied the values of the global Manning's n value. It was found that these trials yielded poorer fits with the observed data. For example, increasing the Manning's n would increase the flood levels in Wandoan Creek, but the peak flow at the Windamere Stream gauge would decrease.

Results – summary

The MIKE11 model results indicate that the observed extents of inundation for the 1983 flood event lies within the range for the 0.095%AEP to 1%AEP events, as expected (to be confirmed when we have plots of the extents of inundation)

The MIKE11 model results indicate reasonable matches to the observed extents of flooding and the observed flood heights (corrected to the DTM) for the February 1991 event.

The MIKE11 model results at the gauge at Windamere show higher flood levels and lower flows than the recorded levels and flows. Further work may be required to reconcile the model results with the recorded gauge data.

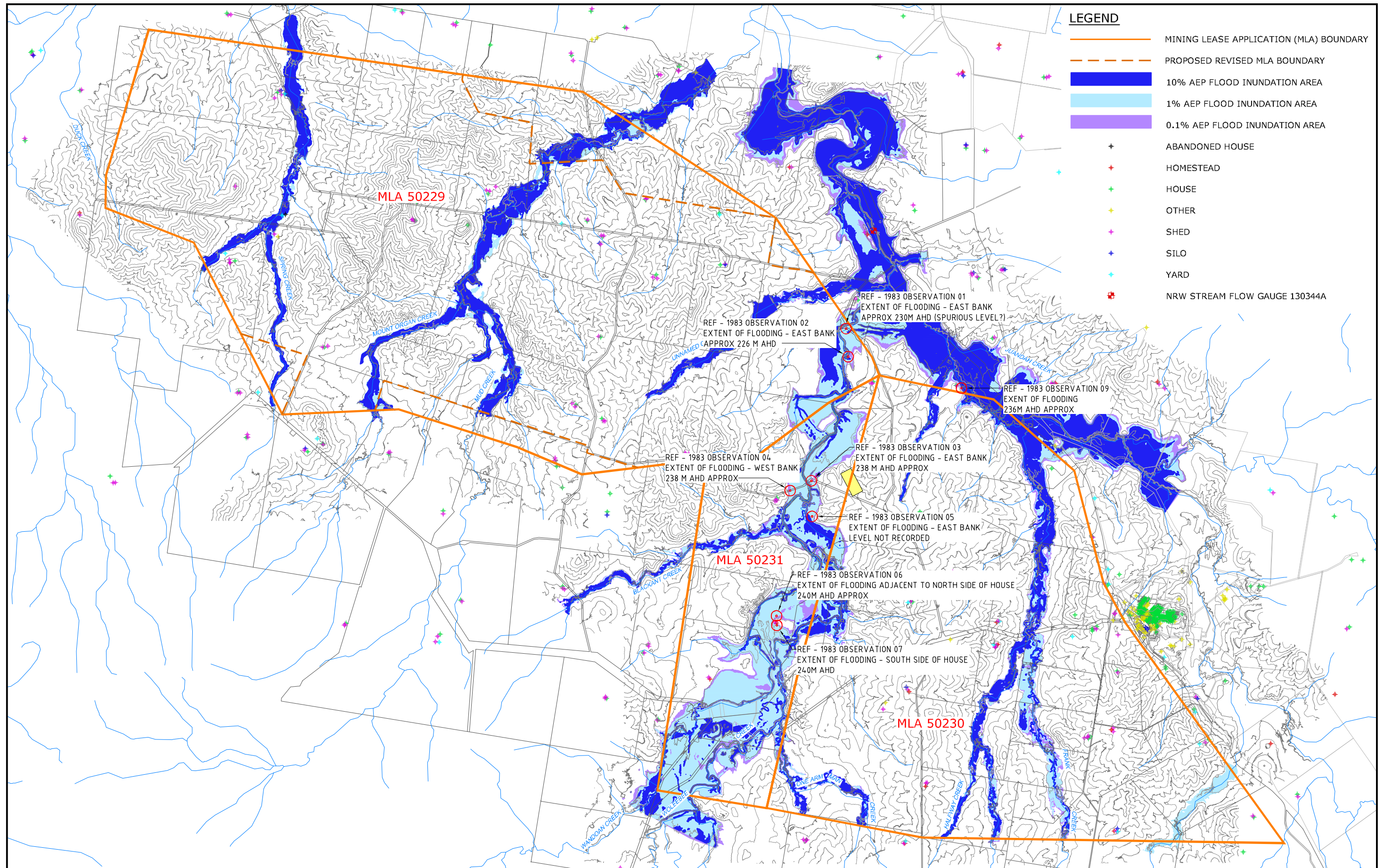
It is considered that, given the quality of the input data, the model represents the best practical fit to the observed data.

1983 Validation – Global Manning’s $n = 0.06$

As noted in the previous report, the 1983 flood could not be modelled accurately because of a lack of rainfall data. The annual exceedence probability (AEP) of the 1983 flood has been estimated previously, based on its peak flow. These estimates yield an AEP of 2% to 5% not including the confidence limits, and an AEP of 2% to 10% including the confidence limits. The observed extents of inundation were compared with the computed extents of inundation for design events to assess whether they fell within this range.

Results – extents of inundation

The modelled extents of inundation are shown in Figure G-2. This figure shows that the observed extents of flooding lie between the model results for the 9.5%AEP and 1% AEP floods, which are within the expected range for the 1983 flood event.



Plotted By: hamillm Plot Date: 07/08/09 - 10:34 Cad File: J:\A442-ENG\PROJ\2133006K_Wandoan_Supple\09_CADD\DRAWINGS\Figures_TechReport\Figure G-2.dwg

Note: Overbank Flood Storage within the Hydraulic Model

The MIKE11 model accounts for flood storage, including overbank storage, within its calculations at each cross section. Incremental flood storage in the overbank areas is calculated by considering the width of inundation at the cross section and the distance between cross sections.

Where the flood plain is relatively straight, cross sections can be drawn as an approximately parallel series. In this case, the lengths of the channel and overbank areas and channel are similar and the model provides a good estimation of flood storage.

Where the river passes around pronounced bends, however, the distance between cross sections in the overbank areas differs from the distance between cross sections in the channel, which introduces approximations into the estimation of overbank flood storage.

In the current model, cross sections have been defined as approximately parallel series as far as practicable. The areas where parallel series are not possible are the big bends in Juandah Creek downstream of the Windamere Stream Gauge and the anabranches of Woleebee Creek. Whilst every effort has been made to avoid the overlapping of cross sections, it is acknowledged that the non-parallel nature of cross sections in these areas may not accurately model overbank flood storage. It is considered that the behaviour of flooding in these areas is at the limits of a one-dimensional analysis, such as MIKE11. Further detailed analysis of these areas should proceed with two dimensional modelling.

Regarding the big bends in Juandah Creek downstream of the stream gauge, it is noted that the extension of the model over this area was not intended to accurately model the bends themselves, but to move the model boundary away from the stream gauge.

Attachment B

Addendum to surface water quality
technical report

Wandoan Coal Project Addendum to the Surface water quality impact assessment

September, 2009

Wandoan Joint Venture



Parsons Brinckerhoff Australia Pty Limited ABN 80 078 004 798

Level 4, Northbank Plaza

69 Ann Street

Brisbane QLD 4000

GPO Box 2907

Brisbane QLD 4001

Australia

Telephone +61 7 3854 6200

Facsimile +61 7 3854 6500

Email brisbane@pb.com.au

NCSI Certified Quality System ISO 9001

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Author: Dr Sanja Oldridge — Senior Water Engineer

Signed:

Reviewer: Helen D'Arcy — Senior Environmental Engineer

Signed:

Approved by: Helen D'Arcy — Senior Environmental Engineer

Signed:

Date: September 2009

Distribution: Wandoan Joint Venture, PB File, PB Library

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Glossary

<i>Al</i>	Aluminium
<i>ANZECC</i>	Australian and New Zealand Environment and Conservation Council
<i>As</i>	Arsenic
<i>Cd</i>	Cadmium
<i>Cr</i>	Chromium
<i>Cu</i>	Copper
<i>DO</i>	Dissolved Oxygen
<i>EC</i>	Electrical Conductivity
<i>Electrical Conductivity</i>	Electrical conductivity or salinity are measures of the total concentration of inorganic ions (salts) in the water.
<i>ESCP</i>	Erosion and Sediment Control Plan
<i>Fe</i>	Iron
<i>Hg</i>	Mercury
<i>Hardness as CaCO₃</i>	Hardness is expressed in mg/L as CaCO ₃ . Increasing calcium and magnesium in waters (hardness) is usually associated with increases in alkalinity. Changes in alkalinity will directly affect metal speciation.
<i>MLA</i>	Mining Lease Application
<i>Mn</i>	Manganese
<i>Ni</i>	Nickel
<i>NO₂ and NO₃</i>	Nitrite and nitrate
<i>OC</i>	Organochlorine Pesticides
<i>OP</i>	Organophosphorus Pesticides
<i>Pb</i>	Lead
<i>Stream order</i>	Streams in geographic terms are awarded order designations. A stream of the first order is a stream which does not have any other stream feeding into it. A stream of the second order is one which is formed by the joining of two or more streams. A third-order stream is one below the confluence of two or more second-order streams; a fourth-order stream is formed by the confluence of at least two third-order streams, and so forth.
<i>TP</i>	Total Phosphorus
<i>TN</i>	Total Nitrogen
<i>Tributary</i>	A tributary is a stream which flows into another river (a key stream).
<i>TSS</i>	Total Suspended Solids
<i>Turbidity</i>	The turbidity or 'muddiness' of water is caused by the presence of suspended particulate and colloidal matter consisting of suspended clay, silt, phytoplankton and detritus.
<i>WQOs</i>	Water Quality Objectives
<i>Zn</i>	Zinc

Executive summary

This Addendum to the surface water quality impact assessment for the Wandoan Coal Project EIS includes the following updates to the original surface water quality impact assessment technical report:

- further water quality data made available since mid September 2008 to March 2009
- responses to submissions on the EIS relevant to the surface water quality impact assessment
- results, findings and mitigation measures associated with received submissions on the EIS and new water quality data.

As mentioned in the original technical report, the water quality assessment examined the following datasets:

- historical data from the following sources:
 - MIM Holdings, from June 1985 to January 1987
 - Department of Natural Resources and Water (NRW), from 1964 (earliest) to 2005 (latest).
- recent data including two water quality datasets provided for assessment by the Wandoan Joint Venture, being:
 - daily log data recorded at generally four hourly intervals and/or during a change in (stream) water level; measuring temperature, conductivity, pH, DO and turbidity parameters, recorded from March 2008 to March 2009. This report includes and builds on the previous technical report that assessed water quality data from March 2008 to mid September 2008
 - event data sampling undertaken by Ecowise Environmental supplied by the WJV, with dates outlined in Table 4-2.

The recent daily log and event datasets, not included in the original technical report, were merged with the original EIS datasets. The combined data for daily log and event water quality datasets range from March 2008 to March 2009. These latest datasets were used to update results and findings associated with the description of the background surface water quality values examining recent water quality datasets.

The updated findings associated with the most recent datasets of the daily and event sampling indicate the following changes to the original technical report:

- elevated heavy metal levels particularly iron, copper and zinc, with exceedences for chromium, copper, lead, nickel and zinc at some sites compared to ANZECC guidelines
- elevated Chlorophyll a at some sites.

The assessment of water quality in relation to variations with water level were assessed for DO and pH levels, because *'extreme flow conditions affect water quality in streams, mainly DO and pH levels'* (EPA 2006, p. 50). High pH levels generally occur in natural ecosystems when aquatic vegetation uses CO₂ for photosynthesis, causing hypoxic conditions (i.e. low O₂). Streams in the Project area are ephemeral, with streams indicating more basic pH levels in stagnant water, however DO percentage concentrations do not follow a particular trend in relation to water level.

The event water quality data indicates the following water quality exceedances compared to ANZECC guidelines:

- physical and chemical stressors and toxicants:
 - TSS, turbidity, TN, TP, Iron at all sites
 - chlorophyll a at Woleebee Creek (downstream) and Mud Creek (upstream)
 - chromium, copper, lead, nickel and zinc at Woleebee Creek (upstream)
 - chromium, copper, lead and zinc at Woleebee Creek (downstream), Juandah (downstream and upstream), Mud Creek (upstream), and Spring Creek (upstream)
 - chromium, copper, and zinc at Mud Creek (downstream)
 - copper and zinc at Frank Creek (downstream) and Mount Organ Creek (upstream).

A submission on the EIS relevant to the surface water quality impact assessment identified an important water quality issue that came out of the *'Report to Queensland Premier Review of the Fitzroy River Water Quality Issues'*, and that is the need to include monitoring and analysis of selenium in both water and sediment samples. Selenium has been included in the list of recommended parameters for water quality monitoring for the Project.

A site assessment of each daily monitoring site should be undertaken to identify a water level in a stream that indicates extreme low flow condition (i.e. waterhole with stagnate water affecting DO and pH levels). This change should be reflected in the water quality monitoring procedure.

The potential impacts and mitigation measures section discussed in the original technical report is still valid for this addendum to the EIS technical report.

Due to the changes to the Project scope, changes are recommended to the locations of water quality monitoring sites for the upstream Frank Creek and the upstream Juandah Creek sites, as shown in Figure 5-1 to cater for the addition of Wubagul Pit. Implementation timeframes for water quality monitoring sites have been provided for all monitoring sites, given in Table 5-5.

As part of the proposed augmentation of the Wandoan wastewater treatment plant, monitoring of water quality at the discharge point associated with the wastewater treatment plant is proposed. It is recommended that this water quality monitoring point be part of the compliance licence for the wastewater treatment plant.

Following the adoption of the recommended mitigation strategies and measures, surface waters that leave the MLA areas are not likely to be degraded compared to existing water quality. Current and future water quality will be maintained at levels that are acceptable for users downstream of the MLA areas.

1. Introduction

This technical report is an addendum to the surface water quality impact assessment presented in the EIS Volume 1, Chapter 11 Water Supply and Management as TR 11-1-V1.5, Attachment B and should be read in conjunction with the original technical report. This addendum examines additional surface water quality data made available since the preparation of the original technical report, thereby assessing approximately 12 months of recent surface water quality data.

The purpose of the surface water quality impact assessment is to describe the existing environment for water resources, considering relevant components of Section 3.4 of the Terms of Reference that may be affected by the Project Mining Lease Application (MLA) areas in the context of environmental values as defined in such documents as the EP Act, Environmental Protection (Water) Policy 2009 (EPP(Water)) and ANZECC 2000.

1.1 Project background

The Project background was outlined in the original surface water quality impact assessment technical report.

1.2 Description of Project area

The description of the study area in relation to the surface water quality assessment is largely as described in the original technical report.

Key changes to the Project description in relation to the surface water quality environment are associated with:

- the addition of Wubagul Pit to the south-east corner of MLA 50230, affecting an unnamed creek north of Two Mile Creek, and potentially affecting Two Mile Creek
- refinement of the design of Wandoan Wastewater Treatment Plant, with discharge to the Wandoan showgrounds, golf course and Juandah Creek
- discharge to Juandah Creek from the wastewater treatment plant is approximated at 510 kL/day.

2. Methodology of assessment

The original technical report discussed the following in relation to the surface water quality impact assessment:

- review of legislation and guidelines potentially relevant to the surface water assessment
- description of the background surface water quality values examining available historic and recent water quality datasets
- description of the features and activities of the Project in the MLA areas that are relevant to the surface water quality assessment
- description of potential impacts
- description of mitigation strategies and measures required to manage the potential impacts on surface water quality.

In addition to the methodology and limitations discussed in the original technical report of the surface water quality impact assessment for the MLA areas, some further limitations to the available datasets were found, and are discussed further in section 4.3.2 below.

3. Relevant legislation and guidelines

The review of relevant legislation and guidelines relevant to the surface water assessment was discussed in the original technical report, with the Environmental Protection (Water) Policy 2009 (EPP (Water)) coming into force on 28 August 2009 since the preparation of the original technical report.

To clarify some points raised in submissions on the EIS, the following provides a brief summary of environmental values and water quality objectives, considering the original technical report.

The EPP (Water) states legally binding standards for water quality. Environmental Values (EVs) and Water Quality Objectives (WQOs) for surface water have been established under the EPP (Water).

Environmental Values (EVs) for the MLA areas have not been identified specifically in the EPP (Water), being within the Fitzroy River catchment. Western Downs Regional Council is in the process of developing EVs for the area. The environmental values are further outlined in the original technical report.

None of the affected waters are scheduled in Schedule 1 of the EPP (Water) through the corresponding EVs and WQOs report(s). The EVs of the receiving waters are therefore deemed to be considered by addressing water quality guidelines.

The EPP (Water) indicates that the Queensland Water Quality Guidelines (QWQG 2009) and the Australian and New Zealand Environment and Conservation Council (ANZECC) 2000 Guidelines are appropriate to be used to decide water quality trigger values for EV indicators for a waterbody. At the time of preparing this report, the Queensland water quality guidelines (QWQG) 2009 referred to in the EPP (Water), Schedule 2, had not been published by the Department of Environment & Resources Management (DERM), and the QWQG 2006 are to be used (pers. comms 16/09/09).

4. Existing environmental values for water quality

4.1 Introduction

This Addendum builds on the data and information provided in the original technical report. Where data and information have not changed since the original technical report, reference should be made to the original technical report.

4.2 Historical monitoring program

In addition to the historical monitoring programs discussed in the original technical report (i.e. MIM Holdings, from June 1985 to January 1987, and Department of Natural Resources and Water (NRW), from 1964 earliest to 2005 latest), water quality monitoring was undertaken by the members of Taroom Shire community, as part of the Upper Dawson Sediment watch program during 2007/2008, where the Taroom Shire Landcare Group undertook water quality sampling at a number of sites above Glebe Weir. This dataset was supplied to the WJV following the Taroom Landcare submission on the EIS.

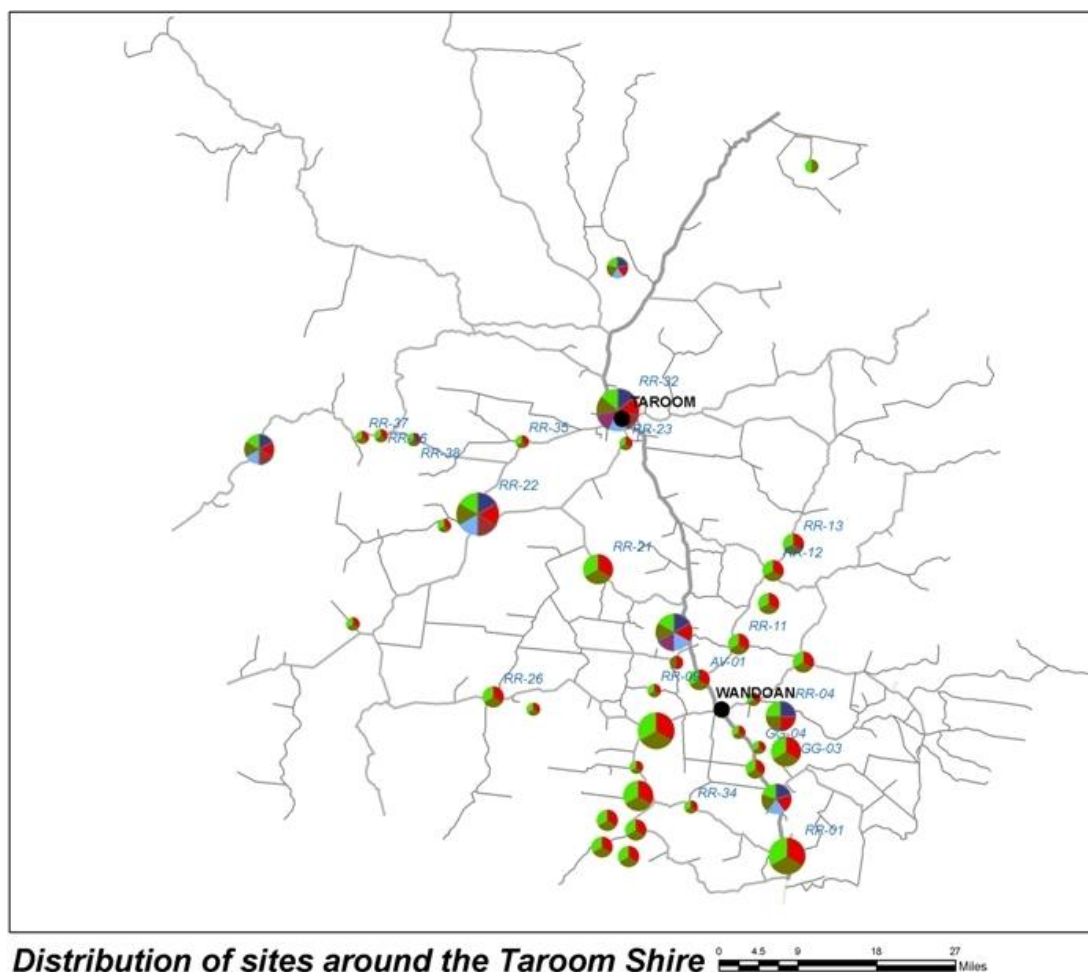


Figure 4-1: Taroom Shire Landcare water quality sampling sites adapted from Landcare Taroom Shire (2009a)

Although 150 sites have been sampled, Taroom Shire Landcare outlined water quality data registered at five key sites situated near NRW gauging stations (Landcare Taroom Shire 2009). The Taroom Landcare group has kindly supplied water quality dataset with the purpose of informing on the historical water quality levels for the Project. The data indicated many similarities in comparison with other historical data (i.e. discussed in the original technical report).

In summary, as outlined on the 'Taroom Landcare' website, the results associated with the water quality monitoring undertaken by the members of Taroom Shire community are as follows (Landcare, 2008):

- turbidity values
 - Woleebee Creek has an average of 4,200 NTU
 - Dawson River at Taroom has an average of 800 NTU during the event periods, while background values range from 200 NTU to 400 NTU.
- nitrogen under low flow conditions in the Upper Dawson was well above the trigger level compared to ANZECC guidelines, although this varied with seasons and flow volume; and Phosphorous levels are naturally high in the Upper Dawson and vary with seasons and flow rates
- electrical conductivity (EC) for a Juandah Creek sample was five times the ANZECC guideline of 110 $\mu\text{S}/\text{cm}$.

This sampling undertaken by Taroom Landcare group was considered to provide important indicative information on water quality at Woleebee Creek and Dawson River at Taroom.

Data supplied by the Taroom Landcare group has not been included in the technical water quality assessment for the Project, but provides useful indicative information on the general area surrounding Wandoan.

4.3 Recent water quality assessment

As mentioned in the original technical report, the water quality assessment examined the following recent datasets:

- historical data from the following sources:
 - MIM Holdings, from June 1985 to January 1987
 - Department of Natural Resources and Water (NRW), from 1964 (earliest) to 2005 (latest).
- recent data including two water quality datasets provided for assessment by the Wandoan Joint Venture, being:
 - daily log data recorded at generally four hourly intervals and/or during a change in (stream) water level; measuring temperature, conductivity, pH, DO and turbidity parameters, recorded from March 2008 to March 2009. This report includes and builds on the previous technical report that assessed water quality data from March 2008 to mid September 2008. Data sampling was undertaken by Ecowise Environmental and supplied by the WJV for assessment

- ▶ event data sampling undertaken by Ecowise Environmental supplied by the WJV, with dates outlined in Table 4-1.

The recent daily log and event datasets, not included in the original technical report, were merged with the original datasets. The combined data for daily log and event water quality datasets range from March 2008 to March 2009. These latest datasets were used to update results and findings associated with the description of the background surface water quality values examining recent water quality datasets.

The historical water quality assessment including MIM Holdings and NRW datasets was discussed in the original report.

4.3.1 Surface water quality monitoring program

The monitoring program prepared for the Project, as presented in Attachment A of the original technical report, was developed to provide a greater understanding of water quality in the Project area, and to allow assessment of water quality changes during the mine operation.

The purpose of the monitoring program is to 'minimise the risks of falsely detecting a disturbance or environmental impact when one has not occurred (giving a false alarm, a Type I error), or alternatively missing an environmental impact if it has occurred (giving a false sense of security, a Type II error)' (ANZECC, 2000, p.11).

These errors would occur if the monitoring program is as follows:

- monitoring sites are located too close to each other. If sampling sites are too close together, they can be autocorrelated, making statistical data analysis invalid if assumed that the sites are independent of each other (ANZECC 2000, p. 10)
- monitoring sites only partially cover the Project area. Therefore the monitoring sites would not represent all streams and catchments located within the Project area.

4.3.2 Equipment and methods used in monitoring and sampling

The original EIS technical report, section 4.3.2, discusses the equipment and methods used in water quality monitoring and sampling.

The review of water quality data found limitations in the use of the organochlorine pesticides (OC) and organophosphorus pesticides (OP) data stemming from sampling and analysis activities. Event sample volumes for OC and OP for the Project were not sufficient to undertake analysis to the standard laboratory Limit of Reporting (LOR). Limitations in laboratory capacity to also analyse samples for OC Chlorande, DDT and Endrin, and OP Azinphos methyl, Dimethoate and Fenitrothion and Parathion for comparison against ANZECC guidelines were also found.

Sample collection and analysis procedures have been amended since June 2009 to cater for additional volumes for all event sampling to ensure the LOR is low enough for comparison against ANZECC Guidelines. Event sampling with a lower LOR will be incorporated into the WJV's datasets for providing the baseline of water quality prior to mine construction and operation.

4.3.3 Water quality characteristics of recent data

Daily monitoring

The daily water quality sampling was undertaken from March 2008 to March 2009 for temperature, conductivity, pH, DO and turbidity. No flow data was recorded, but water level data was provided as part of daily monitoring.

The daily water quality monitoring data recorded via loggers at monitoring sites for the Project is presented as follows:

- downstream Frank Creek
- upstream Frank Creek
- downstream Juandah Creek
- upstream Juandah Creek
- downstream Woleebee Creek
- upstream Woleebee Creek
- downstream Mud Creek
- upstream Mud Creek
- upstream Mount Organ
- downstream Spring Creek.

The water quality data recorded at these sites was taken in generally four hourly intervals and/or during a change in (stream) water level.

The water quality data validated by Ecowise Environmental as 'good data' in their monthly reports for the Project, was used for the water quality assessment.

The water quality data is assessed in Section 4.3.4 against the guideline values, with further discussion provided in Section 4.4.

Event monitoring

The event monitoring associated with each monitoring site is outlined in Table 4-1. Event sampling was undertaken in association with rainfall events.

Table 4-1: Event monitoring (dates and monitoring sites)

Event monitoring	Monitoring sites								
13-Mar-08	1	2	3						9
24/25-July-08	1	2	3						9
10-Nov-08						6	7		
13-Nov-08	1	2	3	4	5	6	7		9
21-Nov-08	1		3						9
4-Dec-08	1							8	

Event monitoring	Monitoring sites								
5-Dec-08	1	2	3					8	
8-Jan-09			3	4		6	7		9
24-Feb-09	1	2		4	5	6	7	8	

Legend:

Downstream Mud Ck '1'	Upstream Mount Organ Ck '6'
Downstream Woleebee Ck '2'	Upstream Spring Ck '7'
Upstream Juandah Ck '3'	Upstream Juandah Ck '8'
Upstream Woleebee Ck '4'	Downstream Frank Ck '9'
Upstream Mud Ck '5'	

The event sampling included the following parameters:

- temperature, conductivity (EC), DO, pH and turbidity for the comparison with daily monitoring data
- nutrients, organics and toxins
- organochlorine pesticides (OC)
- organophosphorus pesticides (OP).

The water quality data was assessed in Section 4.3.4 against the guideline values, and further discussed in Section 4.4.

4.3.4 Water quality compliance

The water quality compliance assessment outlined the performance of the ambient water quality of key creeks that flow through the Project area in relation to Queensland Water Quality Guidelines (QWQG, 2006) and ANZECC (2000) guidelines.

As the Project area is mainly used for cropping and grazing, these activities can potentially generate related pollutants such as nutrients, suspended sediment, herbicides and pesticides. The future mine operation would introduce use of products containing heavy metals and organics, for example as part of vehicle maintenance and fuelling. Therefore the water quality monitoring parameters include pollutants associated with both land use practices (i.e. agricultural and mining).

Parameters assessed against the guideline values include total suspended solids (TSS), turbidity, EC, hardness as CaCO₃, NO₃, Al, Fe, Mn and Zn recorded at all sites within the study area. The water quality exceedances against trigger values (i.e. WQOs and ANZECC guideline values) are indicated in red bold italic print.

The conditions at the Project study area are typically ephemeral and subject to stagnant water with episodic flows, rather than base flow conditions as described in ANZECC guidelines. Therefore, for pH and DO, QWQG (2006) and ANZECC (2000) guidelines are used for compliance assessment only where base flow conditions can be observed.

Table 4–2 shows a daily monitoring data range and a mean for temperature, conductivity, pH, DO and turbidity, and also indicates the water quality compliance against the ANZECC (2000) and the QWQG (2006).

Streams listed in Table 4-3 that are located within the Project area experience extreme high and low flow conditions. These extreme flow conditions affect water quality in streams, mainly DO and pH levels.

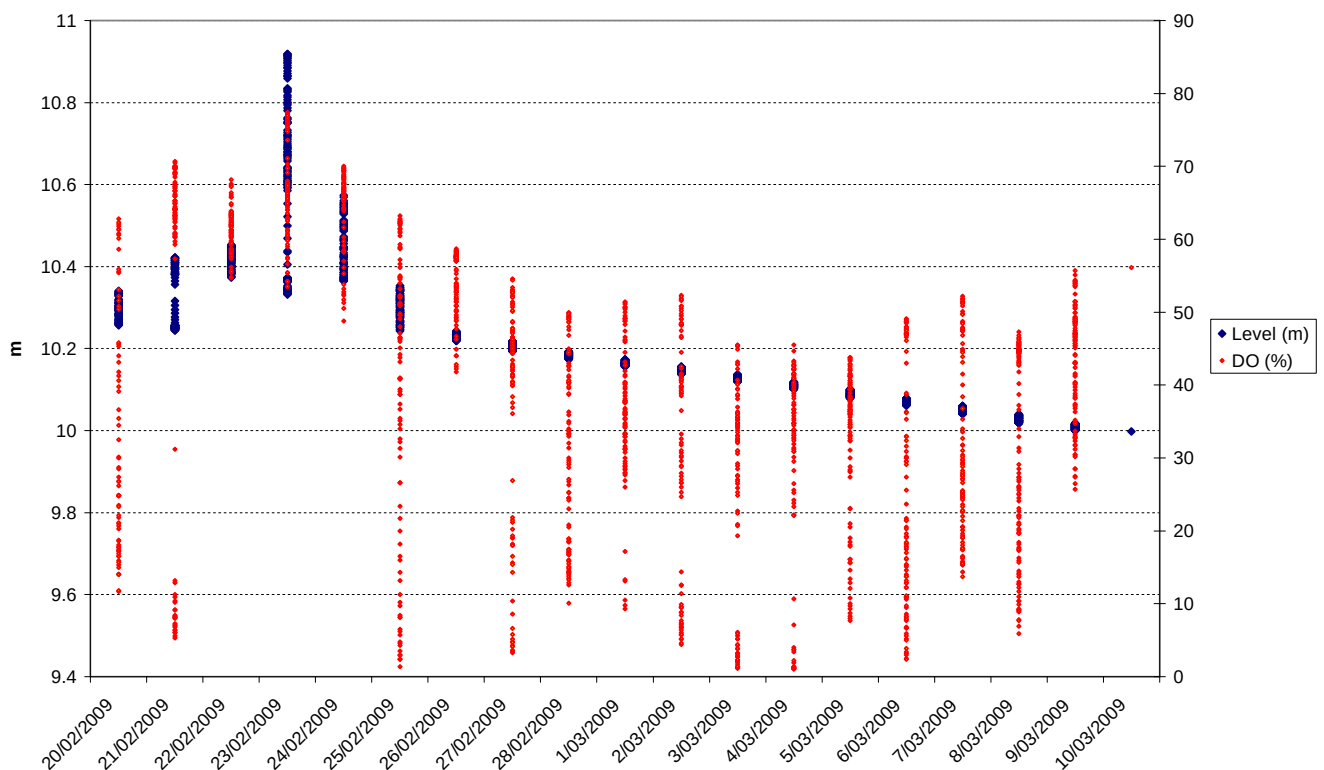
Water quality and variations with flow

Following submissions on the EIS, further discussion on the characteristics of water quality compared to variations in flow (i.e. stream level) is provided. Water levels are used to describe flow variations, as flow readings were not available for the assessment.

To aid in discussion, by example, one watercourse over the duration identified in the relevant figures of variable flow (change in water level) was selected. The reasons for selecting this watercourse for the duration given include:

- variable flow (change in water level) having both changing and near constant water levels for upstream and downstream sites for the same watercourse
- Woleebee Creek dataset demonstrated both changing and near constant water levels characteristics most clearly for the purposes of an example for discussion, however is representative of all watercourses across the study area
- good correlation between sampling intervals (i.e. sampling time) and number of samples between stream levels, DO and pH.

The variations between DO and water levels, likely associated with stagnant water, are shown in Figure 4-2. Figure 4-2 is based on the Woleebee Creek (upstream) monitoring site – the figure shows daily level records (y axis) in relation to daily DO levels (secondary yaxis), over a period characterised by variable water levels (x axis).



Note: Reliable (i.e. good) water quality data for level, DO and pH were recorded from 20 February 2009 to 9 March 2009.

Figure 4-2: Upstream Woleebee Creek DO and water level correlation

Figure 4-2 shows that in this case DO percentage concentration does not follow a particular trend in relation to water level.

The variations between pH and water levels, likely associated with stagnant water, are shown in Figure 4-3 and Figure 4-4. Figure 4-3 was based on the Woleebee Creek (upstream) and Figure 4-4 on Woleebee Creek (downstream) monitoring site. The figures show daily level records (y axis) in relation to daily pH levels (secondary y axis), over a period characterised by a variable water level (x axis).

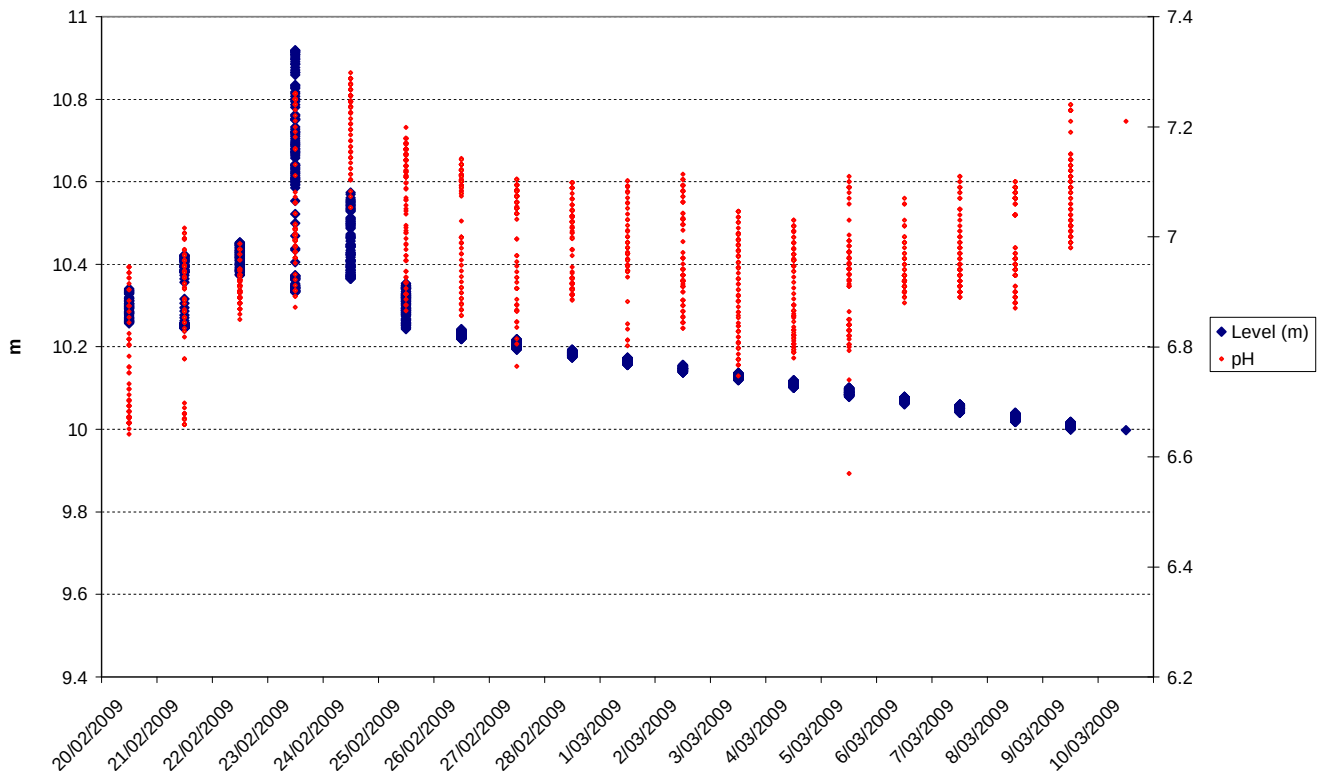
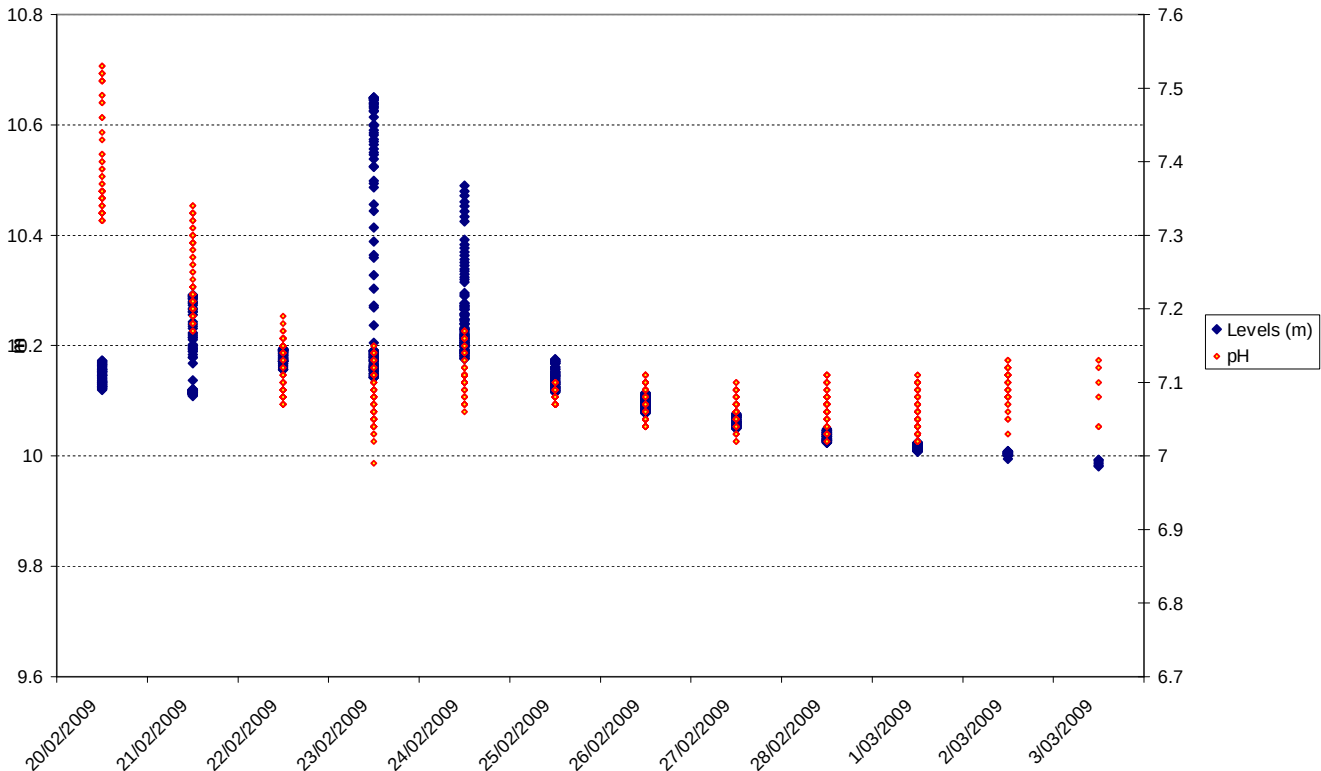


Figure 4-3: Upstream Woleebee Creek pH and water level correlation

Note: Reliable (i.e. good) water quality data for level, DO and pH were recorded from 20 February 2009 to 9 March 2009.



Note: Reliable (i.e. good) water quality data for level and pH was recorded to 2 March 2009.

Figure 4-4: Downstream Woleebee Creek pH and water level correlation

Figures 4-3 and 4-4 show different trends in pH in relation to water levels. Figure 4-3 shows an increase in pH level (from approximately 6.6 to 7.2), and Figure 4-4 shows a decrease in pH level (from approximately 7.5 to 7).

However, both Figure 4-3 and Figure 4-4 show an increase in pH level with a fall in water level (stagnation of water).

High pH levels generally occur in natural ecosystems when aquatic vegetation uses CO₂ for photosynthesis, causing hypoxic conditions (i.e. low O₂). In this case, Figures 4-3 and 4-4 show that pH levels become more basic as waters stagnant.

The findings discussed, in relation to variations with water level, are common for all water quality monitoring sites for the Project area.

EPA (2006) suggests that normal guidelines are 'not applicable to small waterholes' (p. 50) that occur during extreme low flow conditions. Section 5 further discusses suitable mitigation measures to address this issue for ongoing monitoring conditions.

The following was outlined in the EPA Guidelines (2006, p.50) in relation to applying guideline values to non-flowing streams:

- dissolved oxygen (DO) and pH become much more variable during stagnant conditions
- indicators such as nutrients may also be affected by non-flow conditions although there is not much data on this in Queensland waters. Normal guidelines are not applicable to small waterholes

- for toxicants it is appropriate to apply normal guideline values, as the effects on the biota under stagnant conditions will be similar to those during flowing conditions.

Table 4-2: Recent daily monitoring data — water quality compliance with QWQG and ANZECC (2000) guidelines

	Temperature (°C)	Turbidity (NTU)	pH	DO (% of saturation)	EC (µS/cm)
Frank Creek (downstream)					
Range	13.76–29.84	118–2,986	6.71–8.33	1–112.2	31.8–465
Mean	23.07	211	7.24	64.8	186.7
Juandah Creek (downstream)					
Range	0.3–34.1	0.5–3,000	6.92–10.12	29.60–106.20	1–839
Mean	20.8	159	8.69	85	362
Juandah Creek (upstream)					
Range	13.40–42.49	11.9–3,000	6.85–9.71	10.60–79.90	63.20–571.00
Mean	23.6	1,198	7.88	60	345
Woleebee Creek (downstream)					
Range	5.88–35.25	0.5–343.7	6.99–9.03	33.1–95.8	28.95–454
Mean	22.64	43.2	7.72	70.2	287
Woleebee Creek (upstream)					
Range	23.17–25.84	3.6–3,000	6.75–7.44	0.8–77.4	135–714
Mean	23.96	1,573	7.14	16.9	335
Mud Creek (downstream)					
Range	11.37–36.48	0.34–2,265	6.83–9.66	0.7–107.7	91–413
Mean	24.41	11.45	7.41	25.2	307
Mud Creek (upstream)					
Range	15–25.45	107–2,967	6.4–7.0	0.8–80.4	78.3–221.0
Mean	24.01	568	6.6	1.1	108.6
Mt Organ (upstream)					
Range	14.7–32.2	52.9–1,542	6.8–8.9	0.7–202	61.6–304.0
Mean	22.2	148.6	7.3	70	209
Spring Creek (upstream)					
Range	17.48–25.85	51–1,688	6.65–7.48	1.1–71.4	114.9–288
Mean	21.12	391	7.13	22.6	230
Guideline values					
QWQG (2006) – Upland Streams ²	na	2-25	6.5-7.5	85–110% saturation ¹	NA
ANZECC 2000	na	NA	NA	<6 mg/L or QWQG	30–350

¹ The DO guidelines (% saturation) for freshwaters only apply to flowing waters² Upland streams are defined as those at >150 m altitude (ANZECC 2000).

The review of the integrity of water quality data, as outlined by Ecowise Environmental in their monthly reports, indicated that no reliable (good) water quality data was recorded for Frank Creek (upstream). Therefore no assessment on Frank Creek (upstream) has been undertaken. Table 4-3 indicates the following water quality exceedances when compared to QWQG and ANZECC guidelines:

- turbidity exceedances were recorded at Frank Creek (downstream), Woleebee Creek (downstream and upstream), Mud Creek (upstream), Juandah Creek (downstream and upstream), Spring Creek (upstream) and Mount Organ (upstream) monitoring sites
- pH exceedances were recorded Juandah Creek (upstream and downstream) and Woleebee Creek (downstream) monitoring sites
- DO exceedances were recorded at Frank Creek (downstream), Woleebee Creek (downstream and upstream), Mud Creek (downstream and upstream), Juandah Creek (upstream), Spring Creek (upstream) and Mount Organ (downstream and upstream) monitoring sites
- EC exceedances were recorded at Juandah Creek (downstream).

Table 4-3 also demonstrates similar findings as outlined in the original water quality technical report of Table 4-7. The additional data indicates the following changes compared to the original technical report, associated with water quality exceedances:

- turbidity exceedance at Woleebee Creek (downstream), Juandah Creek (downstream and upstream) and Mount Organ (upstream) monitoring sites
- pH compliance at the Woleebee Creek (upstream) and Mud Creek (downstream) monitoring sites
- DO exceedance at Woleebee Creek (downstream), Juandah Creek (upstream), Frank Creek (downstream), Mud Creek (downstream and upstream), Spring Creek (upstream) and Mount Organ (downstream and upstream) monitoring sites
- EC exceedance no longer experienced at Juandah Creek (upstream).

Table 4-4 and Table 4-5 show the water quality compliance against the ANZECC 2000 and the QWQG 2006 for the physical/chemical stressors and toxicants sampled during the sampling events outlined in Table 4-2.

Table 4-6 shows the water quality compliance against the ANZECC 2000 and the QWQG 2006 for the organic toxicants sampled during the sampling events outlined in Table 4-2.

Table 4-3: Event monitoring data — water quality compliance for physical and chemical stressors, and toxicants

	TSS (mg/L)	Turbidity (NTU)	pH	EC (µs/cm)	BOD ₅ (mg/L)	Hardness as CaCO ₃ (mg/L)	Sulphate (mg/L)	TN (mg/L)	TP (mg/L)	Chlorophyll a (mg/m ³)	Fe ³ (mg/L)	As ² (mg/L)	Cd ^{2,4} (mg/L)	Cr ^{2,4} (mg/L)	Cu ^{2,4} (mg/L)	Pb ^{2,4} (mg/L)	Mn ² (mg/L)	Ni ^{2,4} (mg/L)	Zn ^{2,4} (mg/L)	Hg ² (mg/L)
Woleebee Creek (downstream)																				
Range	199–4,228	356–1,430	7.16–7.36	155–286	3–22	51–92	3–10	0.92–2.44	0.29–0.73	<1–10	15.7–60.18	0.004–0.008	<0.0001–0.0005	0.008–0.044	0.01–0.056	0.008–0.055	0.39–1.56	0.011–0.057	0.082–0.246	<0.0001
Median	1,485	400	7.27	166	9.5	61	3.5	1.7	0.48	6.0	22.75	0.005	0.0002	0.016	0.022	0.064	0.62	0.017	0.092	<0.0001
Woleebee Creek (upstream)																				
Range	415–4,740	900–1,100	7.02–7.56	200–288	6.2–9.8	43–75	10–11	1.08–3.94	0.46–0.76	1.4–8.4	3.58–82.76	0.006–0.011	0.0002–0.0004	0.001–0.050	0.013–0.052	0.010–0.065	0.39–1.73	0.010–0.063	0.071–0.349	<0.0001
Median	3,925	1,000	7.24	245	8.5	57	10	2.15	0.56	3.0	60.30	0.008	0.0003	0.043	0.042	0.047	1.24	0.050	0.228	<0.0001
Mud Creek (downstream)																				
Range	67–536	52.4–240	7.11–7.93	122–321	4–8	47–121	2–4	1.3–2.9	0.32–1.09	<1–16	3.03–12.10	0.003–0.004	<0.0001	0.002–0.008	0.004–0.011	0.003–0.006	0.16–0.38	0.003–0.009	0.014–0.070	<0.0001
Median	347	180	7.22	147	5	55	3	2.1	0.81	<1	6.92	0.003	<0.0001	0.004	0.008	0.004	0.26	0.005	0.047	<0.0001
Mud Creek (upstream)																				
Range	102–128	134–230	6.93–7.09	102–128	5–8.8	35–41	1.4–3	1.24–4.28	0.66–0.86	3.8–12.2	6.95–16.54	0.002–0.003	<0.0001–0.0003	0.003–0.007	0.007–0.013	0.004–0.008	0.12–0.40	0.005–0.009	0.028–0.104	<0.0001
Median	118	153	7.00	118	5.5	36	3	1.9	0.74	8.0	10.20	0.002	<0.0001	0.005	0.009	0.006	0.18	0.007	0.041	<0.0001
Juandah Creek (downstream)																				
Range	26–2,658	61.5–790	7.27–7.45	110–282	2.8–10.2	42–134	2–2.4	1.0–4.2	0.17–1.81	<1–3	2.43–21.74	0.003–0.004	<0.0001–0.0002	<0.001–0.014	0.003–0.024	0.001–0.018	0.143–0.82	0.003–0.013	0.014–0.070	<0.0001
Median	1,640	750	7.44	117	4	49	2	3.0	1.37	1.5	14.10	0.003	<0.0001	0.011	0.016	0.013	0.73	0.011	0.051	<0.0001
Juandah Creek (upstream)																				
Range	488–2,972	160–800	7.19–7.51	101–135	3–5	39–53	2–3.8	0.84–3.46	0.21–0.99	<1–2	7.24–28.88	0.002–0.005	<0.0001	0.004–0.018	0.010–0.020	0.006–0.024	0.23–1.19	0.006–0.020	0.034–0.083	<0.0001
Median	1,270	600	7.30	112	4	43	3	1.5	0.55	<1	12.85	0.004	<0.0001	0.008	0.014	0.013	0.37	0.012	0.054	<0.0001
Guideline values																				
QWQG (2006)—Upland Streams ¹	10	2–25	6.5–7.5	na	na	na	na	na	na	na	na	na	na	na	na	na	na	na	na	na
ANZECC 2000	na	na	na	30–350	>15 mg/L		50	0.5	0.05	5	0.34	0.024	0.00020 (H ⁵ 30 mg/L)	0.0033 (H ⁵ 30 mg/L)	0.0014 (H ⁵ 30 mg/L)	0.0034 (H ⁵ 30 mg/L)	1.7	0.011 (H ⁵ 30 mg/L)	0.008 (H ⁵ 30 mg/L)	0.0002 (H ⁵ 30 mg/L)
													0.00047 (H ⁵ 60 mg/L)	0.0058 (H ⁵ 60 mg/L)	0.0025 (H ⁵ 60 mg/L)	0.0082 (H ⁵ 60 mg/L)		0.020 (H ⁵ 60 mg/L)	0.014 (H ⁵ 60 mg/L)	0.0005 (H ⁵ 60 mg/L)
													0.00053 (H ⁵ 90 mg/L)	0.0081 (H ⁵ 90 mg/L)	0.0036 (H ⁵ 90 mg/L)	0.0137 (H ⁵ 90 mg/L)		0.028 (H ⁵ 90 mg/L)	0.020 (H ⁵ 90 mg/L)	0.00084 (H ⁵ 90 mg/L)

¹Upland streams are defined as those at >150 m altitude (ANZECC 2000)²Based on 95% protection level, as recommended in ANZECC (2000) for 'Slightly to Moderately Disturbed' ecosystems³Insufficient data at this stage to derive a reliable trigger value for iron. The current Canadian guideline level is 300µg/L, which could be used as an interim indicative working level but further data is required to establish a figure appropriate for Australian and New Zealand waters⁴Trigger value adjustment required to account for the effect of increased CaCO₃ levels. The ANZECC (2000) values have been calculated using a hardness of 30 mg/L CaCO₃. These were adjusted to the site-specific hardness in Table 4-4 using simple algorithms stipulated in ANZECC (2000) guidelines.⁵Hardness.

Table 4-4: Event monitoring data — water quality compliance for physical and chemical stressors, and toxicants

	TSS (mg/L)	Turbidity (NTU)	pH	EC (µs/cm)	BOD ₅ (mg/L)	Hardness as caco3 (mg/L)	Sulphate (mg/L)	TN (mg/L)	TP (mg/L)	Chlorophyll a (mg/m3)	Fe ³ (mg/L)	As ² (mg/L)	Cd ^{2,4} (mg/L)	Cr ^{2,4} (mg/L)	Cu ^{2,4} (mg/L)	Pb ^{2,4} (mg/L)	Mn ² (mg/L)	Ni ^{2,4} (mg/L)	Zn ^{2,4} (mg/L)	Hg ² (mg/L)
Frank Creek (downstream)																				
Range	50–374	100–164	7.08–7.43	72–194	<2–8.8	42–79	1–3	1.1–4.78	0.25–0.89	<1	4.37–12.9	<0.001–0.004	<0.0001–0.001	0.002–0.008	0.006–0.013	<0.001–0.005	0.13–0.38	0.004–0.006	0.025–0.147	<0.0001–0.0003
Median	259	145	7.27	183	6	65	2	1.7	0.36	<1	4.99	0.004	<0.0001	0.005	0.007	0.004	0.17	0.005	0.066	<0.0001
Mount Organ Creek (upstream)																				
Range	98–460	35–100	7.13–7.43	172–215	5–54	61–96	1.8–3	2–11.8	1.28–2.07	<1	1.38–7.71	0.002–0.003	<0.0001–0.0002	0.002–0.004	0.004–0.009	0.003–0.006	0.02–0.32	0.002–0.006	0.01–0.04	<0.0001–0.0003
Median	299	85	7.32	181	9.5	74	2	5.7	1.42	<1	5.05	0.003	<0.0001	0.003	0.007	0.004	0.18	0.004	0.025	<0.0001
Spring Creek (upstream)																				
Range	222–816	106–164	7.29–7.32	92–100	19–29	29–83	2	1.34–21.84	0.49–0.69	<1	5.38–7.32	0.002–0.003	<0.0001–0.0004	0.003–0.006	0.006–0.008	0.003–0.004	0.12–0.25	0.004–0.007	0.029–0.050	<0.0001–0.0003
Median	551	145	7.30	93	22	36	2	2.2	0.53	<1	5.97	0.002	0.0002	0.005	0.007	0.004	0.15	0.007	0.040	<0.0001
Guideline values																				
QWQG (2006)—Upland Streams ¹	10	2–25	6.5–7.5	na	na	na	na	na	na	na	na	na	na	na	na	na	na	na	na	na
ANZECC 2000	na	na	na	30-350	>15 mg/L		50	0.5	0.05	5	0.34	0.024	0.00020 (H ⁵ 30 mg/L)	0.0033 (H ⁵ 30 mg/L)	0.0014 (H ⁵ 30 mg/L)	0.0034 (H ⁵ 30 mg/L)	1.7	0.011 (H ⁵ 30 mg/L)	0.008 (H ⁵ 30 mg/L)	0.0002 (H ⁵ 30 mg/L)
													0.00047 (H ⁵ 60 mg/L)	0.0058 (H ⁵ 60 mg/L)	0.0025 (H ⁵ 60 mg/L)	0.0082 (H ⁵ 60 mg/L)		0.020 (H ⁵ 60 mg/L)	0.014 (H ⁵ 60 mg/L)	0.0005 (H ⁵ 60 mg/L)
													0.00053 (H ⁵ 90 mg/L)	0.0081 (H ⁵ 90 mg/L)	0.0036 (H ⁵ 90 mg/L)	0.0137 (H ⁵ 90 mg/L)		0.028 (H ⁵ 90 mg/L)	0.020 (H ⁵ 90 mg/L)	0.00084 (H ⁵ 90 mg/L)

¹Upland streams are defined as those at >150 m altitude (ANZECC 2000)²Based on 95% protection level, as recommended in ANZECC (2000) for 'Slightly to Moderately Disturbed' ecosystems³Insufficient data at this stage to derive a reliable trigger value for iron. The current Canadian guideline level is 300µg/L, which could be used as an interim indicative working level but further data is required to establish a figure appropriate for Australian and New Zealand waters⁴Trigger value adjustment required to account for the effect of increased CaCO₃ levels. The ANZECC (2000) values have been calculated using a hardness of 30 mg/L CaCO₃. These were adjusted to the site-specific hardness in Table 4-5 using simple algorithms stipulated in ANZECC (2000) guidelines.⁵Hardness.

Table 4-5: Event monitoring data — water quality compliance for organic toxicants

µg/L	Chlorobenzene	1,2-dichlorobenzene	1,4- dichlorobenzene	Ethylbenzene	Benzo-(a)-pyrene	Toluene
Woleebee Creek (downstream)						
Value	<5	<5	<5	<2	<0.5*	<2
Mud Creek (downstream)						
Value	<5	<5	<5	<2	<0.5*	<2
Juandah Creek (downstream)						
Value	<5	<5	<5	<2	<0.5*	<2
Frank Creek (downstream)						
Value	<5	<5	<5	<2	<0.5*	<2
Woleebee Creek (upstream)						
Value	<5	<5	<5	<2	<0.5*	<2
Mud Creek (upstream)						
Value	<5	<5	<5	<2	<0.5*	<2
Juandah Creek (upstream)						
Value	<5	<5	<5	<2	<0.5*	<2
Mount Organ Creek (upstream)						
Value	<5	<5	<5	<2	<0.5*	<2
Spring Creek (upstream)						
Value	<5	<5	<5	<2	<0.5*	<2
Guideline values						
ANZECC 2000	55	160	60	350	0.2	180

*samples were below detection limit

The event monitoring indicated the following water quality exceedances compared to QWQG and ANZECC (2000) guidelines:

- physical and chemical stressors and toxicants:
 - TSS, turbidity, TN, TP, Iron at all sites
 - chlorophyll a at Woleebee Creek (downstream) and Mud Creek (upstream)
 - chromium, copper, lead, nickel and zinc at Woleebee Creek (upstream)
 - chromium, copper, lead and zinc at Woleebee Creek (downstream), Juandah (downstream and upstream), Mud Creek (upstream), and Spring Creek (upstream)
 - chromium, copper, and zinc at Mud Creek (downstream)
 - copper and zinc at Frank Creek (downstream) and Mount Organ Creek (upstream).

4.4 Water quality findings

The main water quality findings were discussed in section 4.3.4 above with the respective daily and event water quality monitoring.

The assessment of water quality in relation to variations with water level were assessed for DO and pH levels, because '*extreme flow conditions affect water quality in streams, mainly DO and pH levels*' (EPA, 2006, p. 50). High pH levels generally occur in natural ecosystems when aquatic vegetation uses CO₂ for photosynthesis, causing hypoxic conditions (i.e. low O₂). Streams in the Project area are ephemeral, with streams indicating higher pH levels in stagnant water, however DO percentage concentrations do not following a particular trend in relation to water level.

5. Potential impacts and mitigation measures

Having considered the changes to the Project scope and submissions raised on the EIS, the potential impacts and mitigation measures section discussed in the original EIS technical report, as described in Section 5 and Table 5-1, are still valid for this addendum to the EIS technical report.

5.1 Recommended surface water quality monitoring program

A site assessment of each daily monitoring site should be undertaken to identify a water level in a stream that indicates extreme low flow conditions (i.e. waterhole with stagnate water affecting DO and pH levels). This change should be reflected in the water quality monitoring procedure.

As discussed in Section 1.2, with the addition of Wubagul Pit and refinement of the Wandoan Wastewater Treatment Plant augmentation design, additional monitoring sites are proposed as part of the water quality monitoring program.

Changes are recommended to the locations of the upstream Frank Creek and the upstream Juandah Creek, as shown in Figure 5-1 to cater for the addition of Wubagul Pit. Tables 5-2 to 5-5 below outline the recommended parameters for monitoring and associated timeframes for implementation, superseding Tables 5-2 and 5-3 of the original technical report.

A submission on the EIS relevant to the surface water quality impact assessment identified an important water quality issue that came out of the *'Report to Queensland Premier Review of the Fitzroy River Water Quality Issues'*, and that is the need to include monitoring and analysis of selenium in both water and sediment samples. Selenium has been added to the suite of parameters recommended for monitoring.

As part of the proposed augmentation of the Wandoan wastewater treatment plant, monitoring of water quality immediately downstream of the discharge point in the mixing zone associated with the wastewater treatment plant is proposed. It is recommended that this water quality monitoring point be part of the compliance licence for the wastewater treatment plant.

The water quality monitoring sites in Figure 5-1 are recommended to be used to measure compliance with the conditions of the Environmental Authority.

Two types of compliance parameters are proposed in Tables 5-3 and 5-4 below:

- receiving water quality limits — for release management and compliance — exceedance of a receiving water quality limit represents an unauthorised discharge
- trigger investigation levels — for investigation — exceedance of a trigger level will activate an investigation into the cause, and an assessment of the potential for environmental harm.

Due to the ephemeral nature of the watercourses, and resulting variation in parameter characteristics with flow conditions, the establishment of fixed compliance values for parameters will not appropriately protect the environmental values of the site. The recommended trigger values have therefore be based on percentiles of measurements of

water quality parameters at reference monitoring sites which are unaffected by mining operations.

A release will be deemed to be authorised if, at the frequency permitted by the “Limit Type” column according to Tables 5-3 and 5-4, the measured compliance parameter values are within the allowable trigger levels.

The allowable compliance parameter values shown in Tables 5-3 and 5-4 will be continuously updated as more reference data is collected. Minimum receiving water quality limits have been proposed based on previously obtained background monitoring data, as summarised in the EIS and Supplementary EIS water quality technical report and addendum to the technical report. Minimum trigger values have been proposed based on ANZECC (2000) or Queensland Water Quality Guidelines (2006) trigger levels for upland streams in a slightly to moderately disturbed condition.

Table 5-2: Recommended parameters for water quality monitoring

Monitoring sample type	Water quality parameter	Recommended trigger values for parameters	Sample frequency
Fully automated sampling stations	pH Temperature EC Turbidity DO	As per Tables 5-3 and 5-4	At least daily, and more frequently when flow is detected.
Event sampling	Electrical conductivity pH TSS Turbidity TN TP BOD Chlorophyll a Aluminium Arsenic (As) Cadmium (Cd) Copper (Cu) Chromium (Cr) Lead (Pb) Nickel (Ni) Zinc (Zn) Mercury (Hg) Iron (Fe) Manganese (Mn) Selenium (Se) Chlorobenzene 1,2-dichlorobenzene 1,4- dichlorobenzene Ethylbenzene Benzo-(a)-pyrene Toluene OC and OP pesticides Sulfate	As per Table 5-4	During and after major flow events.
Aquatic ecology	Parameters to include, but are not limited to: Taxonomic richness Richness of pollution-sensitive invertebrate taxa (Plecoptera	For taxonomic richness and PET, no lower than existing environment, being, degraded to moderate water quality and	Seasonally as required to ensure a long-term aquatic ecology monitoring program is implemented for the Project, with at least two baseline survey

Monitoring sample type	Water quality parameter	Recommended trigger values for parameters	Sample frequency
	(stoneflies), Ephemoptera (mayflies), and Trichoptera (caddisflies) (PET) DO pH Temperature EC Turbidity. Reference to the aquatic ecology technical report should be made for further mitigation measures associated with aquatic ecology.	habitat. For DO, pH, temperature, EC and turbidity, as per trigger values for fully automated sampling.	events prior to construction.
Wandoan wastewater discharge location	5-day Biochemical Oxygen Demand (five day BOD) TSS pH E.coli Total Dissolved Solids (TDS) DO Aluminium Arsenic (As) Cadmium (Cd) Copper (Cu) Chromium (Cr) Lead (Pb) Nickel (Ni) Zinc (Zn) Mercury (Hg) Iron (Fe) Manganese (Mn) Chlorobenzene 1,2-dichlorobenzene 1,4- dichlorobenzene Ethylbenzene Benzo-(a)-pyrene Toluene	As per table 5-4	Weekly sampling for five day BOD, TSS, pH, E.coli, TDS, DO. Quarterly for heavy metals and hydrocarbons.

Table 5-3: Proposed receiving water quality limits for release management

Water quality parameter	Unit	Recommended receiving water limit	Limit type ³
pH		20 th percentile ¹ of reference monitoring sites ² or 6, whichever is lower 80 th percentile ¹ of reference monitoring sites ² , or 9, whichever is higher	Range
EC	uS/cm	80 th percentile ¹ of reference monitoring sites ² or 1000, whichever is higher	80 th percentile of event samples ^{1 3}
Turbidity	NTU	80 th percentile ¹ of reference monitoring sites ² or 1000, whichever is higher	80 th percentile of event samples ^{1 3}

Notes:

1. Limits and impact monitoring level based on the 80th or 20th percentile to be derived using ANZECC (2000) accepted methodology
2. As measured at monitoring sites as defined in Figure 5-1.
3. The 80th percentile must be determined based on no more than 10 consecutive samples taken at a downstream monitoring station.

Table 5-4: Recommended trigger investigation values for monitoring parameters

Water quality parameter	Unit	Recommended trigger values for parameters ³	Limit type ⁴
Temperature	Degrees C	80 th percentile ¹ of monitoring sites ²	80 th percentile ^{1 4}
DO	mg/L	80 th percentile ¹ of monitoring sites ²	80 th percentile ^{1 4}
TSS	mg/L	80 th percentile ¹ of monitoring sites ² or 2000, whichever is higher	80 th percentile ^{1 4}
TN	mg/L	80 th percentile ¹ of monitoring sites ²	80 th percentile ^{1 4}
TP	mg/L	80 th percentile ¹ of monitoring sites ²	80 th percentile ^{1 4}
BOD	mg/L	80 th percentile ¹ of monitoring sites ² or 15, whichever is higher	80 th percentile ^{1 4}
Chlorophyll a	mg/L	80 th percentile ¹ of monitoring sites ² or 5, whichever is higher	80 th percentile ^{1 4}
Aluminium	mg/L	80 th percentile ¹ of monitoring sites ² or 0.055 whichever is higher (at pH>6.5)	95 th percentile ^{1 4}
Arsenic (As)	mg/L	80 th percentile ¹ of monitoring sites ² or 0.024, whichever is higher	95 th percentile ^{1 4}
Cadmium (Cd)	mg/L	80 th percentile ¹ of monitoring sites ² or 0.00047, whichever is higher	95 th percentile ^{1 4}
Copper (Cu)	mg/L	80 th percentile ¹ of monitoring sites ² or 0.0025, whichever is higher	95 th percentile ^{1 4}
Chromium (Cr)	mg/L	80 th percentile ¹ of monitoring sites ² or 0.0058, whichever is higher	95 th percentile ^{1 4}
Lead (Pb)	mg/L	80 th percentile ¹ of monitoring sites ² or 0.0082, whichever is higher	95 th percentile ^{1 4}

Water quality parameter	Unit	Recommended trigger values for parameters ³	Limit type ⁴
Nickel (Ni)	mg/L	80 th percentile ¹ of monitoring sites ² or 0.020, whichever is higher	95 th percentile ^{1 4}
Zinc (Zn)	mg/L	80 th percentile ¹ of monitoring sites ² or 0.014, whichever is higher	95 th percentile ^{1 4}
Mercury (Hg)	mg/L	80 th percentile ¹ of monitoring sites ² or 0.0005, whichever is higher	95 th percentile ^{1 4}
Iron (Fe)	mg/L	80 th percentile ¹ of monitoring sites ² or 0.34, whichever is higher	95 th percentile ^{1 4}
Manganese (Mn)	mg/L	80 th percentile ¹ of monitoring sites ² or 1.7, whichever is higher	95 th percentile ^{1 4}
Selenium (Se)	mg/L	80 th percentile ¹ of monitoring sites ² or 0.011 whichever is higher	95 th percentile ^{1 4}
Chlorobenzene	µg/L	55	95 th percentile ^{1 4}
1,2-dichlorobenzene	µg/L	160	95 th percentile ^{1 4}
1,4-dichlorobenzene	µg/L	60	95 th percentile ^{1 4}
Ethylbenzene	µg/L	350	95 th percentile ^{1 4}
Benzo-(a)-pyrene	µg/L	0.2	95 th percentile ^{1 4}
Toluene	µg/L	180	95 th percentile ^{1 4}
OC and OP pesticides	µg/L	ANZECC 2000 guidelines	95 th percentile ^{1 4}
Sulfate	mg/L	50	95 th percentile ^{1 4}

Notes:

1. Trigger levels based on the 80th or 20th percentile to be derived using ANZECC (2000) accepted methodology
2. Reference sites as defined in Figure 5-1.
3. ANZECC (2000) or Queensland Water Quality Guidelines (2006) trigger levels for upland streams in a slightly to moderately disturbed condition.
4. The 80th and 95th percentile must be determined based on no more than 10 consecutive samples.

Table 5-5: Recommended implementation timeframes of water quality monitoring sites

Catchments	Sites	Code	Comments
Juandah Creek catchment	Upper Juandah Creek (US Upper Juandah Creek)	J-UJ	Already installed site will cease to be effective in Year 3, and be superseded by the new US Juandah Creek as shown in Figure 5-1, to be installed no later than one year prior to operations commencing in Wubagul Pit (e.g. no later than the beginning of Year 2) so to provide at least one year's water quality monitoring data prior to operations commencing in Wubagul Pit.
	Upper Frank Creek Dam (US Frank Creek Dam)	J-UF	Already installed site will cease to be effective in Year 3, and be superseded by the new US Frank Creek as shown in Figure 5-1, to be installed no later than one year prior to operations commencing in Frank Creek Pit (e.g. no later than the beginning of Year 2) so to provide at least one year's water quality monitoring data prior to operations commencing in Frank Creek Pit.

Catchments	Sites	Code	Comments
	Downstream Juandah Creek (DS Juandah Creek)	J-DJ	Already installed as a permanent water quality monitoring station. Monitoring to continue.
Woleebee Creek catchment	Upper Woleebee Creek (Alternative US Woleebee Creek)	AltW-UW	Installation of a permanent water quality monitoring station to be one year prior to construction commencing of Woleebee Creek Diversion and/or one year prior to operations commencing at Woleebee Creek Pit (e.g. no later than the beginning of Year 9), so as to provide at least one year's water quality monitoring data prior to construction of the creek diversion of Woleebee Creek.
	Wandoan Creek (Alternate US Wandoan Creek)	AltW-UWa	Installation of a permanent water quality monitoring station to be one year prior to construction commencing of Woleebee Creek Diversion and/or one year prior to operations commencing at Woleebee Creek Pit (e.g. no later than the beginning of Year 9), so as to provide at least one year's water quality monitoring data prior to construction of the creek diversion of Wandoan Creek as part of the Woleebee Creek diversion.
	Upper Woleebee Creek (US Woleebee Creek)	W-UW	The already installed Woleebee Creek upstream monitoring site will be able to provide water quality data of Woleebee Creek until Years 11/12, when diversion of Woleebee Creek will impact on or reduce the effectiveness of the site an upstream monitoring location.
	Downstream Woleebee Creek (DS Woleebee Creek)	W-DW	Installation of a permanent water quality monitoring station to be one year prior to construction commencing (e.g. no later than mid Year -3), so as to provide at least one year's water quality monitoring data prior to construction commencing in Year -2. Site will supersede already installed Woleebee Creek Downstream.
	Woleebee Creek Downstream		Already installed site will cease to be effective in Year -2, and be superseded by DS Woleebee Creek as shown in Figure 5-1.
Mud Creek catchment	Upper Mount Organ Creek	M-UM	Already installed as a permanent water quality monitoring station. Monitoring to continue.
	Upper Mud Creek	M-UMu	Already installed as a permanent water quality monitoring station. Monitoring to continue.
	Downstream Mud Creek (DS Mud Creek)	M-DMu	Installation of a permanent water quality monitoring station to be one year prior to operations commencing in Mud Creek Pit (eg. no later than the beginning of Year 3), so as to provide at least one year's water quality monitoring data prior to construction commencing of infrastructure associated with Mud Creek Pit.
	Mud Creek downstream		The already installed monitoring site will cease to be effective prior to Year 4 when infrastructure associated with mining of Mud Creek Pit (conveyors and ROM dump) commences construction. Site will be superseded by DS Mud Creek, as shown in Figure 5-1.
Spring Creek catchment	Spring Creek Downstream		The already installed monitoring site will cease to be effective prior to Year 9 when infrastructure associated with mining of Summer Hill and Turkey Hill Pits (conveyors and ROM dump) commences construction.
	Spring Creek (DS Spring Creek)	S-DS	Installation of a permanent water quality monitoring station to be one year prior to operations commencing at Summer Hill and Turkey Hill Pits (eg. no later than the beginning of Year 8), so as to provide at least one year's water quality monitoring data prior to construction commencing of infrastructure associated with Summer Hill and Turkey Hill

Catchments	Sites	Code	Comments
			Pits.
Duck Creek catchment	Duck Creek	D-DS	Installation of a permanent water quality monitoring station to be one year prior to operations commencing at Turkey Hill Pit (eg. no later than the beginning of Year 8), so as to provide at least one year's water quality monitoring data prior to mining of Turkey Hill Pit.

As part of the surface water monitoring program, during operations it is recommended that water quality information be collected from each of the on-site storage dams, with the following quality characteristics measured:

- pH
- EC
- DO
- Turbidity
- TSS
- Sulphate
- Fluoride
- Aluminium
- Arsenic
- Cadmium
- Cobalt
- Copper
- Lead
- Nickel
- Zinc.

Data is recommended to be collected on at least a quarterly basis during normal operations. For environmental dams, the frequency of testing should be increased to monthly if the stored volume encroaches to within 0.1 AEP 72 hour runoff volume of the spillway crest, and weekly samples taken if the stored volume encroaches to within 0.1 AEP 24 hour runoff volume of the spillway crest.

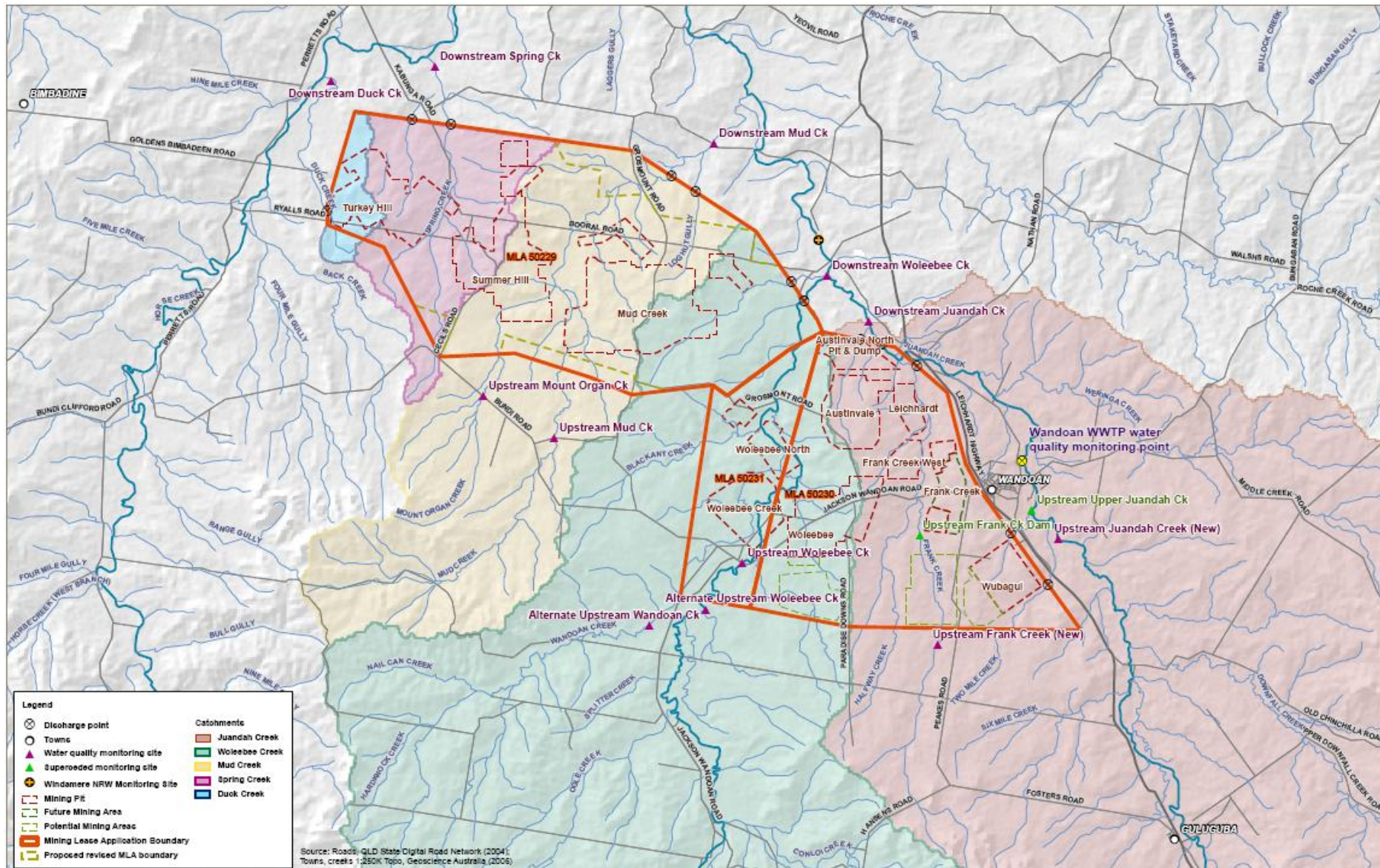


Figure 5-1: Recommended water quality monitoring sites

6. Residual impacts

Having considered the changes to the Project scope and submissions raised on the EIS, no changes to the residual impacts described in the original water quality technical report have arisen due to the findings of this addendum.

Following the adoption of the recommended mitigation strategies and measures, surface waters that leave the MLA areas are likely not to be degraded compared to existing water quality. Current and future water quality will be maintained at levels that are acceptable for users downstream of the MLA areas.

7. Summary of mitigation measures

The potential impacts and mitigation measures section was discussed in the original report, with no amendments to the mitigation measures resulting from the changes in Project scope or submissions on the EIS as described in this addendum.

Tables 5-2 to 5-5, and Figure 5-1 should be referred to for amendments to the recommended water quality monitoring sites and parameters for water quality monitoring.

References

Australian and New Zealand Environment and Conservation Council, and Agriculture and Resource Management Council of Australia and New Zealand (ANZECC), 2000, *National Water Quality Management Strategy*.

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Landcare Taroom Shire. 2009, Water Quality findings available on the Website:

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http://www.taroom-wandoan-landcare.com.au/WQ_Interpretation_2005_601-1.jpg

Attachment C

Addendum to site water
management system technical report

Wandoan Coal Project

Addendum to site water management system report

September, 2009

Wandoan Joint Venture



Parsons Brinckerhoff Australia Pty Limited ABN 80 078 004 798

Level 4, Northbank Plaza

69 Ann Street

Brisbane QLD 4000

GPO Box 2907

Brisbane QLD 4001

Australia

Telephone +61 7 3854 6200

Facsimile +61 7 3854 6500

Email brisbane@pb.com.au

NCSI Certified Quality System ISO 9001

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Author: Adam Anderson, Marina Reichstetter

Signed:

Reviewer: Dr. Mohand Amghar, Michael Batchelor

Signed:

Approved by: Helen D' Arcy.....

Signed:

Date: June 2009.....

Distribution: Wandoan Joint Venture, PB File, PB Library.....

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Glossary

AEP	Annual exceedance probability
ARI	Average recurrence interval
AWBM	Australian Water Balance Model
CHPP	Coal Handling and Preparation Plant
IQQM	Integrated Quality Quantity Model
DME	The former Department of Minerals and Energy
DSA	Design Storage Allowance
EIS	Environmental Impact Study
EPA	Environmental Protection Agency
MIA	Mine Industrial Area
MLA	Mining Lease Application
Mtpa	Million tonnes per annum
NHMRC	National Health and Medical Research Council
NRW	The Department of Natural Resources and Mines
PET	Potential Evapo-Transpiration
PPD	Patched Point Dataset
ROM	Run of mine

Executive summary

This addendum report addresses concerns raised in submissions on the Wandoan Coal Project EIS. It also describes updates to the proposed water management system and associated impact assessment and water balance, as described briefly below:

- conceptual water management system design – a number of changes have been made to the number, size and location of water management dams to cater for changes made to the mine layout. As a result of these changes, the number of potential discharge points from the MLAs has been increased to 12, and a total of up to 46 sediment dams and seven environmental dams (at any time) are now required for protecting downstream water quality
- historical simulation water balance and impact assessment – the assessment has been updated for the revised water management system layout, and additional details have been provided on downstream impacts and anticipated water quality.

The proposed mine water management system design will be further refined and optimised in later stages of design. The leading industry practice design objectives applied in conceptual design will be maintained. They are reiterated below:

- minimise the volume of pit water (surface runoff reporting to pit and groundwater seepage) generated by the Project
- avoid planned discharge of pit/process water under normal operating conditions through preferential on-site reuse of site water stores. Following rainfall, if stored water quality, potential diluting flows and receiving water quality allows, releases may be made to reduce on-site water inventories
- provide sufficient on-site storage to give an acceptable level of risk of accidental off-site discharge of pit/process water during significant rainfall events (no unplanned discharge under modelled historical conditions)
- provide sufficient on-site storage to settle coarse suspended solids from mine area runoff (from overburden dumps and other disturbed areas) during significant rainfall events, through the application of the relevant guideline sediment dam storage capacity.

These objectives are to be achieved by providing three separate water management systems:

- pit/process water management system – managing water captured in pits and running off the MIA, dump stations, and where there is potential to contribute high concentrations of dissolved salts, such as the Coal Handling and Preparation Plant (CHPP) and coal product stockpiles
- overburden runoff water management system – treating water running off overburden dumps and other disturbed areas of the site with the potential to have high concentrations of suspended solids
- clean water system –water from undisturbed areas of the MLAs and the large catchments discharging through the site from upstream.

The adopted water management system design criterion is for no planned discharge of contaminated water under historical conditions. This is to be achieved via the following actions, consistent with the risk management approach set down in Technical Guidelines for the Environmental Management of Exploration and Mining in Queensland (DME 1995) (the Technical Guidelines) as required under the Project Terms of Reference:

- pit/process water management system – sized and operated to ensure environmental dams would not overflow under measured historical climate conditions based on water balance simulation modelling. Releases will only be allowed if the discharge water quality is such that the licensed receiving water quality limits are met. If water levels exceed defined operational storage allowances, pumping to the dams will cease
- overburden runoff management system – provide sedimentation dams sized in accordance with the recommendations in the Technical Guidelines – the design storage will be the 10% AEP time of concentration runoff volume. These dams (designed as ‘dry’ sediment basins) will allow coarse sediment to settle before discharging from the site over 10 days following rainfall. This design AEP is considered appropriate for the discharge of uncontaminated runoff to non-environmentally sensitive receiving waters.

An indicative assessment of expected water quality in the site water management system has been prepared for determining dam hazard category and potential impacts on downstream water users.

In the absence of comprehensive runoff quality data from existing mining areas on the site, electrical conductivity (as an analogue for salinity (total dissolved salts)) has been used as an indicator for compliance with guideline limits (limited salinity data is available from geochemical testing of overburden and groundwater).

The following guideline concentrations are of interest comparison with the toxicity criteria in the Technical Guidelines and the proposed licensed receiving water quality limits.

	Guideline	TDS (mg/L)	EC (µS/cm)
Toxic	>100 x drinking water standard (NHMRC)	1,000,000	1,600,000
Sub-lethal	10-100 x drinking water standard (NHMRC)	100,000	160,000
Low-toxicity	<10x drinking water standard (NHMRC)	10,000	16,000
Stock watering	Beef Cattle	4,000-6,000	5,970-7,460
Drinking water	NHMRC	1,000	1,600
Licensed receiving water quality limit	Proposed in the EM plan based on local water quality observations and objectives		1000

The water quality analysis results showed the following:

- for sediment dams, it is expected that water will not be stored for extended periods, so there will be no increase in concentration of dissolved contaminants over time in these dams. However, a likely upper bound for the salinity of sediment dam discharge has been made based on the ratios of disturbed catchment areas and their corresponding estimated runoff salinity. Conservatively assuming all overburden runoff has an electrical conductivity (EC) of 1,000 µS/cm (the maximum level measured in the overburden geochemical analysis), the highest combined sediment dam salinity likely to be discharged to a stream is 710 µS/cm well below the proposed licensed receiving water quality limit

- for the pit/process water management system, saline pit/process water could potentially concentrate and accumulate in the environmental dams. The conservative solute modelling capabilities of the IQQM have been used to estimate a possible range of salinities in the environmental dams over the project life, using the above assumptions on runoff quality. The results show:
 - TDS levels would only meet sub-lethal or toxic levels when stored water volumes drop to very low levels due to evaporation and subsequent concentration below the level of the pump inlet
 - following large rainfall events, when the risk of discharge is highest, the salinity drops significantly. The maximum salinity level is variable across the various environmental dams, but typically, when water levels are less than 100 ML from the spillway crest, the modelled TDS is slightly above drinking water standard and the stored water would therefore be considered uncontaminated or low-toxicity waste
 - typical modelled environmental dam salinity levels rarely exceed 5,000 mg/L and would therefore generally be suitable for all onsite uses. All significant volumes of water will be suitable for reuse at the CHPP after blending with raw water from the raw water dams.

While most conclusions of the original EIS analysis water balance modelling are still valid, the changes to the water management system have resulted in minor changes to the impacts on flows in the downstream watercourses, as summarised below:

- for the purposes of this assessment, most dams have been initially sized for the ultimate Year 30 scenario, in which disturbed areas are largest, and captured runoff volumes are high. During earlier years, the captured volumes, downstream impacts and risk of discharge are lower than they otherwise would be
- during wet periods, up to 1,440 ML of runoff could be expected to accumulate in each of the larger operational mine pits. However, for 90% of the modelled climate record, surface water in the operational pits would be minimal. Operation of the proposed pit/process water management system would see the pits dewatered to operational in-pit storage levels that would keep mining interruptions to an acceptable frequency
- if extreme wet weather occurs in later years of the mine development (when the area of disturbance and open pits is largest). The capacity of the environmental dams may not be sufficient to drain the active pits. Large water storage volumes and/or reuse will be required to manage the risk of interruptions to mining. Significant water storage is likely to be available in disused pits so that active pits can be drained to operational levels in an acceptable timeframe. During wet periods similar to the wettest on record, up to 18,000 ML of additional water storage could be required for this purpose. If the quality of overburden runoff is such that water captured in sediment dams is unsuitable for release except in larger flows, this additional volume requirement increases to 33,000 ML. In either case, this volume is likely to be available in inactive mine pits. An alternative solution for MLA 50229 would be to provide a high capacity water pipeline to rapidly deliver water from the active pits for use in the CPP
- the mine and associated WMS is expected to reduce mean annual flows immediately downstream of the MLA areas by approximately 3.2% by Year 30 of the Project. The greatest impact is at the small Duck Creek catchment, where mean annual flow will be reduced by 33%. Further downstream, at the Horse Creek/Juandah Creek confluence, where the nearest licensed surface water entitlement (a water harvesting entitlement) is held, the total impact is reduced to a decrease of 2.3% of mean annual flow by Year 30 of the Project

- the modelling indicates the Project will cause almost no change to the seasonal variation of average median flow in the receiving waters. A very small change in some low flow spell statistics is expected. The number of flow spells for Spring Creek and Mud Creek will decrease slightly and their mean duration will increase slightly as a result of the Project. The largest impact is on Spring Creek where there is a reduction in the number of flow spells of 16% and a corresponding increase in low flow spell duration of 19%. At Mud Creek, the change in these statistics is a reduction of 2% and an increase of 2% for the frequency and duration respectively. The impacts on flows in other creeks other creeks is minimal
- the total flow from sediment dams make up a relatively small proportion of flows near the MLA boundaries. By Year 30 of the Project, they will make up 4.1% of flows in the creeks immediately downstream. At Duck Creek, where the proportion of flow is greatest, overflows will make up 33% of total catchment flow at the MLA boundary. Nine kilometres downstream, at the Horse Creek/Duck Creek confluence, the sediment dam overflows would make up only 0.4% of the mean annual Horse Creek flow.
- the proposed water management system concept is robust enough to ensure that environmental harm can be avoided even if overburden runoff quality is much worse than expected. A possible contingency plan for this scenario would be to store contaminated water transferred from sediment dams for reuse. If this was to occur near Year 30, up to 33,000 ML of reserve water storage would be required. A number of potential storage locations throughout the site are likely to be available for this excess water. It must be emphasised that based on the available overburden salinity data, this outcome is very unlikely. It has only been discussed here for the purposes of demonstrating that unexpected water quality can be managed through appropriate contingency measures.

1. Introduction

1.1 Background

1.2 Scope

This report forms part of the technical studies supporting the Supplementary Project Environmental Impact Study (EIS).

This report provides the additional background details addressing those components of the EIS Terms of Reference relating to management of surface water on the MLA.

It addresses concerns raised in submissions on the EIS, and revises the assessment where the project description has changed since EIS publication, as described briefly below:

- conceptual water management system design – a number of changes have been made to the number, size and location of water management dams to cater for changes made to the mine layout
- historical simulation water balance assessment – the assessment has been updated for the revised water management system layout, and additional details have been provided on downstream impacts.

Other aspects of water management (including on-site raw water storage and reticulation) are dealt with in the Addendum to the Water Supply and Surface Water Management Technical Report TR-11-1-V1.5 and its attachments. Flooding and diversion design is covered in the Addendum to the flood study technical report TR-11-2-V1.5, and water quality is described in the Addendum to TR-11-3-V1.5. Ecological aspects are discussed in detail in Chapter 17.

2. Methodology

2.1 Water management system design objectives

A proposed water management system conceptual design is described, which evolves with the mine development to:

- protect downstream watercourses
- maintain quality and quantity of surface waters for existing beneficial downstream uses.

In line with leading industry practice, the objective of the adopted water management system design is to:

- minimise the volume of pit water (surface runoff reporting to pit and groundwater seepage) generated by the Project
- avoid planned discharge of pit/process water under normal operating conditions through preferential on-site reuse of site water stores. Following rainfall, if stored water quality, potential diluting flows and receiving water quality allows, releases may be made to reduce on-site water inventories
- provide sufficient on-site storage to give an acceptable level of risk of accidental off-site discharge of pit/process water during significant rainfall events (no unplanned discharge under modelled historical conditions)
- provide sufficient on-site storage to settle coarse suspended solids from mine area runoff (from overburden dumps and other disturbed areas) during significant rainfall events, through the application of the relevant guideline sediment dam storage capacity.

These objectives are achieved by providing three separate water management systems:

- pit/process water management system – managing water captured in pits and running off the MIA, dump stations, and where there is potential to contribute high concentrations of dissolved salts, such as the Coal Handling and Preparation Plant (CHPP) and coal product stockpiles
- overburden runoff water management system – treating water running off overburden dumps and other disturbed areas of the site with the potential to have high concentrations of suspended solids
- clean water system – water from undisturbed areas of the MLAs and the large catchments discharging through the site from upstream.

Under this arrangement, twelve potential discharge points from the MLAs are nominated. The adopted water management system design criterion is for no planned discharge of contaminated water under historical conditions.

Proposed dams for each of the water management systems were sized through the application of a historical simulation water balance model. The adopted sizes were then shown to comply with the guidance provided in the Technical Guidelines for the Environmental Management of Exploration and Mining in Queensland (DME 1995) as required under the Project Terms of Reference.

A sensitivity analysis was performed to demonstrate the system is robust over a range of reasonable runoff quality and quality scenarios.

2.2 Relevant legislation and guidelines

In undertaking these assessments, the key relevant Acts are the *Water Act 2000* (Water Act) and the *Environmental Protection Act 1994* (EP Act). Other applicable legislation includes:

- Water Regulation 2002
- Water Resource (Great Artesian Basin) Plan 2006
- Great Artesian Basin Resource Operations Plan 2006
- Water Resource (Fitzroy Basin) Plan 1999
- Fitzroy Basin Resource Operations Plan 2006
- *Water Supply (Safety and Reliability) Act 2008*.

In the absence of comprehensive runoff quality data from existing mining areas on the site, electrical conductivity (as an analogue for salinity (total dissolved salts)) has been used as an indicator for compliance with guideline limits (limited salinity data is available from geochemical testing of overburden and groundwater).

The following guideline concentrations are of interest comparison with the toxicity criteria in the Technical Guidelines and the proposed licensed receiving water quality limits.

Table 2-1: Guideline toxicity criteria and receiving water quality limits

	Guideline	TDS (mg/L)	Approx equiv. EC (μ S/cm)
Toxic	>100 x drinking water standard (NHMRC)	1,000,000	1,600,000
Sub-lethal	10-100 x drinking water standard (NHMRC)	100,000	160,000
Low-toxicity	<10x drinking water standard (NHMRC)	10,000	16,000
Stock watering	Beef cattle	4,000-6,000	5,970-7,460
Drinking water	NHMRC Guideline	1,000	1,600
Licensed receiving water quality limit	Proposed in the EM plan based on local water quality observations and objectives		1,000

2.2.1 DME guideline uncontaminated runoff criteria

2.2.2 DME guideline contaminated runoff criteria

2.2.3 Code of environmental compliance for high hazard dams containing hazardous waste

2.3 Adopted design criteria

The following criteria have been adopted for the purposes of this assessment:

Sediment dams

Based on the risk management approach set down in the DME Technical Guidelines for Water Management, the design storage volume of sediment dams will be the 10% AEP time of concentration runoff volume. This criterion is nominated to allow the settlement of coarse sediment before discharge, and is considered appropriate for the discharge of uncontaminated runoff to non-environmentally sensitive receiving waters.

The dams will be designed as 'dry' sediment basins; that is they will have permanently open, low-level outlet works sized to ensure they are emptied within 10 days of filling. Provision will be made in the outlet works to allow a control valve to be fitted to store or release as stored water quality and receiving water conditions dictate. Water may then be pumped out for dust suppression on haul roads or dump stations, or delivered to the CHPP if required.

Natural suspended solids concentrations are observed to be high (see TR 11-3-V1.5), so it is likely that once coarse sediment has been removed, the proposed licensed receiving water quality limits will be met. However, if the measured turbidity is higher than licensed levels, due to the presence of dispersive clays in the overburden, one of the following corrective actions will be taken:

- the water may be used on site for haul road or dump station dust suppression
- the water may be returned for use in the CHPP via an environmental dam
- flocculant may be added to sediment dams to treat the water before release – this is likely to be feasible for small volumes only, and will be contingent on an effective, non-toxic flocculant being identified.

The quality of water in the sediment dams will be monitored during the early years of mining. In the event that captured water has higher than anticipated salinity, the volume of these dams may be increased.

Environmental dams

The final design criteria will be agreed with the DERM through the risk management approach described in the DME Technical Guidelines. As demonstrated in Section 6.0, water discharged from pit water dams in rainfall events is likely to be classified as sub-lethal, based on the expected concentrations of potential contaminants, but may meet the low-toxicity criteria.

For the purposes of this conceptual design and impact assessment, all environmental dams have been sized to achieve ***no discharge when operated as part of the overall site water management system under historical climate conditions, as determined through water balance modelling.***

The adopted design volumes have then been compared to the DSA and Mandatory Reporting Level requirements defined in the Technical Guidelines, for the appropriate design risk level (see Section 4.6). Acceptable design criteria will be agreed with DERM, and the water balance modelling will be revised at detailed design to establish design dam volumes.

2.4 Historical simulation water balance modelling

The simulation modelling has been extended to include an indicative assessment of expected water quality in the site water management system for comparison with the proposed licensed receiving water quality limits for salinity. For the pit/process water management system, saline pit/process water could potentially concentrate and accumulate in the environmental dams. The conservative solute modelling capabilities of the IQQM have been used to estimate a possible range of salinities in the environmental dams over the project life, using the above assumptions on runoff quality

In the absence of measured local mine runoff data, a simplified approach was adopted in which constant runoff salinity levels were applied to runoff from each catchment type. This was represented by the following adopted total dissolved solids concentrations:

- undisturbed natural catchment runoff and rehabilitated area runoff – 200 mg/L conservatively based on observations in the nearby creeks of greater than 230 $\mu\text{S/cm}$
- pit runoff – 13,800 mg/L based on observed groundwater quality of between 8,000 $\mu\text{S/cm}$ and 23,000 $\mu\text{S/cm}$. This is likely to be a very conservative assumption, as runoff water will not always come into contact with the coal seam
- overburden runoff – 1,000 mg/L – conservatively based on geochemical testing data indicating a maximum of 1,100 $\mu\text{S/cm}$ in the weathered rock itself.

It must be emphasised that the on-site surface and groundwater quality data is limited, and as a result there are large uncertainties in forecast site runoff characteristics.

2.5 Referable dams

Since July 2008, referable dams have been legislated under the *Water Supply (Safety and Reliability) Act 2008*.

3. Existing environment

3.1 Climate data

The EIS quoted initial dam volumes based on the following design rainfall depths:

- 1% AEP Nov-Feb rainfall depth: 772 mm – derived from decile analysis of daily Datadrill rainfall dataset from 1889 to 2007 for Wandoan
- 10% AEP Nov-Feb rainfall depth: 511 mm – derived from decile analysis of daily Datadrill rainfall dataset from 1889 to 2007 for Wandoan
- 1% AEP 72 hour rainfall depth: 261 mm derived from intensity-frequency-duration curve from Bureau of Meteorology for Wandoan
- 10% AEP 24 hour rainfall depth: 127 mm derived from intensity-frequency-duration curve from Bureau of Meteorology for Wandoan
- 10% AEP one hour rainfall depth: 57 mm derived from intensity-frequency-duration curve from Bureau of Meteorology for Wandoan.

3.2 Groundwater inflows

3.3 Existing catchments and rainfall runoff behaviour

3.3.1 Sacramento model

3.3.2 AWBM

Figure 3-1: Existing major catchments and proposed discharge points

4. Proposed surface water management system

A surface water management system has been devised to mitigate the potential impacts of the Project on receiving water quality, and to protect the operation of the Project from interruptions due to flooding.

The overall guiding principle in the development of the water management system is to wherever possible, separate water of varying quality to minimise the stored volumes of water with high concentrations of contaminants. For the purposes of the Project, three separate water management systems (in descending order of potential contamination) are considered:

- pit/process water management system – managing water captured in pits and running off dump stations and other areas with the potential to contribute high concentrations of dissolved salts, such as from the CHPP and coal product stockpiles
- overburden runoff water management system – treating water running off overburden dumps and other disturbed areas of the site with the potential to have large concentrations of suspended solids
- clean water system – separating runoff from undisturbed areas from the above systems.

A number of changes have been made to the mine layout and schedule, and as a result, the water management system layout has been modified to suit. Under this revised arrangement, 12 potential discharge points from the MLAs are nominated as shown in Figure 3-1. Downstream of the discharge points, these waterways make their way to one of the four major stream systems near the site.

4.1 Pit/process water management system

4.2 Overburden runoff water management system

4.3 Clean water management system

4.4 Staging of mine water management system

The components of the water management system will evolve as the Project expands, to be compatible with the proposed pit layout and mine schedule. This development of the mine water management system over the 30 year mine life is illustrated through snapshots at five stages of the mine layout:

- A. Year 1
- B. Year 5
- C. Year 10
- D. Year 20
- E. Year 30.

As per chapters 13 and 15 of the EIS, these snapshots were adopted as representative of the Project during the life of the operation. The amended Figures 4-1 to 4-5 show the mine progression, the areas of disturbance, and the required water management structures at each stage. The extent of rehabilitation at each mine stage is described in the excavation and refill plans provided in Chapter 6 of the EIS (Figure 6-6-V1-3 to Figure 6-13-V1-3).

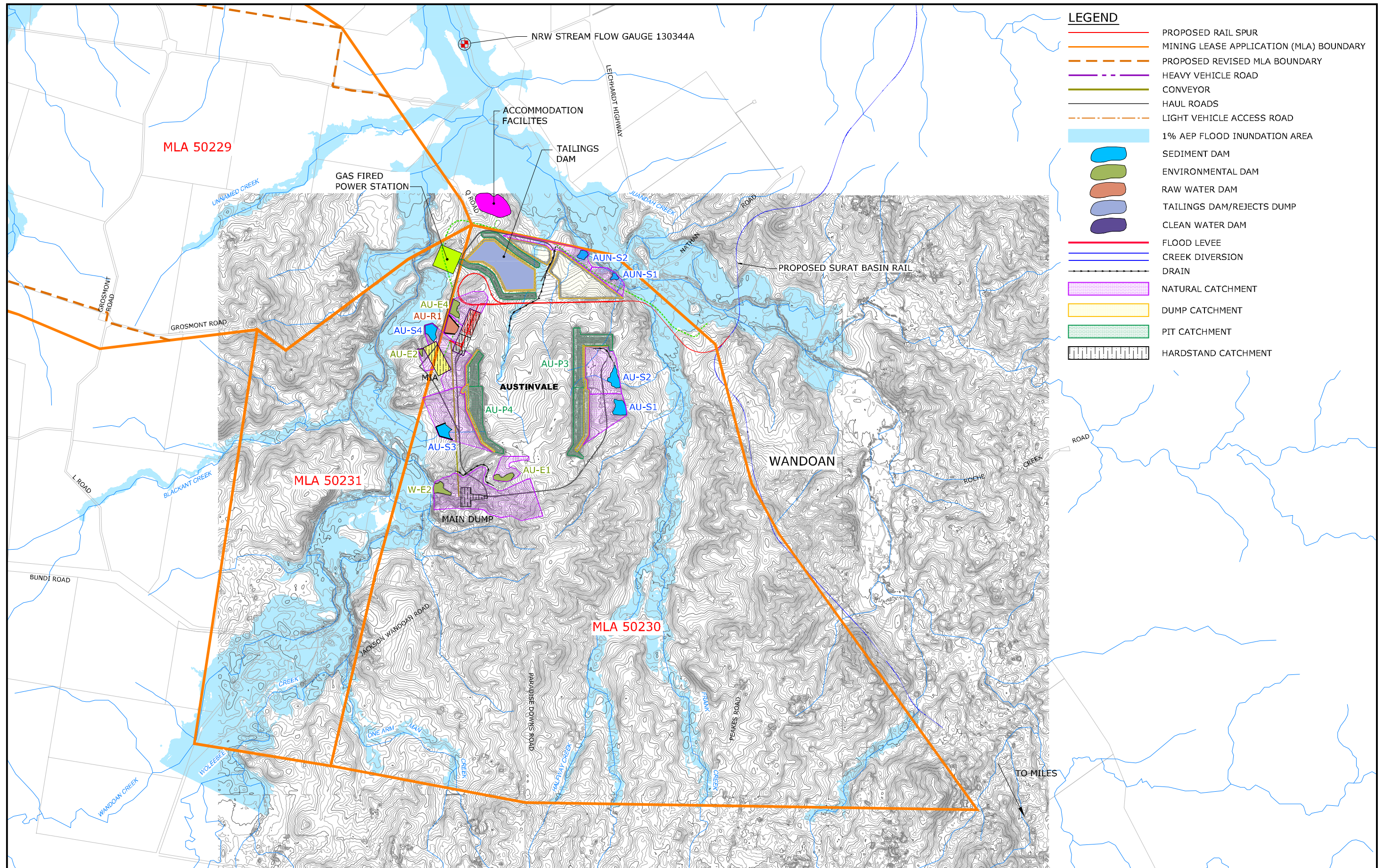
Excluding in-pit pump sumps, a maximum total of 53 water management dams are required to manage runoff from disturbed areas at any one time during the life of the project. The number of dams increases over time as summarised in the table below.

Table 4-1: Total number of water management dams over project life

Year	Sediment dams	Environmental dams	Pit pump sumps	Total
5	20	5	10	35
10	27	7	9	43
20	35	7	14	56
30	46	7	16	69

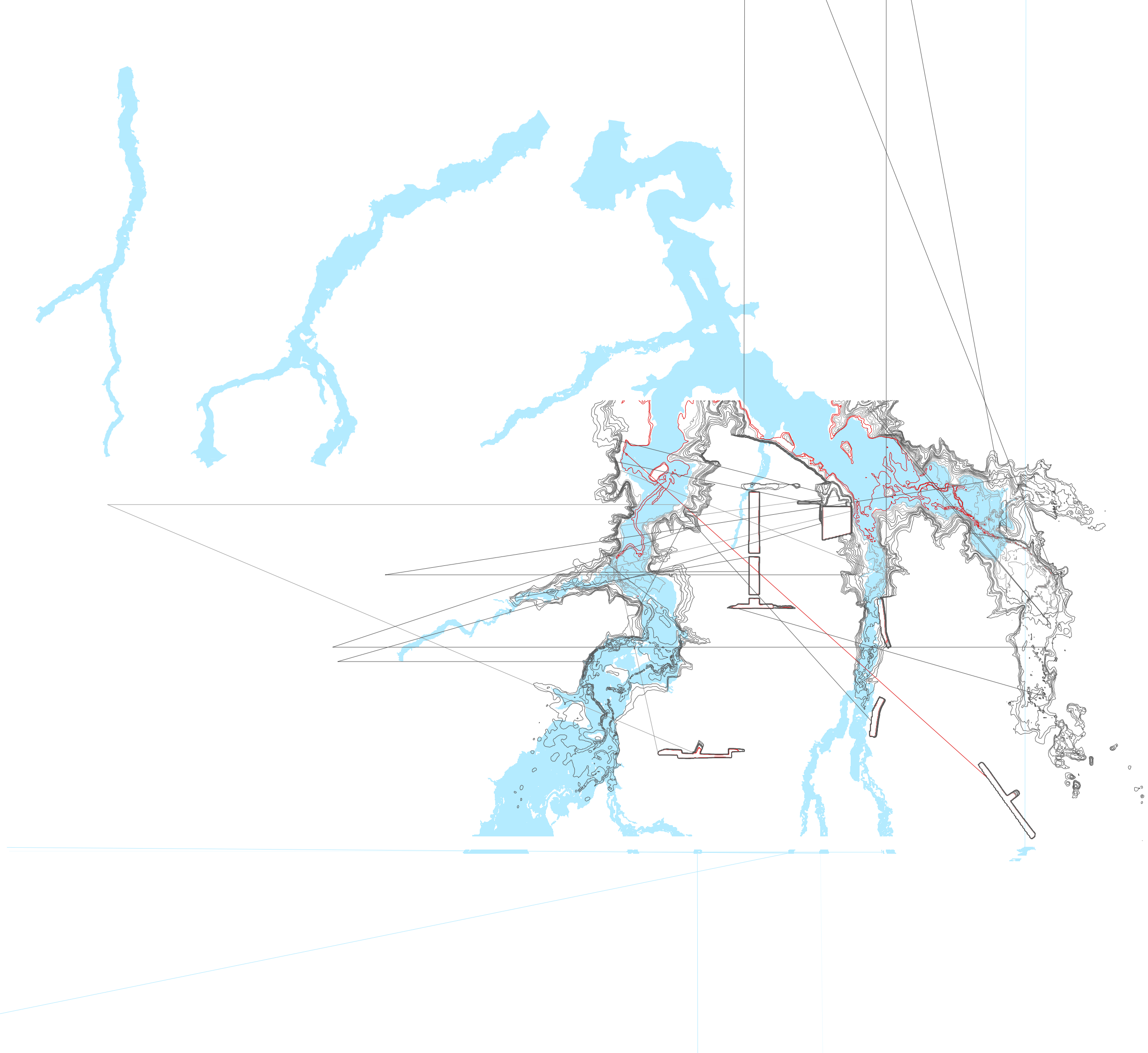
Dams were sited to ensure that runoff from all disturbed areas can be intercepted before it discharges off site. In most cases, dams were located such that they were off-stream from nearby watercourses, to reduce the proportion of undisturbed catchment captured.

All dams will be embankment dams constructed from suitable engineered earthfill, compacted to a suitable specification to be impermeable and durable. Where possible, embankment fill material will be obtained from within the storage impoundment area to minimise haul distances and maximise storage volume. Where required, a cutoff trench will be provided to prevent excessive seepage through the foundation.

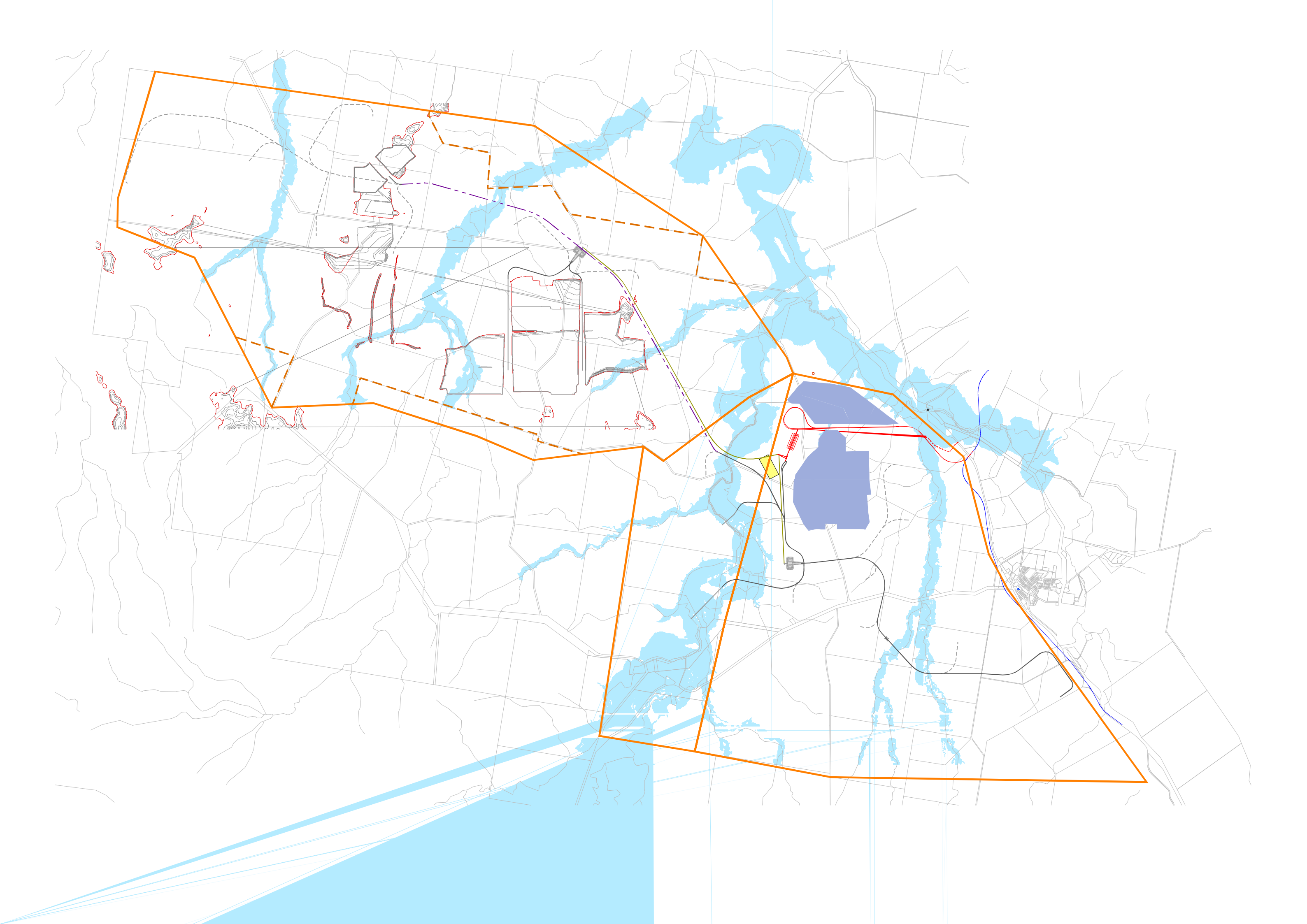


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4.5 IQQM model schematisation

IQQM models were prepared for each snapshot of the mine operation.

Each dam was added to the model with a catchment inflow, a spillway overflow, and a pumped outflow for water demands. Evaporation is calculated on the basis of the dam surface area at each time step using evaporation estimates provided by NRW's Data Drill. Seepage from each dam was set at 0 mm/d, which is the conservative approach for assessing risk of surface discharge.

4.5.1 Water demands and pump rates

For the purposes of this assessment, water demands from the system during mine operations were input as follows – based on the demand estimates presented in the Water Supply and Water Management Technical Report (TR 11-1-V1.5).

- CHPP makeup and other nearby MIA uses:
 - Year 1 — 2,920 ML/a
 - Year 5 to 20 — 9,227 ML/a
 - Year 30 — 7,008 ML/a.
- haul road dust suppression: at each environmental dam — a demand of 1 ML/d was drawn from each environmental dam whenever storage was available.

A key component of the proposed system is a pipeline linking the western pits to the CHPP, so that excess water can be disposed of during very wet periods to reduce the risk of discharge. Following further investigations into acceptable pit availability, the adopted pipeline capacity has been resized as follows, to allow both Turkey Hill and Mud Creek environmental dams to be dewatered at 100 L/s (8.6 ML/d):

- from Turkey Hill Dump to Mud Creek Dump 100 L/s
- from Mud Creek Dump to the CHPP 200 L/s.

The adopted pump rate from each pit to its respective environmental dam is 200 L/s. Pipelines are shown in the schematic diagrams, along with the receiving watercourse.

4.5.2 Storage of excess water during wet periods

Later in the mine life, when numerous pits are operating, the rate of pit dewatering during very wet periods may exceed the capacity of the CHPP to reuse the water. In this case, once all environmental dams are full, additional storage will be required to accept the excess pit water until it can be used by the CHPP. A number of options may be available:

- pits around the site may be inactive for periods while mining focuses on other areas , and these may be used to store water if necessary
- there may be excess storage space available in the tailings and rejects storage facility during earlier years of production.

A notional central pit water storage was added to the model to allow the volume of this additional storage requirement to be estimated. The storage was sized to minimise the period of interruption to mining following wet periods.

Under this proposed arrangement, the environmental dams will tend to be emptied by pumping to the CHPP before all haul road water demands are satisfied, and during dry periods, water will need to be returned from the CHPP to the environmental dams to meet haul road watering demand. During later stages of design, consideration will be given to optimising the distribution of this storage around the site to better balance supplies and demands and reduce the transfer requirements.

Uncontaminated runoff from overburden dumps is collected in dry sediment dams. A return water pump system is not modelled for dry sediment dams, and all runoff entering them was assumed to pass downstream without evaporation or seepage loss.

For the original EIS modelling, the site was split into two parts — one for areas in the vicinity of the CHPP (Central), the other for Turkey Hill, Summer Hill and Mud Creek Pits (Western). These two models have been combined for this supplementary EIS.

These model layouts are illustrated in the schematic diagrams shown in Figures 4-6 and Figure 4-7.

4.5.3 Catchment representation

The contributing catchment inflow was modelled for each storage using an inflow file developed by summing the products of unit runoff depth time-series (derived using the rainfall-runoff models) and the corresponding partial catchment areas.

Catchment areas for each catchment type were derived from the water management system layout shown in Figures 4-1 to 4-5. Rehabilitated areas were derived from the refill plans

Table 4-2 summarises the total catchment areas modelled at each mine phase.

Table 4-2: Change in surface water management catchments through Project life

Year	Natural (ha)	Pit-ramp (ha)	Spoil (ha)	Rehab (ha)	Total area (ha)
5	1,601	659	631	566	3,457
10	2,031	629	1,237	1,297	5,195
20	3,182	1,349	2,288	3,571	10,390
30	3,352	2,058	1,110	7,391	13,912

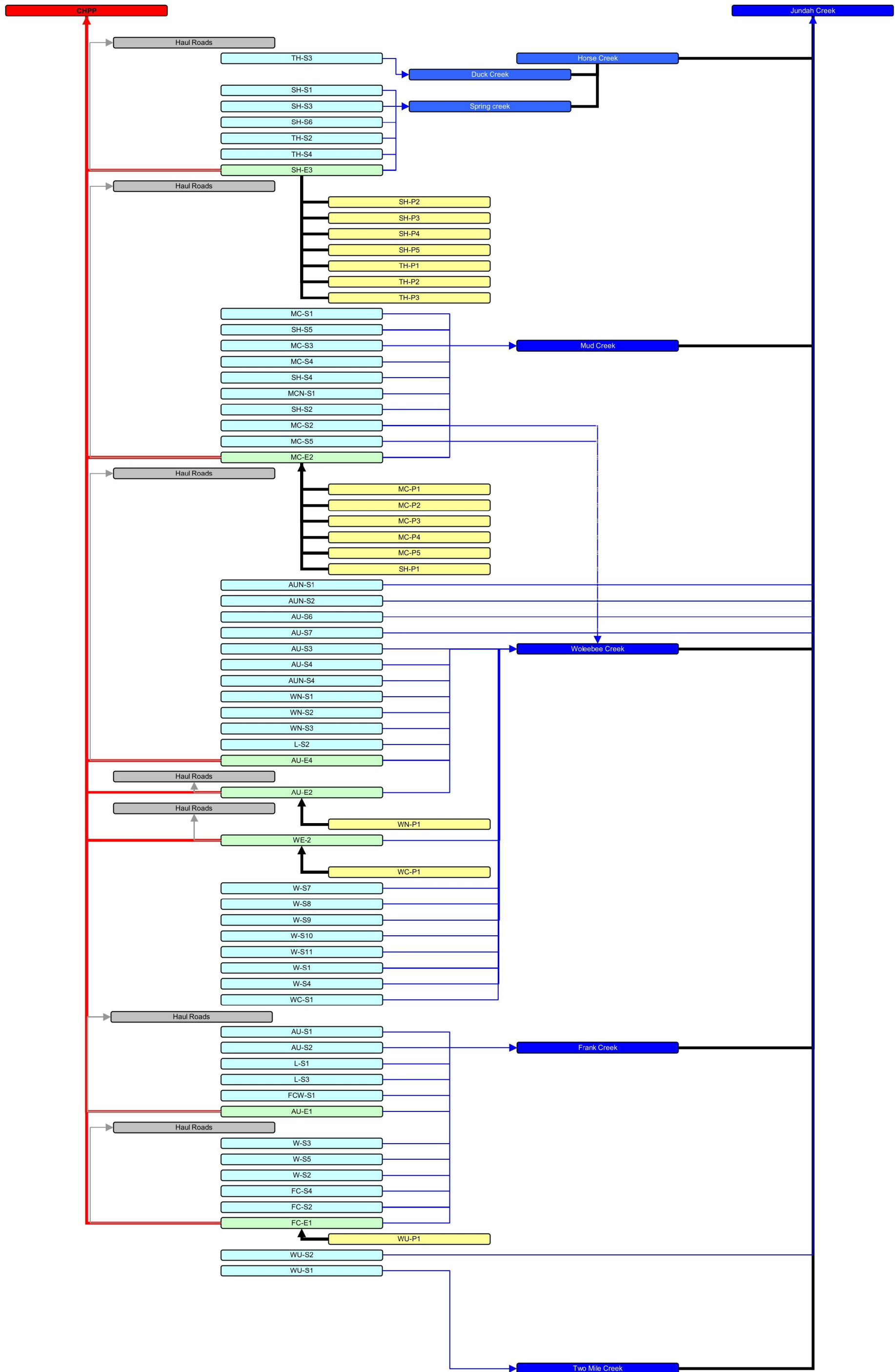


Figure 4-6: Central IQQM model schematic - Year 30

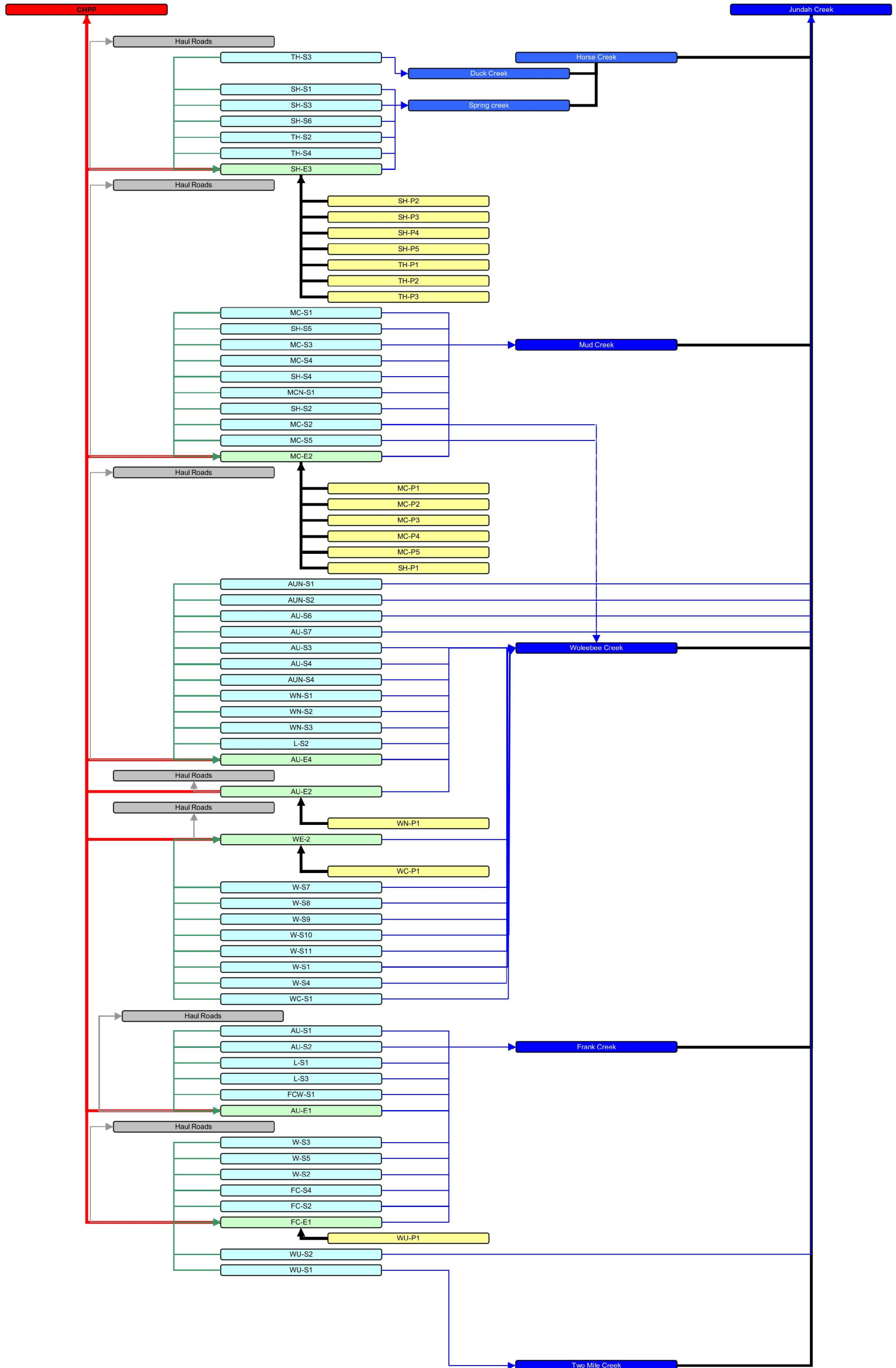


Figure 4-7: Western IQQM model schematic - Year 30

4.6 Catchment delineation and initial dam sizing

Dam sites were chosen to take advantage of the natural topography where possible, with most of the storage volume provided above the natural surface. Suitable material may be extracted from borrow pits located within the proposed storage and this may allow embankment heights to be reduced slightly.

Catchments reporting to each structure were delineated using the conceptual layouts and making reasonable assumptions about the likely destination of runoff from out of pit dumps.

The catchments reporting to each water management structure are shown in Figures 4-1 to 4-5 and also tabulated in Table 4-3 and, Table 4-4. These tables also show the dam volumes adopted for the purposes of the IQQM modelling (10% AEP 24 hour). If initial overburden runoff quality monitoring indicates acceptable dissolved solids, the lesser standard will be adopted for all sediment dams (the one hour duration has been shown as indicative of the time of concentration duration event). Initial water quality modelling indicates this is likely to be the case.

Table 4-3: Sediment dam catchment areas and guideline volume requirements

Catchment area (ha)					Design volume options (based on maximum catchment area)	
Dam	Y5	Y10	Y20	Y30	10% AEP 1 hour	10% AEP 24 hour
					ML	
AU-S1	102	112	55	51	39	85
AU-S2			50	70	24	53
AU-S3	75	93	703	699	242	535
AU-S4	63	101	43	48	35	77
AU-S6	130	315	470	467	162	358
AU-S7			86	125	43	95
AUN-S1	38	39	39	39	14	30
AUN-S2	172	172	172	172	59	131
AUN-S4	72	72	72	72	25	55
FC-S1		75	73		26	57
FC-S2		84	7	152	52	116
FC-S3	58	28			20	44
FC-S4	74	12	163	163	56	124
FCW-S1			210	227	78	173
L-S1			60	310	107	236
L-S2				91	31	69
L-S3				35	12	26
MC-S1	283	318	334	399	137	304

Catchment area (ha)					Design volume options (based on maximum catchment area)	
Dam	Y5	Y10	Y20	Y30	10% AEP 1 hour	10% AEP 24 hour
					ML	
MC-S2				101	35	77
MC-S3				287	99	219
MC-S4	70	522	529	920	317	701
MC-S5			148	227	78	173
MCN-S1				317	109	242
SH-S1			61	169	58	129
SH-S2		111	228	137	79	174
SH-S3				274	94	208
SH-S4		123	188	219	75	167
SH-S5				143	49	109
SH-S6		133	237	139	82	180
TH-S2		92	276	317	109	242
TH-S3		257	259	333	115	254
TH-S4			143	146	50	111
WC-S1				130	45	99
W-S1	143	406	485	477	167	369
W-S2	73	23	23	23	25	56
W-S3	38	148	24	24	51	113
W-S4	30	270	477	411	165	364
W-S5	11	11	42	25	15	32
W-S6	33	45			16	35
W-S7		143	119	108	49	109
W-S8	119		28	28	41	91
W-S9				95	33	73
W-S10				89	31	68
W-S11				12	4	10
WN-S1			516	562	194	428
WN-S2			377	168	130	287
WN-S3				2	1	1
WU-S1	77	6	83	81	29	63
WU-S2	91	22	92	95	33	73
Total	1,753	3,732	6,871	9,178		

Table 4-4: Environmental dam catchment areas

	Catchment area (ha)			
Dam	Y5	Y10	Y20	Y30
AU-E1	33	33	30	30
AU-E2	33	33	33	33
AU-E4	83	94	83	83
FC-E1		10	10	10
MC-E2	44	37	37	37
SH-E3		68	68	68
W-E2	111	111	111	113
Total	305	387	372	375

Following the initial water balance simulation, some environmental dams showed an increased risk of spill compared to others, and the total storage capacity and maximum normal storage capacity were adjusted to prevent unplanned discharge. The adopted environmental dam volumes are summarised in Table 4-5.

It should be noted that in some areas, the full storage requirement is not required until late in the project life, and the dam construction could be staged to match increasing contributing catchment areas.

Table 4-5: Environmental dam volumes compared to design event volumes

Dam	Volume adopted from modelled behaviour (ML)	10%AEP 24 hour	10%AEP 72 hour	1%AEP 72 hour	10%AEP 4 month	1%AEP 4 month
AU-E1	400	42	54	87	170	257
AU-E2	222	42	54	87	171	258
AU-E4	522	120	152	246	481	727
FC-E1	200	13	16	26	52	78
MC-E2	600	56	71	115	226	341
SH-E3	1,000	86	110	178	348	525
W-E2	622	144	183	297	580	876

Pit catchments are summarised in Table 4-6. Water collected in pits will drain to in-pit sumps located at low points away from operational areas. No limit has been applied in the modelling on the volume of in-pit sump storage, but for most of the large pits nominated for this project, of the order of 200 ML could potentially be stored without interfering with mining.

Table 4-6: Pit sump catchment areas

Catchment area (ha)				
Dam	Y5	Y10	Y20	Y30
AU-P1		42		
AU-P2	237			
AU-P3	292			
AU-P4				
AU-P5	44			
FC-P1		29	113	
FC-P2	25	113		
FCW-P1			151	
MC-P1				73
MC-P2	121			253
MC-P3	108	84	590	596
MC-P4				44
MC-P5				105
SH-P1		201	593	629
SH-P2				359
SH-P3			204	172
SH-P4				145
SH-P5				33
TH-P1		27	347	223
TH-P2			225	285
TH-P3		36	151	165
W-P1	248	284	335	
W-P2	84			
W-P3	119		62	
WC-P1			101	516
WN-P1			129	314
WN-P2			25	
WU-P1	121	261	121	448
Total	1,399	1,076	3,147	4,359

4.7 Sensitivity analysis

The robustness of the water management system was tested through sensitivity analysis under different runoff quantity and quality scenarios, as described in the following sections.

4.7.1 Sensitivity to uncertainty in runoff quantities

While previous experience suggests the adopted runoff characteristics are reasonable, there is little data available to categorically support the above parameters. Research on spoil runoff characteristics has indicated high variability from site to site, and from pit to pit. The presence of preferential flow paths from the surface to pits and surface sealing of spoil material can result in relatively large volumes of runoff making its way to the pit floor. Consequently, a sensitivity analysis is normally required to better understand the potential variance from the modelled behaviour.

For the purposes of this analysis, a second runoff scenario was examined to check the robustness of the proposed system with higher than expected runoff inflows. For the purpose of this analysis it was assumed that:

- natural or undisturbed catchments are unchanged
- runoff from open pits was a factor of 1.7 higher than under the base case
- runoff from rehabilitated spoil was a factor of 2.6 higher than under the base case
- unrehabilitated spoil would respond to rainfall in the same way as open pit areas, a factor of 10.5 higher than the base case. (Note this was erroneously reported as a factor of 7 in the original EIS report).

4.7.2 Sensitivity to operational changes due to runoff quality characteristics

As described in Section 2.3, it is likely that runoff from overburden dumps would have high suspended solids concentrations only, and consequently the proposed sediment dams will be designed as 'dry' sediment basins. However, in the event that overburden runoff has very poor quality, the water will be captured and stored for reuse on site. A second model operations scenario (Case B) was developed to assess the project's impacts under these conditions.

In Case B sediment dams volumes are increased to the 10 year ARI 24 hour runoff volume. In this case, all dams are operated as 'wet basins', with pit water and sediment dam water pumped to environmental dams for use in haul road watering, or the CHPP for preferential reuse. Again it is emphasised that based on the available overburden salinity data, this outcome is very unlikely. It has only been discussed here for the purposes of demonstrating that the water management system is robust and that unexpected water quality can be managed through appropriate contingency measures. Section 6.4 demonstrates that given the dilution ratios at each dam outlet, and the expected runoff quality, the sediment dam discharge water is likely to be well below the nominated licensed receiving water quality limit for salinity.

5. Water balance simulation results

5.1 Site water balance

The overall results of the water balance modelling are summarised in Table 5-1 below. Note that the tailings management system is excluded from this analysis, as the CHPP makeup demand is net of tailings return water, and the tailings management system is treated as a separate closed system

The table shows mean annual flows for each component of the water balance (based on 108 years of historical simulation over each design stage).

Throughout the project, on average, there is a large site water deficit, with most water imported via the raw water pipeline to make up the shortfall. The average imported volume reduces as the disturbed area captured by the water management system increases.

The modelled operating strategy assumes that water is pumped rapidly from the environmental dams to the central water storage for use in the CHPP. In practice, a greater proportion of the haul road demands may be supplied using water from this central storage in preference to water imported via the raw water pipeline. During detailed design, there will be opportunities for optimising the location of on-site storage, so that it is located close to the sources of demand throughout the site.

Table 5-1: Total mean annual site water balance

		Year 0	Year 5	Year 10	Year 20	Year 30
Catchment area						
Water management system	ha		3,457	5,195	10,390	13,912
Undisturbed MLAs (exc WMS)	ha	32,200	28,743	27,005	21,810	18,288
Total over MLAs	ha	32,200	32,200	32,200	32,200	32,200
Proportion of MLA in WMS		0.0%	10.7%	16.1%	32.3%	43.2%
Runoff						
Natural	ML/a	-	442	561	878	925
Pit/Ramp	ML/a	-	763	728	1,562	3,882
Spoil	ML/a	-	121	236	437	212
Rehabilitated areas	ML/a	-	156	358	986	2,040
Total	ML/a	-	1,481	1,883	3,863	7,059
Undisturbed MLAs (exc WMS)	ML/a	8,883	7,933	7,454	6,020	5,048
Total runoff on MLAs	ML/a	8,883	9,415	9,337	9,882	12,107
Outflows from WMS						
Dam evaporation (net of rain)	ML/a	-	14	34	46	80
Haul road fill pts (worked water)	ML/a	-	101	120	211	334
CHPP makeup (worked water)	ML/a	-	921	790	1,807	2,628
Sed dam release and overflow	ML/a	-	451	934	1,789	2,506
Raw water pipeline inflows						
Haul road fill pits deficit	ML/a	-	927	752	680	243
CHPP makeup deficit	ML/a	-	8,306	8,437	7,420	4,380

6. Potential impacts and mitigation measures

6.1 Wet-weather impacts on mining

During the later stages of the mine life, the potential catchment area draining to the pits is at its maximum. Small water volumes will be able to be contained in in-pit sumps without interruption to mining activities, however during extended wet periods, with standard capacity dewatering systems, relatively large volumes of water will accumulate in-pit. Table 6-1 shows the maximum water volumes in each dam over the 108 year simulation period for the Year 30 catchment layout. Pits MC-P3, SH-P1 and SH-P2 accumulate the most water, with maxima of 1,046 ML, 1,440 ML and 610 ML respectively.

Table 6-1: Maximum simulated in-pit water volume for Year 30 catchment layout

Pit	Y5	Y10	Y20	Y30
AU-P1		47		
AU-P2	248			
AU-P3	277			
AU-P5	51			
FC-P1		45	71	
FC-P2	47	64		
FCW-P1			244	
MC-P1				139
MC-P2	84			375
MC-P3	77	129	579	1,046
MC-P4				82
MC-P5				208
SH-P1		251	597	1,440
SH-P2				610
SH-P3			160	163
SH-P4				98
SH-P5				34
TH-P1		24	498	509
TH-P2			161	313
TH-P3		39	104	139
W-P1	222	244	197	
W-P2	28			
W-P3	80		78	
WC-P1			143	474
WN-P1			67	220
WN-P2			22	
WU-P1	212	223	119	411
Total	1,326	1,066	3,040	6,261

The environmental dam sizes proposed here have been chosen to provide a reasonable level of pit availability without pit/process water spills from the site. The volume adopted to meet these operational criteria is in excess of the 10% AEP DSA volume recommended in the Technical Guidelines (and in most cases the design volume exceeds the 1% AEP DSA).

During very wet periods, the combined capacity of the environmental dam return water system pumps will exceed the CHPP makeup demand. Additional storage will be required to temporarily hold the excess water for later reuse. Modelling of the system indicates that the maximum volume required in this storage would be of the order of 18,000 ML.

A number of options may be available to provide this storage:

- a number of other pits around the site will be inactive for periods while mining focuses on other areas, and these may be used to store water if necessary
- there may be excess storage space available in the tailings and rejects storage facility during the earlier years of production
- additional environmental dam storage could be provided by the construction of new excavated storages around the site if required.

The results show that for some of the time, haul road demands can be supplied from the local water management system. However, during dry periods, water will need to be transferred between process/pit water storages to make up shortfalls — and in very dry periods, imported raw water will be used for road dust suppression.

6.2 Environmental dam performance

Pumping from pits to each environmental dam will be stopped when a specified maximum operating level is reached to ensure that a reserve storage allowance is maintained below the spillway crest level. This will ensure that following an episode of prolonged pit dewatering, small runoff events from the catchment draining directly into the environmental dams does not cause the dam to overflow.

Examples of the simulated historical storage behaviour are shown in Figure 6-1 and Figure 6-2. The plots show the dams are typically close to empty — with generally short periods during which they store significant volumes of water before being pumped out to meet water demands. The exception is during extended wet periods such as during the mid 1950s, when the dams would have remained close to full for periods of up to three months, as water collected in the nearby pits is gradually transferred out.

The environmental dams and the pumping rules were configured to ensure the dams did not overflow during the simulation period so that downstream water quality is maintained. Any overflows from the WMS occur from the sediment dams only. The maximum normal operating levels used in the analysis were generally higher than mandatory reporting levels calculated as recommended in the Technical Guidelines. Acceptable maximum operating levels will be agreed with the DERM.

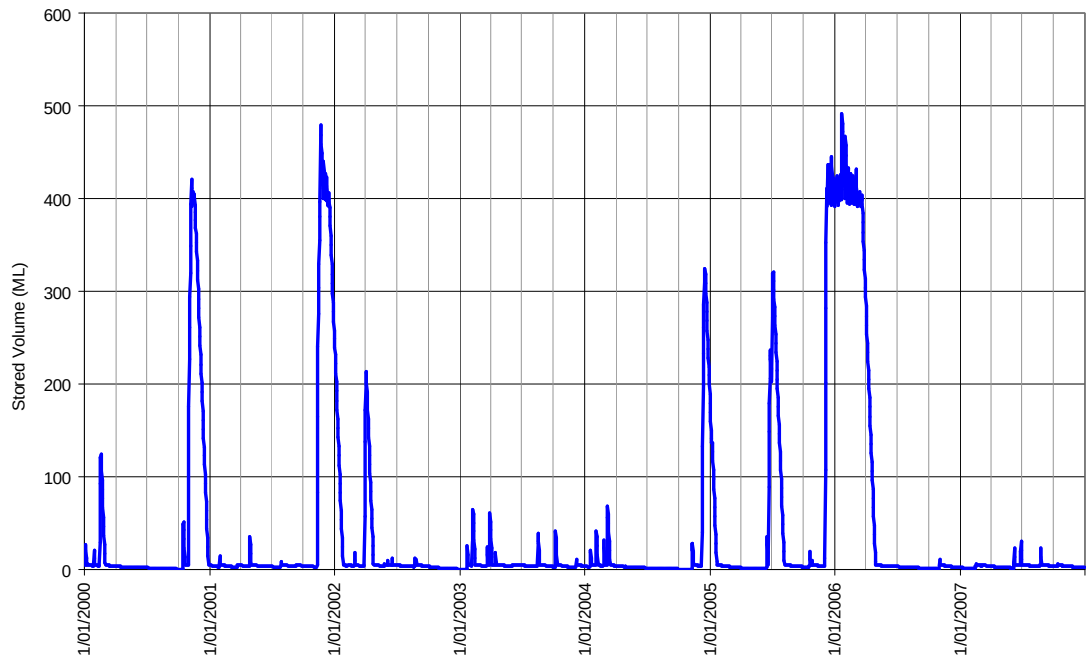


Figure 6-1: Simulated historical storage behaviour at MC-E2 environmental dam (Post 2000) (Year 30 catchments)

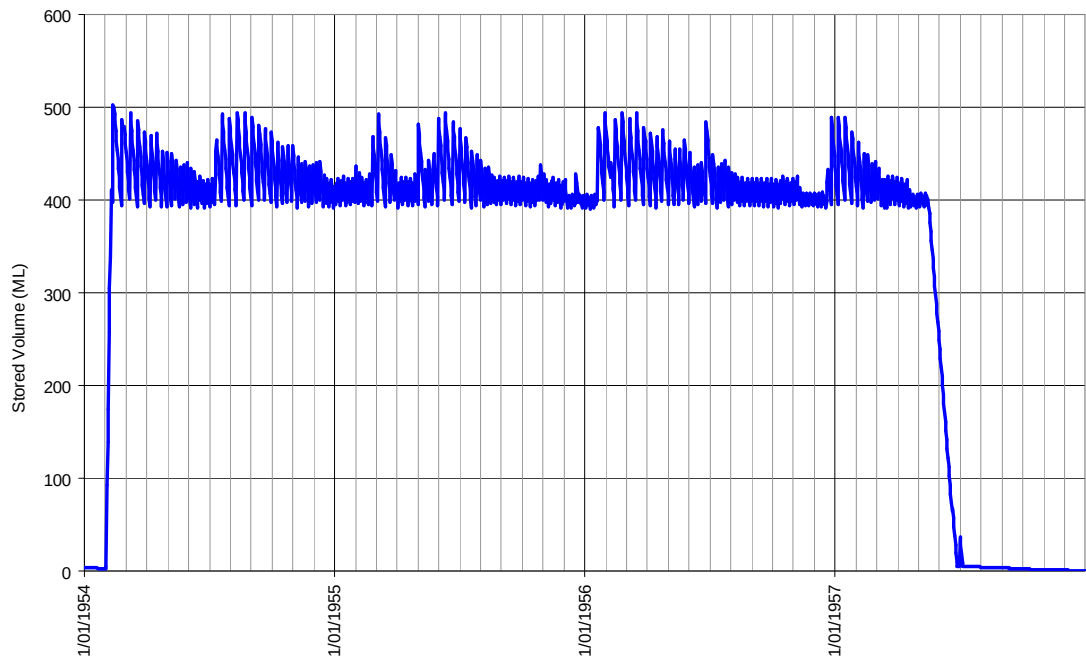


Figure 6-2: Simulated historical storage behaviour at MC-E2 environmental dam (1950s wet period) (Year 30 catchments)

For the pit/process water management system, saline pit/process water could potentially concentrate and accumulate in the environmental dams. The conservative solute modelling capabilities of the IQQM have been used to estimate a possible range of salinities in the environmental dams over the project life, using the above assumptions on runoff quality. The results show:

- TDS levels would only reach sub-lethal levels when stored water volumes drop to very low levels due to evaporation and subsequent concentration below the pump inlet level
- following large rainfall events, when the risk of discharge is highest, the salinity drops significantly. The maximum salinity level is variable across the various environmental dams, but typically, when water levels are less than 100 ML from the spillway crest, the modelled TDS is slightly above drinking water standard and the stored water would therefore be considered uncontaminated or low-toxicity waste
- typical modelled environmental dam salinity levels rarely exceed 5,000 mg/L and would therefore generally be suitable for all onsite uses. All significant volumes of water will be suitable for reuse at the CHPP after blending with raw water from the raw water dams.

6.3 Impacts on downstream flow

The use and evaporation of water captured in the pit/process water management system sees a reduction in the volume of runoff from the site. However, this is offset by an increase in runoff from the disturbed areas (due to changed catchment characteristics). Table 5-1 shows that the changes to the runoff characteristics of catchments around the site increases runoff from the MLAs over the life of the project. The effect is small early in the project, but by Year 30, when 43% of the MLAs are captured in the water management system, the mean annual runoff will have increased from 8,883 ML/a to 12,107 ML/a.

Table 6-2 and Table 6-3 show the impact the project will have on mean annual runoff, in the major catchments crossing the MLAs, at key reporting points. The reporting points are shown graphically in Figure 3-1 and represent locations where flow analysis has been undertaken. The reporting points for Duck Creek, Mud Creek, Spring Creek and Woleebee Creek are at the same locations as the discharge points for these creeks. The reporting points for Juandah Creek downstream of Two Mile Creek confluence (Juandah Creek immediately downstream of MLA boundary and Horse Creek downstream of Duck Creek confluence) are provided specifically for reporting purposes.

Assessment has also been undertaken directly downstream of the Juandah Creek-Horse Creek confluence (shown in Figure 3-1 as a reporting point). This reporting point encapsulates the total impact of the development and results in a reduction in the mean annual flow of 1.7%. The nearest surface water entitlement to the development (a water harvesting licence) is held just upstream of this confluence on Juandah Creek and the impact on the mean annual flow at this location is a decrease of 2.3%.

Table 6-2: Reduction in mean annual flow from MLA area (ML/a)

Creek	Year 5	Year 10	Year 20	Year 30
Juandah Creek	184	264	336	471
Two Mile Creek	3	0	3	0
Woleebee Creek	358	187	284	302
Mud Creek	77	81	428	694

Creek	Year 5	Year 10	Year 20	Year 30
Spring Creek	0	100	236	208
Duck Creek	0	71	71	92
Total	619	703	1,363	1,767

Table 6-3: Percentage reduction in mean annual flow near downstream boundary of MLA.

Creek	Year 5	Year 10	Year 20	Year 30
Juandah Creek	0.8%	1.1%	1.4%	2.0%
Juandah Creek d/s of Two Mile Creek	0.0%	0.0%	0.0%	0.0%
Woleebbee Creek	1.7%	0.9%	1.3%	1.4%
Mud Creek	1.5%	1.6%	8.3%	13.4%
Spring Creek	0.0%	6.6%	15.7%	13.8%
Duck Creek	0.0%	4.5%	0.0%	0.1%
Horse Creek d/s of Duck Creek	0.0%	0.1%	0.0%	0.0%
Total	1.2%	1.2%	2.5%	3.2%

Note: Horse Creek downstream of Duck Creek confluence figures not included in totals.

For most of the project life the loss of runoff to the pit/process water system is offset by the temporary increase in runoff from disturbed areas, resulting in a very slight increase in typical downstream flow as the project proceeds. However, by Year 30 the area intercepted by pits becomes large enough to impact downstream flows, and in Mud Creek and Spring Creek the impact is relatively significant, with reductions of 13.4% and 13.8% respectively in mean annual flow. The percentage reduction is greater in these creeks due to the number of pits in their catchment areas. In all other creeks the impact is minimal.

Mean annual flow can be influenced by a small number of very large flow events and as such, its relevance as a representative statistic is limited. A more descriptive assessment of the changes in the flow regime throughout the year is provided by looking at the monthly distribution of flow. One measure is the average median (also referred to as the mean of the median) flow for each month, as depicted in the figures presented in Attachment C. This statistic is determined by calculating the median flow for each month of the simulation period and then determining the mean of these results for each individual month. Flows of this magnitude are of interest to stock and domestic users as they are potentially important for filling waterholes following dry periods.

Results have been provided for Juandah Creek (downstream of MLA boundary), Woleebbee Creek, Mud Creek, Spring Creek and downstream of the site at the Horse Creek/Juandah Creek confluence. The charts in Attachment C show that the project does not effect the seasonal distribution of flow and in most cases there is a small increase in monthly flow as a result of the increased runoff from disturbed areas.

Another statistic that is suitable in assessing the impact of a development on downstream flows is the low flow spells statistic. This statistic looks at the number of spells of a given

duration (31 days in this case) below a specified threshold (200 ML/d). Low flow spells can also be of interest to stock and domestic users as they will highlight if the impact on the flow regime will cause longer, or a greater number of periods of low flow that might impact on their operations. The low flow spells have been assessed at the same reporting points as the average median assessment above and the results have been provided in Attachment D.

The results presented in attachment D show very little impact except at Spring Creek, and to a lesser extent, Mud Creek. At both of these locations there is a decrease in the number of flow spells and an increase in the spell duration. Closer investigation of the results confirmed that this was a result of a reduction in the number of flow events greater than 200 ML/d as a result of the proposed development. If a flow event is lost from the system, what was previously seen as two low flow spells interspersed by that flow event, will then become one long low flow event. The duration of the one low flow event will then be equivalent to the sum of the two original low flow events. Thus, the results in these two creeks indicate that a small number of flow events (200 ML/d) have been lost from the system at these locations.

The largest impact is on Spring Creek where there is a reduction in the number of flow spells of 16% and a corresponding increase in spell duration of 19%. This reduces to 2% for both statistics for Mud Creek, and for the remaining creeks is much less again

6.4 Water management system outflow

Tables 6-4 and 6-5 below show that discharges from the water management system (sediment dam releases and overflows) grow over time, but make up only a small proportion of flows at the MLA boundary. By Year 30, they make up only 4.1% of flows in the creeks downstream of the MLA. This potential dilution indicates that even if sediment dam water quality is worse than expected, the impact on downstream concentrations of potential contaminants is likely to be relatively small, if the receiving streams are flowing. At Duck Creek however, where the proportion of disturbed land is greatest, overflows make up 33% of total catchment flow at the MLA boundary. Nine kilometres downstream, at the Horse Creek/Duck Creek confluence, the sediment dam overflows would make up only 0.4% of the mean annual Horse Creek flow.

Table 6-4: Mean annual outflow from water management system (ML/a)

Stream	Year 5	Year 10	Year 20	Year 30
Juandah Creek	116	151	227	248
Two Mile Creek	18	2	20	22
Woleebee Creek	123	277	779	889
Mud Creek	96	249	318	649
Spring Creek	0	54	179	285
Duck Creek	0	58	71	92
Total	335	788	1,575	2,162

Table 6-5: Proportion of mean annual flow near MLA boundary that is WMS outflow

Stream	Year 5	Year 10	Year 20	Year 30
Juandah Creek	0.5%	0.6%	1.0%	1.0%
Juandah Creek d/s of Two Mile Creek	0.1%	0.0%	0.1%	0.1%
Woleebee Creek	0.6%	1.3%	3.6%	4.1%
Mud Creek	1.9%	4.8%	6.1%	12.5%
Spring Creek	0.0%	3.6%	11.9%	19.0%
Duck Creek	0.0%	21.0%	25.7%	33.1%
Horse Creek d/s of Duck Creek	0.0%	0.3%	0.3%	0.4%
Total	0.6%	1.5%	3.0%	4.0%

Note: Horse Creek d/s of Duck Creek figures not included in totals.

It is expected that water will not be stored in sediment dams for extended periods, so there will be no increase in the concentration of dissolved contaminants over time. However, a likely upper bound estimate for the salinity of sediment dam discharge has been made based on the ratios of disturbed catchment areas and their corresponding estimated runoff salinities (as ascribed to various catchment types in Section 2.4).

Even where sediment dam overflows make up a large proportion of overflows, the impact on downstream water quality is likely to be small. The resultant estimated overflow salinities at the MLA boundary presented in Table 6-6 are well within the proposed licensed receiving water quality limits for salinity, even before allowing for further dilution between these flows and other contributing flows from outside the MLA area.

Table 6-6: Indicative EC ($\mu\text{S}/\text{cm}$) of sediment dam overflow (all dams overflowing)

Stream	Year 5	Year 10	Year 20	Year 30
Two Mile Creek	617	200	619	200
Frank Creek	442	268	200	200
Juandah Creek	272	200	316	200
Woleebee Creek	529	414	338	200
Mud Creek	238	668	484	282
Spring Creek	n/a	592	466	229
Duck Creek	n/a	710	200	200

6.5 Sensitivity analysis

The robustness of the water management system under a range of conditions was tested through sensitivity analysis as described in Section 4.7. The results are presented in this section.

6.5.1 Sensitivity analysis – high runoff scenario

In the year 30 high runoff scenario, large water volumes could potentially be supplied from the water management system following wet periods. The proportion of CHPP demand supplied would ultimately be limited by the return water pipeline capacity from the west of the site. During very dry periods, most of the site water demand would need to be delivered via the pipeline, and sufficient on-site storage will be needed to meet haul road watering requirements. The total site water balance is summarised in Table 6-7.

Table 6-7: Total mean annual site water balance — high runoff sensitivity analysis

		Year 0	Year 5	Year 10	Year 20	Year 30
Catchment area						
Water management system	ha		3457	5195	10390	13912
Undisturbed MLAs (exc WMS)	ha	32,200	28,743	27,005	21,810	18,288
Total over MLAs	ha	32,200	32,200	32,200	32,200	32,200
Proportion of MLA in WMS		0.0%	10.7%	16.1%	32.3%	43.2%
Runoff						
Natural	ML/a		442	561	878	925
Pit/Ramp	ML/a		1,340	1,279	2,743	4,186
Spoil	ML/a		1,284	2,516	4,655	2,259
Rehabilitated areas	ML/a		409	938	2,582	5,344
Total	ML/a	-	3,474	5,294	10,858	12,714
Undisturbed MLAs (exc WMS)	ML/a	8,883	7,933	7,454	6,020	5,048
Total runoff on MLAs	ML/a	8,883	11,408	12,748	16,878	17,761
Outflows from WMS						
Dam Evaporation (net of rain)	ML/a	-	16	42	121	238
Haul road fill pts (worked water)	ML/a	-	175	258	516	706
CHPP makeup (worked water)	ML/a	-	1,944	1,515	4,806	6,096
Sed dam release and overflow	ML/a	-	1,335	3,465	5,389	5,630
Raw water pipeline inflows						
Haul road fill pts deficit	ML/a	-	853	614	375	129
CHPP makeup deficit	ML/a	-	7,283	7,712	4,421	912

The environmental dams are able to be operated without overflows, but additional on-site storage would be required to manage interruptions if conditions similar to the wettest on record were encountered during operations. In-pit storage in disused pit would be used for this purpose.

Table 6-8 shows the impact of the project on downstream flows with a slight increase evident in flows from Year 10 onwards.

Table 6-8: Percentage reduction in mean annual flow near downstream boundary of MLA – high runoff sensitivity analysis

Stream	Year 5	Year 10	Year 20	Year 30
Juandah Creek d/s of Two Mile Creek	-0.5%	0.0%	-0.5%	-0.2%
Juandah Creek	-1.9%	-1.0%	-2.2%	-2.5%
Woleebee Creek	0.1%	-2.4%	-4.9%	-2.2%
Mud Creek	0.9%	-18.2%	-8.5%	-6.7%
Spring Creek	0.0%	-5.3%	-17.0%	-6.3%
Duck Creek	0.0%	-92.0%	-29.5%	-38.8%
Horse Creek d/s of Duck Creek	0.0%	-1.2%	-0.4%	-0.5%
Total	-0.7%	-3.9%	-4.5%	-3.1%

Sediment dam discharges would be significantly increased compared to the base case scenario. At Duck Creek, they would make up 72% of Duck Creek flows at the MLA boundary, but only 10.4% overall.

Table 6-9: Proportion of mean annual flow near MLA boundary that is WMS overflow – high runoff sensitivity analysis

Stream	Year 5	Year 10	Year 20	Year 30
Juandah Creek d/s of Two Mile Creek	0.6%	0.0%	0.7%	0.4%
Juandah Creek	3.1%	2.7%	4.6%	5.5%
Woleebee Creek	2.2%	4.6%	9.9%	7.8%
Mud Creek	2.4%	24.6%	22.9%	32.6%
Spring Creek	0.0%	15.5%	44.7%	39.1%
Duck Creek	0.0%	117.5%	55.3%	72.0%
Horse Creek d/s of Duck Creek	0.0%	1.5%	0.7%	0.9%
Total	2.6%	6.6%	10.0%	10.4%

6.5.2 Sensitivity analysis – reuse overburden runoff water

For the purposes a sensitivity analysis to check the robustness of the WMS, and the effectiveness of contingency planning, assessment of the impacts on downstream flows was also performed assuming that if runoff quality was unexpectedly poor, the dams would be sized to contain runoff from the 10% AEP 24 hour rainfall event. The 10% AEP standard is considered appropriate when the receiving waters are not environmentally sensitive, and the stored water is less than 10 times the NHMRC drinking water standard (as is very likely to be the case).

It should be noted that based on the available overburden salinity data, this outcome is very unlikely. It has only been contemplated here for the purposes of demonstrating that the water management system is robust and that unexpected water quality can be managed through appropriate contingency measures. Section 6.4 demonstrates that given the dilution ratios at each dam outlet, and the expected runoff quality, the sediment dam discharge

water is likely to be well below the nominated licensed receiving water quality limit for salinity.

For the purposes of this sensitivity analysis, it was assumed that the design environmental dam capacities would be unchanged. The additional water would be accommodated by reducing the volume that could be pumped from pits, so that environmental dam's are operated as per the base case. Storage exceedance curves for SH_P1 and MC_P3 are presented in Figure 6-6 demonstrating the impact this scenario would have on pit water levels. The risk of discharge of pit/process water from environmental dams will be essentially unchanged.

There will be a number of consequences of these changes:

- the duration of in-pit inundation will be increased as illustrated in Figure 6-3
- the reliability of water supplies captured on site will increase
- the reduction in downstream streamflow will be greater than predicted in the base case, as shown in Table 6-10 below
- the proportion of the mean annual flow at the MLA boundary that is WMS overflow will be less than predicted in the base case, as shown in Table 6-11 below
- occasional unplanned overflow from sediment dams will represent a smaller proportion of downstream streamflow
- additional temporary storage will be required to store the maximum volume in excess of the CHPP and dust suppression requirements – under the wettest period on record during the 1950s – the storage requirement would have been 33,000 ML.

If pit availability is reduced unacceptably, a mitigation measure may be to provide additional environmental dam storage to allow pits to be dewatered more rapidly.

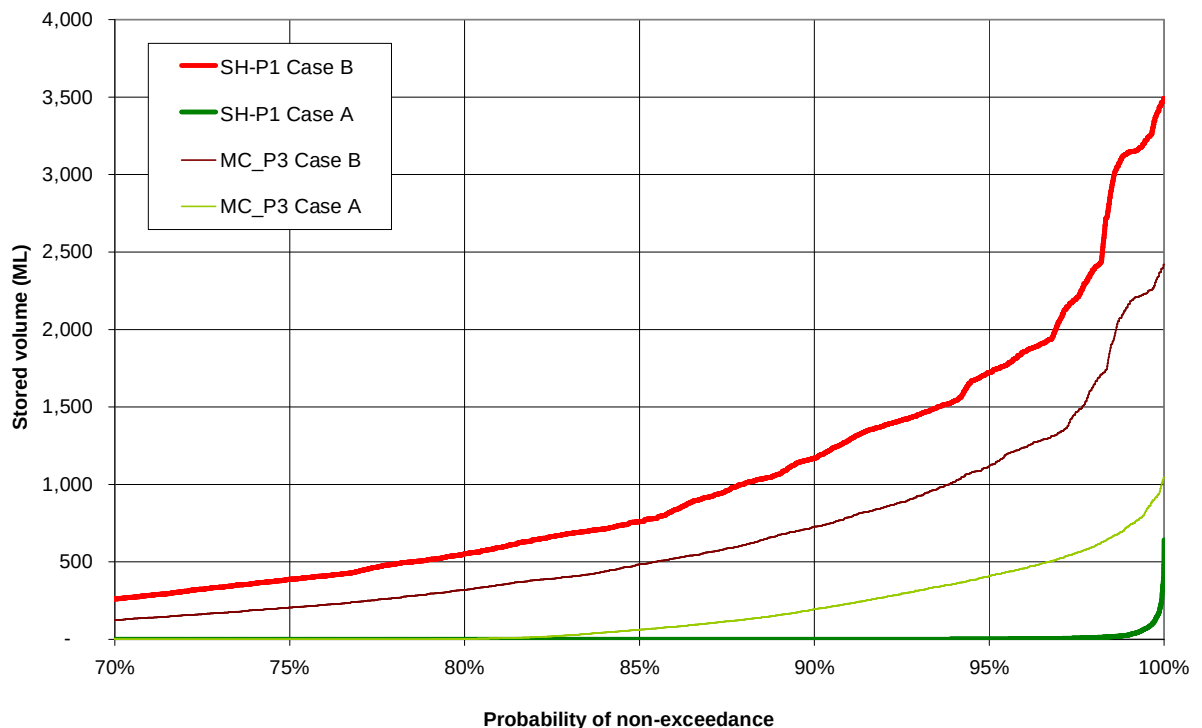


Figure 6-3: Sensitivity analysis - simulated duration of pit flooding at MC-E2 environmental dam (Year 30 catchments)

Table 6-10: Percentage reduction in mean annual flow near downstream boundary of MLA – low runoff quality sensitivity analysis

Stream	Year 30
Juandah Creek d/s of Two Mile Creek	0.1%
Juandah Creek	3.0%
Woleebee Creek	5.5%
Mud Creek	25.2%
Spring Creek	32.4%
Duck Creek	32.6%
Total	7.2%

Table 6-11: Proportion of mean annual flow near MLA boundary that is WMS overflow – low runoff quality sensitivity analysis

Stream	Year 30
Juandah Creek d/s of Two Mile Creek	0.1%
Juandah Creek	0.1%
Woleebee Creek	0.1%
Mud Creek	0.8%
Spring Creek	3.5%
Duck Creek	0.5%
Total	0.1%

7. Conclusions

The proposed conceptual water management system balances the need to maintain the downstream flow regime while managing the quality of water discharged from disturbed areas of the site.

Sediment dams to capture runoff from mine areas not impacted by pit/process water or mine industrial area (i.e. from overburden runoff) will be provided to allow coarse sediment to settle before discharging from the site. There is to be no planned release of any other water captured within the site, unless the stored water quality and receiving water flows are such that the receiving water quality limits can be met.

The results of the historical simulation modelling indicate that under historical climate extremes, all pit/process water runoff could be captured and contained on the site for reuse. The volume of water captured and reused in the pit/process water management system will increase as the project proceeds, due to the increase in captured catchment areas and the increased runoff due to catchment disturbance draining to the pits.

The capture and reuse of runoff in pits and environmental dams will result in a reduction in runoff from the MLAs. However, because the major catchments crossing the site are diverted through the site, the impact on mean annual flow in major creeks is relatively small. By Year 30, the mean annual flow immediately downstream of the MLAs could be reduced by 3.2%. Further downstream, on Juandah Creek upstream of the Horse Creek/Juandah Creek confluence, the simulated impact on mean annual flow is less than 2.3%.

In later years of the mine development, if an extreme or prolonged wet period occurs, re-use of large quantities of water captured in the pit/process water management system will be critical to managing the risk of interruptions to mining and off-site discharge. The system can be managed to maximise the reliability of haul road dust suppression supply by limiting CHPP reuse of site water. However, this results in an increase the maximum volume of water accumulated during wet periods. A high capacity water pipeline will be required to return water to the CHPP for this purpose. This water could eventually be used within the CHPP without off-site discharges and would reduce the volume imported from the raw water pipeline. Based on the 1950s wet period, up to 18,000 ML of temporary water storage, in addition to the proposed environmental dams, could be required to manage the excess. A number of potential options for this additional storage have been identified. In later stages of design, the location and capacity of this storage, and the associated transfer pipelines will be refined and optimised.

For sediment dams, it is expected that water will not be stored for extended periods, so there will be no increase in concentration of dissolved contaminants over time in these dams. However, a likely upper bound for the salinity of sediment dam discharge has been made based on the ratios of disturbed catchment areas and their corresponding estimated runoff salinity. Conservatively assuming overburden runoff has an electrical conductivity (EC) of 1,000 $\mu\text{S}/\text{cm}$ (the maximum level measured in the overburden geochemical analysis); the highest combined sediment dam salinity likely to be discharged to a stream is 710 $\mu\text{S}/\text{cm}$ well below the proposed licensed receiving water quality limit for salinity.

For the pit/process water management system, saline pit/process water could potentially concentrate and accumulate in the environmental dams. The conservative solute modelling capabilities of the IQQM have been used to estimate a possible range of salinities in the environmental dams over the project life, using the above assumptions on runoff quality. The results show

TDS levels would only meet sub-lethal or toxic levels when stored water volumes are drop to very low levels due to evaporation and subsequent concentration below the pump inlet level.

Following large rainfall events, when the risk of discharge is highest, the salinity drops significantly. The maximum salinity level is variable across the various environmental dams, but typically, when water levels are less than 100 ML from the spillway crest, the modelled TDS is slightly above drinking water standard and the stored water would therefore be considered uncontaminated or low-toxicity waste.

Typical modelled environmental dam salinity levels rarely exceed 5,000 mg/L and would therefore generally be suitable for all onsite uses and stock watering. All significant volumes of water will be suitable for reuse at the CHPP after blending with raw water from the raw water dams.

8. Limitations

The IQQM model has been prepared for the purposes of assessing the change in flows from the site due to the mine development, and should not be used for any other purpose without consideration of the inherent modelling uncertainties and simplifications.

One limitation of the approach here is that rainfall is assumed to be spatially uniform. This assumption does not affect many of the conclusions drawn in this report; however, it may affect the assumed levels of dilution during small locally intense rainfall events.

The model runoff parameters have been selected using experience on other similar projects with limited quantitative data to assess the runoff-characteristics of disturbed mine site catchments.

The indicative water quality analysis has been undertaken based on very limited surface water and groundwater quality data collected as part of the EIS baseline studies. Further data from disturbed areas of the site will be needed to reduce uncertainties in the water quality analysis.

While the model assesses the performance of the system under historical extremes which may reasonably be expected to re-occur in the future, it does not specifically quantitatively incorporate the impact of future climate change on runoff.

The model does not specifically include the tailings and coarse rejects disposal systems, and the runoff water which would be contained and reused within the tailings return water system.

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

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Attachment A

Annual water balance summaries for
median, 10th percentile and
90th percentilile flows

Attachment B

Catchment types draining to each
dam

Year 5

		Rehab (ha)	Spoil (ha)	Pit-Ramp (ha)	Natural (ha)
JUNDAH CREEK	AU-S6	130	0	0	0
	AUN-S1	16	0	0	22
	AUN-S2	138	0	0	34
	WU-S2	34	34	0	22
	AU-P5	0	0	44	0
	TOTAL	319	34	44	78
DUCK CREEK	-	0	0	0	0
	TOTAL	0	0	0	0
SPRING CREEK	-	0	0	0	0
	TOTAL	0	0	0	0
MUD CREEK	MC-S1	0	8	0	274
	MC-S4	0	7	0	64
	MC-E2	0	0	16	28
	MC-P2	0	0	30	91
	MC-P3	0	0	30	78
	TOTAL	0	15	75	535
WOOLEEBE CREEK	AU-S3	0	8	0	67
	AU-S4	0	14	0	49
	AUN-S4	19	0	0	53
	W-S1	0	42	0	101
	W-S4	0	0	0	30
	W-S8	0	119	0	0
	AU-E1	0	0	0	33
	AU-E2	0	0	21	13
	AU-E4	0	0	30	54
	W-E2	0	0	14	97
	AU-P2	43	43	115	36
	AU-P3	79	79	111	23
	W-P1	0	99	94	55
	W-P2	0	0	14	70
	W-P3	0	37	43	40
	TOTAL	141	441	441	719
FRANK CREEK	AU-S1	0	0	0	102
	FC-S3	25	11	0	22
	FC-S4	44	11	0	18
	W-S2	0	51	0	23
	W-S3	0	14	0	24

		Rehab (ha)	Spoil (ha)	Pit-Ramp (ha)	Natural (ha)
TWO MILE GULLY	W-S5	0	0	0	11
	W-S6	0	18	0	15
	FC-P2	0	0	25	0
	WU-P1	0	0	73	48
	TOTAL	69	105	98	263
	WU-S1	36	36	0	6
	TOTAL	36	36	0	6

Year 10

		Rehab (ha)	Spoil (ha)	Pit-Ramp (ha)	Natural (ha)
JUNDAH CREEK	AU-S6	221	0	0	93
	AUN-S1	16	0	0	23
	AUN-S2	138	0	0	34
	WU-S2	0	0	0	22
	TOTAL	376	0	0	172
DUCK CREEK	TH-S3	0	146	0	111
	TOTAL	0	146	0	111
SPRING CREEK	SH-P1	0	51	122	28
	SH-S6	0	43	0	90
	TH-S2	0	55	0	37
	SH-E3	0	0	16	52
	TH-P1	0	0	27	0
	TH-P3	0	0	36	0
	TOTAL	0	149	201	207
MUD CREEK	MC-S1	0	108	0	210
	MC-S4	0	353	0	169
	SH-S2	0	60	0	52
	SH-S4	0	38	0	86
	MC-E2	0	0	16	21
	MC-P3	0	0	84	0
	TOTAL	0	559	100	537
WOOLEEBE CREEK	AU-S3	27	0	0	66
	AU-S4	51	0	0	50
	AUN-S4	19	0	0	53
	W-S1	306	0	0	100
	W-S4	115	115	0	40
	W-S7	0	143	0	0

		Rehab (ha)	Spoil (ha)	Pit-Ramp (ha)	Natural (ha)
FRANK CREEK	AU-E1	0	0	0	33
	AU-E2	0	0	21	13
	AU-E4	0	0	30	64
	W-E2	0	0	14	97
	AU-P1	0	0	42	0
	W-P1	0	16	96	173
	TOTAL	518	273	201	688
	AU-S1	50	0	0	62
	FC-S1	18	18	0	38
	FC-S2	22	22	0	40
	FC-S3	0	0	0	28
	FC-S4	0	0	0	12
	W-S2	0	0	0	23
	W-S3	124	0	0	24
	W-S5	0	0	0	11
	W-S6	31	0	0	15
	FC-E1	0	0	0	10
	FC-P1	0	0	29	0
	FC-P2	88	0	25	0
	WU-P1	70	70	73	48
	TOTAL	403	111	127	310
TWO MILE GULLY	WU-S1	0	0	0	6
	TOTAL	0	0	0	6

Year 20

		Rehab (ha)	Spoil (ha)	Pit-Ramp (ha)	Natural (ha)
JUNDAH CREEK	AU-S6	383	0	0	88
	AUN-S1	16	0	0	23
	AUN-S2	138	0	0	34
	WU-S2	35	35	0	22
	AU-S7	0	76	0	10
	TOTAL	572	111	0	177
	TH-S3	184	0	0	76
DUCK CREEK	TOTAL	184	0	0	76
	SH-S1	0	0	0	61
SPRING CREEK	SH-S6	43	107	0	87
	TH-S2	189	0	0	87

		Rehab (ha)	Spoil (ha)	Pit-Ramp (ha)	Natural (ha)
MUD CREEK	TH-S4	0	104	0	39
	SH-E3	0	0	16	52
	TH-P1	78	45	225	0
	TH-P2	0	163	62	0
	TH-P3	0	104	47	0
	TOTAL	309	524	348	326
	MC-S1	102	22	0	210
	MC-S4	141	234	0	154
	SH-S2	0	148	0	80
	SH-S4	41	0	0	147
WOOLEEBE CREEK	MC-E2	0	0	16	21
	MC-P3	0	441	149	0
	SH-P1	192	192	166	44
	SH-P3	134	0	70	0
	TOTAL	610	1,036	401	657
	MC-S5	0	68	0	80
	AU-S3	636	0	0	67
	AU-S4	0	0	0	43
	AUN-S4	19	0	0	53
	WN-S1	80	153	0	283
	WN-S2	0	54	0	323
	W-S1	384	0	0	101
	W-S4	225	178	0	74
	W-S7	119	0	0	0
	W-S8	28	0	0	0
	AU-E1	0	0	0	30
	AU-E2	0	0	21	13
	AU-E4	0	0	30	53
	W-E2	0	0	14	97
	WC-P1	0	0	101	0
	WN-P1	0	57	52	20
	WN-P2	0	0	25	0
	W-P1	38	68	42	188
	W-P3	0	0	62	0
	TOTAL	1,528	579	345	1,424
FRANK CREEK	AU-S1	0	0	0	55
	AU-S2	0	0	0	50
	FC-S1	0	0	0	73
	FC-S2	0	0	0	7

		Rehab (ha)	Spoil (ha)	Pit-Ramp (ha)	Natural (ha)
	FC-S4	155	0	0	8
	FCW-S1	61	0	0	150
	L-S1	0	0	0	60
	W-S2	0	0	0	23
	W-S3	0	0	0	24
	W-S5	32	0	0	11
	FC-E1	0	0	0	10
	FC-P1	83	0	31	0
	FCW-P1	0	0	151	0
	WU-P1	0	0	73	48
	TOTAL	330	0	254	517
TWO MILE GULLY	WU-S1	39	39	0	6
	TOTAL	39	0	0	6

Year 30

		Rehab (ha)	Spoil (ha)	Pit-Ramp (ha)	Natural (ha)
JUNDAH CREEK	AU-S6	382	0	0	85
	AUN-S1	16	0	0	23
	AUN-S2	138	0	0	34
	WU-S2	74	0	0	22
	AU-S7	109	0	0	16
	TOTAL	719	0	0	180
DUCK CREEK	TH-S3	242	0	0	91
	TOTAL	242	0	0	91
SPRING CREEK	SH-S1	44	33	0	91
	SH-S3	80	0	0	193
	SH-S6	79	0	0	60
	TH-S2	230	0	0	88
	TH-S4	108	0	0	39
	SH-E3	0	0	16	52
	TH-P1	10	0	213	0
	TH-P2	223	0	63	0
	TH-P3	108	0	57	0
	TOTAL	881	33	349	523
MUD CREEK	MC-S1	198	0	0	201
	MC-S3	56	153	0	78
	MC-S4	810	0	0	110
	MCN-S1	158	0	0	159

		Rehab (ha)	Spoil (ha)	Pit-Ramp (ha)	Natural (ha)
	SH-S2	81	0	0	56
	SH-S4	0	68	0	151
	SH-S5	115	0	0	28
	MC-E2	0	0	16	21
	MC-P1	0	0	73	0
	MC-P2	0	122	130	0
	MC-P3	0	328	268	0
	MC-P4	0	0	44	0
	MC-P5	0	0	105	0
	SH-P1	254	0	374	0
	SH-P2	195	0	164	0
	SH-P3	102	0	70	0
	SH-P4	0	101	44	0
	SH-P5	0	0	33	0
	TOTAL	1,970	773	1,321	804
WOOLEEBE CREEK	MC-S5	169	0	0	58
	MC-S2	42	0	0	59
	AU-S3	634	0	0	65
	AU-S4	0	0	0	48
	AUN-S4	19	0	0	53
	WC-S1	0	0	0	130
	WN-S1	90	0	0	472
	WN-S2	138	0	0	30
	WN-S3	0	0	0	2
	W-S1	381	0	0	96
	W-S4	346	0	0	64
	W-S7	108	0	0	0
	W-S8	28	0	0	0
	W-S9	95	0	0	0
	W-S10	89	0	0	0
	W-S11	12	0	0	0
	AU-E1	0	0	0	30
	AU-E2	0	0	21	13
	AU-E4	0	0	30	53
	W-E2	0	0	14	100
	WC-P1	190	190	135	0
	WN-P1	114	114	65	21
	TOTAL	2,456	304	264	1,294
FRANK CREEK	AU-S1	0	0	0	51

		Rehab (ha)	Spoil (ha)	Pit-Ramp (ha)	Natural (ha)
	AU-S2	24	0	0	46
	FC-S2	152	0	0	0
	FC-S4	155	0	0	8
	FCW-S1	78	0	0	148
	L-S1	297	0	0	13
	L-S2	86	0	0	5
	L-S3	0	0	0	35
	W-S2	0	0	0	23
	W-S3	0	0	0	24
	W-S5	14	0	0	11
	FC-E1	0	0	0	10
	WU-P1	243	0	124	81
	TOTAL	1,049	0	124	454
TWO MILE GULLY	WU-S1	75	0	0	6
	TOTAL	75	0	0	6

Attachment C

Impact of mine on typical monthly
flows in downstream watercourses

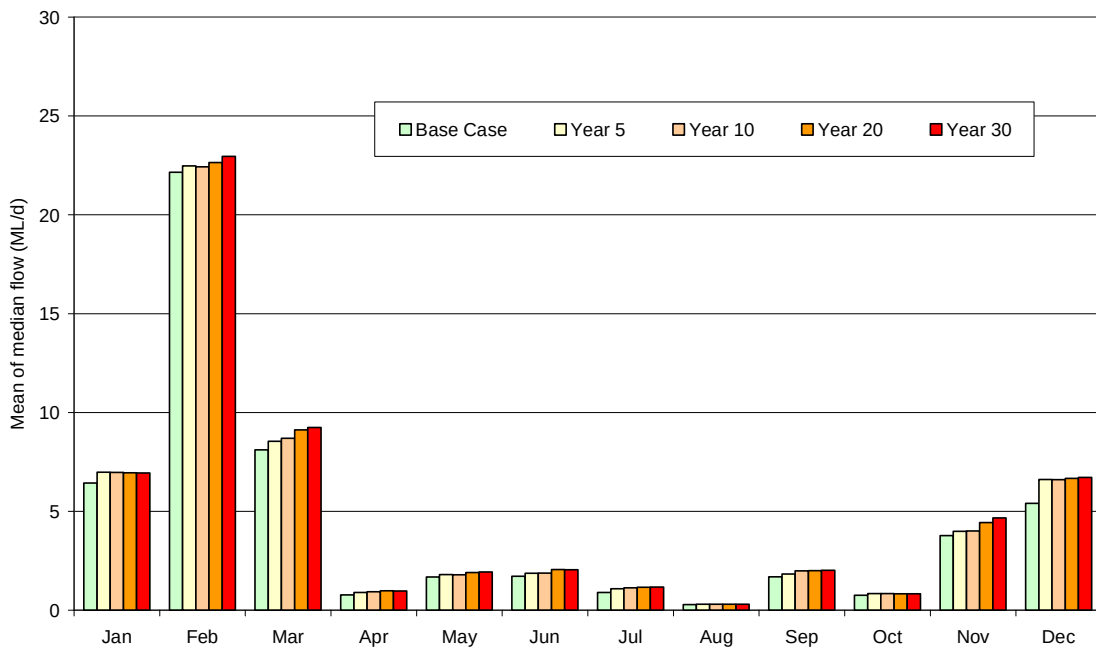


Figure C-1: Impact of mine on typical monthly flows - Juandah Creek

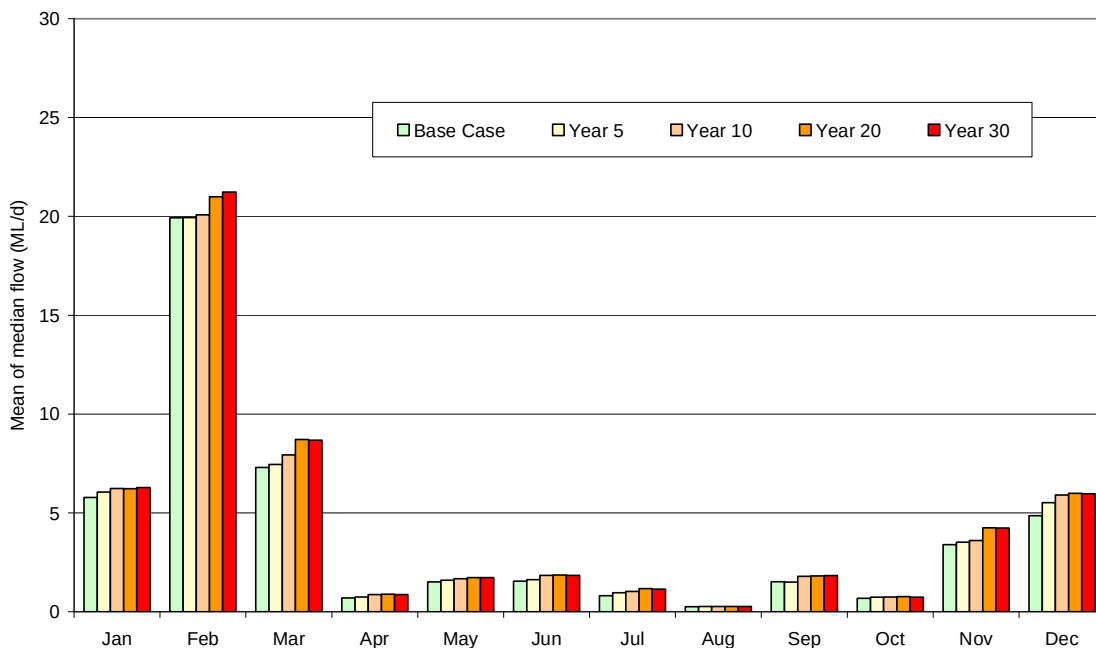


Figure C-2: Impact of mine on typical monthly flows - Woleebbee Creek

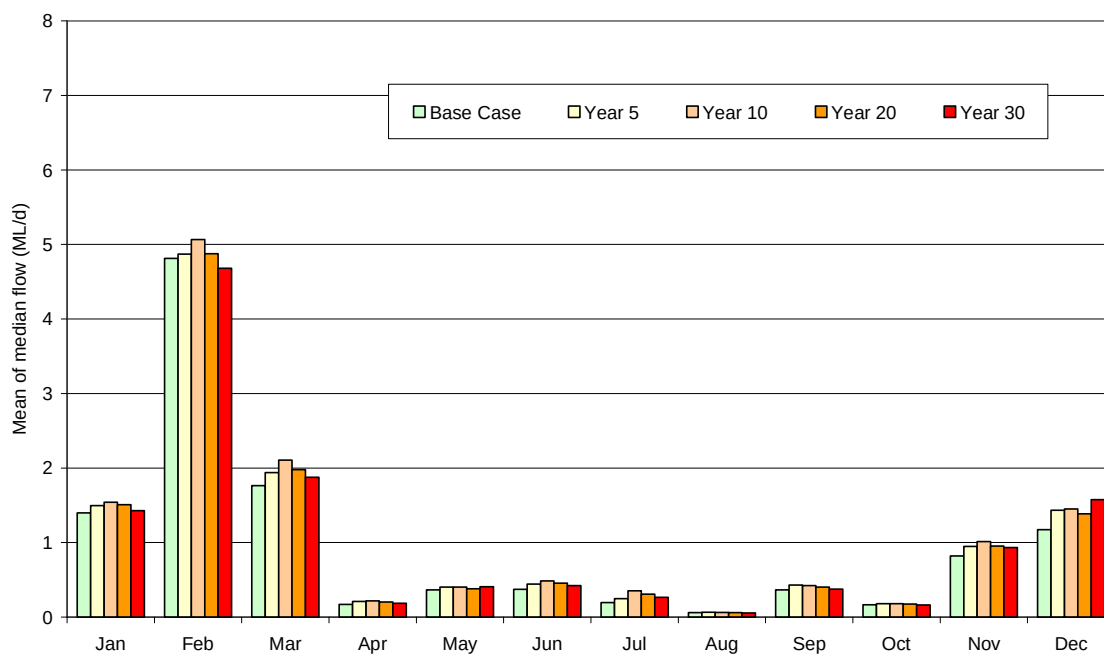


Figure C-3: Impact of mine on typical monthly flows - Mud Creek

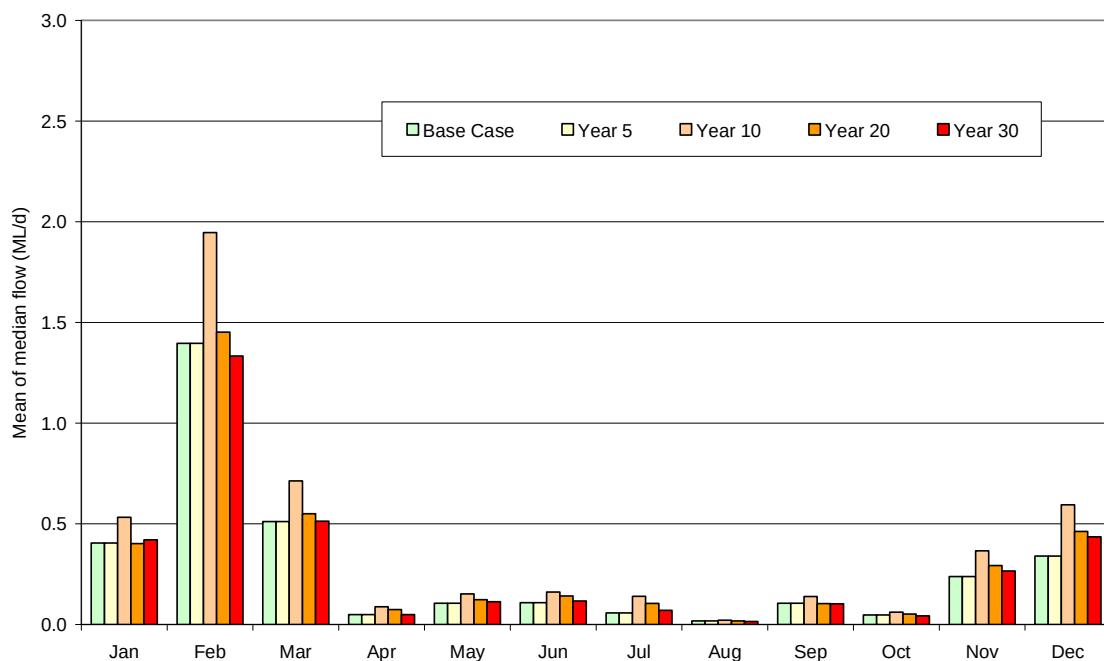


Figure C-4: Impact of mine on typical monthly flows - Spring Creek

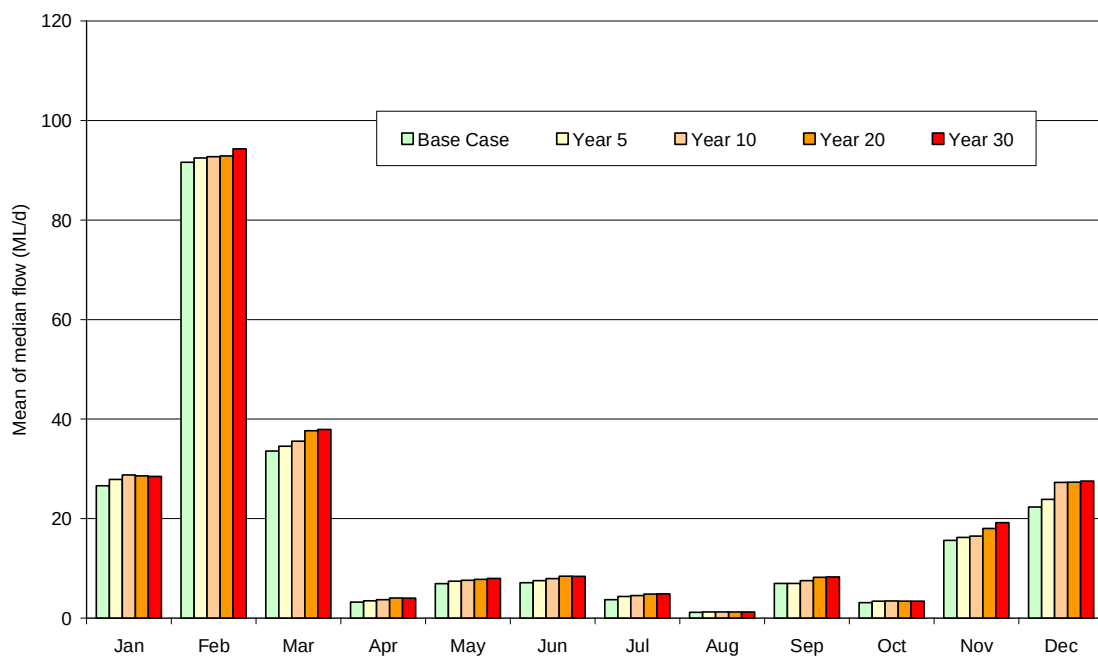


Figure C-5: Impact of mine on typical monthly flows - Horse Creek/Juandah Creek confluence

Attachment D

Impact of mine on number of low
flow spells and the mean duration of
flow spells

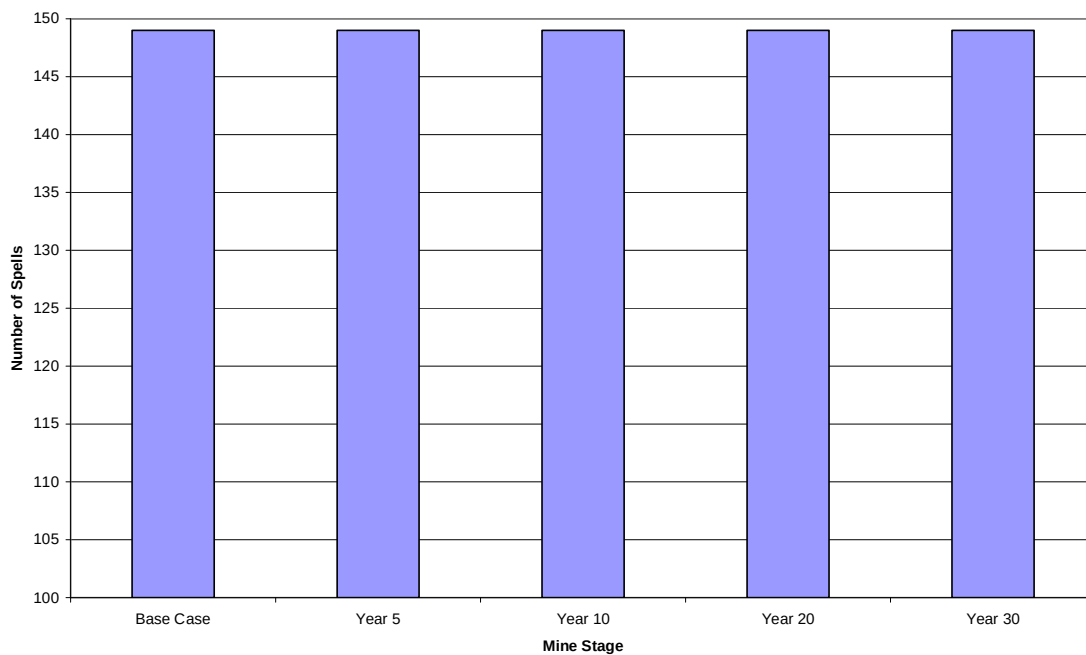


Figure D-1: Number of low flow spells - Juandah Creek (flow spells at least 31 days in duration)

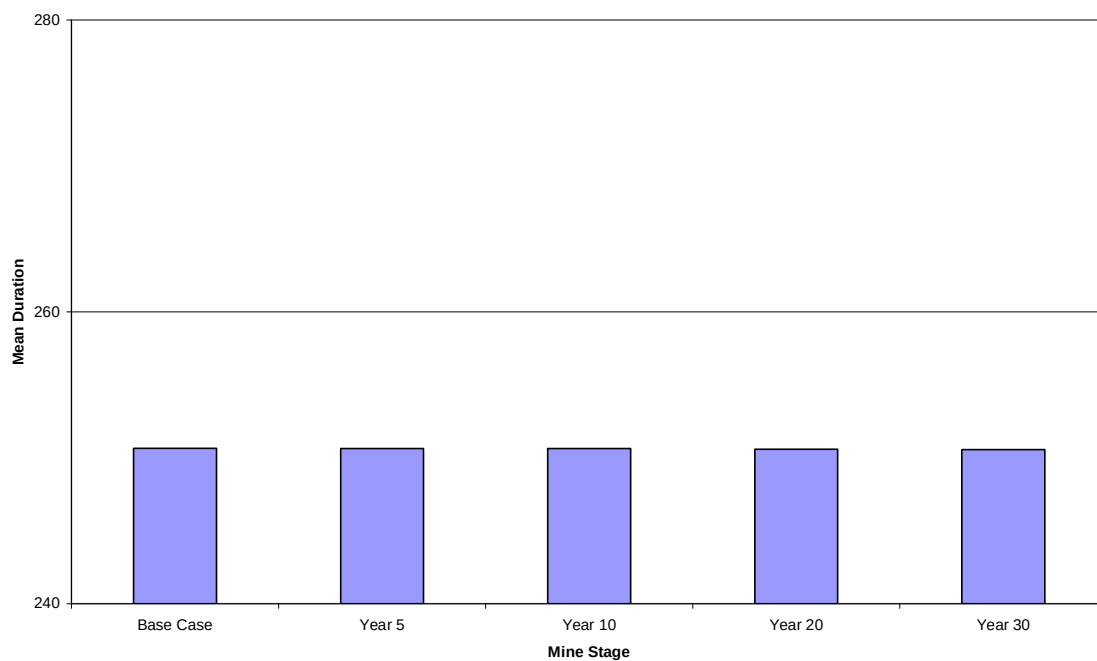


Figure D-2: Mean duration of low flow spells – Juandah Creek

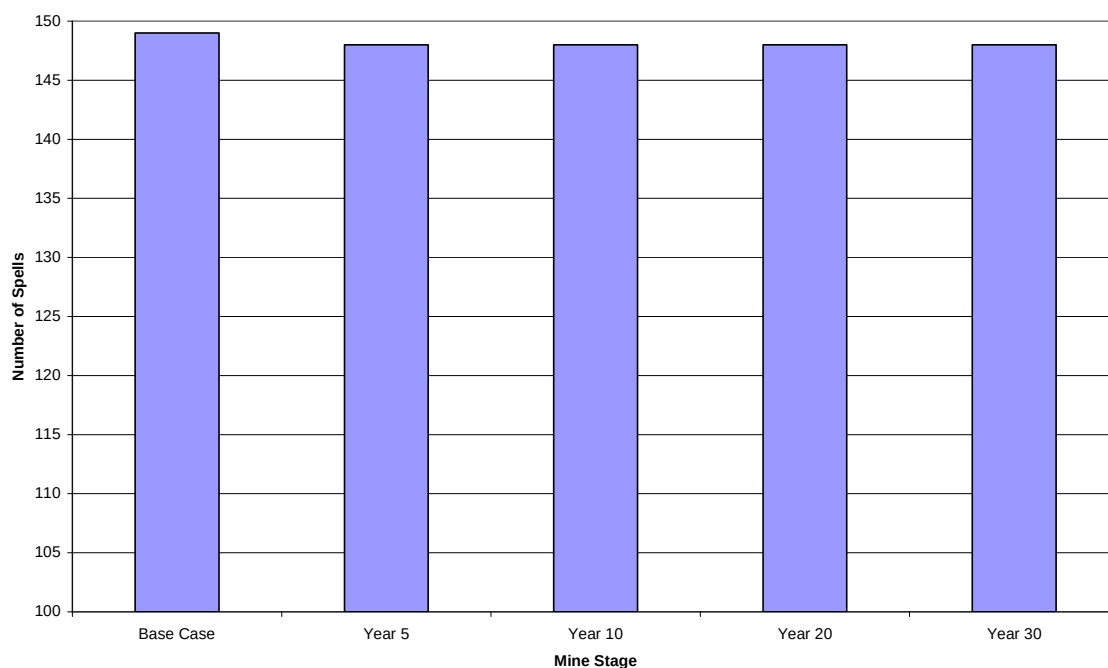


Figure D-3: Number of low flow spells - Woleebee Creek (flow spells at least 31 days in duration)

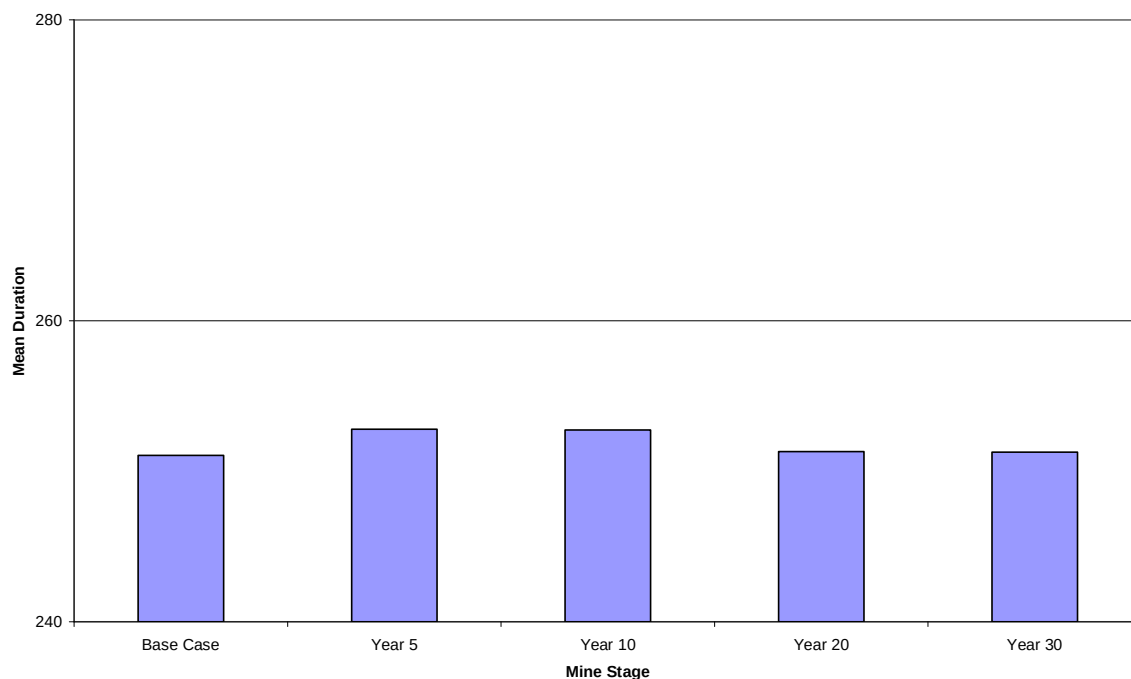


Figure D-4: Mean duration of low flow spells – Woleebee Creek

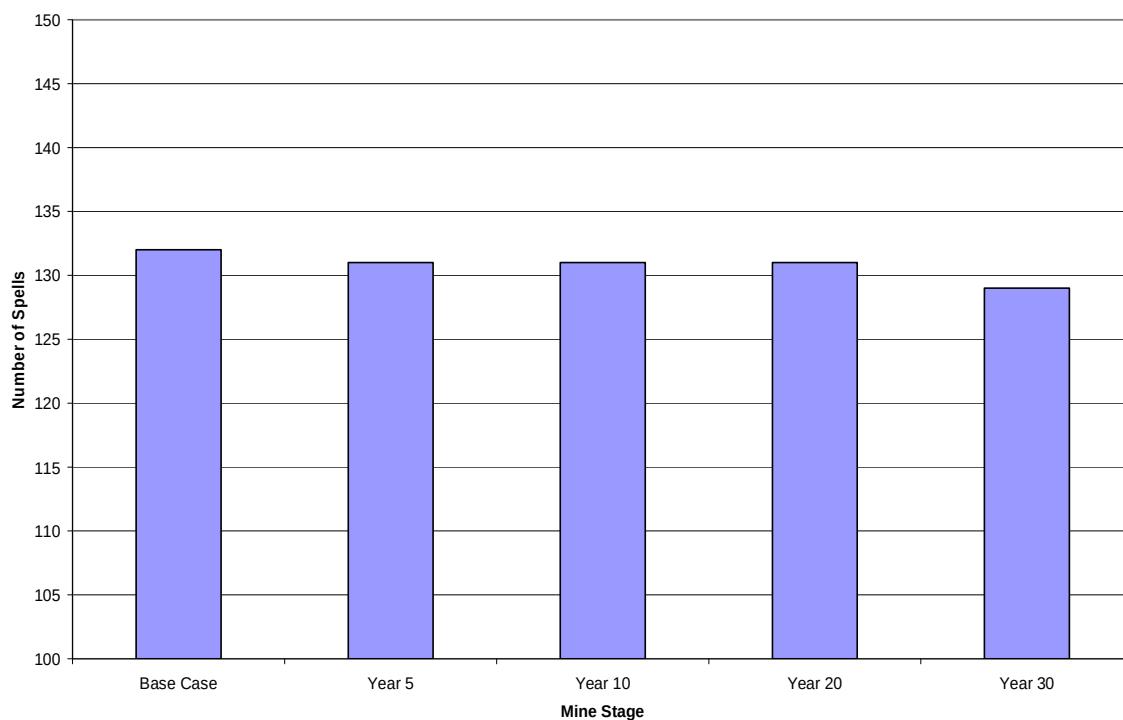


Figure D-5: Number of low flow spells - Mud Creek (flow spells at least 31 days in duration)

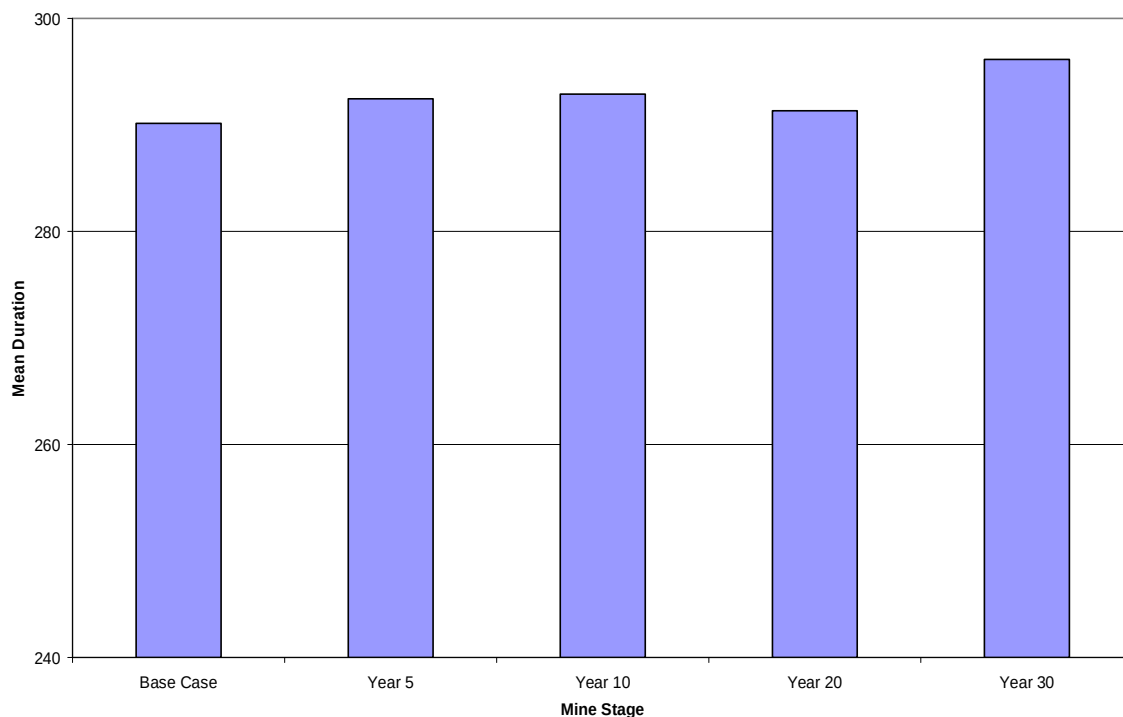


Figure D-6: Mean duration of low flow spells – Mud Creek

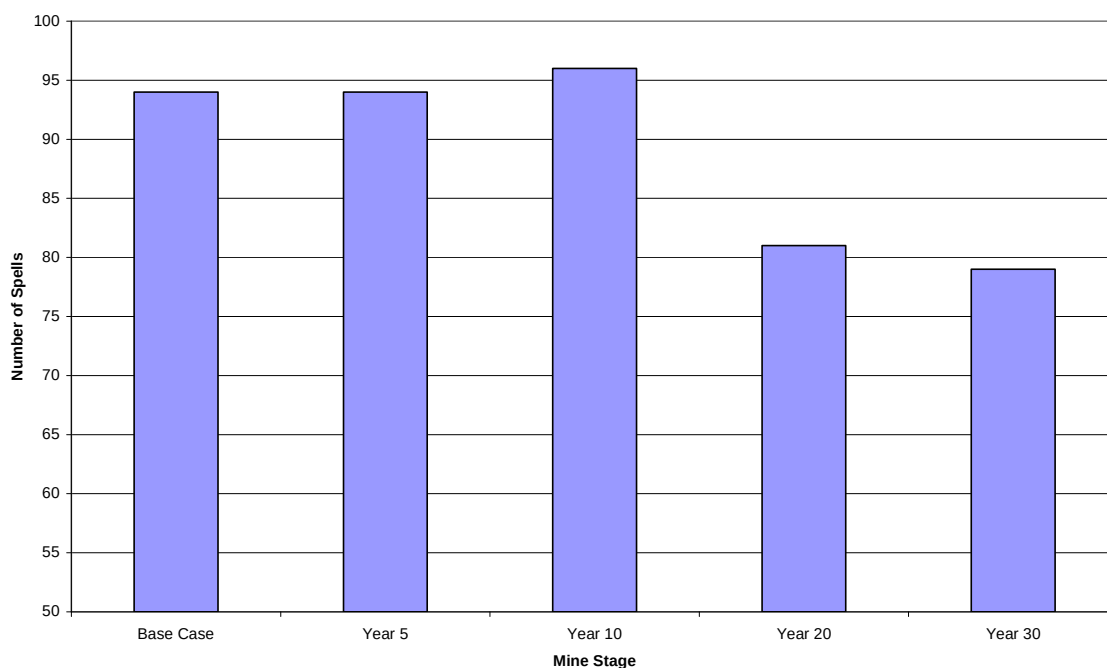


Figure D-7: Number of low flow spells - Spring Creek (flow spells at least 31 days in duration)

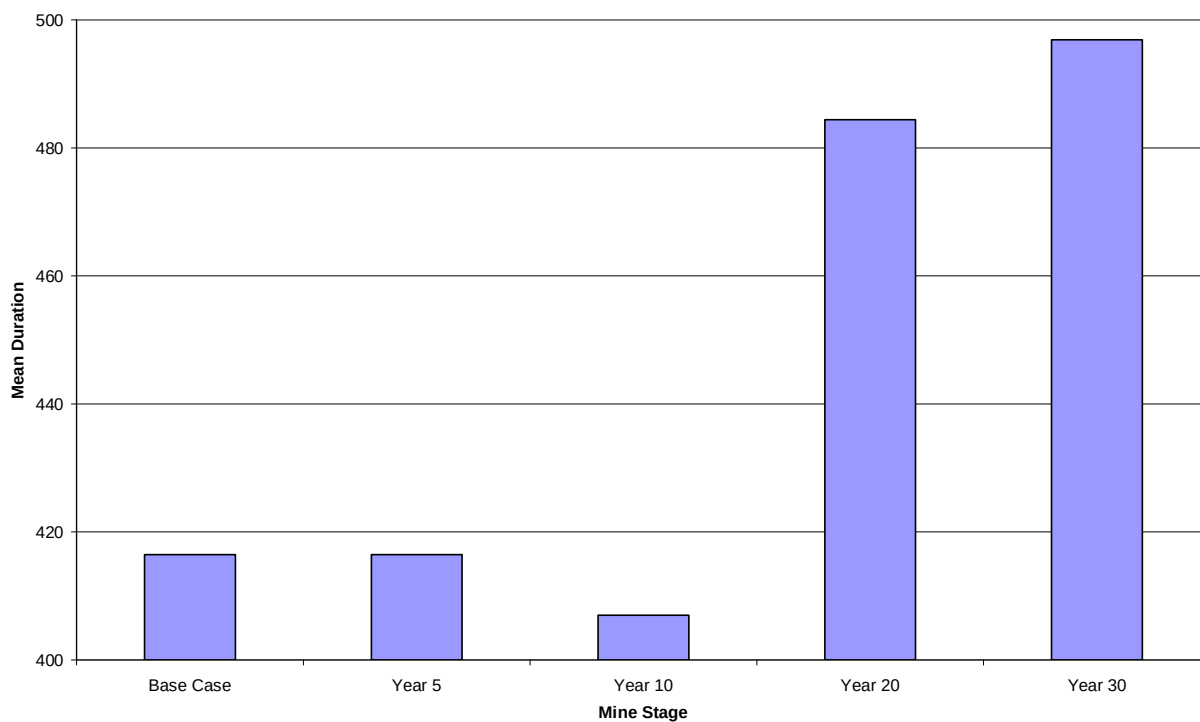


Figure D-8: Mean duration of low flow spells – Spring Creek

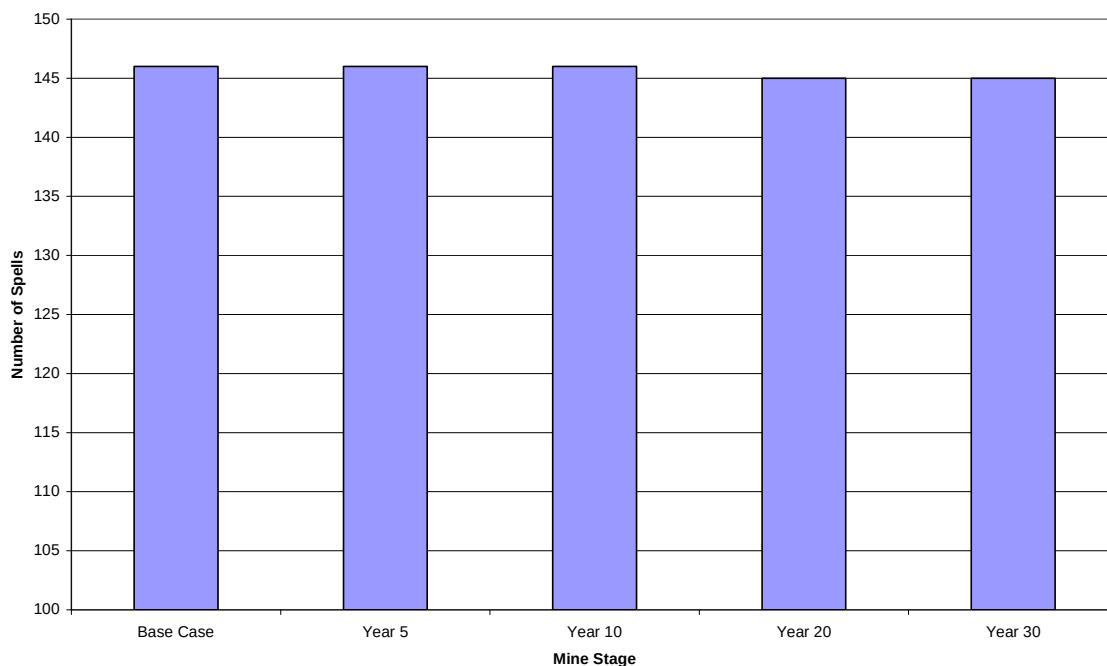


Figure D-9: Number of low flow spells - Horse Creek/Juandah Creek confluence (flow spells at least 31 days in duration)

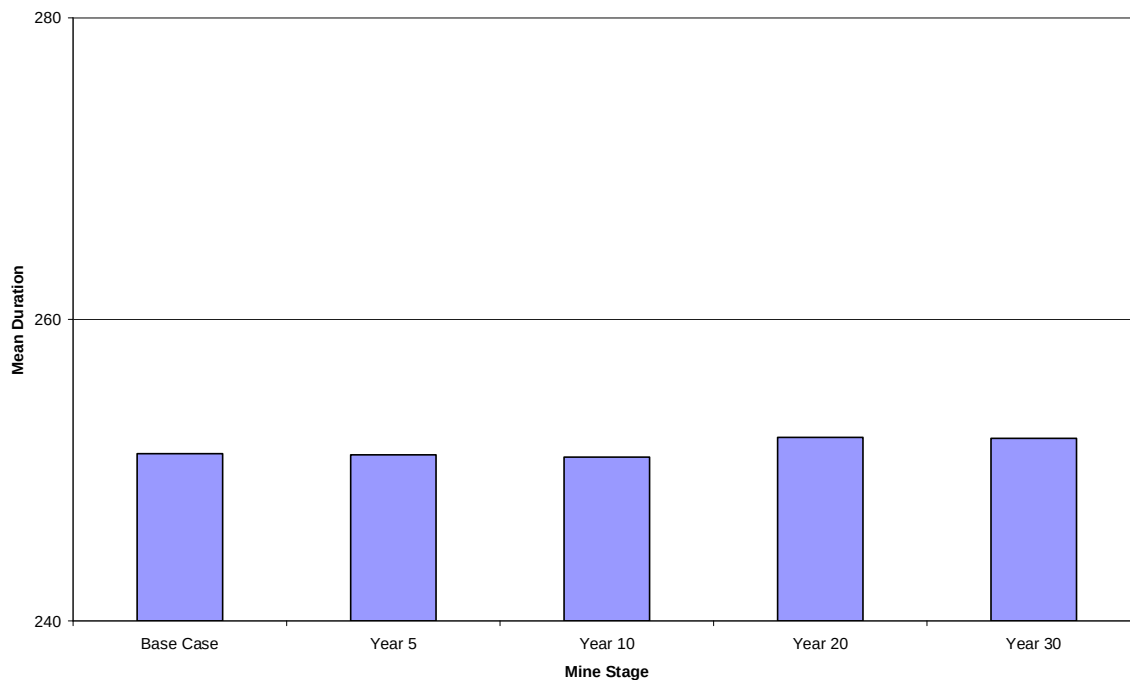


Figure D-10: Mean duration of low flow spells – Horse Creek/Juandah Creek confluence

Attachment D

Wandoan effluent reuse study

Wandoan Coal Project Wandoan Sewage Treatment Plant Effluent Disposal Study

July 2009

Wandoan Joint Venture



Document10



Parsons Brinckerhoff Australia Pty Limited
ABN 80 078 004 798

Level 4, Northbank Plaza
69 Ann Street
BRISBANE QLD 4000
GPO Box 2907
BRISBANE QLD 4001
Australia

Telephone +61 7 3854 6200
Facsimile +61 7 3854 6500
Email brisbane@pb.com.au

Certified to ISO 9001, ISO 14001, AS/NZS 4801

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Author: Susan Kizito,

Signed:

Reviewer: Phil Selmes, Mohand Amghar

Signed:

Approved by: Michael Batchelor

Signed:

Date: July 2009.....

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Appendices

Appendix A	Amalgamated permit, development application permit number ENDC00610406
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Executive summary

Treated effluent inflows from the upgraded Wandoan Wastewater Treatment Plant will increase as a result of the proposed Wandoan Project. Average dry weather effluent flows will peak during the construction phase, when the population served will increase from 440 to 2026, with corresponding effluent flows of 104 kL/d and 520 kL/d respectively. Operational flows from a population of 1256 people would peak at 322 kL/d.

This MEDLI modelling study addresses the management of this effluent by using irrigation as a disposal means in accordance with EPA's Operational Policy, which requires demonstration of at least 90% beneficial reuse of the treated reclaimed water.

The irrigation areas considered in this study are:

- Wandoan showground – we have assumed the entire showground (15.9 ha) will be irrigated whilst taking into account various legislative requirements. It should be noted that Council is yet to make a decision on increasing irrigation of the showground due to the extra maintenance costs that are associated with having all the facility watered all the time
- Wandoan Golf course – consideration had been given to irrigating 13 ha of the course with treated effluent. This modelling study has assumed the entire area will be available for irrigation whilst taking into account the legislative controls on recycled wastewater irrigation. It is understood that the WJV is yet to enter into an agreement with Wandoan Golf Club for irrigation of the golf course.

In this study, an average irrigation rate of 5.6 ML/ha/yr was adopted. Irrigation would be triggered when the soil moisture deficit fell below 40 mm.

Conclusions that can be drawn from the modelling study are:

- the proposed combined irrigation area of 28.9 ha at the showground and golf course would increase the percentage of effluent used while also reducing discharges to Juandah Creek
- however, the estimated existing storage volume of 4.3 ML is a limiting factor for managing overflows in wet weather. The resultant discharges reduce the percentage of effluent reused for irrigation purposes. As a consequence, the EPA Operational Policy of achieving at least 90% effluent reuse would not be realised with the existing 4.3 ML storage in either the construction or operational phase. Additional irrigation area would therefore be required to comply with the EPA Operational Policy compared to if the storage volume was increased
- a 20 ML wet weather storage would enable the EPA Operational Policy to be met in the operational phase if both the showground and golf course were irrigated
- in the operational phase of the mine, a 20 ML storage would manage effluent overflows within the currently licensed discharge limit of 220 kL/d. However in the construction phase, the effluent overflows would temporarily exceed the discharge limit
- overflows could also be reduced to below the existing licensed discharge limit of 220 kL/d for both the 4.3 ML and 20 ML storage if additional irrigation area was provided

1. Introduction

1.1 Background

The Wandoan Coal Project (the Project) comprises the development of thermal coal resources situated immediately west of the Wandoan township, located in Dalby Regional Council (DRC). The Project is located approximately 350 km northwest of Brisbane and 60 km south of Taroom. The coal reserves for this Project exist within three mining lease applications, Mine Lease Application (MLA) 50229, 50230 and 50231. The coal resources will be developed by open cut mining with related infrastructure. The Project covers an area of approximately 32,000 ha.

The mining of the coal will be undertaken using a combination of truck, shovel, dozer and dragline mining equipment. Coal will be mined at a rate of around 30 million tonnes per annum (Mtpa) run of mine (ROM) coal. The coal will be crushed, sized and washed before being transported by rail to ports in the Gladstone area.

The Project will be developed by the Wandoan Joint Venture (WJV). The joint venture partners are Xstrata Coal Queensland Pty Ltd (XCQ), ICRA RPW Pty Ltd and Sumisho Coal Australia Pty Ltd.

1.2 Scope

This report forms part of the supplementary technical studies supporting the Project Supplementary Environmental Impact Study (EIS). This study addresses issues relating to the disposal of treated sewage effluent from proposed upgrades to the Wandoan Sewage Treatment Plant (STP). Specifically, the following issues in relation to the proposed methods of reducing dry weather discharges to Juandah Creek are described in detail in this report:

- the existing legislative provisions for the disposal of effluent to land
- existing effluent disposal practices at Wandoan
- the expected changes to the quantity and quality of wastes generated as a result of the proposed plant upgrades
- options for the irrigation of treated effluent to land
- details of an assessment using the Model for Effluent Disposal using Land Irrigation (MEDLI) to assess sustainable irrigation rates, as per Amalgamated Permit, Development Application permit number ENDC00610406
- mitigation measures to manage of treated effluent discharge during wet weather

All sewage generated at the MIA, CHPP and accommodation village will be transferred back to the upgraded Wandoan Town STP by way of a dedicated series of pump stations and rising mains. All off-site infrastructures will be owned and maintained by the DRC.

The STP upgrade has been designed to treat sewage to a Class B recycling quality. During dry weather, effluent will be disposed of by way of irrigation locally and during wet weather effluent will be stored in additional storage to be constructed as part of the proposed upgrade. Excess wet weather flows will be discharged to the creek as per the existing license.

1.3 Objectives

The underpinning criteria for this study was the management of effluent from Wandoan STP by using irrigation as a mean of disposal, minimising discharge to Juandah Creek and meeting the EPA Operational Policy (Refer to Section 2.5). In achieving these criteria, the following were considered:

- ensuring the effluent disposal area is not degraded
- maximising effluent reuse
- minimising impacts to the shallow aquifer by using sustainable irrigation rates
- maintaining reliability of irrigation water supply.

The resulting objectives from these criteria are as follows:

- determination of a suitable irrigation rate
- maximisation of effluent reuse
- utilisation of available irrigable land
- compliance with relevant legislation.

1.4 Population and Effluent Flows

Information supplied by DRC shows that the current population of Wandoan is 440 people and is not expected to increase. The construction and operation of the mine will however cause a rise in the population. Onsite mine population will be higher during the construction phase and reduce during the operational phase.

During the construction phase the mine population is projected to be 1425 people. In the operational phase, there will be 880 people employed at the mine site, of which 130 will be administration staff and 750 will be workers on a two shift rotations; 375 workers on-site and 375 workers off-site. It is envisaged staff not working will live off site. This assessment has assumed an additional 25% non-working staff remaining on site. Table 1-1 shows the projected dry weather flows estimated for the Wandoan STP based on the projected flows.

Table 1-1: Current and projected population and wastewater flows

	Capita Construc.	Capita Ops 1	Capita Ops 2* ¹	Estimated ADWF Construc. (kL/day)	Estimated ADWF Ops 1* ² (kL/day)	Estimated ADWF Ops 2* ² (kL/day)
Existing town	440	440	440	113	113	113
Town growth	15	15	15	4	4	4
Support population	146	146	146	37	37	37
Mine village	1425	505	655	366	130	168
Total	2026	1106	1256	520	284	322

* ADWF – average dry weather flows

*¹ Includes 25% non-working staff remaining on site

*² Based on effluent production of 257 l/cap/day

The effluent production (for the combined mine and town population) of 257 l/cap/day was calculated from 520 kL/d wastewater flow which include a 15% contingency flow. This effluent production was applied to operational phase in calculation of the estimated average dry weather flows (ADWF).

1.5 Existing discharge licence

The current discharge license allows for the release of 220 kL/d into Juandah Creek. The Wandoan Infrastructure DRC reports states that 104 kL/d is treated at Wandoan STP. Some effluent is pumped to the earth dam at the Council owned showground for irrigation; however effluent is often discharged to Juandah Creek.

Table 1-2 shows the existing environmental discharge allowance for the Wandoan STP.

Table 1-2: Environmental discharge licence allowance

Parameter	Unit	Release limit	Limit type
Licensed effluent release volume	kL/day	220	Maximum
5 day BOD	mg/L	20	80 th percentile
Suspended solids	mg/L	30	80 th percentile
pH		6.0 – 8.5	Range
<i>E. coli</i>	organisms/100 mL	<1000	Maximum
Total dissolved solids		1000	Maximum
Dissolved oxygen	mg/L	2	Minimum

Treated effluent from Wandoan STP during the construction and operation of the mine will exceed the allowable discharge limit.

The current treatment process at Wandoan STP is not a nutrient removal process. Concentration of nitrogen and phosphorus are unknown. Water quality allowances appear to conform to Class C recycled water standards, as per the *Queensland Water Recycling Guidelines* (Wandoan DRC Infrastructure Report)

1.6 Irrigation practices

1.6.1 Current irrigation practices

Approximately 25% of the showground is currently irrigated (pers. comms, T Fagg, DRC, May 2009). Additional irrigation occurs at the polocrosse field, car park and racecourse. The irrigation rate, irrigation frequency and size of the preceding irrigation areas are not known.

1.6.2 Proposed irrigation area

The irrigation areas considered in this study are:

- showground – This modelling study has assumed the entire showground (15.9 ha) will be irrigated whilst taking into account the legislative requirements discussed in Chapter 2. It should be noted that Council is yet to make a decision on increasing irrigation of the showground due to the extra maintenance costs that are associated with having all the facility watered all the time (pers. comms, T Fagg, DRC, and May 2009)
- Wandoan Golf course – The study has examined irrigating the entire 13 ha of with treated effluent. This modelling study has assumed the entire area will be available for irrigation whilst taking into account the legislative requirements discussed in Chapter 2. It is understood that The WJV is yet to enter into an agreement with Wandoan Golf Club for irrigation of the golf course.

2. Relevant legislation

Legislation and guidelines, relevant to the use of effluent for land application is listed below.

- Amalgamated Permit, Development Application permit no. ENDC00610406
- Environmental Protection Act 1994 and subsequent Operational Policy
- Queensland Water Recycling Guidelines (2005)
- Queensland Plumbing and Wastewater Code – March 2006 (Department of Local Government, Sports and Recreation).

2.1 Amalgamated permit, development application permit number ENDC00610406

The sections relevant to this study (subject to the approval of a new development permit) are:

- schedule C – Water. Sub-section C2, C3 and C4
- schedule E – Land Application. Sub-section E1, E3, E4, and E13. Additional sections that will require implementation at the commencement of the irrigation scheme are sub-section E2, E5 – E12. An Irrigation Management Plan (IMP) will be required to meet sub-sections E10 – E12.

Refer to Appendix A for a full copy of the Amalgamated Permit.

2.2 Environmental protection Act 1994

The principal legislation for the use of recycled water in Queensland is the Environmental Protection Act 1994 and its subordinate legislation, the Environmental Protection Regulation 1998 and the Environmental Protection (Water) Policy 1997.

Under this legislation, if an operator of the STP commences the supply of recycled water to an external user or to a site separate from the STP, this may require an amendment to existing approval conditions. If the recycled water is supplied to user (Wandoan Golf Club, *added*), the Environmental Protection Agency (EPA) will require the supplier (Dalby Regional Council, *added*) to enter into a formal Recycled Water Agreement with the user. The Recycled Water Agreement commits the third party to only use the recycled water in a certain way for certain specified purposes (QWRG, 2005).

2.3 Queensland water recycling guidelines (2005)

The Queensland Water Recycling Guidelines are designed to provide guidance on water recycling that is appropriate to Queensland conditions and to provide a road map to other resources that can support water recycling.

Table 2-1 (as an extraction from QWRG (2005)) outlines the recommended class and monitoring required for irrigation purposes.

Table 2-1: Recycled water uses, recommended class and recommended monitoring

Recycled water use	Class	Recommended monitoring
Irrigating public open space and golf course - Above ground open space irrigation, uncontrolled access - Controlled access or subsurface irrigation	A C	<i>E.coli</i> weekly, turbidity ² continuous, disinfection ³ continuous, pH weekly <i>E.coli</i> weekly, SS monthly, disinfection ³ pH monthly
Industrial purpose - Closed system (low human contact) - Irrigation of “no public access” areas	C	<i>E.coli</i> weekly, disinfection ³ weekly, pH weekly

Source: Extract from Table 6-3 p 50. Queensland Water Recycling Guidelines (2005)

Notes: 2. Turbidity should be monitored at the stage in the production process for recycled water that is most relevant. In other words, if turbidity levels are important to the disinfection process, turbidity should be measured immediately before disinfection.

3. See comments on chlorine disinfection in Section 6.6.3 of the guidelines

QWRG stipulate that spray irrigation of Class B, C or D can be used where public access can be prevented during irrigation and long after the wetted surface has dried. Where subsurface irrigation is used, Class B or C may be used in uncontrolled access areas however it is essential that the irrigation does not lead to surface ponding of water. Additional treatment is required for Class C recycled water if used within recycled water pipes to prevent fouling, scaling, corrosion, foaming or biological growth.

2.4 Queensland plumbing and wastewater code

The Queensland Plumbing and Wastewater Code is a subordinate legislation to the Plumbing and Drainage Act 2002 which is administered by the Department of Local Government, Planning, Sport and Recreation.

The Act and Code regulate the design and operation of on-site sewerage facility used for the disposal of effluent by a common effluent drainage or collection from an onsite storage tank (QWRG (2005)

Relevant to this study, the code requires that a distance of 30 m is maintained between the land application area and a watercourse which includes open stormwater drainage channel, dam or bore. It also requires a minimum unsaturated vertical soil depth of 0.6 m is maintained between the land application area and permanent water table.

Additional horizontal separation distances are required as follows:

- 2 m for property boundaries, pedestrian paths and walkways
- 6 m for water edge of a swimming pool
- 10 m for dwellings, recreation area.

2.5 EPA operational policy

The EPA Operational Policy approval of sewage treatment plants including options for use of reclaimed water (EPA 2005a) describes the approach EPA takes to approve new, upgraded or augmented sewage treatment plants that have a peak design capacity to treat sewage from 1000 EP or more. The policy been developed in consultation with the Great Barrier Reef Marine Park Authority and is used in the evaluating and setting of release standards for WWTPs in Queensland. The policy focuses on environmental management decisions that will achieve the best net environmental outcome while providing flexibility to the operator.

The goal is to achieve 100 per cent beneficial reuse (based on average dry weather flow (ADWF) of treated water reclaimed form STPs. STPs not achieving 90 percent beneficial reuse will be required to investigate options for further effluent reuse. The policy also indicates that any release of contaminants, including treated water from the WWTP must be managed to minimise environmental harm. The nutrient release objectives should be mass based unless concentration limits are more beneficial to the local receiving environment benefits.

3. MEDLI analysis

This section details the methodology that was used to undertake the assessment for effluent disposal via land application using the Model for Effluent Disposal using Land Irrigation (MEDLI). MEDLI is an interactive computer model that is used for designing and analysing effluent disposal systems. The model was developed jointly by Cooperative Research Centre (CRC) for Waste Management and Pollution Control, the Queensland Department of Natural Resources and Queensland Department of Primary Industries. MEDLI replicates the effluent from a wide range of industries such as feedlots, piggeries, dairy and municipal sewage treatment plants including user defined waste streams. The model takes into account climatic conditions in the analysis of effluent disposal (by irrigation), storage capacity, the fate of nitrogen, phosphorus and soluble salts and predicts the reuse by irrigators.

3.1 Methodology

The following methodology was adopted for the assessment:

- undertake lab analysis to identify soil characteristic of the proposed irrigation areas
- determination of a suitable irrigation rate
- determining optimum storage capacity
- maximisation of effluent reuse by utilising available irrigation area and storage capacity
- determining excess effluent volumes.

3.2 MEDLI assessment

3.2.1 Input parameters and assumptions

The following input parameters were used in the modelling study:

- population – Population data was sourced from information Wandoan DRC Infrastructure Report
- storage – The storage volume was assumed to be 4.3 ML. This was based on information (supplied by pers. comms, T Fagg, DRC, May 2009) that the surface area of the dam was estimated to be 1000 m² and the likely depth of the dam was 3 m and not greater than 5 m. A combined storage of 20 ML (including the existing 4.3 ML storage) has been proposed in the study and it is assumed this maximum storage capacity that will be constructed
- irrigation area – Additional irrigation areas (irrigation demands) of polocrosse field, car park, racecourse straight were not modelled as the combined area is estimated at 2 acres and excess effluent would be sufficient to meet this additional demand

- climate data was obtained from Department of Natural Resources and Water using SILO weather station located at 26.12 S and 149.95 E. The data ranged from 1980 – 2008. The average rainfall and pan evaporation data is shown in Figure 1
- soil type – Gray clayey soil parameters were adapted where the required MEDLI parameters could not be determined from the lab analysis
- crop type – Tropical pasture was the default crop chosen for both the golf course and showground crop type
- aquifer thickness – MEDLI default value of 10 m was assumed for the shallow aquifer
- nutrient concentrations – Nitrogen (N) and phosphorus (P) for the modelling study have been assumed at 14 mg/L and 4 mg/L. These values were assumed based on typical values obtain from treatment plants that did not have a nutrient removal process.

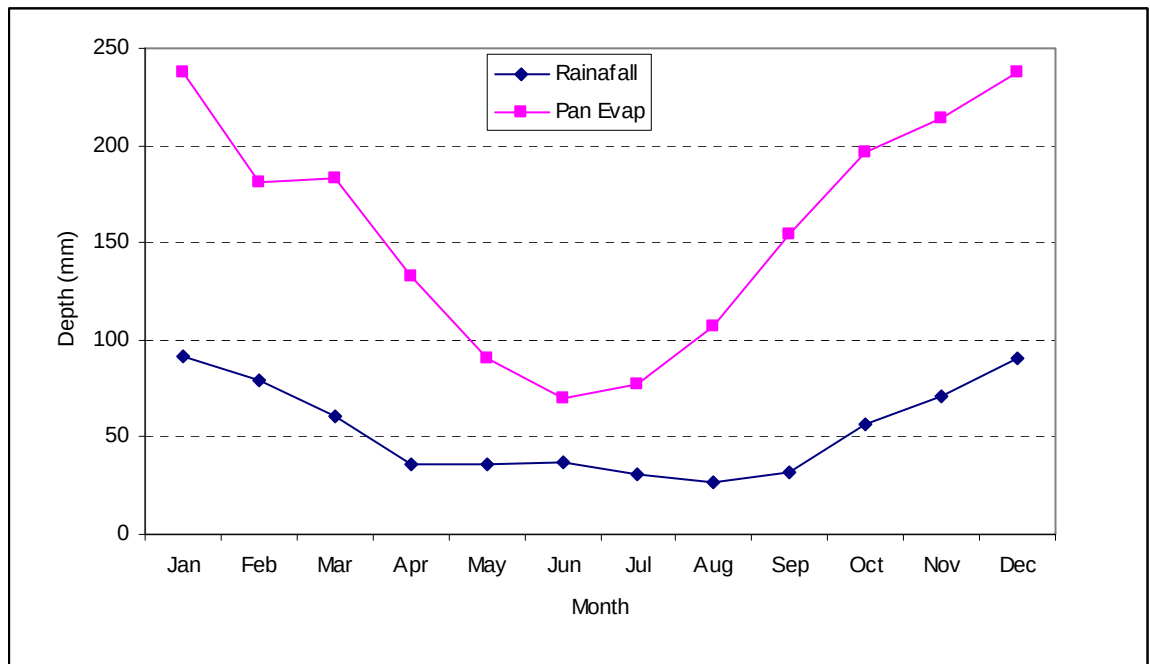


Figure 3-1: Average climate data for Wandoan Town

Figure 3-1 shows the mean monthly evaporation rate exceeds the mean monthly rainfall throughout the year providing opportunities for irrigation.

Additional assumptions are:

- the shallow aquifer is at a sufficient depth that would not cause pollution of movement of the irrigation water in the soil.

A base case model with the above described input parameters was developed and used to determine a sustainable irrigation rate for the modelling exercise.

3.3 Modelled scenarios

The modelling accounted for the entire irrigation area however this is dependent on the property boundary and nearby watercourse or channels (see Section 2.4).

- scenario 1 – Construction Phase. The construction phase will have a total of 2026 people which includes the town population with a corresponding flow of 520 kL/d (ADWF)
- scenario 2 – Operation Phase. In the operation phase it is currently estimated that the mine working (on-shift) population and town population will be 1106 with a corresponding flow of 284 kL/d (ADWF)
- scenario 3 – Operation Phase. This scenario has been developed to account for rotational shift workers who may potentially remain at the mine site when not working. An additional 25% has been estimated for these workers resulting a projected population of 1256 people with a corresponding flow of 322 kL/d (ADWF)

For each of the scenarios, three models were developed with the following known irrigation areas:

- showground – 15.9 ha
- golf course – 13 ha
- combined showground and golf course area – 28.9 ha.

3.4 Sustainable Irrigation Rate

It is reported in the DRC Wandoan Infrastructure Report the lower limit and upper limit irrigation rate is 3 ML/ha/yr and 6 ML/ha/yr for the showground and golf course. A sensitivity analysis was undertaken to determine the optimum irrigation rate for the golf course and showground. The analysis included developing two base cases with the input parameters described in Section 3.2. Each base case was used to determine a lower and upper limit irrigation rate. The following irrigation schedules were tested for validation:

- irrigation trigger schedule of 20 mm, 30 mm, 40 mm and 50 mm soil water deficit with and application rate of 10 mm/d
- irrigation trigger schedule of 20 mm, 30 mm, 40 mm and 50 mm soil water deficit at 0 mm beyond the drained upper limit
- irrigation trigger schedule of 20 mm, 30 mm, 40 mm and 50 mm soil water deficit with periods of no irrigation,

From the results, the soil water deficit of 40 mm soil water deficit was established as the trigger schedule for irrigation to occur. This trigger level is supported in literature for crop types in the study area (Anon.1991, Stone and Nofziger, 1993, Tsegaye et.al., 1993 and Yates, 1967). A lower limit average irrigation rate of 3.5 ML/ha/yr was determined from the modelling and no impacts of phosphorus, nitrate and salinity concentrations in groundwater were noted from the results. The second base model was used to determine an upper irrigation rate. The modelling results showed that an average irrigation rate of 5.6 ML/ha/yr could be used to sustainable supply to the golf course and showground. The adopted irrigation rate for the modelling study is the upper limit irrigation rate of 5.6 ML/ha/yr.

3.5 Optimization of effluent irrigation using storage capacity

To optimise effluent reuse, adequate storage is required to accommodate inflows, outflows and minimize overtopping. The storage area was modelled as two options using the existing dam at the showground and the nominated 20 ML storage (dam).

3.5.1 Option 1 – Existing storage

The existing Council owned 4.3 ML storage (earthen dam) was used in this option for each of the scenarios outlined in Section 3.3. The results are presented in Figure 3-2 which shows the relationship between the irrigation area and percentage of effluent reuse for all the scenarios modelled. Refer to Appendix B for the tabulated results.

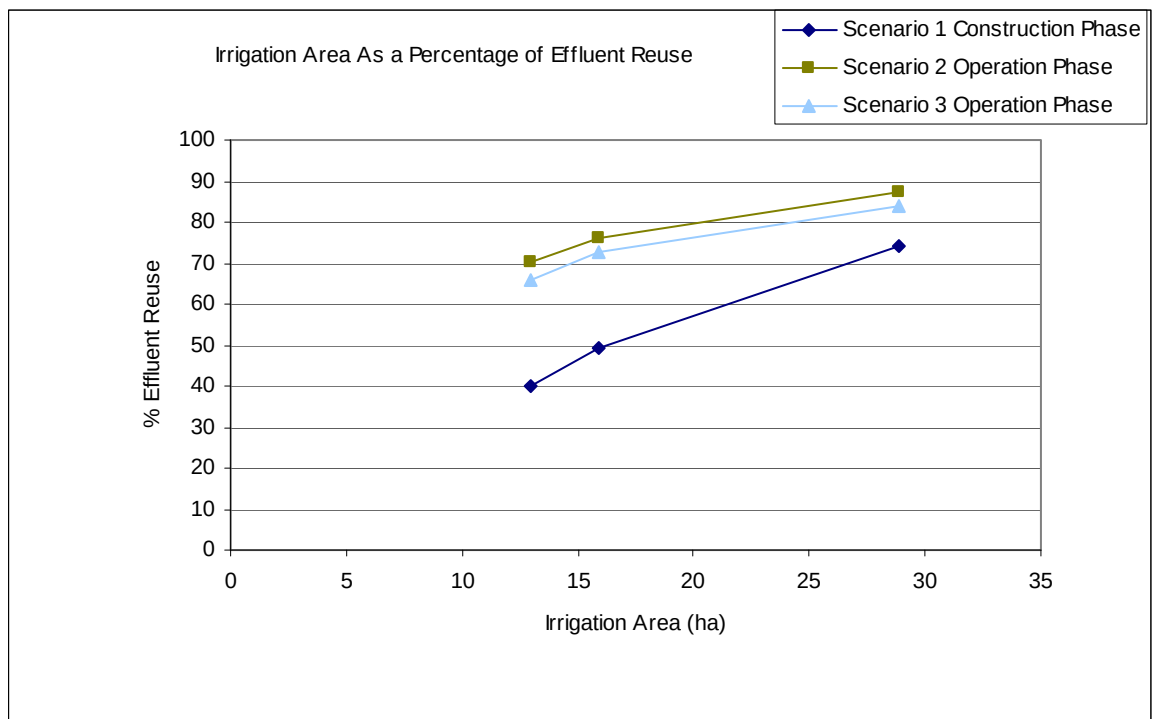


Figure 3-2: Irrigation area as a percentage of effluent reuse using 4.3 ML storage

The results show the highest percentage of effluent reuse occurs in Scenario 2 for all irrigation areas modelled while the lowest percentage of effluent reuse occurs in Scenario 1.

The results also show the EPA Operational Policy requirements of 90% effluent reuse will not be met for all scenarios modelled.

3.5.2 Option 2 – Construction of a new storage

A proposed combined storage of 20 ML (existing 4.3 ML storage and additional new storage) was modelled for this option. Figure 3.3 shows the relationship between the effluent reuse and irrigation area using the 20 ML storage for all the scenarios.

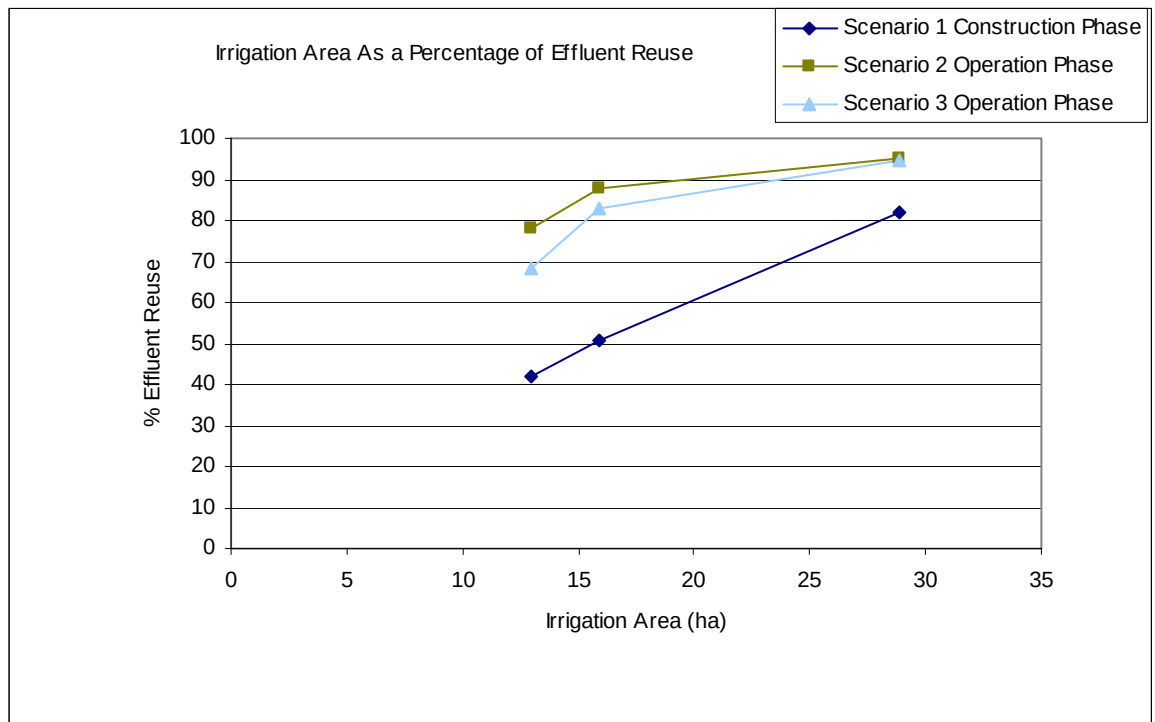


Figure 3-3: Irrigation area as a percentage of effluent reuse using the proposed 20ML storage

The results show that 95% and 94% effluent reuse is achieved for Scenario 2 and 3 respectively meeting the EPA Operational Policy requirements of 90% effluent reuse.

3.6 Management of excess effluent

Excess effluent is the effluent that cannot be used for irrigation due to lack of available irrigable land and storage capacity. Figures 3.4 – 3.5 show the effluent overflows that would result for the modelled options above. Refer to Appendix B for the tabulated results.

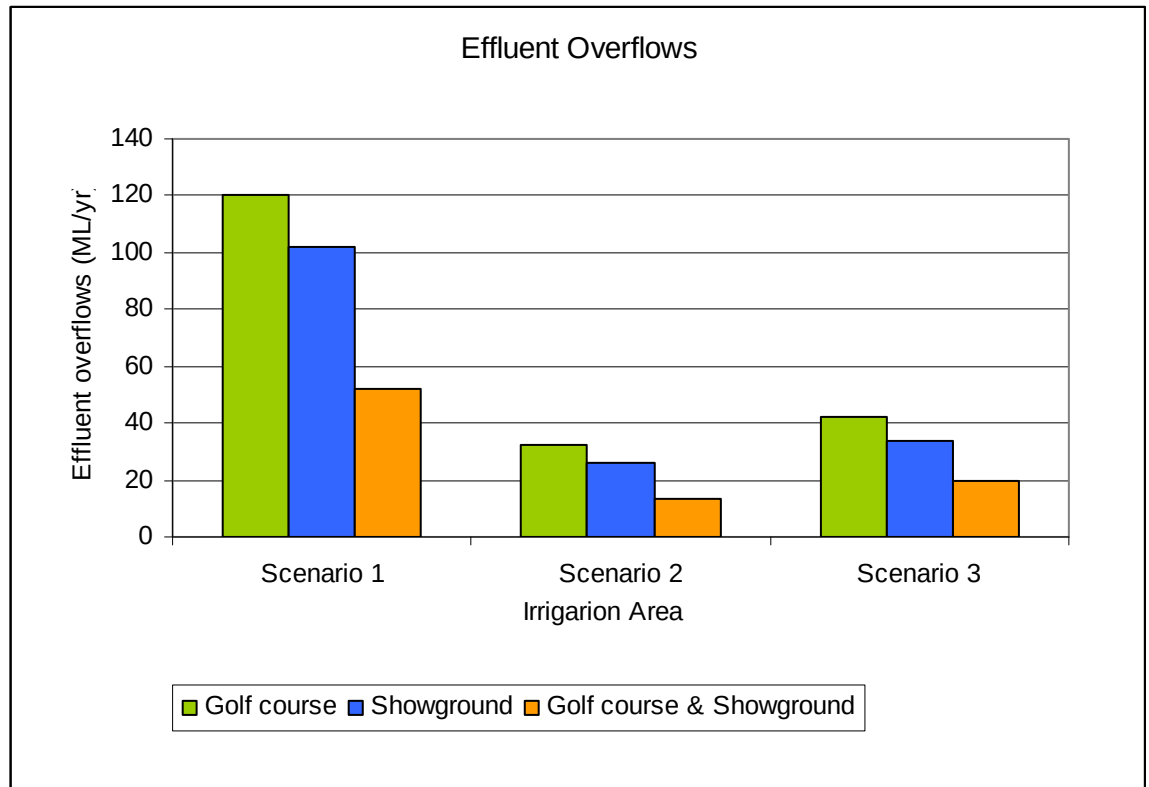


Figure 3-4: Overflows using 4.3 ML storage

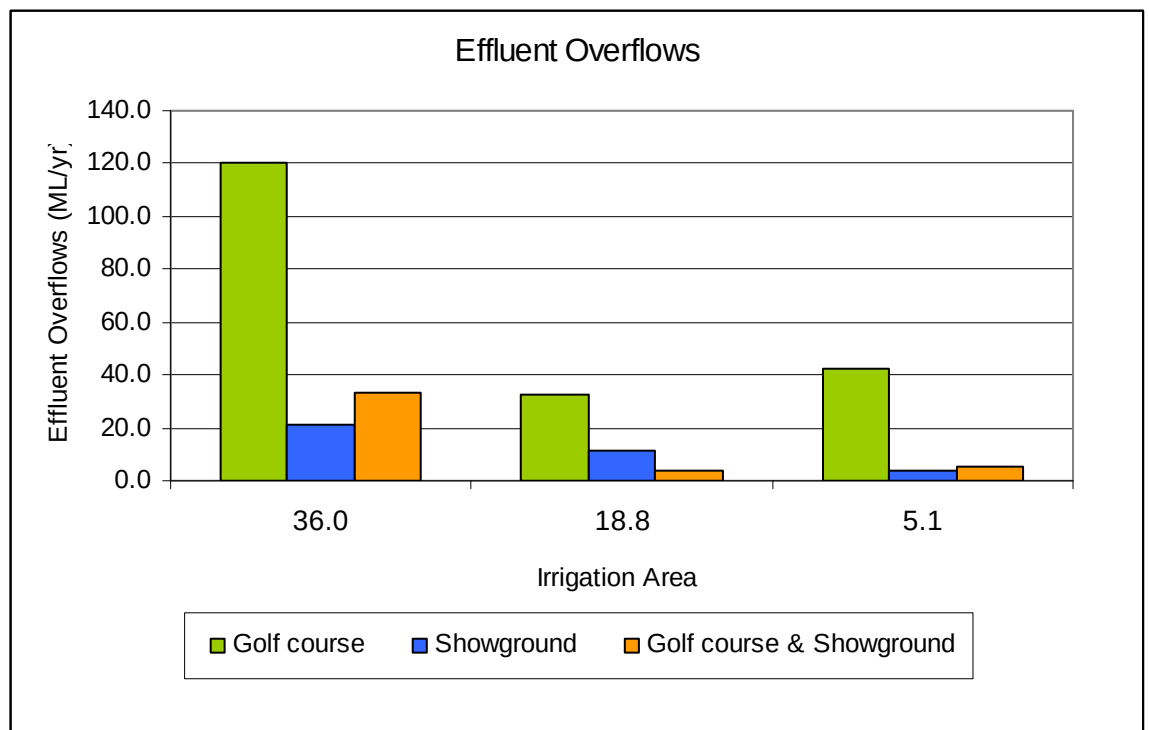


Figure 3-5: Overflows using 20 ML storage

The above results shows the highest effluent overflows will be experienced during the construction phase for both options modelled.

The results also show that across the three scenarios in both options, effluent overflows are highest when irrigation occurs on the golf course only and lowest when irrigation occurs on both the showground and golf course. The results indicate that a large irrigation area is required to limit overflows and achieve a highest percentage of effluent reuse.

3.6.1.1 Reduction of excess effluent

Modelling was undertaken to determine the additional irrigation area required to reduce the overflows from the storage. Table 3-1 – 3-2 show the additional area that would be required to further reduce overflows from the storage.

Table 3-1: Additional irrigation area using 4.3 ML storage

Scenario	Additional Irrigation Area (ha)	Effluent reuse (%)	Overflows (ML/a)
Scenario 1	116.1 – 132	88.1	23.6
Scenario 2	51.1 – 67	91	9.5
Scenario 3	66.1 – 82	90.6	11.4

Table 3-2: Additional irrigation area using 20 ML storage

Scenario	Additional Irrigation Area (ha)	Effluent reuse (%)	Overflows (ML/a)
Scenario 1	66.1 - 82	95.5	7.5
Scenario 2	46.1 - 62	97.3	1.3
Scenario 3	56.1 - 72	97.2	1.9

The results show that when using the proposed combined 20 ML storage, following are observed:

- less irrigation area is required
- greater that 90% effluent reuse is achieved for all modelled scenarios
- less overflows will result.

It should be noted that the additional irrigation area will result in a higher irrigation (water) demand. An average irrigation rate of 1.5 ML/ha/yr was observed from the modelling results. Large irrigation areas and higher effluent overflows result from using the 4.3 ML storage.

3.6.2 Water and nutrient balance

The tabulated water and nutrient balance for each scenario from the analysis in Section 3.5 can be found in MEDLI results in Appendix B. The results indicate that the rate of irrigation applied to the soil does not degrade the disposal area. Initial soil organic nitrogen and initial

soil inorganic nitrogen is greater than the final soil organic nitrogen and final soil inorganic nitrogen which indicates the irrigation rate applied does not increase the initial nutrient content of the soil.

3.7 Pre-treatment requirements

Water quality requirements for irrigation described in Section 2.3 will be dependent on the irrigation method to be adopted. The current effluent quality from Wandoan STP appears to be Class C (see Section 1.5). The modelling study has assumed surface irrigation will be adopted and therefore Class C water quality parameters will need to be maintained. Testing and monitoring of the effluent from the STP would be required to confirm if the water quality meets these requirements and subsequently improve treatment process if required.

4. Discussion

The increase in population in Wandoan due to the proposed construction of the 32 million ton per annum coal mine adjacent to Wandoan, will result in a rise in population and subsequently increase in wastewater and effluent flows. This MEDLI modelling study addresses the management of the effluent by using irrigation as a disposal means and determining the volume of effluent not utilised for irrigation purposes. The higher wastewater flows to Wandoan STP will require an upgrade to the existing plant. The EPA's Operational Policy requires demonstration of at least 90% beneficial reuse of the treated reclaimed water.

The proposed irrigation areas in this study are the showgrounds (15.9 ha), golf course (13 ha) and combined showground and golf course (28.9 ha) and these were modelled for 3 scenarios (see Section 3.3). The study shows that the highest inflows or volume of effluent available for irrigation will occur during the construction phase. In this phase, the projected population will result in increase the town population by from 440 people to 2026 people with corresponding flows of 104 kL/d and 520 kL/d. In this study, the irrigation rates were represented using 40 mm SWD which achieves on average irrigation rate of 5.6 ML/ha/yr. Irrigation would be triggered when the soil moisture deficit fell below 40 mm.

Modelling of the existing 4.3 ML storage in Scenario 1 2, 3 showed that the percentage of effluent reuse when irrigating the golf course and showground individually and irrigation on the both areas ranged from 40.2%, to 74%, 70.1%, to 87.4% and 65.9 to 83.8% respectively. Modelling of the existing 20ML storage for Scenario 1 2, 3 showed that the percentage of effluent reuse when irrigating the golf course and showground individually and irrigation on the both areas ranged from 41.8% to 82.2%, 77.9% to 95.2% and 68.4% to 94.4% respectively. A higher percentage of effluent reuse and lower volume of effluent overflows was typical of the 20 ML storage in comparison to the 4.3ML storage.

In the management of effluent overflows, it was observed from the results the higher effluent overflows resulted with the 4.3 ML storage in comparison to the 20 ML storage. The current discharge limit of 220 kL/d will be exceeded during the construction phase when irrigating the showground and golf course individually using the 4.3 ML storage and 20 ML storage however when additional irrigation area was modelled to reduce the overflows, the overflows in the operational phase were below the discharge limit for both storages. A higher percentage effluent reuse was also observed. The results indicated that with the combined proposed 20 ML storage, less additional irrigation area would be required to managed the effluent overflows in comparison to the using the existing 4.3 ML storage. Refer to Section 3.6 for the tabulated results

The water quality required for irrigation was determined from the Queensland Recycled Water Quality Guidelines (2005). Class C water quality parameters are the minimum requirements that would need to be met for irrigation on the golf course and showground. Additional requirements would be controlled access during irrigation use as per the legislative requirements outlined in Section 2.4.

5. Conclusions

Conclusions that can be drawn from the modelling study are:

- The proposed combined irrigation area of 28.9 ha at the showground and golf course would increase the percentage of effluent used while also reducing discharges to Juandah Creek.
- However, the estimated existing storage volume of 4.3 ML is a limiting factor for managing overflows in wet weather. The resultant discharges reduce the percentage of effluent reused for irrigation purposes. As a consequence, the EPA Operational Policy of achieving at least 90% effluent reuse would not be realised with the existing 4.3 ML storage in either the construction or operational phase. Additional irrigation area would therefore be required to comply with the EPA Operational Policy compared to if the storage volume was increased.
- A 20 ML wet weather storage would enable the EPA Operational Policy to be met in the operational phase if both the showground and golf course were irrigated.
- In the operational phase of the mine, a 20 ML storage would manage effluent overflows within the currently licensed discharge limit of 220 kL/d. However in the construction phase, the effluent overflows would temporarily exceed the discharge limit.
- Overflows could also be reduced to below the existing licensed discharge limit of 220 kL/d for both the 4.3 ML and 20 ML storage if additional irrigation area was provided

6. Recommendations

The following recommendations have been drawn from the modelling study:

- it is recommended that The WJV should endeavour to enter into an agreement with Wandoan Golf Club and Dalby Regional Council to irrigate effluent from Wandoan STP over the entire showground (15.9 ha) and golf course (13 ha)
- it is proposed that an additional 15.7 ML storage is constructed to manage excess effluent in the construction (with combined irrigation on the showground and golf course) phase and construction phase.

References

Dalby Regional Council Wandoan Infrastructure Report

Queensland Water Recycling Guidelines 2005.

Appendix A

Amalgamated permit, development
application permit number
ENDC00610406

Appendix B

A MEDLI Results