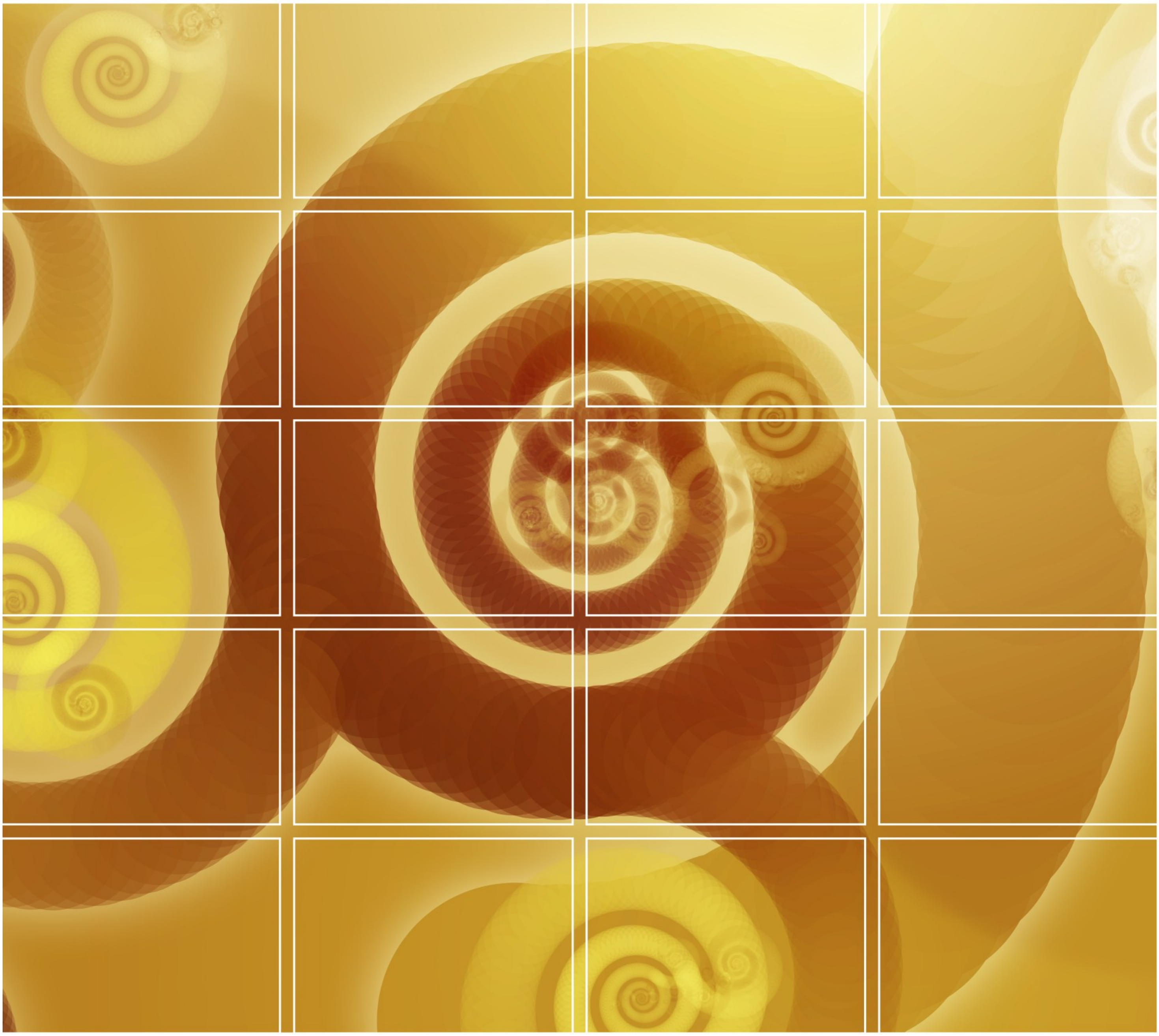

Ammonium Nitrate Storage and Distribution Facility

Environmental Impact Statement —Final Volume III

Crawfords Freightlines

October 2012

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ANNEX LIST

Annex E

Water/Flooding Impact Assessment

A part of BMT in Energy and Environment

AN Storage and Distribution Facility, Sandgate, NSW - Stormwater, Flooding and Receiving Water Quality Assessments

R.N2303.001.02.docx

October 2012

AN Storage and Distribution Facility, Sandgate, NSW - Stormwater, Flooding and Receiving Water Quality Assessments

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Title :	AN Storage and Distribution Facility, Sandgate, NSW - Stormwater, Flooding and Receiving Water Quality Assessments
Authors :	Michael Barry, Jane Macarthur, Philip Haines, Mark Wainwright
Synopsis :	*This report outlines an assessment of the potential impacts of a proposed ammonium nitrate storage facility at Sandgate, NSW on stormwater, flooding and receiving water quality.*

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1 INTRODUCTION

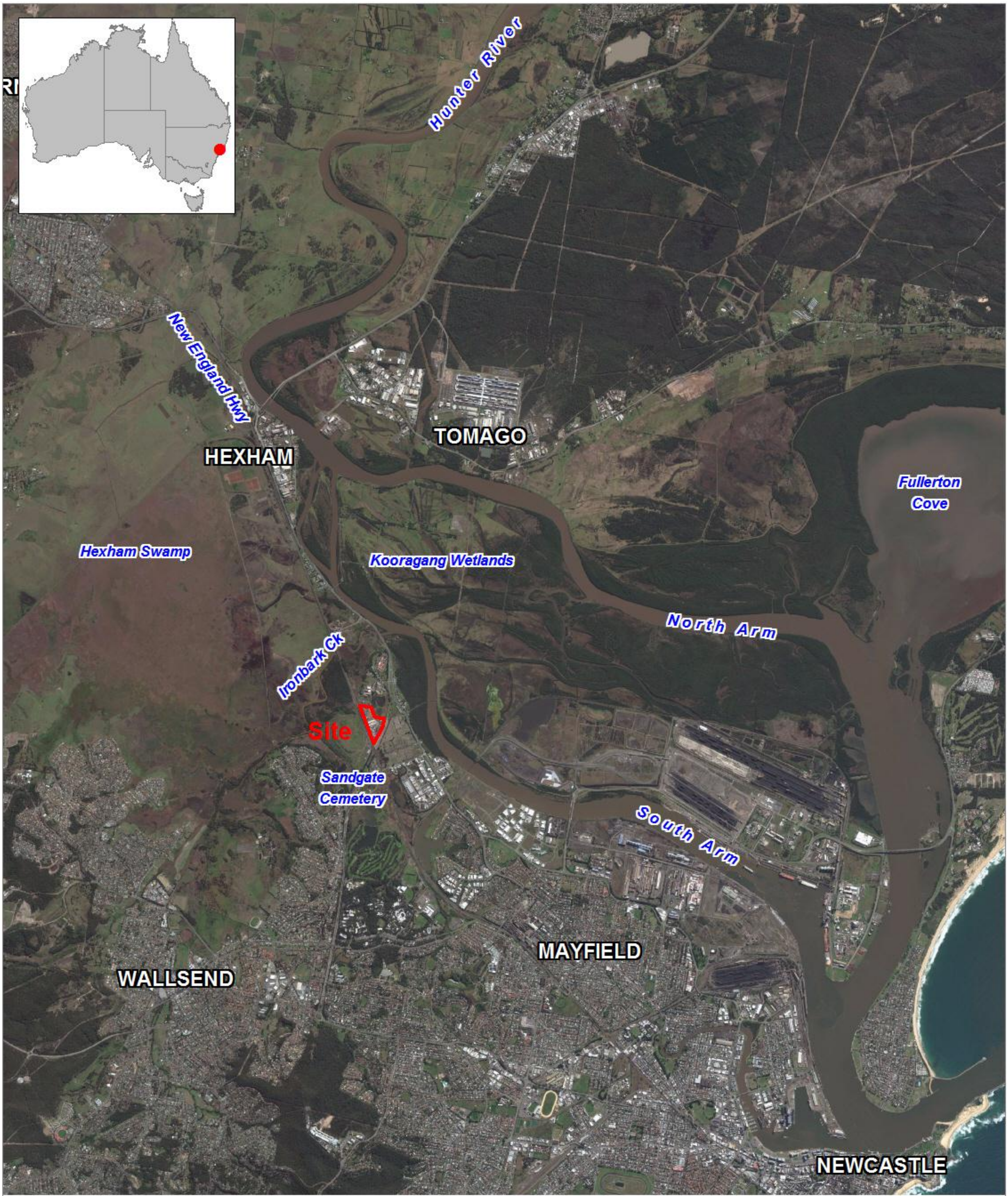
The proposed development comprises an application to use an existing site at 158 Maitland Road (Lot 12 DP 625053), Sandgate, NSW “the Site” to store ammonium nitrate. The Site locality and surrounding environment is shown in Figure 1-1. Crawfords Freightlines Pty Ltd is seeking approval to store up to 13,500 tonnes of ammonium nitrate within the Site. The Site is currently storing a maximum of 2,000 tonnes of ammonium nitrate under an interim arrangement with the NSW EPA in advance of receiving a licence for this activity.

Unloading, handling and loading of ammonium nitrate would occur undercover within three existing sheds (known as Sheds A, B and C, refer to Figure 1-2). The majority of the ammonium nitrate will be stored in bags in a stacked arrangement within the sheds. The bags each have a capacity of approximately one tonne and comprise a polyethylene bag filled with ammonium nitrate which is inserted inside a woven polypropylene ‘bulka bag’. The polyethylene bag is clasped at the top by a plastic fastener.

In some circumstances, ammonium nitrate is to be distributed in loose form, for which bulker bags will be opened within the sheds and transferred by a conveyer into trucks for transport from the Site. Additional bulker bags are proposed to be stored in sealed shipping containers outside of the sheds. Maximum ammonium nitrate quantities of 4,500 tonne, 3,500 tonne and 3,500 tonne are proposed to be stored internally within Sheds A, B and C respectively. An additional 2,000 tonnes of ammonium nitrate is proposed to be stored in sealed shipping containers within compounds adjacent to Sheds B and C (up to 1,000 tonnes in each compound). We understand that the proposed development does not involve the construction of any additional infrastructure external to the existing sheds with the exception of environmental controls.

Storage of ammonium nitrate within the Site has occurred for a number of years, although the Site is currently not licenced for this activity. Environmental controls for water management are currently limited within the Site. It is likely that past ammonium storage activities have resulted in elevated concentration of ammonia and nitrate discharging from the Site. Groundwater and surface water quality monitoring within the Site and adjacent environments supports this assumption.

The classification of the proposed development as a State Significant Development initiates the requirement for preparation of an Environmental Impact Statement (EIS). Director General Requirements (DGRs) have been issued by the NSW Department of Planning and Infrastructure. The DGRs outline a range of issues to be considered for the development including stormwater, flooding and receiving water quality issues identified by the government authorities. This report outlines the findings of stormwater, flooding and receiving water quality assessments for the proposed development.



Title:

Site Locality

Figure: **1-1**

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Approx. Scale

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Figure 1-2 Shed Locations

2 STORMWATER MANAGEMENT

2.1 Natural Resources Commission Objectives

The Natural Resources Commission (NRC) was tasked with recommending state-wide standards and targets for natural resources management to the NSW Government in 2005. The NRC identified 13 state-wide targets for natural resource management, including 5 specific water management targets:

- Target 5: By 2015 there is an improvement in the condition of riverine ecosystems;
- Target 6: By 2015 there is an improvement in the ability of groundwater systems to support groundwater dependent ecosystems and designated beneficial uses;
- Target 7: By 2015 there is no decline in the condition of marine waters and ecosystems;
- Target 8: By 2012 there is an improvement in the condition of important wetlands, and the extent of those wetlands is maintained; and
- Target 9: By 2015 there is an improvement in the condition of estuaries and coastal lake ecosystems.

Existing stormwater management practices within the Site are currently likely to be contributing to a lowering of water quality in the adjacent receiving environments. The proposed development provides an opportunity to incorporate improved stormwater management practices within the Site and subsequently contribute to achieving the NRC targets.

2.2 Hunter River Water Quality and Flow Objectives

The Hunter River Water Quality and River Flow Objectives are the agreed environmental values and long-term goals for the Hunter River. The Objectives reflect the community's desired values and uses of the Hunter River (i.e. healthy aquatic ecosystems and water suitable for recreational activities including swimming and boating). The Site drains to a reach of the Hunter River that is categorised as “waterways affected by urban development”.

Water Quality Objectives for waterways affected by urban development include aquatic ecosystems protection, visual amenity, secondary contact recreation (medium term objective) and, primary contact recreation (long term objective). River Flow Objectives are provided for maintaining wetland and floodplain inundation, mimicking drying in temporary waterways and wetlands, maintaining natural flow variability, maintaining natural rates of change in water levels, and minimising effects of weirs and other structures.

With regards to highly modified waterways (e.g. urban catchments), the following applies:

- *“Even in areas greatly affected by human use, continuing improvement is needed towards healthier, more diverse aquatic ecosystems.*
- *Water quality in artificial watercourses (e.g. drainage channels) should ideally be adequate to protect native species that may use them, as well as being adequate for the desired human uses. However, full protection of aquatic ecosystems may not be achievable in the short-term in some artificial watercourses.*

- *Artificial watercourses should meet the objectives (including protection of aquatic ecosystems) applying to natural waterways at any point where water from the artificial watercourse flows into a natural waterway."*

The established water quality objectives are shown in Table 2-1. The water quality objective criteria summarised are the more stringent for the water quality objectives being protected.

Table 2-1 Hunter River Water Quality Objectives

Indicator	Protection of:	Numerical criteria / trigger value
Total phosphorus	aquatic ecosystems	0.03 mg/L
Total nitrogen	aquatic ecosystems	0.3 mg/L
Chlorophyll-a	aquatic ecosystems	4 µg/L
Turbidity	aquatic ecosystems	0.5 to 10 NTU
Salinity	aquatic ecosystems	up to 2200 µS/cm
Dissolved oxygen	aquatic ecosystems	80 to 110% saturation
pH	aquatic ecosystems	7.0 to 8.5
Temperature	aquatic ecosystems	ANZECC Guidelines, table 3.3.1.
Chemical toxicants	aquatic ecosystems	ANZECC Guidelines, chapter 3.4 and table 3.4.1.
Biological assessment indicators	aquatic ecosystems	Many potential indicators exist and these may relate to single species, multiple species or whole communities. Recognised protocols using diatoms and algae, macrophytes, macroinvertebrates, and fish populations and/or communities may be used in NSW (e.g. AusRivAS).
Faecal coliforms	primary contact recreation	Median over bathing season of < 150 faecal coliforms per 100 mL, with 4 out of 5 samples < 600/100 mL (minimum of 5 samples taken at regular intervals not exceeding one month).
Enterococci	primary contact recreation	Median over bathing season of < 35 enterococci per 100 mL (maximum number in any one sample: 60-100 organisms/100 mL).
Algae & blue-green algae	primary contact recreation	< 15 000 cells/mL
Turbidity	primary contact recreation	A 200 mm diameter black disc should be able to be sighted horizontally from a distance of more than 1.6 m (approximately 6 NTU).
Protozoans	primary contact recreation	Pathogenic free-living protozoans should be absent from bodies of fresh water. (Note: it is not necessary to analyse water for these pathogens unless temperature is greater than 24 degrees Celsius).
Nuisance organisms	primary contact recreation	Macrophytes, phytoplankton scums, filamentous algal mats, blue-green algae, sewage fungus and leeches should not be present in unsightly amounts.
Temperature	primary contact recreation	15°-35°C for prolonged exposure.
Visual clarity and colour	visual amenity	Natural visual clarity should not be reduced by more than 20%. Natural hue of the water should not be changed by more than 10 points on the Munsell Scale. The natural reflectance should not be changed by more than 50%.
Surface films and debris	visual amenity	Oils and petrochemicals should not be noticeable as a visible film on the water, nor should they be detectable by odour. Waters should be free from floating debris and litter.

2.3 Newcastle City Council Objectives and Targets

2.3.1 Overview

Newcastle City Council (NCC) has identified improved stormwater management within the Site to prevent contamination of nearby sensitive waters from spilt ammonium nitrate as a key environmental issue for the proposed development.

Local stormwater management objectives and targets for development are outlined in Section 7.06 of the Newcastle City Council DCP 2012 and discussed in further detail within the associated Stormwater and Water Efficiency Technical Manual (the "Technical Manual"). Whilst Council's DCP does not apply under the provisions of *SEPP State and Regional Development*, the DCP outlines objectives and targets that are considered relevant for development within the Site.

Council's stormwater management objectives include:

- Set a minimum standard for the collection and management of stormwater on development sites;
- Minimise the potential impacts of development and other associated activities on the aesthetic, recreational and ecological values of receiving waters;
- Prevent pollutants such as litter, sediment, nutrients and oils from entering waterways;
- Ensure stormwater is controlled in a way that minimises nuisance to neighbouring properties; and
- Ensure appropriate easements are provided over existing drainage systems on private property.

Although the proposed development is unlikely to result in any significant additional stormwater pollution from the Site (provided effective hazard risk management controls are in place), the existing stormwater management system within the Site warrants improvement to address current deficiencies.

2.3.2 Stormwater Collection

The NCC DCP 2012 indicates that for development other than houses, the following drainage system requirements are to be met:

- Surface levels are to be graded such that sites are generally free draining with sufficient overflow capacity to ensure that waters do not enter buildings when underground drainage systems are beyond their capacity;
- Drainage pits are to be installed so that nuisance water does not collect at low points; and
- Gutters, downpipes and pits are to be connected to the stormwater management system.

Our understanding is that the intent of the policy is to minimise nuisance, disruptions and inconvenience to site activities. For an industrial development site the selection of an appropriate design standard should be undertaken considering the potential disturbance to operations within the site.

The major drainage system is to be designed to convey the 100 year ARI flow. Flow paths are to be provided to direct flow around buildings without relying on underground pipes. Surface levels are to be graded such that sites are free draining with sufficient overflow capacity to ensure that overland flow does not enter buildings.

Existing stormwater management within the Site appears to have been undertaken in a relatively ad-hoc manner since development commenced within the Site. Overland flow within the Site during runoff events is largely uncontrolled and the relatively flat surface gradients currently results in shallow pooling of water throughout the Site. Specific details of the existing drainage system are provided in Section 2.4.1.

2.3.3 Flooding and Runoff Regimes

The NCC DCP 2012 sets out the requirement to replicate natural conditions and manage peak runoff. The DCP requires that:

- Development is to be designed so that runoff from low intensity, common rainfall is equivalent to the runoff from a natural catchment. This can be achieved by intercepting and storing 12 mm of rainfall from a minimum of 90% of the impervious area.
- Runoff generated by more intense rainfall needs to be managed so that downstream drainage systems are not compromised beyond their design criteria. Development is to be designed so that peak runoff from the site for all events from the 1 year ARI to the 100 year ARI is not greater than for the 'natural' drainage conditions. For sites less than 50% impervious area, this can be achieved by providing 12mm of storage.

2.3.4 Storage Drawdown and Site Discharge Controls

The NCC DCP 2012 outlines the requirements placed in order to ensure storage tanks have capacity to store runoff for successive rainfall events. The DCP stipulates:

- In order to provide sufficient capacity to accommodate subsequent rainfall events, the stored water must be drawn down at a minimum rate of 2 mm of rainfall per day (0.023 litres per second per 1000m^2 of contributing catchment). In general this can be achieved by using the water internally in the development or by disposing to groundwater.
- Alternatively, the stored water may be released back to the catchment. In order to ensure flows do not form erosive velocities downstream, the maximum discharge rate must not exceed 2mm of rainfall per hour (0.5 litres per second per 1000 m^2 contributing catchment).
- The above solutions relating to storage and drawdown can be achieved by installing 'site discharge controls'. Selection of appropriate 'site discharge controls' will largely depend on the constraints and opportunities presented by the site and are a matter for the designer to integrate within the development proposal.

2.3.1 Stormwater Pollutants

The NCC DCP 2012 specifies pollutant reductions targets for post-construction stormwater runoff. These are presented in Table 2-2.

Table 2-2 NCC DCP Pollutant Targets

Parameter	Target
Total Suspended Solids	85% reduction in the average annual load of Total Suspended Solids.
Total Nitrogen	45% reduction in the average annual load of Total Nitrogen.
Total Phosphorus	65% reduction in the average annual load of Total Phosphorus.
Gross Pollutants	90% reduction in the average annual load of Gross Pollutants

The NCC targets have been adopted for sizing of stormwater quality management measures. Rather than sizing the measures for only the proposed development, the approach adopted was to size measures that would be sufficient to achieve the targets for the entire Site (i.e. to treat all existing developed areas to this standard).

It is important that the stormwater strategy incorporates appropriate measures to intercept and remove a high proportion of the nitrogen currently being conveyed in the stormwater system. It will also be important that sources of nitrogen are managed to minimise the exposure of these sources to rainfall-runoff that has the potential to convey elevated nitrogen loads into the drainage system.

2.3.2 Overflow disposal

Development is to be designed so that overflows do not adversely affect neighbouring properties by way of intensification, concentration or inappropriate disposal across property boundaries. This can be achieved by securing appropriate easements over downstream properties or discharging overflows directly to the street system where feasible.

2.4 Site Characteristics

2.4.1 Overview

The following sections summarise the key existing site characteristics that currently influence and potentially constrain the application of environmental controls for water management within the Site.

There are a number of site constraints which potentially limit the feasibility of retro-fitting water quality measures, including the flat nature of the Site, vehicle movement paths and existing groundwater characteristics. The following constraints have been taken into consideration when developing the surface water drainage strategy.

2.4.2 Land Use / Surface Types

The existing Site comprises 8.77 ha of previously developed industrial land. The existing surfaces broadly include roof areas, unsealed road/storage areas, sealed roads/storage areas and grassed landscaping areas. The proposed development will not change the existing composition of land uses and associated surface types within the Site. The distribution of existing surfaces within the site is shown in Figure 2-1.

The Site is surrounded by sensitive environments including the adjacent wetlands to the north-east and west, and the SEPP 14 listed Hexham Swamp. The Site discharges stormwater directly to these areas. Ironbark Creek is located approximately 400 metres downstream of the Site to the north, and the south arm of the Hunter River estuary is located approximately 700 metres to the north-east. Both watercourses experience tidal influence. The Site is bound on the western side by the Great Northern Railway and the 2HD ponds to the north-east.

2.4.1 Topography and Drainage

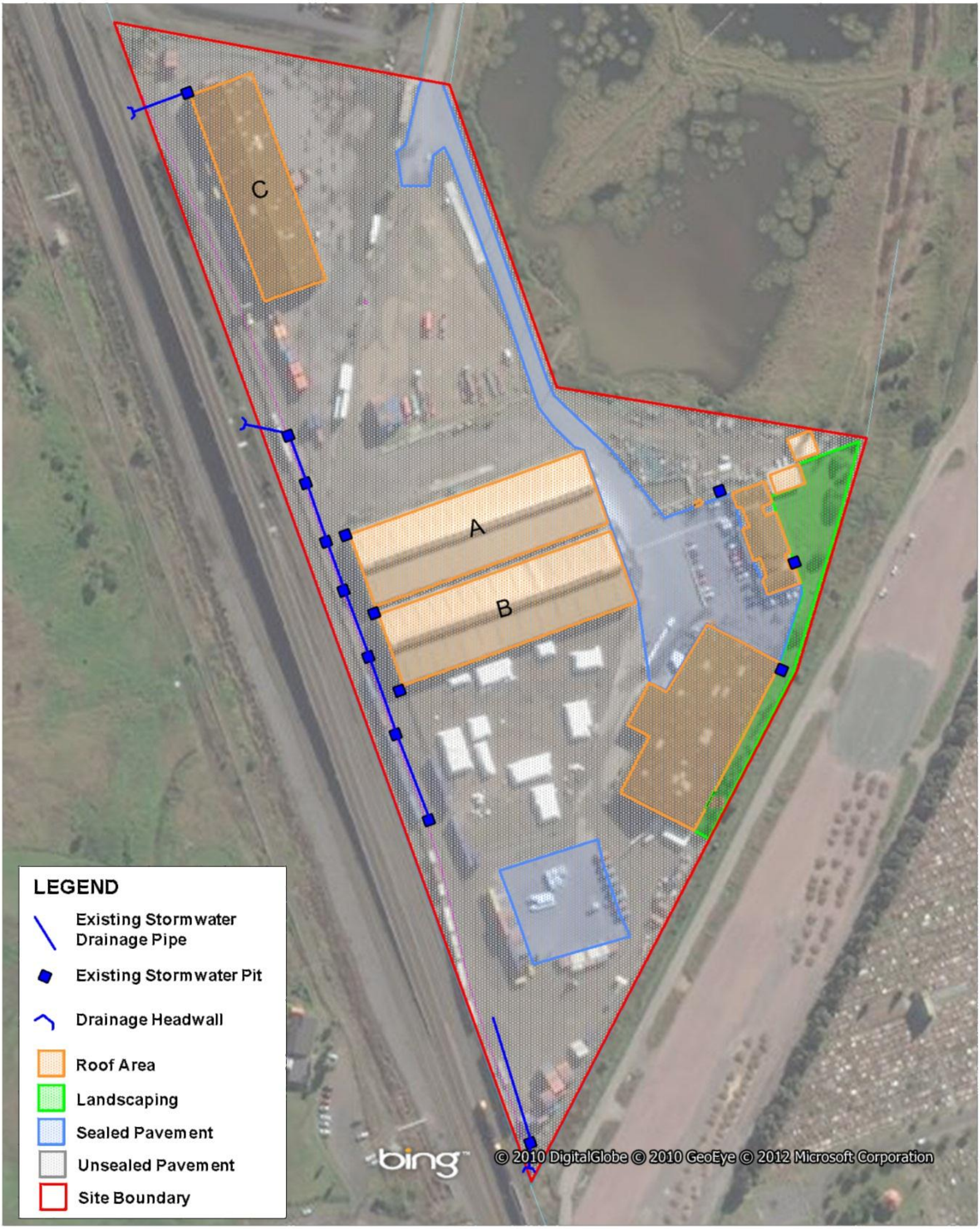
The existing site topography is relatively level with typical surface gradients being less than 1%. The Site has been extensively filled to elevate the building slabs and adjacent trafficable areas above the surrounding floodplain and wetlands.

Although no detailed modelling has been completed, it is considered that the existing stormwater drainage system would be inadequate for the purpose of capturing and conveying stormwater from the Site to Newcastle City Council's standards. In particular, it is considered that there is currently insufficient inlet capacity within the Site to capture the design minor flows. These deficiencies have been inherited from historical developments within the Site and may be cost prohibitive to fully resolve as a component of this development. The development proposal aims to improve the existing drainage system wherever practicable, however, the primary focus is on improving water quality from the Site.

The existing shed roofs are drained through a number of parallel downpipes. It appears that downpipes previously were allowed to discharge directly onto a gravel layer adjacent to the building slab from midway up the side of each shed. The downpipes have been extended and either connected directly to below ground stormwater drainage pipes or allowed to discharge onto the gravel layer from a lower height. A typical example of the shed roof drainage configuration is shown in Figure 2-2.

A high proportion of the Site currently drains to the west through an existing stormwater drainage line constructed when the adjacent rail line was upgraded. Available survey data for this line (Parker Scanlon, 2012) indicates that it currently grades at approximately 0.2% to a recently constructed open drainage channel located in the adjacent railway land. It appears that the drainage line is currently not free grading or self-cleansing due to the low gradient, and is also functioning as a sediment trap for runoff from the Site. Site observations and available survey suggest that the roof drainage lines from Sheds A and B also connect to this line. Management of runoff draining to this drainage line should be a high priority in order to improve runoff quality from the Site.

Other roof and ground surfaces primarily around the fringes of the Site are currently drained informally as overland flow from the Site or to excavated channels within the Site. It is our understanding that the north-eastern side of the Site drains to three separate outlets. These outlets drain to the 2HD ponds and minor watercourses located east of the Site. Other fringe areas on the Site currently drain informally as overland flow onto adjacent properties.



Title:
Existing Site Configuration

Figure: 2-1

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Figure 2-2 Existing Roof and Stormwater Drainage

2.4.2 Soils and Groundwater

Available geotechnical data indicates the existing site soils typically comprise a surface layer (asphalt, gravel or topsoil) underlain by fill material to an approximate depth of 2 metres below ground level (ERM, 2012a). The surface layers comprise material typically described as: sandy silty gravel, loose, fine to coarse gravel, poorly sorted, sub-rounded to sub-angular, with some occurrences of slag-like material and concrete pieces. The fill material comprises highly compacted silts, clays and gravels, with localised areas of slag and concrete. Although no specific permeability testing has been completed, the unsealed parts of the Site where heavy vehicle traffic occurs are expected to have low permeability.

Potential Acid Sulphate Soils (PASS) exist in-situ below the fill (ERM, 2012a). To avoid exposure of the PASS (which may or may not be acidic), excavation of the natural estuarine sediments is typically avoided. As the existing fill within the Site has depths exceeding 1m at most locations, it is considered unlikely that additional drainage works will require excavation of natural sediments. However, further investigation is warranted before any major excavations are commenced.

The Site is located in an estuarine floodplain where groundwater exists as a shallow unconfined water zone within the fill material and estuarine sediments (ERM, 2012a). During drilling, groundwater was encountered at depths of 0.5 to 2.2 metres below ground level across the Site. It is likely that the performance of any stormwater management measures will at times be influenced by elevated groundwater levels, although, these measures could also assist to intercept elevated nutrients within groundwater during these times.

2.4.3 Vehicle Movements

Vehicle movements within the Site are relatively uncontrolled, with most parts of the site accessible to vehicles. This relatively uncontrolled movement of vehicles has resulted in many unsealed areas within the site being destabilised by vehicles. This has the impact of increasing surface erosion potential during storm events.

The unsealed pavement between Sheds A and B, and the railway line is a relatively highly trafficked area as it forms the key access between the railway line and the storage sheds. This area is also close to a number of existing drainage system inlets and destabilisation of the pavement due to vehicle movements within this area is likely to have resulted in elevated loads of sediment being transported into the drainage system.

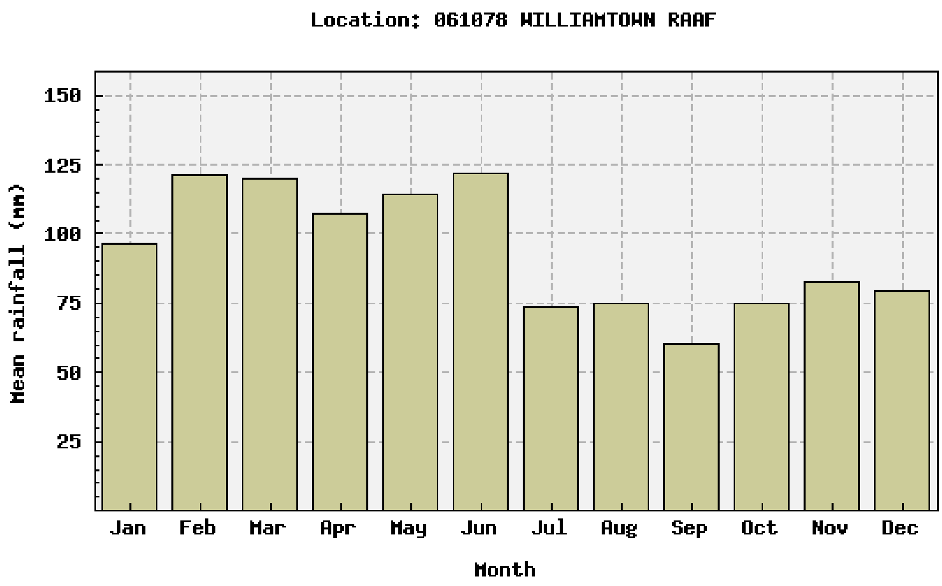
Vehicle movements within the site have been considered in developing the stormwater management strategy. Key treatment areas will be positioned in areas that can be avoided by vehicular traffic. In addition, it is proposed to undertake some minor regrading and protection of unsealed surfaces to reduce the potential for eroded sediment to discharge directly into the drainage system. Bollards will provided around treatment measures to protect them from damage by vehicles.

2.4.4 Rainfall and Evapotranspiration

The nearest long-term Bureau of Meteorology rainfall station is located at Williamtown (Stn 61078) approximately 15km north-east of the Site. Rainfall data has been recorded continuously at this Site since 1942. The average annual rainfall is 1126 mm and annual pan evaporation is 1716 mm. Monthly rainfall is typically higher over the January to June period when compared to the July to December period. Average monthly rainfall during these periods is similar for each month. Pan evaporation rates are more variable with rates being significantly higher during summer. The monthly distributions of rainfall and evaporation are shown in Figure 2-3.

The mean annual number of days where rainfall exceeds 0 mm, 10 mm and 25 mm at Williamtown are 138, 29 and 10 respectively (BoM, 2012). This indicates that, on average, 80% of days where rainfall occurs can be effectively managed by providing a retention volume equivalent to a runoff depth of 10mm from impervious or otherwise low permeability site surfaces. On average, 93% of days where rainfall occurs, runoff quality can be managed by providing a 25mm runoff depth storage volume. This conservatively assumes that runoff does not discharge from the storage during a storm event as is the case for a first flush retention system. Current best practice stormwater quality management measures will continuously filter flows throughout a storm event and consequently will be more hydrologically efficient for a similar storage volume as a first flush system.

It is envisaged that provision of basic stormwater management measures would be effective at treating more than 95% of runoff events from the development. For this Site, it is considered that larger highly effective treatment measures are warranted.



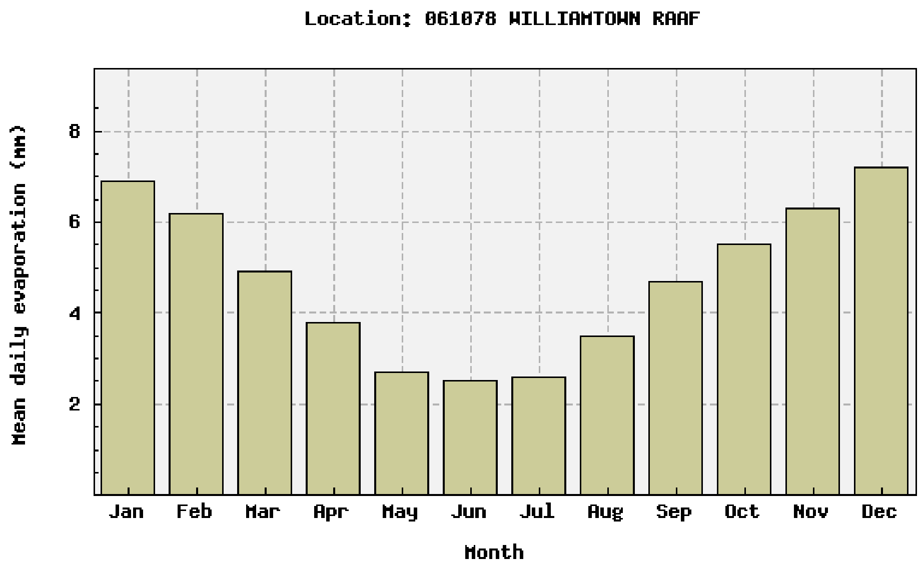


Figure 2-3 Monthly Rainfall and Daily Pan Evaporation Averages at Williamtown (BoM, 2012)

2.4.5 Surface Water Quality

No historical surface water quality data are available for sampling locations within the Site, however, groundwater quality data are available within the Site, and for surface waters adjacent to the Site. These data are discussed below.

2.4.5.1 Environmental Site Assessment (ERM, 2012a)

This assessment was completed as a background study for the proposed development to ascertain the background pollutant levels in soil and groundwater across the Site. Groundwater was sampled on one occasion, with one sample taken from each of five monitoring wells established across the Site. The monitoring wells were positioned at the following locations (with observed ammonia concentrations noted in brackets):

- Two sites just south of Shed B (16.4mg/L and 4.62mg/L);
- One site adjacent to the north-western corner of Shed C (1.28mg/L);
- One site on the eastern side of the office building (0.12mg/L); and
- One site in the southern most corner of the Site (0.51mg/L).

Based on this limited sampling, it is expected that existing concentrations of nitrogen in surface runoff will be elevated over typical industrial conditions. It appears from the limited data available that surface runoff concentrations are likely to be elevated in areas adjacent to the storage sheds. As concluded in that assessment, very high ammonia observations were observed just south of Shed B in an area where ammonium nitrate has previously being handled and stored outside buildings. The potential for elevated nitrogen concentrations in surface runoff and baseflow has been considered in the MUSIC modelling completed for the Site.

The maximum reported concentrations of ammonia (as nitrogen) within the fill material were between 110-130 mg/kg, within a reported historical storage area. Concentrations of total oxidised nitrogen (TON) up to 510 mg/kg were recorded within the Site (ERM, 2012a).

2.4.5.2 Hexham Swamp Rehabilitation Water Quality Monitoring

Recently water quality has been monitored within the adjacent environment as part of on-going ecological monitoring associated with the opening of the Ironbark Creek floodgates to increase tidal flows into Hexham Swamp. Water quality sampling has been undertaken regularly at 13 sites since December 2008. Sampling has typically coincided with periods of dry weather, although two sampling events have coincided with periods where rainfall exceeding 80mm occurred over the 72hr period prior to sampling.

Three of these sampling sites are considered to be particularly relevant to the current study. These sites include two on Ironbark Creek (King Street and floodgates (upstream)) and one at the 2HD ponds immediately adjacent to the Site. The King Street sampling site is located upstream of the confluence with a minor tributary that conveys runoff from the Site into Ironbark Creek. The floodgates (upstream) site is located downstream of this confluence.

Water quality sampling has been completed on up to 12 occasions for each parameter since December 2008, with the latest sampling round data available from September 2011. A range of physical, chemical and biological water quality parameters were sampled as part of this monitoring. For this study, discussion focuses on key parameters that are particularly relevant to the development proposal and past site activities. A summary of median concentrations for the key parameters is shown in Figure 2-4.

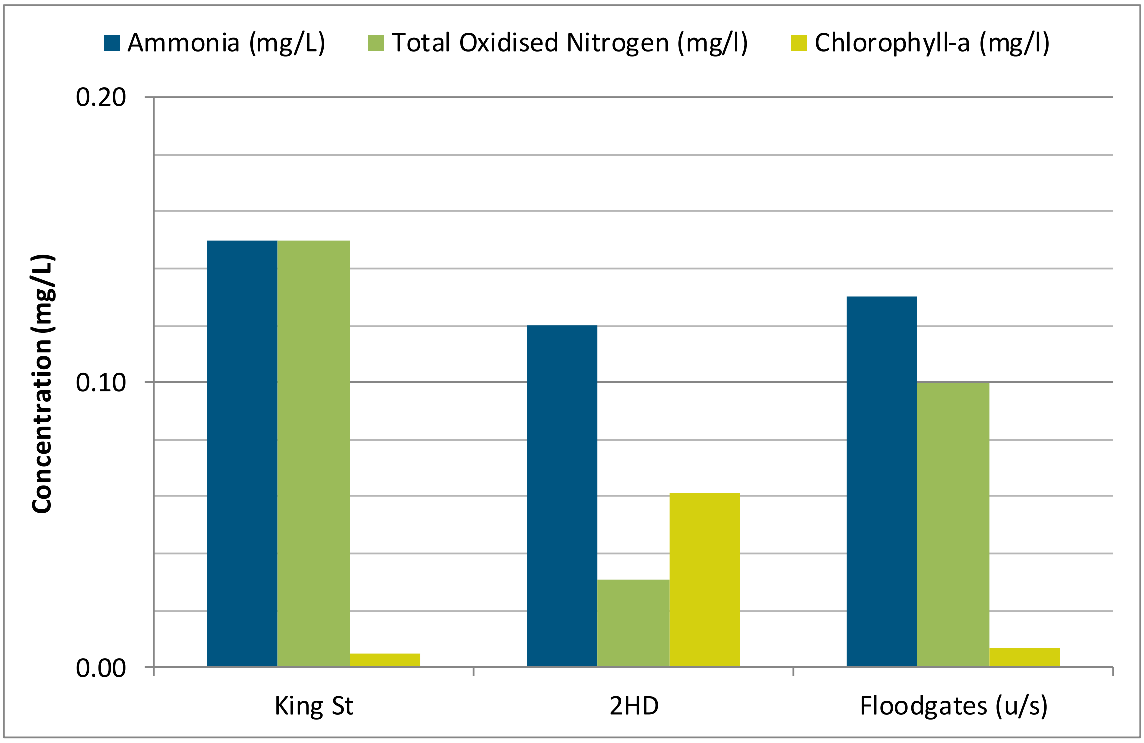


Figure 2-4 Hexham Swamp Monitoring Data – Median Concentrations (source: HCRCMA, 2012)

Although the available data is limited, the median concentrations available for the King Street and floodgates (upstream) site indicate similar ambient water quality conditions at these sites. Concentrations of ammonia and TON are slightly lower in the 2HD ponds, although chlorophyll-a concentrations are significantly higher. Chlorophyll-a is essentially a measure of microalgae concentration in the water, with higher concentrations representing higher biological productivity which is often due to elevated nutrients.

2.4.5.3 Hunter River Estuary Water Quality Data Review and Analysis

This report provides an overview of the water quality along the Hunter River Estuary including river reaches in the vicinity of the Site. The report analyses water quality monitoring observations along the Hunter River Estuary that have been gathered by the Hunter Water Corporation and the NSW EPA (OEH) over the 25 years prior to the preparation of the report (Sanderson, 2001). The analysis highlights spatial patterns of nutrients and biota along the estuary. The report concludes that concentrations of NO_3 and Total Kjeldahl Nitrogen (TKN) in the Hunter River substantially exceed default trigger values for NSW estuaries outlined in ANZECC (1992).

Key observations outlined in that report for the Hunter River reach adjacent to the Site include:

> *'Total Oxidised Nitrogen and NH_3 have increased concentrations in the lower estuary, which is incongruous with the relatively low nutrient status of the nearby shelf waters'*

> *'The tendency for NH_3 concentrations to be much higher at the downstream end of the estuary is puzzling. While it would seem that there is some source elevating NH_3 in the lower estuary, side creeks in the lower estuary do not have particularly high NH_3 levels.'*

‘The fact that NH_3 levels are very much higher in Zones B and C indicates there may be a source in this region.’

Without targeted surface runoff monitoring data, it is not possible to ascertain the origins of the potential sources of the elevated nutrient concentrations in the Hunter River immediately downstream of the Site, however, the findings from that study highlight the importance of seeking to reduce ammonia and nitrate loadings to the Hunter River Estuary from developments in the nearby catchments when opportunities arise.

2.5 Proposed Design Responses

2.5.1 Overview

Conventional stormwater quality management practices on industrial sites have typically focused on the capture of the “first flush” (i.e. interception of the initial runoff from a storm event). For the Site, an improved approach to the management of stormwater is proposed using treatments that not only treat the “first flush”, but to continue to function throughout a range of storm events of varying intensities and durations. The approach adopted for the Site initially targets the interception of coarser sediments entrained in stormwater runoff from unsealed surfaces prior to the discharge to measures that remove finer and dissolved pollutants including suspended solids, heavy metals and nutrients.

The treatment measures proposed include combinations of measures that retain and/or filter stormwater runoff. Coarse sediments will be captured in pre-treatment sediment basins, whilst finer pollutants would be intercepted within following biofiltration basins. Bypass, overflow or filtered flow from these systems is proposed to then be discharged into the existing drainage system.

In addition, non-structural source controls including improved housekeeping, minor site regrading, surface protection and a wheel wash bay are proposed to reduce the loads of potential stormwater pollutants closer to the sources. Further details of the proposed measures, and the modelling approach followed to assess their performance is outlined in the following sections.

2.5.2 Stormwater Drainage Improvements

The surveyed location of existing drainage system elements is shown in Figure 2-1. No additional development is proposed that would result in surface runoff from the Site being increased above existing conditions. Although no additional surface runoff will be generated by the development, the development provides an opportunity to improve the way that stormwater is currently drained within the Site.

The ability to regrade existing surfaces within the Site to ensure they are free draining is highly constrained by the location of existing fixed infrastructure. Any significant modifications to the existing site grading, fixed infrastructure and piped drainage systems is expected to be costly and potentially result in additional unforeseen impacts. The approach taken for this proposed development is to augment the existing drainage system; undertake minor site regrading; provide surface protection in highly trafficked unsealed areas and optimise the interception of coarse pollutants wherever possible to prevent pipe blockage due to sedimentation.

Review of the existing surveyed ground levels indicates that the shed slab levels are elevated above the adjacent external paved areas in most cases. Where adjacent ground levels are within 150mm of the floor slab level, minor regrading is proposed to ensure that local runoff is unable to enter the sheds. Minor regrading of the existing ground surface levels away from the sheds will also be undertaken to increase the proportion of the Site directed to stormwater management measures for treatment.

Additional drainage inlets will be provided at strategic locations to reduce overland flows during frequent runoff events, and piped drainage systems will be extended to connect these drainage inlets to the existing drainage system. Whilst increasing the number of inlets will improve drainage of the Site and reduce surface ponding, increasing the number of drainage inlets would (without mitigation) also increase the efficiency of the connection between stormwater pollutants stored on the surfaces within the Site and the receiving environments. Therefore, any augmentation of the existing drainage system would only be undertaken where existing nuisance ponding of stormwater interferes with the site operations and appropriate environmental controls can be provided adjacent to the inlets to manage runoff quality.

Existing roof drainage downpipes for the buildings and sheds within the Site are currently either connected to underground piped systems or discharged onto the ground surfaces adjacent to the structures (refer Section 2.4.1 for further details). Roof drainage pipes would need to be connected to a stormwater drainage system to achieve Council's objectives. Whilst this is preferable to reduce mixing of cleaner roof runoff with 'dirty' runoff from the ground surfaces, existing infrastructure constraints in some circumstances will prevent this occurring. In addition, direct connection of additional downpipes to the below ground drainage system may result in the existing pipe capacity being exceeded. It is proposed to initially connect roof drainage systems to rainwater tanks within the Site to reduce the volume of clean runoff discharged directly into the stormwater drainage system and onto ground surfaces. Provision of rainwater tanks would also provide retention/detention storage to mitigate the impacts of increased connections to the drainage system on peak discharges in the existing pipes.

2.5.3 Flooding and Runoff Regimes

Although this application does not involve any change to the runoff regime when compared to existing conditions, the requirement to replicate 'natural' conditions will be addressed to extent possible due to site constraints. The existing Site is approximately 34% impervious and to retrospectively achieve Council's requirements, rainwater tanks with a permanent storage volume equivalent to $320m^3$ would be required to achieve the 12mm runoff depth target. Rainwater tanks have been incorporated into the MUSIC models.

2.5.4 Storage Drawdown and Site Discharge Controls

Rainwater tanks would be utilised for harvesting of stormwater for dust suppression within the Site. The storage drawdown target would be achieved through the use of harvested roof runoff to meet the dust suppression demand.

It is considered preferable to utilise the rainwater tank captured runoff for appropriate internal uses within the Site in lieu of discharging and treating these flows within filtration measures. Reducing recharge to groundwater should also be avoided due to the high ground water table present and existing groundwater contamination issues. Measures should be lined with an impermeable liner to minimise the potential for interactions between contaminated groundwater and surface water within the site.

2.5.5 Stormwater Pollutants

A range of Water Sensitive Urban Design (WSUD) measures are proposed within the Site to retain and filter stormwater runoff to reduce the concentrations and loads of stormwater pollutants discharging from the Site. The performance of the treatment measures was assessed using the industry standard MUSIC software and the results of this analysis are presented in this assessment.

It is the intention of this stormwater strategy to include all reasonable and practical measures to reduce pollutant loads/concentrations to acceptable levels consistent with the objectives. Current available research on the best practice measures outlined in the strategy indicates that considerable stormwater pollutant load reductions would be achieved.

2.5.6 Overflow Disposal

The proposed development includes options to intercept and manage concentrated surface runoff and roof water discharges to reduce impacts on adjacent properties. Overflow from paved areas adjacent to property boundaries is to be directed by kerb or low bunds to away from neighbouring properties. Diverted runoff would be treated in stormwater management measures prior to discharge at existing outlets.

2.6 Numerical Modelling

2.6.1 Overview

The performance of the proposed stormwater management strategy was assessed using the industry standard MUSIC software. The software has been specifically designed to allow for comparisons to be made between different stormwater management scenarios to assist with decision making. Stormwater quality was modelled considering water quality constituents including TN, TP, and TSS.

For the Site's proposed function for storage of ammonium nitrate, TN is considered a critical pollutant. Also, due to the large proportion of unsealed area on the Site, TSS is similarly a key pollutant for the design of stormwater management measures.

The scenario modelled for the Site was the existing site configuration (which is the same as the proposed site configuration) with all proposed stormwater management measures installed and functional. The key model inputs and MUSIC modelling approach are described in the following sections.

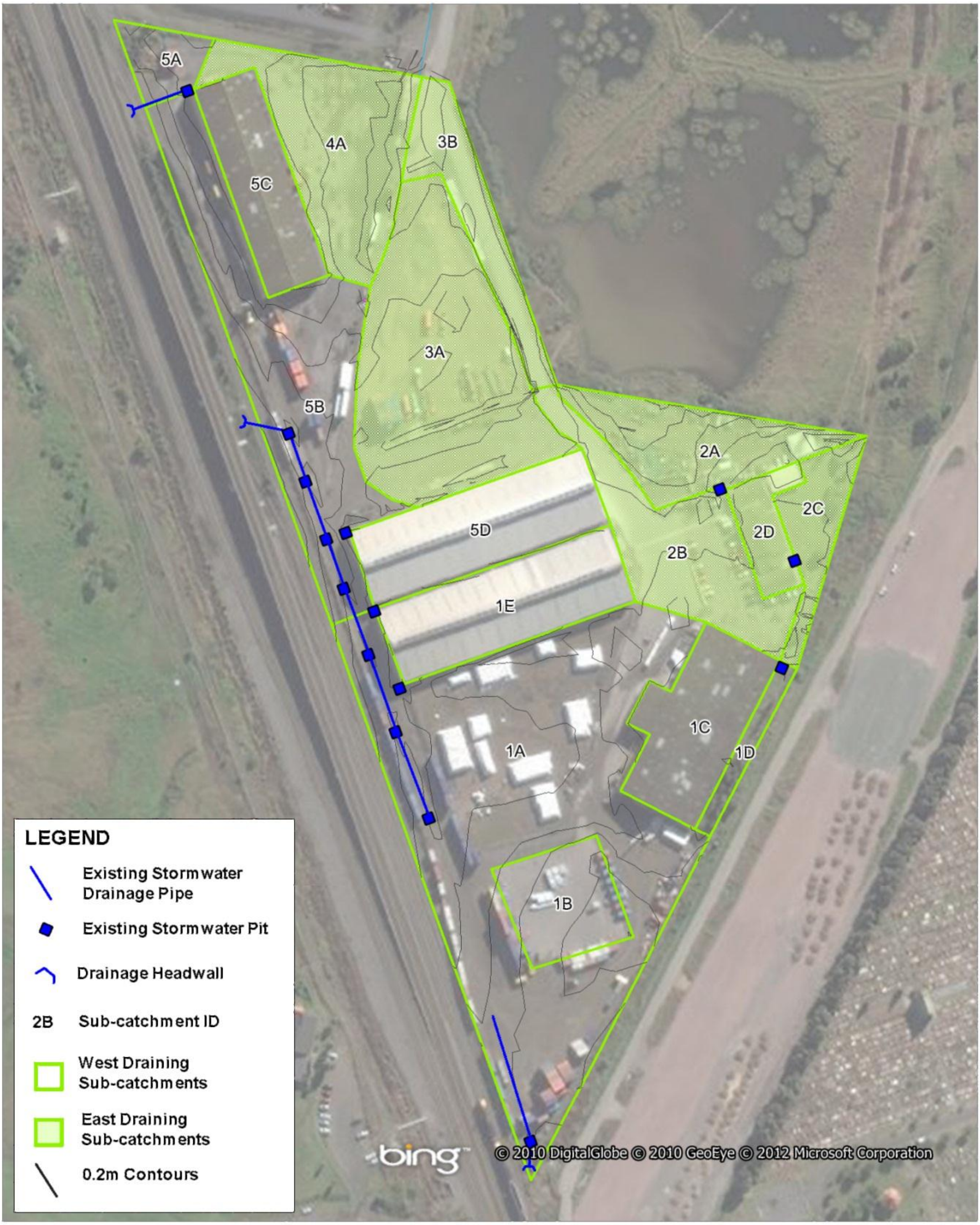
2.6.2 Sub-catchments

The Site was divided into 16 sub-catchments based on topography and surface characteristics for the MUSIC modelling. Sub-catchment characteristics are summarised in Table 2-3 and shown in Figure 2-5. The sub-catchments were defined utilising recent ground survey data, aerial photographs, a site inspection and proposed locations of stormwater management measures.

Table 2-3 Sub-catchments Characteristics

Sub-catchment ID	Surface type	Total Area (ha)	Imperviousness[1] (%)
1A	unsealed pavement	2.345	0%
1B	sealed pavement	0.279	100%
1C	roof	0.405	100%
1D	landscaping	0.081	0%
1E	roof	0.476	100%
2A	unsealed pavement	0.404	0%
2B	sealed pavement	0.521	100%
2C	landscaping	0.201	0%
2D	roof	0.137	100%
3A	unsealed pavement	1.028	0%
3B	sealed pavement	0.289	100%
4A	unsealed pavement	0.621	0%
5A	unsealed pavement	0.123	0%
5B	unsealed pavement	0.999	0%
5C	roof	0.342	100%
5D	roof	0.482	100%
	Total	**8.733**	**34%**

1. Unsealed pavement areas whilst pervious have a higher potential for generating surface runoff due to the compacted nature of the base layers. This has been allowed for when selecting appropriate rainfall-runoff parameters for modelling (refer Section 0).



Title:

Site Sub-catchments

Figure: 2-5

Rev: A

BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

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0 50 100m

Approx. Scale

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2.7 Meteorological Template

The meteorological template includes the rainfall and areal potential evapotranspiration data. It forms the basis for the hydrologic calculations within MUSIC.

Rainfall data for the Site was obtained from the Bureau of Meteorology (BoM) from the pluviograph at Williamtown RAAF base. The 2001 to 2005 period was assessed to have a mean annual rainfall of 1009mm with a mix of wet and dry years. This period provided rainfall within 2% of the long-term average estimated from BoM SILO grid data covering the Site.

Potential evapotranspiration (PET) rates were applied within the model primarily to simulate the distribution of demand from the rainwater tanks. The values were taken from the Bureau of Meteorology's Climatic Atlas of Australia (Wang et al, 2001) and are presented in Table 2-4.

Table 2-4 Average Monthly Areal Potential Evapotranspiration Rates (Wang et al, 2001)

Month	Average Areal PET rate (mm)
January	186
February	146
March	146
April	95
May	66
June	54
July	56
August	72
September	99
October	136
November	161
December	179

2.8 Annual Sediment Load

The Revised Universal Soil Loss Equation (RUSLE) was applied to estimate average annual soil loss rates from the Site for sizing of sediment basins. The RUSLE is used to estimate soil loss from uniform slopes within a site subject to sheet (including raindrop impact) and rill erosion. This method is considered appropriate for this application as large areas of uniform 'sheet' flow would be typical for this Site. The RUSLE calculates an annual soil loss estimate based on the formula:

A = R.K.LS.C.P where;

A = annual soil loss due to erosion (tonnes/ha/yr);

R = rainfall erositivity factor;

K = soil erodibility factor;

LS = topographic factor derived from slope length and slope gradient;

C = cover and management factor; and

P = erosion control practice factor.

Table 2-5 outlines the adopted values for each RUSLE parameter.

Table 2-5 Adopted RUSLE parameters

Parameter	Value	Source
R	2539	Landcom, 2004: R = 164.74 $(1.1177)^{S} S^{0.6444}$, where S = the 2year ARI, 6 hour duration storm intensity (mm).)
K	0.02	Soil description: Based on soil coverings of gravel and topsoil, underlain by fill materials (ERM, 2012a). (Landcom, 2004: Figure A3: Soil erodibility nomograph in SI units (Foster et al., 1981))
LS	0.21	(Landcom, 2004: Table A1: LS-factors on construction sites using the RUSLE) An average slope of 1% was conservatively assumed for the Site as this is the extent of Table A1 (the average slope
C	1	No measures taken on the Site.
P	0.8	(Landcom, 2004: Table A2: P-factors for construction sites (Goldman et al., 1986))

The estimated annual soil loss from the unsealed areas within the Site based on the RUSLE is 48 tonnes/year, or 8.5 tonnes/Ha/year. The estimated average annual soil loss places the Site in soil loss Class 1, with erosion hazard of Very Low. The estimated soil loss rate was utilised for sizing of sediment basins (refer Section 2.10.6).

The estimated soil loss rate includes fine, medium and coarse grained sediment particles. The sediment basins are proposed to target the removal of medium to coarser sediment particles, whilst finer particles (and attached pollutants) would be managed through higher level treatment measures. The estimation of the potential finer sediment load (i.e. total suspended solids) is discussed in Section 2.9.

2.9 Runoff Pollutant Concentrations

Within MUSIC the user specifies source nodes to represent the pollutant generating potential of different land uses / surface types within a site. MUSIC provides three default source nodes to represent urban, forest and agricultural land uses. The source nodes incorporate parameters for typical wet (storm) and dry (baseflow) weather concentrations.

The option exists within MUSIC for the user to alter the default parameters as required to represent specific land uses or surface types being modelled. This is particularly important when the land use or surface type does not correspond with the typical urban, forest or agricultural defaults supplied in MUSIC.

The Site consists primarily of unsealed gravel surfaces, roofs and some paved areas. To allow for an appropriate pollutant generation and hydrologic assessment of the Site, each sub-catchment was assigned source nodes representing the type of land surface (roof, sealed road, unsealed road or landscaped area), the size of the area and proportion impervious (estimated from aerial photography, site photos and a site visit).

Considering past activities within the Site and the ongoing land use for ammonium nitrate storage / distribution, default parameters have been increased to better represent the pollutant generating potential of the Site. The approach followed to identify reasonable runoff concentrations parameters for the different surface types is described in the following sections.

2.9.1 Roof Areas

To best simulate the land use and pollutant characteristics for the roof surface type, base flow and storm flow concentrations of TSS, TN and TP were sourced from Fletcher et al (2005) which provides values adopted by NSW OEH for site/catchment modelling within NSW. Storm and base flow pollutant export concentrations applied in the model for roof surfaces are presented in Table 2-6.

Table 2-6 Adopted MUSIC Source Pollutant Concentrations for Roofs

Source	Parameter	Base Flow Concentration (Log_{10} mg/L)		Storm Flow Concentration (Log_{10} mg/L)	
		Event Mean Concentration	Standard Deviation	Event Mean Concentration	Standard Deviation
Roofs	Total Nitrogen	0.11	0.12	0.30	0.19
	Total Phosphorus	-0.85	0.19	-0.89	0.25
	Total Suspended Solids	1.20	0.17	1.30	0.32

Roof runoff would typically be a relatively ‘clean’ runoff source compared to runoff from the other site surfaces. Draining roof runoff volumes through treatment measures can often result in increased concentrations from these sources when mixed with ‘dirty’ runoff. It is proposed that the roof runoff will be managed separately from other ‘dirty’ site surface runoff wherever possible.

2.9.2 Un-sealed Pavement Areas

2.9.2.1 Total Suspended Solids

The existing Site operations and the presence of large unsealed pavement areas is currently resulting in high quantities of sediment being generated and discharged into the existing drainage system. As discussed in Section 2.8, the RUSLE was applied and an average annual soil loss of 8.5 tonnes/Ha/yr estimated for unsealed surfaces within the Site. This estimate includes allowance for all coarse, medium and fine sediment particles potentially entrained in runoff from the Site. The TSS concentrations modelled in MUSIC are based on an assumed particle size distribution in the runoff that is dominated by finer sediments up to 150 μm in size (i.e. medium and coarse sediments are assumed to comprise only a minor proportion of the total load).

The Draft NSW MUSIC Modelling Guidelines (BMT WBM, 2010) indicate that a storm flow TSS concentration of 1000mg/L may be appropriate for areas that are not vegetated in the post development state (e.g. unsealed roads). This is equivalent to an average annual load of approximately 3.5 tonnes/Ha of unsealed pavement/yr for this Site. The TSS load based on 1000mg/L is considered to be reasonable when compared with the RUSLE equation estimate of 8.5 tonnes/Ha of unsealed pavement/yr for total sediment load.

2.9.2.2 Total Nitrogen

It is assumed that appropriate source control measures will be in place to prevent ammonium nitrate spilled during loading and unloading from being exposed to wind, rainfall and subsequent entrainment in stormwater runoff. The proposed measures to ensure that this would not occur are outlined in a separate report prepared for the Site (ERM, 2012b).

The main potential for ammonium nitrate to enter the stormwater drainage system will be through spillage from trucks transporting the material within the Site. Operational procedures to clean up potential ammonium nitrate spills during transport within the Site and to minimise the potential for this material to enter the drainage system are documented by others in a separate report prepared for the Site (ERM, 2012b).

Although it is expected appropriate spill containment procedures will be in place for future operations, TN concentrations in surface runoff are likely to remain somewhat elevated above typical industrial concentrations. It is expected that even with stringent spill containment procedures in place, the high solubility of the ammonium nitrate and the likelihood that any spill clean-up in not 100% effective, source concentrations would remain elevated. In addition, ammonia (as N) concentrations of up to 16.4 mg/L have been observed recently in groundwater, and concentrations for TON of up to 510 mg/kg were measured in soil samples from the Site. Consequently, it is considered that historical soil and groundwater contamination within the Site is likely to have an on-going impact on elevating TN concentrations in runoff and baseflow.

A mean storm flow concentration of 10mg/L has been adopted in the MUSIC model as being representative of potential runoff concentrations from the Site. This concentration is similar to what has been observed in the monitoring of runoff from a Sydney market garden (Fletcher et al, 2005). Similarly, due to the existing groundwater contamination discussed above, an elevated base flow concentration of 5 mg/L has been adopted. Baseflow has been modelled as bypassing treatment measures to simulate the persistent degraded condition of the groundwater under the Site.

Table 2-7 Adopted MUSIC Source Pollutant Concentrations for Unsealed Roads

Source	Parameter	Base Flow Concentration (Log_{10} mg/L)		Storm Flow Concentration (Log_{10} mg/L)	
		Event Mean Concentration	Standard Deviation	Event Mean Concentration	Standard Deviation
Unsealed Pavement	Total Nitrogen	0.70	0.12	1.0	0.19
	Total Phosphorus	-0.85	0.19	-0.3	0.25
	Total Suspended Solids	1.20	0.17	3.00	0.32

2.9.2.3 Metals and PAHs

The Environmental Site Assessment (ERM, 2012a) reported PAHs and metals were encountered in the fill material on the Site and minor dissolved metal exceedances were reported in the groundwater samples across the Site.

Although MUSIC does not currently have the specific capability to model heavy metals, an assessment of the potential loads and treatment of total metals, can be made using TSS as a surrogate. The use of TSS as a surrogate was considered appropriate where metallic constituents are bound or adsorbed onto sediment particulates.

2.9.3 Paved Areas

2.9.3.1 Total Suspended Solids

For TSS concentrations from paved areas, a value of 270 mg/L was adopted from the NSW MUSIC Modelling Guidelines for sealed roads.

2.9.3.2 Total Nitrogen

The event flow mean concentration of Total Nitrogen for the paved areas on the Site has been taken as the same as the value used for the unsealed pavement areas, as there is a high likelihood that spills will occur in any trafficked areas.

Table 2-8 Storm and Base Flow Pollutant Export Concentrations adopted in MUSIC

Source	Parameter	Base Flow Concentration (Log_{10} mg/L)		Storm Flow Concentration (Log_{10} mg/L)	
		Event Mean Concentration	Standard Deviation	Event Mean Concentration	Standard Deviation
Paved Areas	Total Nitrogen	0.70	0.12	1.0	0.19
	Total Phosphorus	-0.85	0.19	-0.30	0.25
	Total Suspended Solids	1.20	0.17	2.43	0.32

2.9.4 Pervious Area Rainfall-Runoff Parameters

Parameters adopted to model the catchment response of unsealed pavement and landscaping areas are presented in Table 2-9. The soil moisture storage and field capacity parameters for the unsealed pavement were adjusted from the model defaults to reflect the lower storage and infiltration potential in these areas due to compaction of the road base.

2.9.5 Treatment Nodes

MUSIC requires the user to specify stormwater treatment nodes. These nodes essentially represent the stormwater management measures provided to improve the quality of stormwater discharged from the Site. MUSIC has a range of default treatment nodes including gross pollutant traps, ponds, wetlands, swales, bio-retention systems, sedimentation ponds and buffer strips. Each treatment node has several default parameters that may be altered by the user to allow it to be ‘customised’ to best represent the stormwater management measure proposed for a particular site.

For the Site, it is proposed that stormwater quality would be managed using key stormwater management measures including rainwater tanks; sedimentation basins; and biofiltration basins. Details of treatment measures proposed within the Site, their configuration and the modelling approach applied to estimate sizes are summarised in Section 2.10.

Table 2-9 Adopted Pervious Surface Rainfall-Runoff Parameters

Rainfall-runoff parameter	Adopted Values	
	Unsealed Pavement	Landscaping
Rainfall Threshold (mm/day)	1.5	1.5
Soil Storage Capacity	47	75
Initial Storage (% of capacity)	30	30
Field Capacity	38	49
Infiltration Capacity Coeff. “a”	150	150
Infiltration Capacity Coeff. “b”	3.5	3.5
Initial Groundwater Depth (mm)	10	10
Daily GW Recharge Rate	25%	25%
Daily Baseflow Rate	10%	10%
Daily Deep Seepage Rate	0%	0%

2.10 Proposed Stormwater Management Measures

2.10.1 Overview

The stormwater management measures proposed for this Site include a range of source and conveyance control measures. Source control measures are provided close to the source of pollutant generation and aim to reduce the availability of pollutants exposed to stormwater. Source control measures are typically not enough to avoid pollutants being entrained within stormwater, and typically other structural measures are required to intercept and treat the stormwater. The stormwater management measures proposed for this Site are summarised in the following sections.

2.10.2 Improved Housekeeping

Operations within the Site have the potential to result in ammonium nitrate being deposited onto site surfaces exposed to rainfall. It will be important for minimising environmental impacts that areas where transfer of ammonium nitrate is being undertaken are covered to prevent exposure to rainfall. The measures proposed to ensure that ammonium nitrate will not be exposed to rainfall and stormwater runoff are documented in the hazard risk management plan prepared for the site (ERM, 2012b). The TN concentrations adopted for this assessment assume that the risk management plan includes sufficient barriers to prevent ammonium nitrate being exposed to rainfall and stormwater runoff.

Improved housekeeping is a non-structural practice which is not specifically modelled within MUSIC. Although improved housekeeping is expected to reduce the existing source loads of pollutants available for transport by stormwater, this expected reduction is conservatively not included within the modelled stormwater management strategy.

2.10.1 Drainage System Improvements

To improve the manner in which stormwater is managed within the Site, some adjustments to the existing stormwater drainage system configuration is proposed. The proposed drainage adjustments are primarily to connect proposed stormwater management measures to the existing drainage system. It has been assumed that the recent modifications to the drainage system along the western boundary were designed according to NCC requirements, however, we understand that some blockage has been observed in these pipes and surveyed levels indicate that some sections of the pipe may have reverse grading. Whilst this is not preferred, provision of sediment basins and connection of roof water drainage to these pipes should assist with improving their function and minimising future blockages.

2.10.2 Wheel Wash Bay

A wheel wash bay will be provided at the Site egress to rinse trucks exiting the Site that may pick up loose ammonium nitrate and other stormwater pollutants in their movement through the Site. The rinse water would be pumped daily to an adjacent media filter installed with appropriate media (e.g. zeolite) or biofiltration measure for treatment prior to discharge.

Modelling of the impact of including a wheel wash bay on the loads of stormwater pollutants discharged from the Site is conservatively not included in the MUSIC modelling.

2.10.3 Rainwater Tanks

Rainwater tanks capturing runoff from the existing shed and office building roof areas are proposed to function as both detention and retention measures to achieve NCC storage requirements. Harvested roof runoff would primarily be used to supply water for dust suppression within the unsealed areas of the Site. All shed roof surfaces will be used for harvesting for dust suppression apart from Shed C due to its asbestos roof. Although, Shed C will be connected to rainwater tanks with controlled releases in order to satisfy NCC storage requirements.

The rainwater tank detention storage was calculated according to the minimum NCC requirement to capture the equivalent of 12mm from 90% of impervious surfaces on the Site. For the existing site development this would require a combined active storage of 320 kL. This storage would be distributed across multiple rainwater tanks proportionately according to the contributing roof area. The total tank volumes required to satisfy NCC storage requirements are presented for each roof sub-catchment in Table 2-10. The final distribution of tanks across the Site will depend on configuration of guttering and the location of downpipes which would be confirmed during future detailed design.

The demand for dust suppression on the Site is estimated to be 4L/m^2/day for unsealed surfaces. In order to best represent the varied daily demand for water on the Site within the MUSIC modelling, this demand was scaled according to daily potential evapotranspiration (PET) minus daily rainfall (i.e. dust suppression demand only occurs when PET exceeds rainfall). The demand from each tank was scaled according to the size of the unsealed area surrounding the adjacent shed.

Table 2-10 Proposed Total Rainwater Tank Storage Volumes

Roof Sub-catchment	Total Rainwater Tank Volume (m^3)
1C	80
1E	85
2D	15
5C	55
5D	85

Based on 4L/m^2/day for dust suppression, the average demand rate is equivalent to an average discharge rate of 3.1L/s which satisfies Council's minimum drawdown rate of 2mm per day or 0.023 L/s. The average annual reduction in potable water use across the Site due to harvesting roof runoff for dust suppression demand is estimated as 5.6 ML/yr.

2.10.4 Site Regrading

Minor re-grading across the Site is proposed to maximise the sub-catchment areas draining to the sediment basins. Without extensive regrading (which is not considered feasible for this site) the sediment basin locations are largely constrained by the existing ground levels. The areas proposed for regrading are delineated on Figure 2-7.

2.10.5 Surface Protection

Provision of a primer seal will be considered in areas subject to high traffic movements to assist with reducing the area of unsealed pavement exposed to erosion and disturbance from traffic. Areas that will be regraded to assist with improving surface drainage and increasing the area of the site directed to the treatment zones will also be considered for primer sealing. As an alternative to sealing, a layer of 7mm nominal diameter crushed rock may also be placed over highly trafficked areas to assist with minimising surface erosion. The areas proposed for surface protection are delineated on Figure 2-7

2.10.6 Sediment Basins

Sediment basins are primarily provided to manage sediment from construction sites following clearing of vegetation cover. For this Site, particular areas are susceptible to erosion in the developed state due to the unsealed nature of many of the trafficable areas. Sediment basins are proposed to intercept a high proportion of the sediment eroded from the unsealed surfaces within the Site that is currently either conveyed into existing stormwater drainage pipes (and at times causing blockage) or overland to existing receiving environments. The sediment basins will also function as a pre-treatment measure for other measures performing a high level of treatment.

The design of sediment basins requires an understanding of the characteristics of the eroded soil that the basins are designed to intercept. 'Managing Urban Stormwater' (Landcom, 2004) describes a number of different soil landscapes, with different design considerations applicable to each. The soils within the unsealed areas of the Site are different and highly modified from natural soils found in the local area. The existing soil characteristics that comprise the upper layer of the unsealed pavement areas indicate that the soils are generally of a low dispersive potential. The existing soils in unsealed areas are considered to comprise primarily "*Type C*" soils, where the bulk of the soils are coarse-grained (less than 33 % finer than 0.02 mm) that would settle relatively quickly in a sediment retention basin.

The design procedure requires the sizing of two components of the sediment basin:

- the settling zone, within which water is stored allowing the settlement of suspended sediment. The settling zone is designed to capture most sediment in a nominated design rainfall event and, in turn, a specific discharge water quality.
- the sediment storage zone, where deposited sediment is stored until the basin is cleaned out.

The design requirements for each of these zones for *Type C* soils include:

- The settling zone must have a surface area of 4,100 $m^2/m^3/s$ in the 3 month ARI flow, a minimum depth of 0.6 metres, and a length to width ratio exceeding 3:1; and
- The sediment storage zone must have a capacity to store a minimum two months sediment loss as estimated by RUSLE.

Sediment basins will be provided at five 'treatment zones' within the Site. Table 2-11 summarises the proposed size of sediment basins based on the criteria listed above. A settlement zone depth of 0.6 metres is initially proposed, however this may be refined during detailed design to be smaller as the following proposed biofiltration basins can also capture finer sediments that otherwise could be captured in the sediment basins. The sediment basins outlined in Table 2-11 were incorporated into the MUSIC models.

Table 2-11 Sediment Basin Dimensions

Sediment Basin ID	Treatment Zone[1]	Average Settling Zone Surface Area (m^2)	Minimum Sediment Storage Zone Volume (m^3)
SB1	Zone 1	410	11.5
SB2	Zone 2	95	2.1
SB3	Zone 3	195	5.4
SB4	Zone 4	150	3.4
SB5	Zone 5	215	6.0

1. Refer to Figure 2-7 for treatment zone locations.

2.10.7 Biofiltration Basins

Biofiltration basins comprise an above ground storage and below ground filter media. The above ground storage functions as a sediment basin with the size of particles captured dependent on the hydraulic residence time. The below ground filter acts to intercept finer particles including heavy metals and particulate nutrients. Nutrients are also removed through uptake by appropriate vegetation species planted within the measure.

The filtered stormwater typically either infiltrates through the base and sides of the swale (rapidly in sandy soils or slowly in clay soils). Where the infiltration potential of the in-situ soils is low, or groundwater conditions are unsuitable, sub-soil drainage may be provided at the base of the infiltration storage to collect and convey the filtered stormwater. It is expected that the infiltration potential of the existing soils within the Site will be low and groundwater conditions unsuitable, and that subsoil drainage will be required to discharge flow from the base of the biofilter. Examples of biofiltration systems are shown in Figure 2-6.

Figure 2-6 Biofiltration Basin Examples

Biofiltration basins are proposed at five 'treatment zones' within the Site adjacent to the sediment basins to further improve runoff quality downstream of the sediment basins. Biofiltration basins are considered the best approach for reducing potential nitrogen loads through vegetation uptake. These systems rely heavily on vegetation and related processes with bacteria and fungi for the removal of nutrients such as nitrogen from stormwater.

Each biofiltration basin is proposed to have a nominal above ground extended detention depth of 300 mm and a sandy loam biofilter depth of 400 mm. The depths are proposed to be relatively shallow to account for the high groundwater table and low available surface gradients within the Site. Interconnection of the biofiltration basins with the sediment basins will effectively increase the extended detention volume. At this stage, we have conservatively ignored this additional storage volume in preparing MUSIC models.

The filter media for the biofiltration systems has been modelled as a sandy loam media with an effective particle diameter of 0.5 mm and average saturated hydraulic conductivity of 100 mm/hr.

Table 2-12 Biofiltration Basin Dimensions

Biofiltration Basin ID	Treatment Zone[1]	Average Extended Detention Area (m^2)	Biofilter Area (m^2)	Estimated Total Footprint (m^2)
BB1	Zone 1	250	160	300
BB2	Zone 2	80	55	100
BB3	Zone 3	125	85	150
BB4	Zone 4	65	45	80
BB5	Zone 5	115	75	140

1. Refer to Figure 2-7 for treatment zone locations.

For planting of the biofiltration systems, vegetation that is tolerant to periodic inundation with deep/ spreading root systems would be provided (e.g. *Carex appressa*). Other suitable groundcover plant species that may be provided are listed in Table 2-13.

Table 2-13 Plant Species for Biofiltration Systems – adapted from GCCC (2006)

Scientific Name	Common Name	Form	Height (mm)	Planting Density (No./m^2)
Carex appressa	Tall Sedge	Tufted	1000	6-8
Carex fascicularis	Tassel Sedge	Tufted	1000	6-8
Cyperus polystachyos	Bunchy Sedge	Tufted	600	6-8
Dianella brevipendunculata	Flax Lily	Tufted	500	4-6
Dianella caerulea cv 'Breeze'	Blue Flax-lily	Tufted	600	4-6
Dianella caerulea cv 'Little Jess'	Blue Flax-lily	Tufted	400	4-6
Dianella longifolia var. longifolia	Pale Flax-lily	Tufted	300-800	6-8
Fincia nodosa (Syn. Isolepis nodosa)	Knobby Club Rush	Tufted	600	4-6
Juncus kraussii	Sea Rush	Tufted	600-2300	8-10
Lomandra confertifolia subsp confertifolia	Matting Lomandra	Tufted	300	4-6
Lomandra confertifolia subsp pallida	Mat Rush	Tufted	400	4-6
Lomandra hystrix	Creek Matt Rush	Tufted	1000	4-6
Lomandra longifolia	Matt Rush	Tufted	1000	4-6
Lomandra longifolia cv 'Tanika'	Tanika	Tufted	500	4-6

The vegetation in the biofiltration basins is likely to experience more rapid growth than in other situations due to the likely high availability of nitrogen (particularly in the short-term). Monitoring of the plant growth will be undertaken during the operational phase and plants harvested and new planting undertaken where required to ensure that sufficient plants are available to take up nitrogen.

2.11 MUSIC Modelling Results

Estimated annual average flow and pollutant loads from the Site for the existing and proposed (with treatment) scenarios are provided in Table 2-14. The results shown in Table 2-14 indicate that the stormwater management measures proposed for the Site would achieve the NCC stormwater pollutant load reduction targets for TSS, TP and TN when considering the full site.

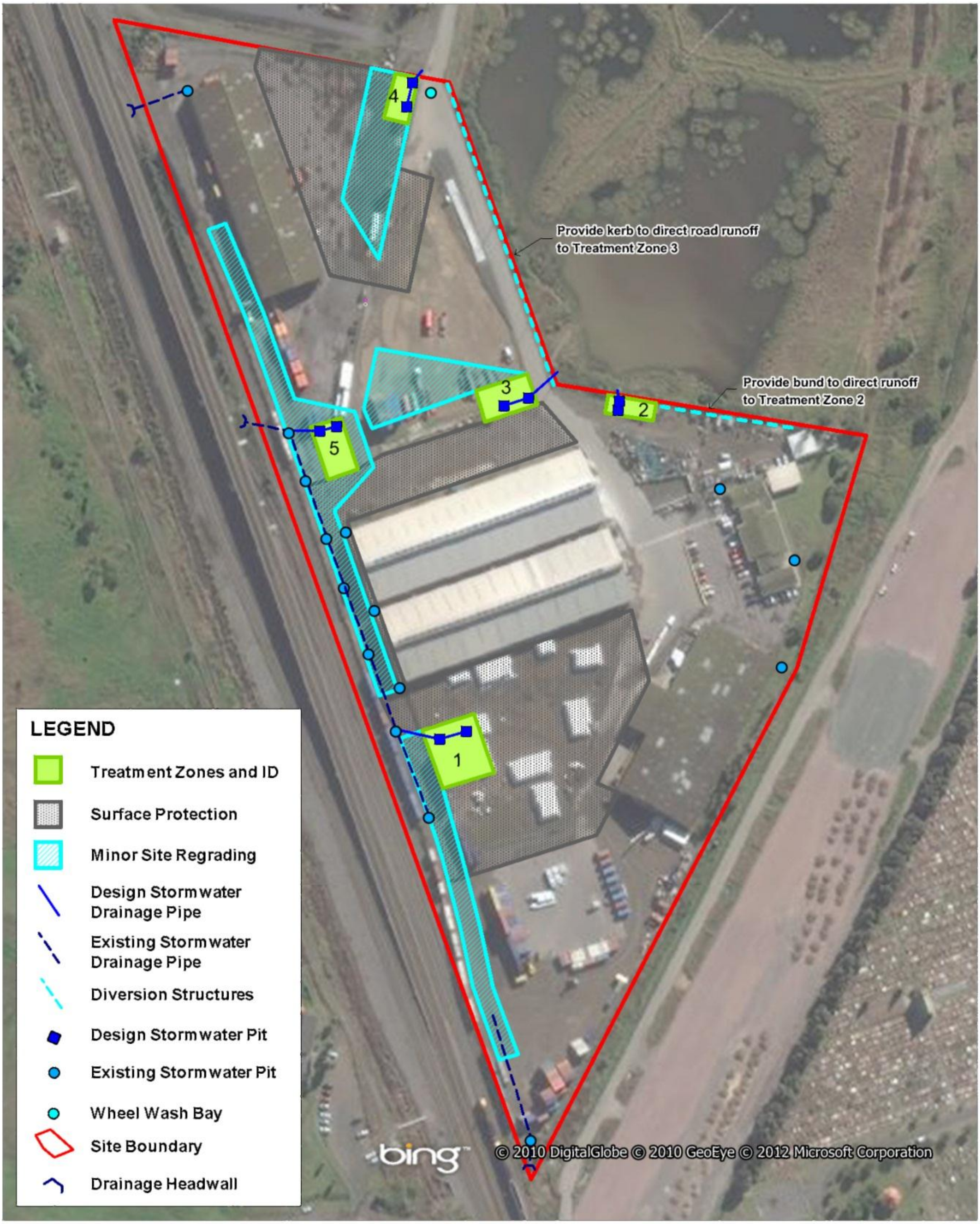
Table 2-14 Predicted Annual Pollutant Loads

Parameter	Existing Load	Post Treatment Load	Reduction	Target
Flow (ML/yr)	43.2	36.7	15%	-
Total Suspended Solids (t/yr)	23.9	1.6	93%	85%
Total Phosphorus (kg/yr)	15.9	5.5	65%	65%
Total Nitrogen (kg/yr)	319	150	53%	45%

Based on the MUSIC modelling results, it is considered that the primary treatment series comprising rainwater tanks, sediment basins and biofiltration basins within the Site, would significantly improve stormwater quality when compared to the existing Site. In addition, it is considered that provision of a secondary series of source controls including improved housekeeping, site regrading, surface protection, improved drainage and a wheel wash bay would further substantially reduce the exposure of pollutants to stormwater. Whilst these source controls are unable to be explicitly modelled in MUSIC, it is considered that further reductions in stormwater pollutant loads discharged from the Site would be achieved above those summarised in Table 2-14.

2.12 Stormwater Management Concept

The proposed stormwater management measures described in Section 2.10 are shown on the stormwater concept plan in Figure 2-7. The concept plan shows the locations of five 'treatment zones'. These treatment zones will function to perform the majority of active stormwater treatment within the Site. Each treatment zone incorporates a sediment basin and biofiltration basin proposed in series. The proposed sediment basin and biofiltration basin sizes for each treatment zone are summarised in Table 2-11 and Table 2-12. The treatment zones shaded on Figure 2-7 include additional allowance for batters and tapering into existing surfaces in order to construct the basins. The typical configuration of the proposed treatment zones is outlined in the concept sketch in Figure 2-8.



Title:

Stormwater Management Concept Plan

Figure: 2-7

Rev: B

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Approx. Scale

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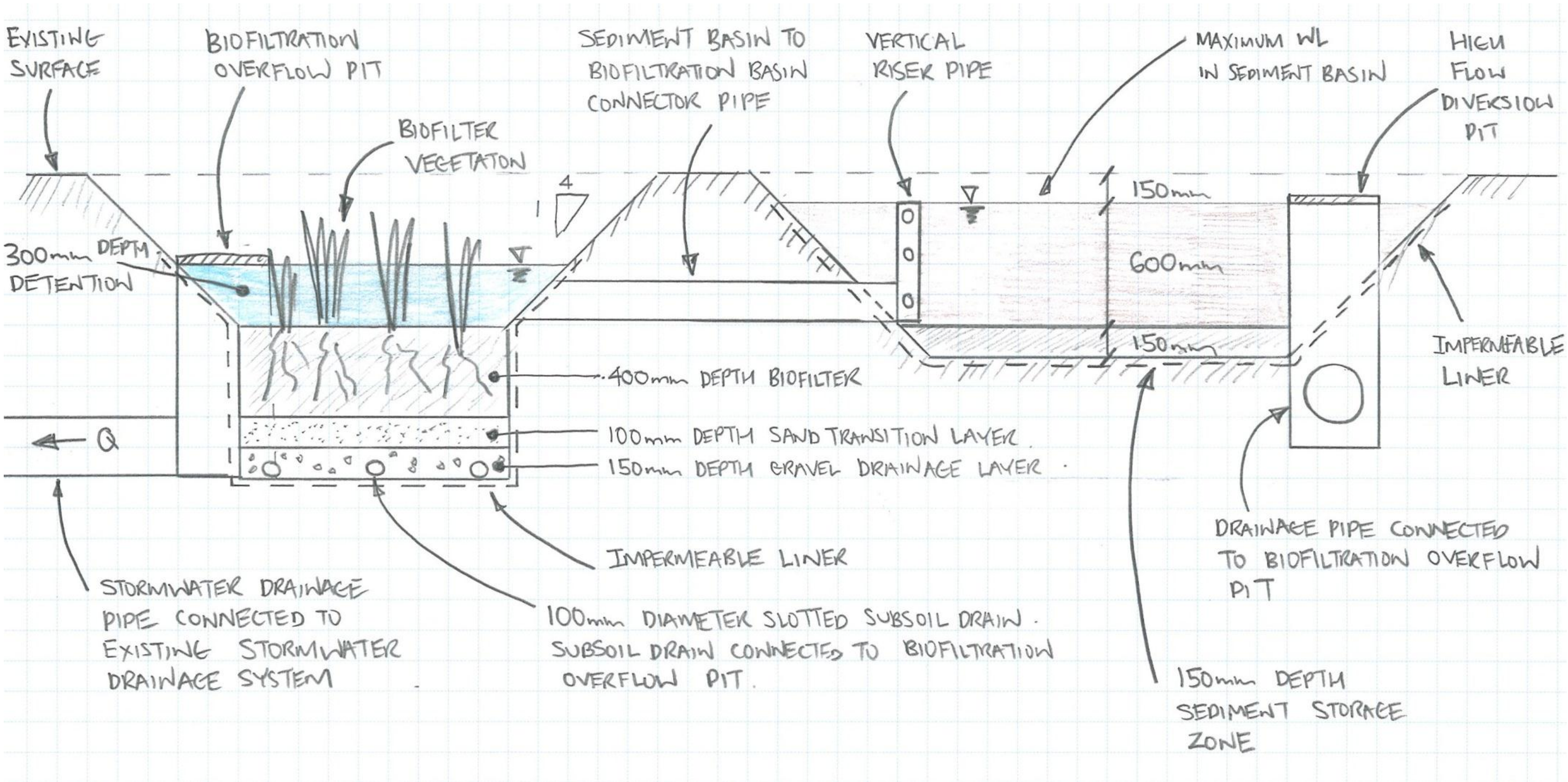


Figure 2-8 'Treatment Zone' – Proposed Sediment Basin / Biofiltration Basin Concept (not to scale)

3 FLOODING

3.1 Hunter River Flooding

Hunter River flooding behaviour has been assessed using an existing TUFLOW flood model of the Hunter and Williams Rivers, set-up and calibrated by BMT WBM on behalf of the (then) Roads and Traffic Authority and Port Stephens Council as part of previous investigations. The flood model was calibrated to the March 1978, February 1990, and May 2001 flood events. In terms of the Lower Hunter flood events relevant to the Site, the February 1990 flood event was the principal event used to calibrate the lower section of the Williams River model and the lower Hunter River model.

The flood levels generated by the model are consistent with the levels generated by the flood model developed on behalf of Newcastle City Council for flood planning purposes. Flood depths and flood level contours across the Site are presented in Figure 3-1, Figure 3-2, Figure 3-3 and Figure 3-4 for the 5%, 2%, 1% AEP and PMF flood events respectively. A summary of these flood depths and levels across the Site is provided in Table 3-1.

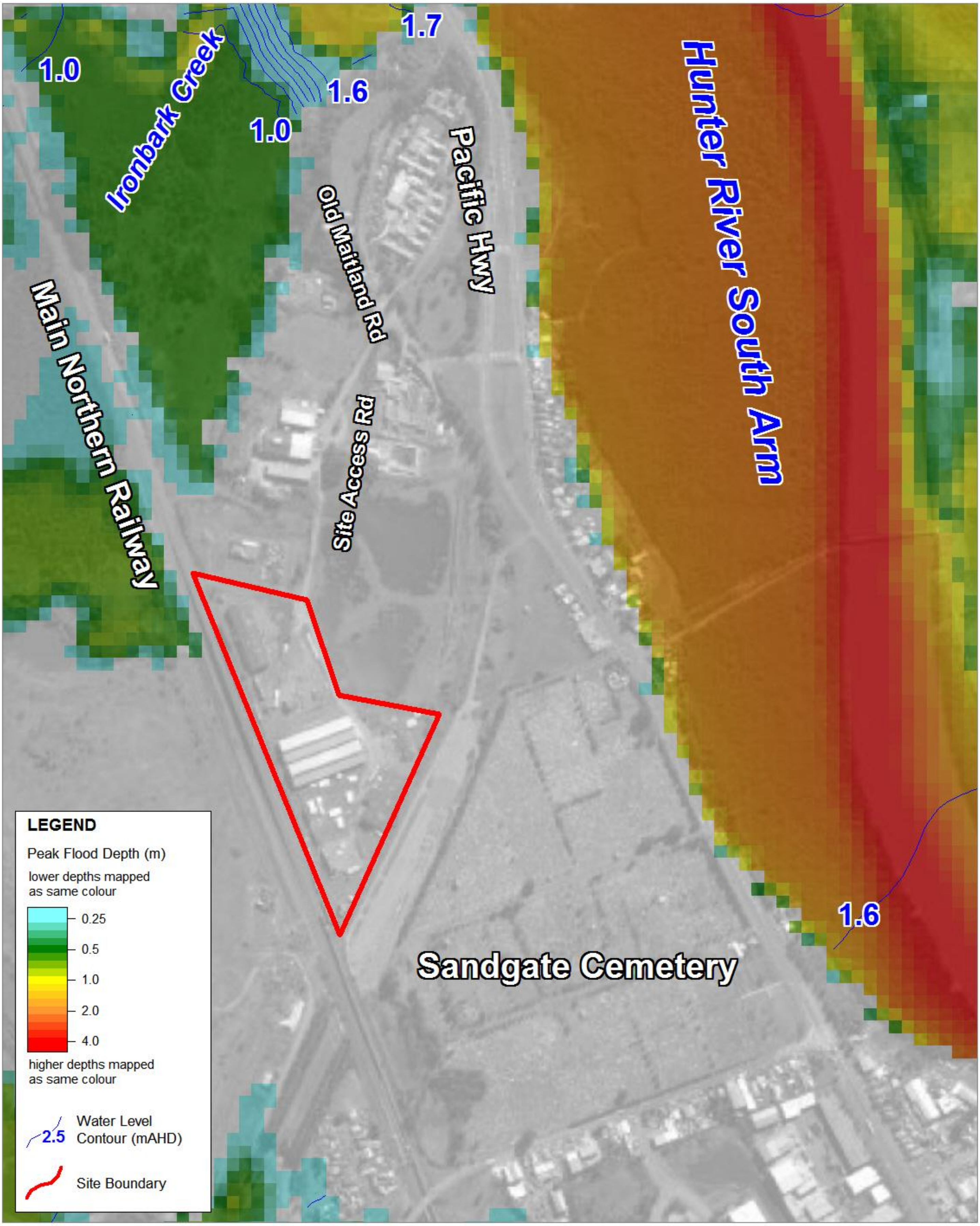
Table 3-1 Design Site Flood Levels and Depths

AEP	Flood Level (m AHD)	Flood Depth Range Within the Site (m)
5%	1.0	0
2%	2.0	0.1 – 0.4
1%	3.5	1.0 – 1.8
PMF	7.6	5.1 – 5.9

As can be seen from the figures and Table 3-1, the Site is not flooded during the 5% AEP design flood event, as ground levels are typically higher than 5% AEP flood levels in the Hunter River. Furthermore, the floodgates on Ironbark Creek would typically be lowered during a Hunter River flood event that would prevent backwater flooding from such events.

For a 2% AEP event, the Ironbark Creek floodgates, along with some low sections of the New England Highway at Hexham, would be overtopped allowing inundation of the Hexham Swamp floodplain, including the Site, located on the fringe of this floodplain. For this size event, flooding would be limited to less than approximately 0.4m on the Site, and would essentially involve backwater inundation from Ironbark Creek. If local rainfall coincided with this flood event, then there may be potential for a small conveyance of floodwaters through the Site.

For a 1% AEP event, the Site would again be inundated primarily from backwater inundation, with Hunter River floodwaters routed through the adjacent Hexham Swamp, as an overbank flow path. The Site is still regarded as Flood Fringe (as per City of Newcastle Flood Mapping) for the 1% AEP event, but with flood depths of 1 – 1.8 metres. Again there may be a small through-flow across the Site associated with local rainfall and drainage of Hexham Swamp post-flood peak, although flood velocities are expected to be very low.



Title:
5% AEP Peak Flood Depths and Levels - Existing Conditions

Figure: 3-1

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BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

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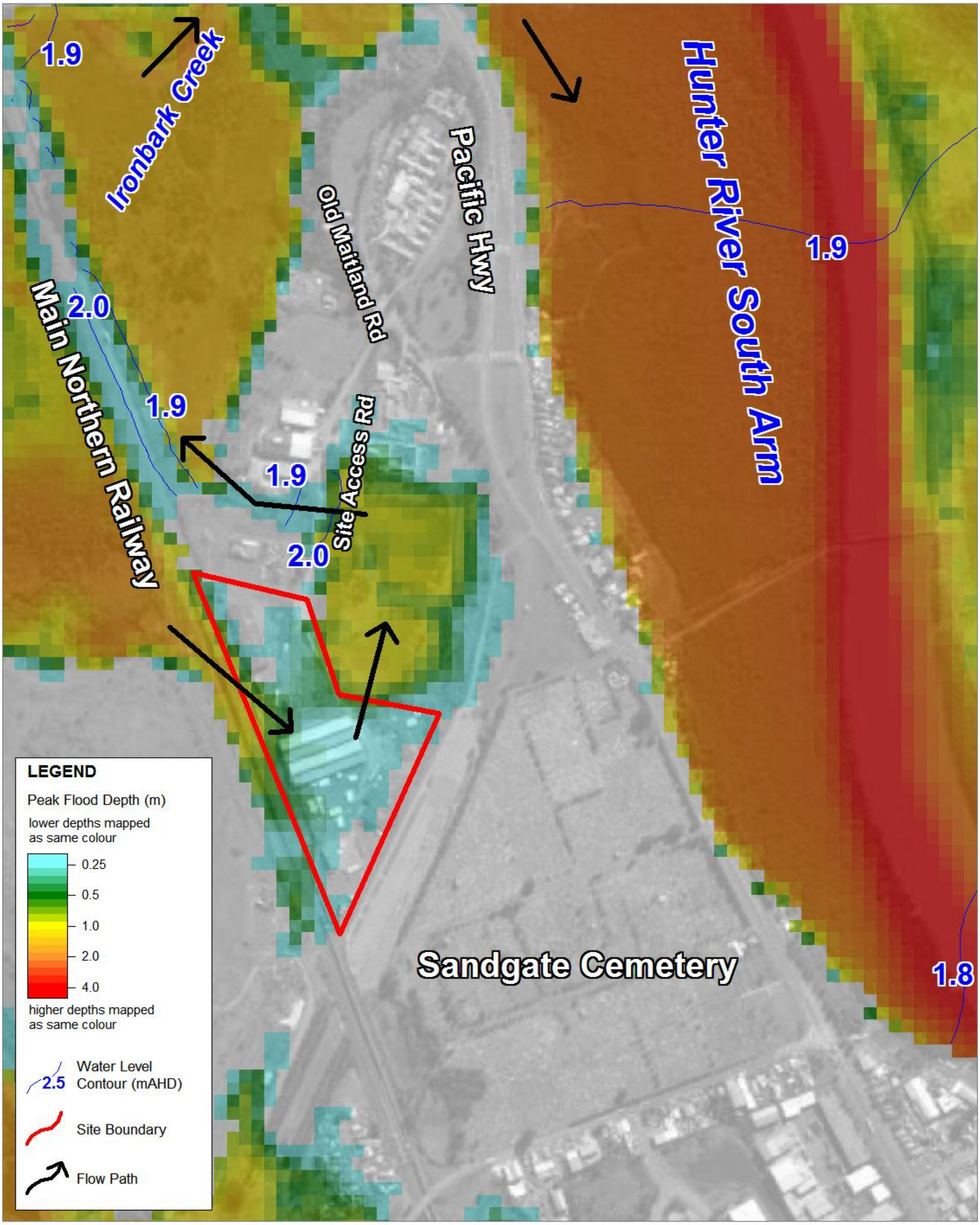

Title:
2% AEP Peak Flood Depths and Levels - Existing Conditions

Figure: **3-2**

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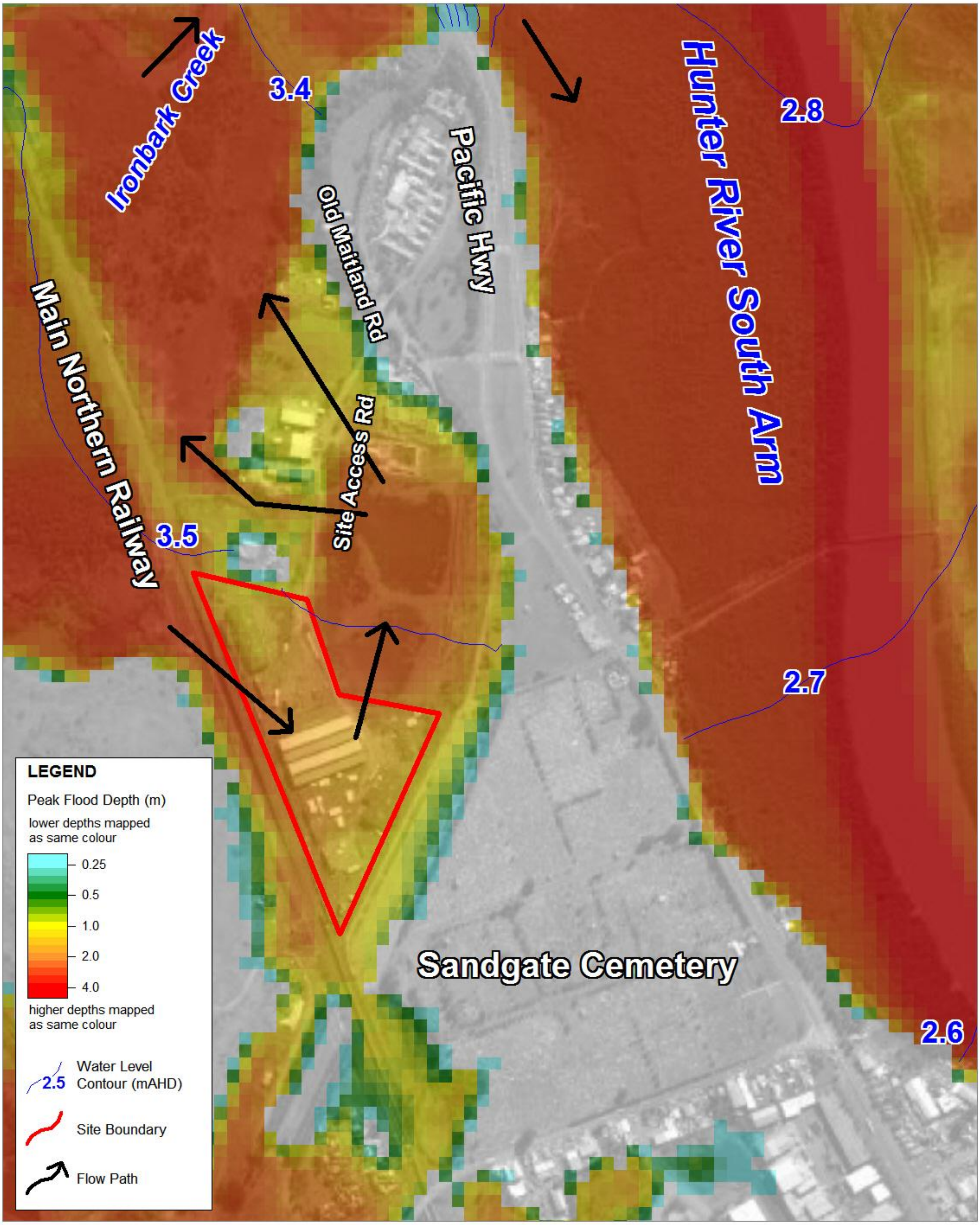

Title:

1% AEP Peak Flood Depths and Levels - Existing Conditions

Figure: 3-3

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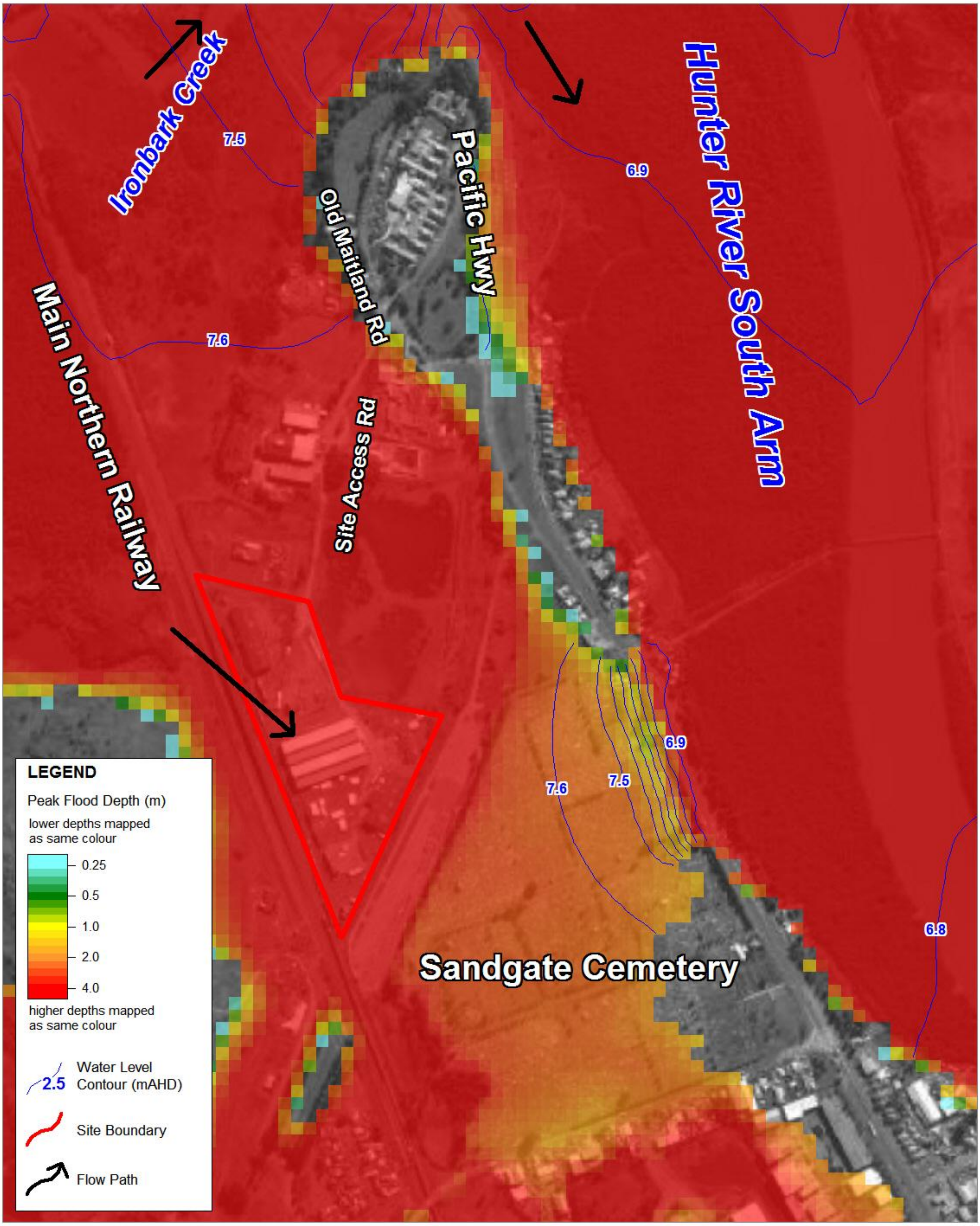

Title:

PMF Peak Flood Depths and Levels - Existing Conditions

Figure: 3-4

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The PMF event in the Hunter River is approximated by use of a standard multiplier to the 1% AEP flood conditions. The PMF event results in very deep floodwaters at the Site. Approaching the peak of the PMF event, floodwaters would also overtop the New England Highway adjacent to the Sandgate Cemetery and flow back into the river, however, the overtopping rate would be low compared to conveyance through the main flow path to Ironbark Creek and further north. As per the City of Newcastle Mapping, the Site would still be considered Flood Fringe in the PMF event.

3.2 Local Catchment Flooding

The local catchment of the Site is essentially restricted to the Site boundaries, with multiple discharge points from the Site. Runoff from higher land to the south-east (Sandgate cemetery) is intercepted by drainage channels associated with the new bypass road works and redirected to the western side of the main northern railway, while runoff from higher land to the east is directed into the 2HD ponds. Thus, there is little, if any, external runoff entering the Site.

Rainfall-runoff on the Site is directed to the north, into the 2HD ponds (and associated downstream drainage channel), and also to the west, where it is drains via a series of pits adjacent to the rail siding and then into a newly constructed open channel on the eastern side of the rail-line, before discharging into a natural channel to the north of the Site. Overflow from the pits, would result in local ponding and then overtopping of the railway siding into the new formal drainage channel on the eastern side of the rail lines.

Given the small local catchment, flooding due to local rainfall would be minor, and generally less than 50 – 100mm. It is considered that the design conditions for flooding on the Site would be driven by backwater flooding from the Hunter River, as discussed in detail above. Management of local flooding issues could be achieved through good site maintenance, including preservation of inflow capacity of the pits along the western boundary and maintaining sufficient grade on ground surfaces to prevent localised ponding. It is considered that local flooding issues could be mitigated to some degree by good practice Stormwater Management Measures, as discussed further in Section 2.

The proposed development does not involve any additional construction of buildings or significant site regrading that would affect the local flood behaviour. As such, it is considered that this proposal will have no measurable impact on local flood conditions.

3.3 Flooding Impacts and Risks

3.3.1 Existing Floodplain Risk Management Plan

The Newcastle City-wide Floodplain Risk Management Plan was adopted by the City of Newcastle on 26 June 2012. The plan is comprehensive in addressing potential flood risks from river, ocean and local catchment flooding. It provides a series of recommended actions that Council and others should pursue over the coming years in order to reduce the potential flood risks (to both property and life) across the City. With respect to the proposed development, there are no specific inconsistencies with the adopted Plan, although a strategic planning review is recommended that may lead to changes in the LEP, DCP or other guiding policies where existing land uses are incompatible with the flood risks.

3.3.2 Flood Hazard of the Site

The Site is subject to flooding from the Hunter River. It is anticipated that there would be several hours warning provided by the Bureau of Meteorology (BoM) before the Site would be inundated or access roads to and from the Site would be cut. The closest BoM flood gauge is at Raymond Terrace. A 'major' flood level for Raymond Terrace is 3.5 m which is slightly below the 2% AEP flood level. The NSW State Flood Sub Plan (SES, 2008) indicates that for floods above 3.5 m AHD at Raymond Terrace, a typical minimum flood warning of 18 hours would be available for areas around Hexham. As the Site is estimated to be only partially flooded during the 2% AEP flood, it is likely that a flood warning based on a BoM warning at Raymond Terrace would provide a minimum of 18 hours to prepare for inundation at the Site.

Given the availability of warning time for evacuation, and in accordance with the City of Newcastle hazard classification, the Site has the lowest (L1) grading for Risk to Life Hazard.

With respect to Risk to Property, it is recognised that flooding at or above the 2% AEP level will result in inundation of the Site and thus potential for property damage. For example, the 1% AEP will lead to inundation of the Site by 1 – 1.8 metres. It is expected that flooding of this depth has the potential for significant damage of fixed property and infrastructure as well as stored goods and materials. The actual risk of property damage due to inundation is not atypical of many industrial developments that fringe the Hunter River floodplain. The broader consequences of inundation are addressed elsewhere in this report. Modelling results indicate that the 1% AEP event would potentially inundate the Site for greater than 72 hours.

3.3.3 Impact of Development on Adjacent Properties

The Site is in a Flood Fringe area. This means that the flooding behaviour is driven primarily by backwater flows. By definition, further infill of the property within Flood Fringe areas is unlikely to have significant impacts on adjacent sites. Furthermore, the proposed development does not propose to create additional fill on the Site, instead just seeking alternative use of existing sheds. If these sheds were completely water-tight, preventing any floodwater ingress, there would be a theoretical loss of floodplain volume, however, this volume would be so small compared to the total volume held within the Hexham Swamp floodplain that it would have no measurable impact on flood levels or behaviour.

3.3.4 Impact of Development on the Environment

As noted above, the development will not affect the behaviour of a Hunter River flood, including affecting the local flood levels, depths or velocities. As such, the development is not expected to cause avoidable erosion, siltation, destruction of riparian vegetation or a reduction in the stability of river banks. The potential impacts of possible leachate of Ammonium Nitrate from the Site during a flood event are discussed further in Section 4.

3.3.5 Measures to Manage Risk to Life

Measures to manage risk to life at the Site would be based on evacuation when the Site and/or the access road (including the New England Highway) are expected to be impacted by inundation. It is anticipated that there would be several hours warning of this occurrence, which would be sufficient for evacuation of the Site. It is noted that the high land at St Joseph's Convalescent Home is flood-free.

Thus, in a worst case scenario where evacuation of the Site is delayed or sufficient warning time is not provided, all people on the Site could take refuge at the St Joseph's flood-free land once inundation of the Site commences or is imminent.

LiDAR survey suggests that a sag point in the private access road to the Site is about 0.6 metres lower than the Site ground levels. Thus, access away from the Site may be difficult if evacuation is delayed until the Site starts to become physically inundated.

Under no circumstances should people take refuge on the Site. It is recommended that the Site-based emergency response plan be modified to include evacuation due to flooding (including any on-site actions that can be undertaken prior to the flooding that would minimise property damage and loss of stored material to the wider environment refer Section 4).

3.3.6 Community Impacts

Social and economic costs to the community as a consequence of flooding would only be tangible if the stored material (ammonium nitrate) was leached from the Site during a flood and was subsequently spread across the Lower Hunter River, having a lasting effect on local agricultural and fisheries resources. This issue is discussed further in Section 4.

4 RECEIVING WATER QUALITY MODELLING

4.1 Introduction

BMT WBM has investigated the potential impacts on water quality in the receiving environment that could result from the release of ammonium nitrate (AN) from the Site under flood conditions. Specifically, BMT WBM has:

- undertaking numerical modelling of the advection and dispersion of AN released from the Site under 1% AEP (annual exceedance probability) flood conditions; and
- assessed the potential water quality impacts associated with the potential release of ammonia from the Site during the 1% AEP flood.

This section describes those works and the potential downstream impacts of different AN release mechanisms during the 1% AEP flood.

4.2 Modelling Software

The study used an existing TUFLOW model of the Hunter River floodplain. This model has been developed by BMT WBM over a number of years and deployed in a wide range of flood studies. A description of the model is provided in Section 3.

The TUFLOW AD (Advection-Dispersion) module within the TUFLOW suite was used to simulate the fate and transport of dissolved AN during and following flooding. TUFLOW AD solves the full two-dimensional, depth-averaged, constituent conservation equation, using the ULTIMATE QUICKEST algorithm of Leonard (1991), Leonard & Niknafs (1991) and Leonard et al. (1993). For this study, a passive tracer was included in TUFLOW AD as a proxy for AN.

No biochemical reactions or transformations were included in the model, therefore only the short-term impacts on water quality during and immediately following flooding were considered. This is appropriate given that the primary impact of concern is the acute toxicity of a potential release during flooding, rather than longer term (chronic) impacts that would be associated with more frequent discharges from the Site (e.g. discharges through the local stormwater drainage system).

4.3 Simulations

4.3.1 Overview

The most appropriate way in which to simulate the release of AN within the TUFLOW modelling framework was to treat it as a localised areal inflow. As such, a polygon through which flow (and mass) was delivered to the model was used. The locations of these are presented in Figure 4-1.

Title: **SA polygons used to assign flows to storage sheds**

Figure: 4-1

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The details of each flow-concentration pair are described below for each scenario, however the key parameter which guided their assignment was release of the correct mass of AN – i.e. at all times the model conserved the released AN mass, regardless of how this was released in time.

The 1% AEP design flood event was set up and executed as a basis for the advection and dispersion simulations. Once this simulation was completed, the output hydraulic results were used as inputs to the TUFLOW AD module to simulate the advection and dispersion of released AN from the sheds.

AN release scenarios during a 1% AEP flood were considered using the TUFLOW AD module, and these are described below. For each scenario, a background ammonia concentration of 0.1 mg/L-N was assumed prior to any AN release from the sheds. Ideally, a background ammonia concentration associated with flood conditions would have been adopted, however to our knowledge no such data are available in the area of interest. In lieu of this data, a typical long term background concentration was adopted. It was taken as the average of all data collected downstream of the floodgates as part of the Hexham Swamp rehabilitation project. The data spanned the period February 2010 to September 2011.

The use of the 1% AEP was identified through consultation with authorities as being the preferred event for consideration (rather than the PMF or some other flooding event). BMT WBM were engaged to simulate a scenario where 1% of the maximum stored AN (from each shed) was dissolved and released during the 1% AEP design flood. This mass percentage was nominated by Crawfords Pty Ltd based on the hazard risk management plan prepared for the Site (ERM, 2012b). The scenarios simulated were (all included the 1% AEP flood):

- 1% of maximum stored solid AN is dissolved and released during the flood falling limb; and
- 1% of maximum stored solid AN is dissolved and released in a single 'slug'.

These two scenarios are described in further detail in the following sections.

There is some uncertainty associated with the proportion of AN that would potentially be dissolved during the 1% AEP flood, and as required by the DGR's, a range of discharge quality/quantity and environmental conditions (in addition to the 'typical' conditions) require consideration (including worst-case scenarios). As such, BMT WBM also completed sensitivity simulations adopting higher dissolution of solid AN and subsequent release from the Site.

4.3.2 Scenario 1: One Percent AN Leak

Under this scenario, it was assumed that:

- Floodwater would flow into the storage sheds during the 1% AEP design flood event;
- 1% of the maximum total AN would be dissolved into the volume of floodwater present in each of the storage sheds at flood peak;
- No releases of AN would occur on the flood rising limb;
- Dissolved AN would be released into the immediate environment during the falling limb of the flood, as water drained from each of the storage sheds; and

- The release would cover a period of 51 hours following the flood peak, which is approximately the time taken for floodwaters to recede from the flood peak to the slab level of the storage sheds.

The masses of AN released were as follows:

- Shed A – 45 tonnes;
- Shed B – 35 tonnes; and
- Shed C – 35 tonnes.

This equates to mean ammonium concentrations within the storage sheds of:

- Shed A – 2.10 g/L;
- Shed B – 1.47 g/L; and
- Shed C – 3.34 g/L.

These concentrations were computed using GIS and other supporting techniques.

4.3.3 Scenario 2: Shed Structural Failure

This scenario was designed to investigate the impact of a shed failure, which would allow for 1% of the maximum stored AN to be dissolved and released in a short period of time. Such an event could occur, for example, under conditions of high winds associated with an extreme weather event related to the modelled flooding.

It was assumed that in the event of all three of the sheds failing, 1% of the AN in each of the 3 sheds would be exposed to the floodwaters and instantaneously released into the surrounding environment. Although this is the same mass of AN released under the 1% leak scenario, in this scenario it was released instantaneously, rather than gradually. Under this scenario, it was considered that the release would occur at flood peak (45 hours). The model was then run for the full 163 hours to examine the distribution of AN throughout the region. The mass of AN released was as follows:

- Shed A – 45 tonnes;
- Shed B – 35 tonnes; and
- Shed C – 35 tonnes.

This equates to instantaneous ammonium concentrations of:

- Shed A – 2.10 g/L-N;
- Shed B – 1.47 g/L-N; and
- Shed C – 3.34 g/L-N.

4.4 Results

4.4.1 Summary

The results from the modelled scenarios are presented below. The figures in each section illustrate the maximum ammonia concentration (in mg/L-N) reached during the 163 hour simulation. The maximum concentrations should be considered against the toxicity trigger value (TTV) of ammonia, which is the level above which ammonia is toxic, even for short exposure times. The guideline toxicity trigger value (TTV) for ammonia in marine or estuarine environments is 0.91 mg/L-N at a pH of 8.0 (ANZECC, 2000).

Two sets of figures are included to illustrate the maximum ammonia concentrations reached during each of the scenarios, with respect to the TTV. Separate scales are provided to illustrate the spatial extent of areas affected by the ammonia discharges, and to highlight the degree to which the ammonia TTV is exceeded. Specifically, Figure 4-2 and Figure 4-4 are scaled to highlight modelled areas that exceed the TTV at some point over the period of the run. Figure 4-3 and Figure 4-5 are scaled to highlight the degree to which the TTV is exceeded, as multiples of the TTV.

4.4.2 Scenario 1: One Percent AN Leak

Figure 4-2 illustrates the maximum ammonia concentration reached during the 163 hours of this scenario run. As can be seen from the ammonia concentrations, the majority of the release from the sheds flows north to the confluence of Hunter River and Ironbark Creek, and then closely follows the western bank of the Hunter River south arm prior to continuing out to sea. The TTV of ammonia (0.91 mg/L-N) is exceeded in the area immediately surrounding the sheds, the downstream reach of Ironbark Creek and the reach of the Hunter River from the confluence with Ironbark Creek to a point approximately 2200 m downstream. However, the signature of the ammonia release from the sheds does extend over a much wider area including the entire south arm from Hexham to the estuary entrance, and from Fullerton Cove to the mouth along the north arm.

The extent by which ammonia concentrations exceed the TTV is illustrated in Figure 4-3. Under this scenario, in regions where the TTV is exceeded, it is exceeded by a factor of between one and two. Concentrations higher than this are only found in the Site immediately surrounding the storage sheds, and not in the aquatic environments of Ironbark Creek and Hunter River.

The majority of the released ammonia is advected beyond the estuary entrance within approximately 20 hours of the initial release.

Title: **Maximum ammonia concentration: 1% leak scenario**

Figure: 4-2

Rev: A

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Title: **Maximum ammonia concentration, relative to TTV: 1% leak scenario**

Figure: 4-3

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Filepath : I:\B19218_I_JLB Sandgate Dispersion Modelling MEB\DRG\003_PT1_Max_s1_DTM.wor

4.4.3 Scenario 2: Shed Failure

Figure 4-4 illustrates the maximum ammonia concentration reached during the shed failure scenario. The spatial extent of impact from a shed failure is higher than that of the leak scenario. The south arm of the river also experiences higher maximum ammonia concentrations than under the leak scenario.

Under this scenario, if the sheds failed and 1% of stored AN were instantaneously released into the floodwaters, the whole of the south arm of the Hunter River from Ironbark Creek to the river mouth would exceed the ammonia TTV of 0.91 mg/L-N. Over the majority of this region, concentrations would be an order of magnitude higher than the TTV, with the reach from Sandgate to Ironbark Creek experiencing concentrations of over fifty times the TTV (refer Figure 4-5).

The majority of the released ammonia is advected beyond the estuary entrance within approximately 12 hours of the modelled shed failure.

4.4.4 Sensitivity Runs

In addition to the two specific scenarios described above, additional sensitivity model runs were completed. These sensitivity runs were undertaken to broadly identify the impact of increasingly higher volumes of AN being dissolved in flood waters during the 1% AEP flood. The results indicated that the following broad trends would occur if progressively higher proportions of stored AN were dissolved during the 1% AEP flood:

- the maximum concentrations along the south arm of the Hunter River would increase considerably and proportionally to the volume of AN dissolved; and
- the proportion of the Hunter River where the ammonia TTV is exceeded would increase.

Title: **Maximum ammonia concentration:shed failure scenario**

Figure: 4-4

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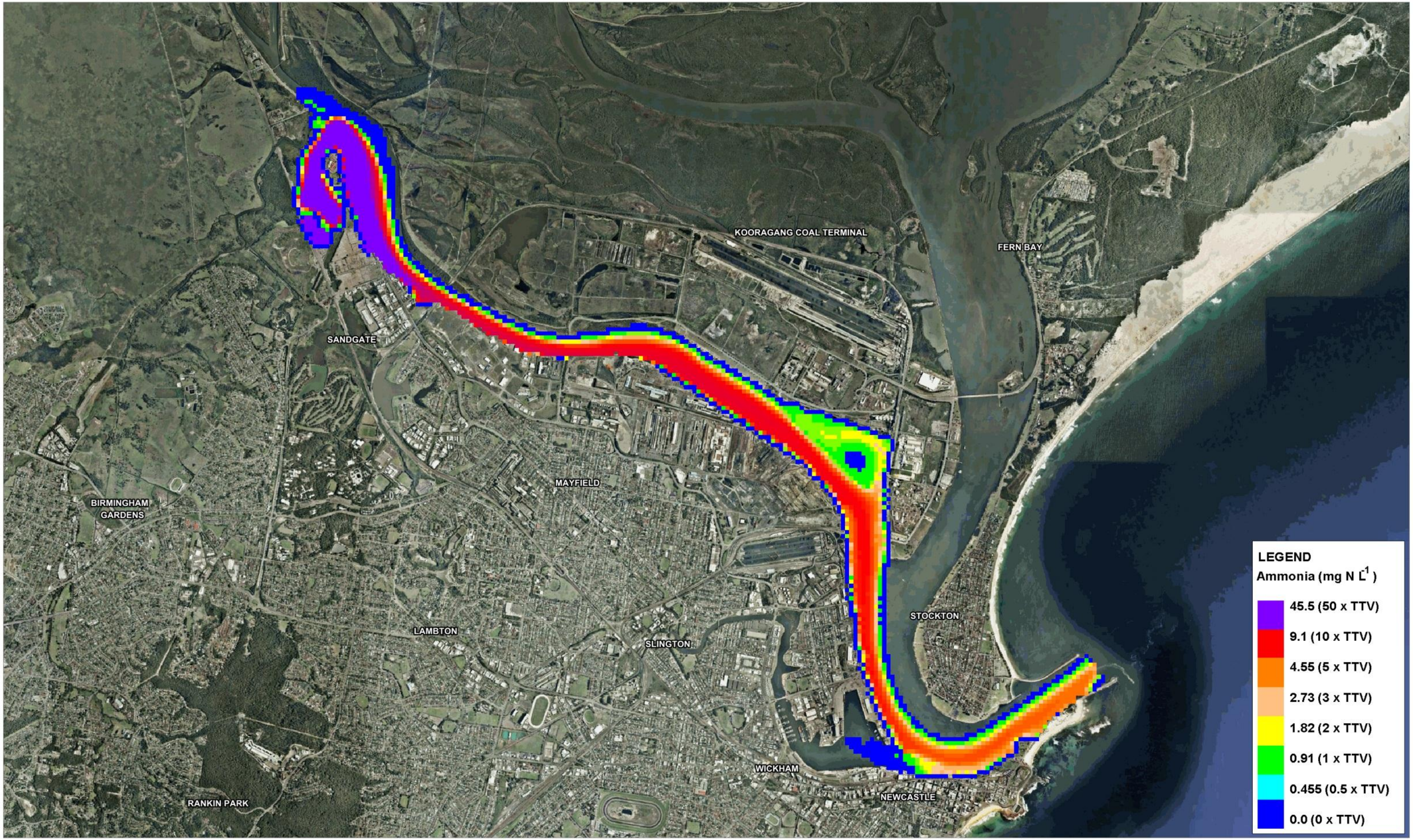

Title: **Maximum ammonia concentration, relative to TTV: shed failure scenario**

Figure: 4-5

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4.4.5 Duration of TTV Exceedence

In addition to the maximum modelled ammonium concentrations described above, time series of ammonia concentrations have been extracted at three distinct locations within the model. These time series are presented in Figure 4-6 and include:

- Point A: at the confluence of the Hunter River and Ironbark Creek;
- Point B: approximately 5km downstream of the confluence; and
- Point C: approximately 9km downstream of the confluence.

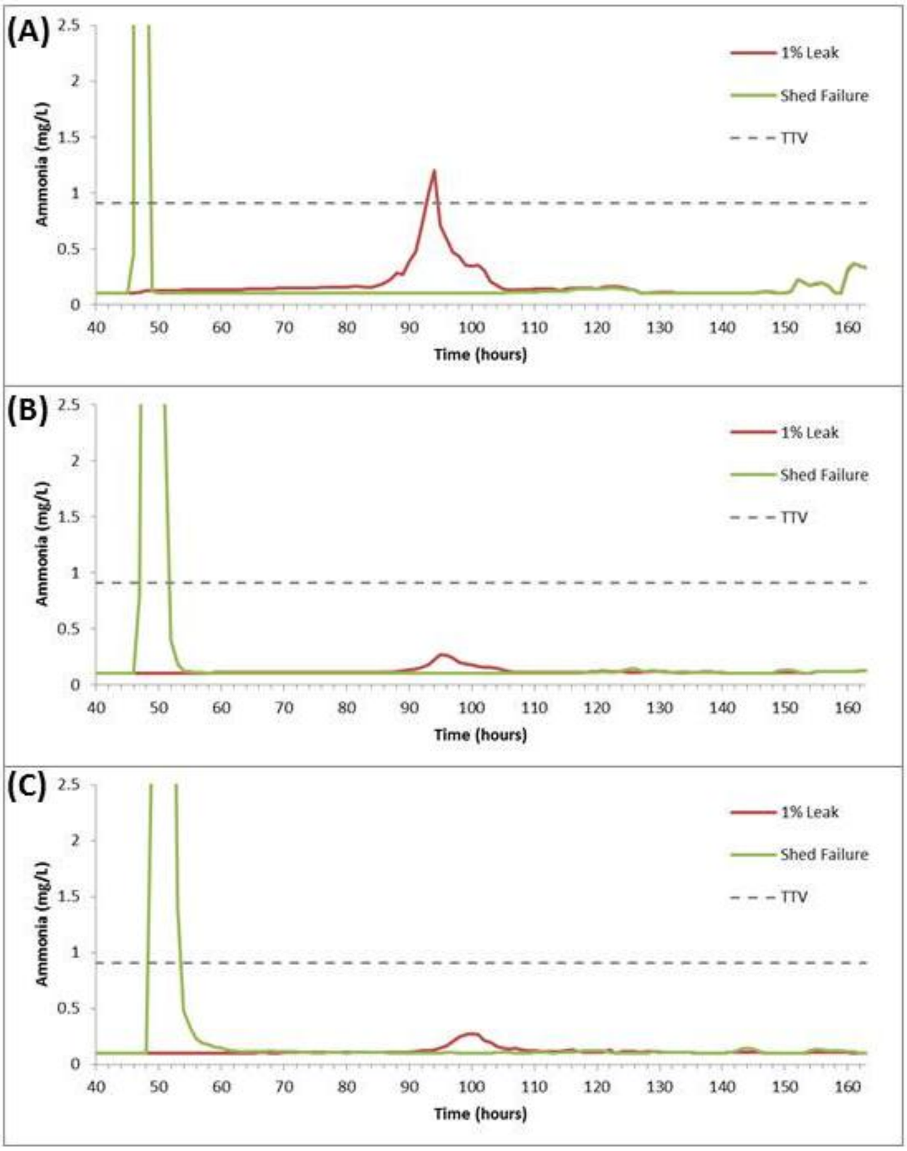

Figure 4-6 Time series of ammonia concentrations at three downstream locations

Figure 4-6 shows that the duration of TTV exceedences for the modelled flood varies from approximately 10 to 20 hours, depending on the scenario and location. Notably, the highest concentrations observed (i.e. the shed failure scenario) persist for approximately 4 to 6 hours at any given point. The sensitivity runs indicated that as the % AN dissolved increases above 1%, the duration of the ammonia TTV exceedance along the river would also increase at each location.

4.5 Summary

BMT WBM has used numerical modelling tools to assess the likely fate and transport of the release of ammonia from three storage sheds at Sandgate, NSW. These scenarios examined the 1% AEP flood, but had differing AN release assumptions. These included:

- 1% of maximum stored AN gradually released over the duration of the flood falling limb; and
- 1% of maximum stored AN instantaneously released due to shed failure under high wind or other such conditions.

Under each scenario, AN release from Sheds A, B and C would result in ammonia concentrations in the local area well in excess of the relevant toxicity trigger value provided by the ANZECC guidelines. In all scenarios, this area of TTV exceedance extended from a minimum of approximately 2,200 metres downstream of the sheds to a maximum extent beyond the mouth of the Hunter River, some 14 kilometres from the Site.

Ammonia is highly toxic to a wide range of aquatic fauna (ANZECC, 2000), and if it were to be released from the sheds as per the scenarios simulated, the consequences for downstream ecosystems are likely to be significant. This is because, even under Scenario 1, the simulations predict that the ammonia TTV would be exceeded for more than two kilometres downstream of the Site. Scenario 2 predicts even larger zones where the TTV is likely to be exceeded, including all the way to the mouth of the Hunter River.

Industry standard modelling tools have been used to assist in the investigation of the likely advection and dispersion of AN released from the three sheds on the Site. Notwithstanding this, the modelling has the following limitations:

- Despite the hydraulic model being developed, tested and applied over several years, there is always some inherent uncertainty in model predictions;
- We did not have data available to calibrate the dispersion coefficients required by TUFLOW AD. As such, these are unknown and there is a degree of uncertainty in this regard. These coefficients govern the rate of spread of dissolved constituents;
- Notwithstanding the above, we adopted minimal values for dispersion coefficients during the modelling, and any increase in these will lead to greater dispersion and lower predicted ammonia concentrations. As such, our model predictions here are likely to be somewhat conservative;
- No biochemical reactions or transformations were included in the model, therefore only the short-term impacts on water quality during and immediately following flooding were considered.

Our modelling results are specific to:

- the 1% AEP design flood;
- the release mechanism and timing assumptions (e.g. falling limb and instantaneous release); and
- The dissolution of 1% AN during the flood.

Advection and dispersion of released ammonia will be different under conditions and assumptions different to the above.

5 SUMMARY AND CONCLUSIONS

5.1 Stormwater

Estimated annual average flow and pollutant loads from the Site were estimated for existing and developed (with treatment) scenarios. The result indicate that the stormwater management measures proposed for the Site would achieve the NCC stormwater pollutant load reduction targets for TSS, TP and TN when considering the full site. Based on the MUSIC modelling results, it is considered that the primary treatment series comprising rainwater tanks, sediment basins and biofiltration basins within the Site, would significantly improve stormwater quality when compared to the existing Site. In addition, it is considered that provision of a secondary series of source controls including improved housekeeping, site regrading, surface protection, improved drainage and a wheel wash bay would further substantially reduce the exposure of pollutants to stormwater. Whilst these source controls are unable to be explicitly modelled in MUSIC, it is considered likely that further reductions in stormwater pollutant loads to that modelled would be achieved.

5.2 Flooding

The Newcastle City-wide Floodplain Risk Management Plan was adopted by the City of Newcastle on 26 June 2012. With respect to the proposed development, there are no specific inconsistencies with the adopted Plan, although a strategic planning review is recommended that may lead to changes in the LEP, DCP or other guiding policies where existing land uses are incompatible with the flood risks.

The Site is subject to flooding from the Hunter River. It is anticipated that there would be several hours warning provided by the Bureau of Meteorology (BoM) before the Site would be inundated or access roads to and from the Site cut. The NSW State Flood Sub Plan (SES, 2008) indicates that for floods above 3.5 m AHD at Raymond Terrace, a typical minimum flood warning of 18 hours would be available for areas around Hexham. It is likely that a flood warning based on a BoM warning at Raymond Terrace would provide a minimum of 18 hours.

Given the availability of warning time for evacuation, and in accordance with the City of Newcastle hazard classification, the Site has the lowest (L1) grading for Risk to Life Hazard. With respect to Risk to Property, it is recognised that flooding at or above the 2% AEP level will result in inundation of the Site and thus potential for property damage. The 1% AEP flood will lead to inundation of the Site by up to 1.8 metres. It is expected that flooding of this depth has the potential for significant damage of fixed property and infrastructure as well as stored goods and materials. The actual risk of property damage due to inundation is not atypical of many industrial developments that fringe the Hunter River floodplain.

Measures to manage risk to life at the Site would be based on evacuation when the Site and/or the access road (including the New England Highway) are expected to be impacted by inundation. It is anticipated that there would be several hours warning of this occurrence, which would be sufficient for evacuation of the Site. It is noted that the high land at St Joseph's Convalescent Home is flood-free. Thus, in a worst case scenario where evacuation of the Site is delayed or sufficient warning time is not provided, all people on the Site could take refuge at the St Joseph's flood-free land once inundation of the Site commences or is imminent.

The sag point in the private access road to the Site is about 0.6 metres lower than the Site ground levels. Thus, access away from the Site may be difficult if evacuation is delayed until the Site starts to become physically inundated.

Under no circumstances should people take refuge on the Site. It is recommended that the Site-based emergency response plan be modified to include evacuation due to flooding (including any on-site actions that can be undertaken prior to the flooding that would minimise property damage and loss of stored material to the wider environment refer Section 4).

The development will not affect the behaviour of a Hunter River flood, including affecting the local flood levels, depths or velocities. As such, the development is not expected to cause avoidable erosion, siltation, destruction of riparian vegetation or a reduction in the stability of river banks. The potential impacts of possible leachate of ammonium nitrate from the Site during a flood event are summarised in Section 5.3.

Social and economic costs to the community as a consequence of flooding would only be tangible if the stored material (ammonium nitrate) was leached from the Site during a flood and was subsequently spread across the Lower Hunter River, having a lasting effect on local agricultural and fisheries resources.

5.3 Receiving Water Quality

Numerical modelling tools were applied to assess the likely fate and transport of ammonium nitrate potentially released from the storage sheds during flooding. These scenarios all considered the 1% AEP flood, but with differing AN release assumptions. The scenarios considered included:

- 1% of stored AN released over the duration of the flood falling limb; and
- 1% of stored AN instantaneously released due to shed failure under high wind or other such conditions.

For these scenarios, AN release from storage sheds A, B and C would result in ammonia concentrations in the local environment well in excess of the relevant toxicity trigger value (TTV) outlined in the ANZECC guidelines. In each scenarios, this area of TTV exceedance extended from a minimum of approximately 2,200 metres downstream of the sheds to a maximum extent beyond the mouth of the Hunter River, some 14 kilometres from the Site.

The duration of TTV exceedences for the modelled flood vary from approximately 10 to 20 hours, depending on the scenario and location. Notably, the highest concentrations observed (i.e. the shed failure scenario) persist for approximately 4 to 6 hours at any given point.

Ammonia is highly toxic to a wide range of aquatic fauna (ANZECC, 2000), and if it were to be released from the sheds as per the scenarios simulated, the consequences for downstream ecosystems are likely to be significant.

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