  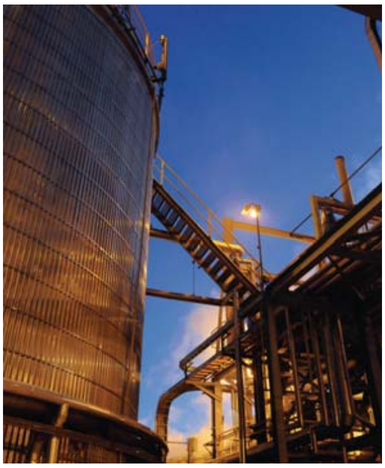

Appendix H

Water Management Report

Appendix H

Surface Water and Wastewater Management Report

27 AUGUST 2012

Prepared for

Incitec Pivot Ltd

39 Heron Road
Kooragang Island
Newcastle NSW

43177771

URS

Project Manager:	William Miles Associate Environmental Consultant	**URS Australia Pty Ltd** **Level 6, 1 Southbank Boulevard** **SOUTHBANK VIC 3006** **Australia** **T: 61 3 8699 7500F: 61 3 8699 7550**
Principal-In-Charge:	David Fuller Senior Principal	
Author:	Cara Van Megchelen Gemma Costin Cameron Purss Elliot Hannan Tung Nguyen Daniel Fokkens	
Reviewer:	Timothy Routley Senior Principal	

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Introduction & Overview

1.1 Background

Incitec Pivot Limited (IPL) is seeking development approval for the development of a Nitric Acid (NA)/Technical Grade Ammonium Nitrate (TGAN) facility (the 'proposed development') at Kooragang Island, Newcastle. The proposed development is required to service the needs of the growing mining industry in the Hunter Valley. Whilst not an explosive itself, ammonium nitrate (AN) is the main raw material used in the manufacture of commercial blasting products used by the mining, quarrying and construction industries. Projections have indicated that by 2012 there will be an AN supply shortfall in the Hunter Valley. The proposed development would help address this shortfall and ensure that the expanding mining operations in the Hunter Valley are not unnecessarily constrained.

IPL's Kooragang Island site (the 'site') has been used as a fertiliser manufacturing facility since its development and was originally owned by Australian Fertilizers Ltd. IPL now owns the site, which is currently used as a fertiliser distribution centre. This existing development is concentrated in the western portion of the site and comprises a number of industrial buildings and facilities such as storage tanks, as well as office buildings and associated infrastructure. The existing operation and the majority of the existing infrastructure would be retained alongside the Project.

The site is located on southern part of Kooragang Island towards Walsh Point. It is surrounded by Newcastle Port Corporation (NPC) land, which in turn borders the South Arm of the Hunter River to the west and the North Arm of the Hunter River to the east. The North Arm and South Arm of the Hunter River meet at Walsh Point before entering the Tasman Sea past Nobby's Head.

The nearest residential properties are located at Stockton, approximately 800 metres (m) to the south east of the Site boundary. Residential properties are also located in Carrington to the south, Fern Bay to the north east and Mayfield to the west, approximately 1.5 kilometres (km), 1.5 km and 2 km from the Site respectively.

This Project requires development approval and is considered to be State Significant Development (SSD) under the provisions of Part 4 of the *Environmental Planning and Assessment Act* (1979) (EP&A Act). The Director General's surface water and wastewater assessment requirements include the following:

- *an assessment of the potential surface water impacts including impacts on Newcastle Harbour;*
- *water supply including options for reuse of process water;*
- *proposed erosion and sediment controls (during construction) and the proposed stormwater management system (during operation); and*
- *potential impacts of flooding, with consideration of climate change and projected sea level rises.*

IPL commissioned URS to provide the necessary assessments.

1.2 Assessment Overview

This technical report was prepared from separate studies completed over the period August 2011 – May 2012. During the development of the report a number of changes were made to the design of the project including the proposed process and stormwater management and site drainage. Each of the Chapters stands in its own right and unless otherwise stated represent the best available information available for the EIS.

Chapter 2 presents the results of the assessment of stormwater volumes and quality. Stormwater from the existing site is discharged to the NPC drainage system on Kooragang Island. This will continue under the proposed development. Stormwater quality from the existing development is in the process of being upgraded to meet licence conditions. For the proposed development stormwater quality will be managed through the use of passive and active systems to separate waters of different quality including the use of roofing, a first flush system, bunds and isolation valves as appropriate. The quality of runoff from the proposed development will be significantly higher than the existing system designed in the 1960s. Where possible, stormwater peak flows will be managed to be no greater than existing levels.

Chapter 3 discusses the flood risk assessment for the proposed development site. This assessment is based upon flood studies completed by the City of Newcastle. The analysis shows the proposed development is either dry or within a “flood fringe” area, and is therefore unlikely to impact on flood levels, or to be impacted by flood inundation.

Chapter 4 provides URS’ independent review of water supply, demand and usage on the proposed development site based on process flow diagrams supplied by IPL’s design engineering consultants, Tecnicas Reunidas. Water supply is to be sought from the Hunter Water Corporation (HWC) and, where practicable, from roof runoff and HWC recycled water.

Chapter 5 provides URS’ independent review of wastewater generation and treatment based on process flow diagrams supplied by Tecnicas Reunidas. The main source of wastewater is cooling tower blowdown. The quality and quantity of the wastewater stream is identified within reasonable ranges.

Chapter 6 discusses the potential impacts of wastewater discharge. Wastewater is proposed to be discharged to the South Arm of the Hunter River. This location has been chosen after taking into account investigations by UNSW Water Research Laboratory who found that discharge at depth would provide the greatest level of mixing. Within the accuracy of the analysis the proposed wastewater discharge will maintain the water quality in the Hunter River estuary at or below existing conditions.

Additional water quality sampling and hydrodynamic modelling is currently being completed to validate the most current water quality dataset for the Lower Hunter Estuary and to confirm the hydrodynamic modelling results. This work is ongoing and will be reported whist responding to submissions following the exhibition period.

Stormwater

2.1 Introduction

This Chapter assesses stormwater behaviour and drainage on the site. The analysis provides:

- Descriptions of existing catchments and sources of material contributing to stormwater at the site.
- Estimates of peak stormwater flows for a range of annual recurrence intervals for catchments contributing to the stormwater drainage system on the site.
- Assessment of the existing drainage system capacity based on limited available drainage system information.
- Estimates of the capacity of the drainage system post development
- Estimates of first flush volumes associated with 10 mm of runoff from the proposed development.
- Estimates of changes in annual runoff from the site pre to post development.
- Description of management of construction stormwater including the management of stormwater and disposal from hydrostatic testing.

2.2 Design Approach

Stormwater from the existing site is discharged to the NPC drainage system on Kooragang Island before ultimately being discharged to the Hunter River. This would continue under the proposed development.

Key design principles for the proposed development are to:

- Manage peak stormwater runoff so that no significant increase occurs compared with current conditions;
- Adopt a 20 year ARI level of service for new drainage infrastructure;
- Adopt a paving design level of 3.5 m AHD, and
- Use passive and active systems to separate waters of different quality so that stormwater quality is significantly improved compared with current conditions.

Areas of the site have been categorised as those that may be considered to:

1. Potentially make a ‘SIGNIFICANT’ contribution to stormwater pollutant levels (i.e. process areas, product loading areas, areas vulnerable to spills, floors of some plant and chemical storage areas);

2. Potentially make a ‘MODERATE’ contribution to stormwater pollutant levels (i.e. runoff from new plant areas where process materials could be present including roadway areas around materials handling); and

3. Contribute ‘CLEAN’ stormwater that is unlikely to be contaminated with process materials, particulate matter or nutrients (i.e. back roads, roofed areas, hard stand areas away from high intensity areas, and grassed areas).

The impact of the stormwater on the receiving water environment has been designed to be minimised where possible through the adoption of sustainable design strategies including:

- Roofing to eliminate contaminated stormwater runoff including areas deemed ‘significant’, where practicable.
- Bunding of all chemical storage and processing areas to provide secondary containment in the event of system failure and to control and minimise areas of significant potential impact.

- Installation of a first flush containment system to capture the first 10 mm of rainfall from areas rated as potentially having a 'moderate' impact of stormwater quality. The contained stormwater will be tested before being discharged appropriately from the site.
- Stormwater from areas classified as 'clean' will be separated from the stormwater from 'moderate' and 'significant' impact areas through careful drainage design. Clean stormwater runoff would be discharged directly to receiving waters.
- Installation of a contaminated water pond to capture contaminated water run-off from a leak, spillage, fire or other emergencies.

2.3 Relevant Guidelines and Legislation

When completing the assessment, the following policies, guidelines and standards were used:

- Environmental Planning and Assessment Act 1979 (NSW);
- Protection of the Environment Operations Act 1997 (NSW);
- Soil Conservation Act 1938 (NSW);
- Urban Stormwater Program - Managing Urban Stormwater, LandCom, 2003;
- Urban Stormwater Program - Managing Urban Stormwater: Council Handbook, EPA, 1997;
- Urban Stormwater Program - Managing Urban Stormwater: Treatment Techniques, EPA, 1997;
- Urban Stormwater Program - Managing Urban Stormwater: Source Control, EPA, 1997;
- The Blue Book - Managing Urban Stormwater: Soils and Construction, 2004. (Construction Phase), LandCom, 2006;
- Best Practice Erosion & Sediment Control (Construction Phase), International Erosion Control Association, 2008;
- Newcastle City Council (NCC) (Operational Phase);
  - Stormwater Management Plan, 2004.
  - Kooragang Port and Industrial Area, DCP 2005.
  - Flood Management Technical Manual, 2005.
- Environmental Compliance Report Liquid Chemical Storage, Handling and Spill Management, Part B, Review of Best Practice and Regulation (fire water storage), DECCW, 2006;
- Urban Stormwater Best Practice Management Guidelines, CSIRO, 1999; and
- Water Sensitive Urban Design (WSUD) Engineering Procedures: Stormwater, CSIRO, 2006.

2.4 Hydrology

2.4.1 Existing Catchment

While the site is generally flat, it is separated by a central ridge, which directs surface water in an easterly and westerly direction. Flows draining west towards Heron Road are collected by one of three NPC's 600mm diameter culverts that subsequently outfall to the Hunter River South Arm. IPL has advised that there is no formal connection between the existing site stormwater drainage system and the drainage system on Greenleaf Road on the eastern side of the site, which discharges into the North Arm of the Hunter River. The only discharge across this boundary occurs through infiltration. The locations of the outfalls are shown in Appendix A.

2 Stormwater

The eastern catchments currently do not have a formal outfall. There is a bank along Greenleaf Road which prevents surface water from leaving the site. The Public Works drainage plans (Appendix A) show drainage outlets to the North Arm of the Hunter River which currently collect surface water from Greenleaf Road and provide legal points of discharge for the properties east of Greenleaf Road.

The catchments falling to the west of the site are collected and discharged from the site by three stormwater outfall points referred to as Points 1, 2 and 7 (JBS Environmental, 2005). These points connect to Newcastle Ports drainage infrastructure along Heron Road. The Public Works plans also show a number of existing 450mm diameter drains near the north-west extent of the site.

Figure 2-1 shows the division of sub-catchments for the site as identified from the contour plan and the surface water flow diagram (see Appendix A). Each catchment has both an upstream and downstream component. Upstream components are labelled A – G and represent those components of the outfall catchments that are on IPL property. Downstream catchments are labelled with a subscript 'DS' and represent NPC land outside IPL property that drains to the NPC outfalls.

Catchment A, C and E discharge through stormwater Points 1, 7 and 2 respectively. Catchment A is predominantly comprised of impervious surfaces and roofed areas. Catchment B includes the majority of the area occupied by Chemtrans as well as an existing grassed field. Catchment C has approximately 50% pervious (including a gypsum stockpile) and 50% impervious surfaces including roofed areas. Catchment D consists of a grassed surfaces with portions of impervious zones including the cycle club. Catchment E consists predominantly of pervious surfaces including stockpiled gypsum and grassed areas. Catchment F includes the area occupied by Air Liquide, a pad and significant grassed areas. Catchment G is the thin strip of land adjacent to Heron Road to the north of the Bagging Plant. Flows from Catchment G are considered to be directed towards a number of culverts adjacent to this strip of land.

The proportions of pervious and impervious areas adopted for the existing conditions of the site are outlined in Table 2-1 below. The proportions of pervious and impervious zones were calculated based on aerial photography for the site and surrounding area, as well as a survey of the upstream catchments. Refer to Table 2-6 for details of downstream catchments.

Table 2-1 Existing IPL Site Catchment Properties

Catchment	Total Area (Ha)*	Percent Impervious	Percent Pervious
A	8.7	78	22
B	3.5	33	67
C	5.0	47	53
D	6.1	11	89
E	6.5	19	81
F	4.3	12	88
G	2.7	24	76
TOTAL	**36.8**		

* Note: areas have been taken from satellite images and available plans. Actual areas may vary slightly from what is shown.

Based on the subregions categorised by JBS Environmental (2005) (see Appendix A), with the addition of the Chemtrans and Air Liquide areas, the catchment that each building lies within is identified in Table 2-2.

2 Stormwater

Figure 2-1 Existing Catchment Plan

Table 2-2 Existing Structures Within Each Catchment

Subregion	Catchment						
	A	B	C	D	E	F	G
1 – Concrete Slab and vehicle washbay	X						
2 – Warehouse, IBC Storage, Bagging / Despatch Plant	X						
3 – Fertiliser storage sheds 1,2 & 3 and blending plant	X						
4 – Decommissioned wax storage area	X						
5 – Shed 4 - AN store	X		X				
6 – Shed B and external AN storage	X		X				
7 – Mobile equipment washbasin, sulphuric acid tanks, wastewater tanks			X				
8 – Compressor house, decommissioned control room			X				
9 – Seminar Centre			X		X		
10 – Decommissioned Double Super Plant, Rock Mill, Trace Element Plant			X		X		
11 – Grassed area					X		
12 – Decommissioned external storage area			X		X		
13 – Weighbridge, operations office, carpark			X		X		
14 – Rail lines easements	X		X		X		
Chemtrans occupied area		X					
Air Liquide occupied area						X	
P & O occupied area							X

2.4.2 Developed Catchment

The basis of design for the stormwater system for the proposed site is described in Section 2.1. The low impact catchment stormwater will be directed to existing NPC outfall drainage which discharges to the Hunter River. The first flush of medium impact catchment stormwater will be collected and treated as described in Section 5.1.3. Significant impact stormwater will be contained and separated from other flow through control devices such as roofing and bunding.

The catchment delineation is shown in Figure 2-2 based on limited civil grading of the site during development and the location of downpipes from proposed buildings directing flow into adjacent catchments.

These changes are unlikely to affect the timing of runoff. However, the increased area of impervious surfaces is likely to change the volume of stormwater runoff. The proportion of pervious and impervious surfaces estimated for the proposed site is estimated in Table 2-3.

Table 2-3 Proposed Project Site Catchment Properties

Catchment	Total Area (Ha)*	Percent Impervious	Percent Pervious
A	8.1	76	24
B	4.2	59	41
C	3.0	60	40
D	5.1*	15	85
E	6.9	28	72
F	4.1	16	84
G	2.7	24	76
TOTAL	**33.9**		

Note: * Excludes bunded area + first flush area = 1.80 ha + 0.72 ha = 2.52 ha

2 Stormwater

Figure 2-2 Post-Development Catchment Plan

2 Stormwater

The majority of the proposed development would drain to the eastern side of the site which is reflected in the increase in percentage impervious surfaces for catchments B, C[1] and D (refer to Appendix B for proposed site layout). For catchment D bunded and first flush areas area excluded from Table 2-3 so apparent fraction impervious /pervious for peak flow estimates are similar to pre-development conditions. There is only limited development expected on the western side of the site with some of the development occurring on surfaces already classified as impervious.

Table 2-4 lists existing and proposed infrastructure post development.

Table 2-4 Proposed Project Site Catchment Use

Subregion	Catchment						
	A	B	C	D	E	F	G
Ammonium Nitrate Storage and Load-out and Bagging Plant		X					
Truck parking / workshop (Chemtrans)		X					
First flush area		C					
Nitric Acid Plant				C*			
Nitric Acid Tank				C			
Ammonium Nitrate Plant				C*			
ANSOL Storage and Loading				C			
NH_3 Storage Tank				C			
Emergency Generator					X		
Control Room/ Laboratory					X		
Administration Building					X		
Workshop Warehouse					X		
NH_3 Refrigeration Package					X		
NH_3 Flare					X		
Substation and MCC 1					X		
Substation and MCC 2				X			
Auxiliary Boiler					X		
Nitrogen System					X		
Fire Fighting System					X		
Wastewater System					X		
Demineralised Water System					X		
Service Water System					X		
Cooling Tower				C			

Note: C – Fully Contained, C* - Contained with some roofed area, X – contributes stormwater flow

2.5 Water Quality

2.5.1 Existing Development

The site is currently operating as a fertiliser bagging and distribution centre. Fertilisers currently handled include both phosphorus (e.g. super phosphate) and nitrogen based products (e.g. mono and diammonium phosphates, urea). Approximately half of the site on the western side is developed and being used for this purpose. The site has been used for a range of fertiliser manufacturing and storage purposes over the last forty years.

[1] Although the percentage of impervious area for catchment C increases, the total area of catchment C decreases. This reduces the total runoff per year.

2 Stormwater

The existing stormwater pipe network drainage to the western boundary collects stormwater from the site and discharges through three discharge points known as Points 1, 2 and 7 (JBS Environmental, 2005), which represent the North, South and Central Drains respectively, and connect to Newcastle Port's drainage infrastructure along Heron Road. There are corresponding EPA designated stormwater monitoring points on each of these drains, in accordance with Environment Protection Licence (EPL L11781). These monitoring points monitor runoff from the catchments effectively corresponding to A (point 1), C (point 7), and E (point 2), indicated in Table 2-1 and shown in Figure 2-1. Ultimately the stormwater discharges to the South Arm of the Hunter River, which is the ultimate receiving water body. However this stormwater mixes with the stormwater from the roadway and potentially other sites on Kooragang Island prior to discharge to the river.

As discussed in Section 2.4.1, there is currently no formal connection between the existing site drainage system and the stormwater drains running along Greenleaf Road, which discharge into the North Arm of the Hunter River.

EPL L11781 requires (conditions P1.2, P1.3 & M2.1), at each of the three monitoring locations (1, 2 & 7), the collection and analysis of grab samples during discharge via an event activated automatic sampler. The samplers automatically collect a grab sample of stormwater following 2mm of rainfall at monitoring points 2 and 7, and following 3mm of rainfall at point 1. An additional sample is collected an hour later if the same conditions are present. Samples are only collected in the period two hours either side of low tide to eliminate any potential estuary water interference. Collected samples are removed from the sampler and submitted for analysis on a weekly basis with results averaged and reported monthly. As the samples collected are grab samples within the first 10mm of any storm-event, the measured quality is effectively more representative of the first flush storm water quality (designated as the first 10mm of a storm-event), rather than of the overall stormwater load.

The current EPL does not impose any concentration or load limits on the discharge of stormwater from the site.

The monitoring data for the stormwater currently discharging from the existing site, for the period May 2009 until December 2011, is summarised in Table 2-5.

Table 2-5 Summary of Stormwater Quality Monitoring

	Concentrations								
	EPL 1			EPL 2			EPL 7		
Parameter	Max	Min	Mean	Max	Min	Mean	Max	Min	Mean
Total Nitrogen (as N) (mg/L)	488	8.8	97	383	2.3	55	746	5.2	99
Phosphate (mg/L)	760	7.4	145	488	8.3	100	342	14.0	104
pH (pH units)	7.8	5.9	6.44	7.3	2.9	5.81	8.8	2.6	6.10
Zinc (mg/L)	3.69	0.12	0.85	3.0	0.071	0.64	2.94	0.22	0.96
Total Suspended Solids (mg/L)	680	14	136	1126	7	176	498	14	122

Stormwater quality from the existing site is in the process of being improved under a pollution reduction plan to reduce the runoff of nutrients and suspended solids. Some of the improvements at the site include removal of stockpiles no longer in use, cleaning of collection pits and drains, more regular street sweeping, improved work practices, including management of spills, and further isolation of significant contamination sources. A recent audit of drains indicated significant sediment build-up probably reflecting work practices at the site over a long period. The current quality of the stormwater is likely to be adversely impacted by the sedimentation from historical operations therefore existing data is unlikely to be wholly representative of first-flush runoff from current operations.

2.5.2 Proposed Development

The stormwater drainage system for the proposed development has been designed in accordance with the principles described in Section 2.2, with a view to minimising impacts on the Hunter River water quality. There would be connections to the drainage systems on both the eastern and western sides of Kooragang Island as part of the proposed development.

Areas with significant contamination potential and the first flush (first 10mm of rainfall) from catchments with moderate potential to impact on water quality would be collected for recycling or management in the wastewater system, or discharged to the stormwater system if of suitable quality.

The separation of different classes of stormwater quality and the use of first flush and other systems provides a high degree of confidence that the stormwater quality from the proposed development would be significantly lower than existing onsite stormwater systems even after the pollution reduction program is implemented.

No water quality modelling was undertaken due to data limitations.

2.5.3 Construction Phase

Potential impacts on stormwater runoff quality during the construction phase of the project will be managed in accordance with a Construction Phase Environmental Management Plan (CEMP).

All construction works would be undertaken in a manner to minimise the potential for soil erosion and sedimentation and in accordance with the measures outlined in the Managing Urban Stormwater – Soils and Construction Volume 1 (NSW Department of Housing, 2004) (commonly referred to as the Blue Book guidelines). Areas which are disturbed would be managed with appropriate erosion and sedimentation control devices installed and maintained in line with the Blue Book guidelines. This may include limiting slope length, the installation of sediment filters and the construction of a sedimentation basin downstream of the construction area. These devices would remain in place until the surface is restored. These devices would also capture any gross pollutants.

Temporary containment bunds would be constructed with provision for collection of any spilt construction material. Waste collection areas would be designated. Appropriate bunding would be installed and appropriate containers would be provided. Waste collection and disposal would be properly undertaken by licensed contractors. All vehicle and equipment maintenance and washing would be undertaken offsite.

Staff facilities would be provided and installed and maintained so that pollutants are not conveyed from the site in stormwater.

During the construction phase water may be required for dust suppression.

2.6 Assessment of Drainage Capacity

DRAINS models were developed to estimate the capacity of the drainage network under existing and proposed site conditions. Based on Australian Rainfall and Runoff 1987, DRAINS performs design and analysis calculations for urban stormwater drainage systems. Peak flow rates were estimated using the Rational Method and supplied to the DRAINS model. The Rational Method peak flows were derived for a range of rainfalls based on Intensity Frequency Duration (IFD) data extracted from the Bureau of Meteorology (Appendix D).

The DRAINS model was constructed based on the information summarised in Sections 2.2 and 2.3 and 2.4.

2.6.1 Assumptions

The following assumptions were made for the drainage network modelling. Changes to these assumptions will change the results from the analysis. For example, if more detailed survey data is made available for the pipe inverts, the pipe grades may differ to those assumed and therefore the capacity of the pipes will be different.

1. The side entry pits, grate structures and pipes (including any associated blockage) that contribute to the internal network do not govern the capacity of the external drainage network, i.e. they do not choke the flow below the capacity of the external culverts.
2. There is no head loss across pit structures (details of type and quantity of pit structures are unknown).
3. There is no blockage of the external pipe network.
4. With only limited information available on the existing drainage network, the following assumptions were made:
   - Pipe material: concrete;
   - Pipe roughness: Manning's n = 0.015;
   - Pipe grade: 1 in 200 – analysis relatively insensitive to this parameter as network surcharges and operates under pressurised conditions when near capacity;
   - Depth to pipe invert: 1.5m;
   - Pipe vents to atmosphere (assumed that river level does present a tail water condition).
5. Time of concentration was estimated for developed catchments based on time of pipeflow and overland flow, for undeveloped catchments time of concentration was estimated using Australian Rainfall and Runoff (AR&R) Guidance for NSW: $t_c=0.76A^{0.38}$ where t_c = the time of concentration (min) & A = catchment area (km^2)

2 Stormwater

Table 2-6 Estimated Times of Concentration

Catchment	Time of Concentration (min)
A	15.0
B	13.7
C	11.0
D	17.4
E	12.0
F	13.6

Notes:
1) Time of concentration is not based on site data, for example, location of pipe entrances.
2) Land use for Area G will remain unchanged. It is therefore excluded from the drainage analysis areas have been taken from satellite images and available plans. Actual areas may vary slightly from what is shown.

The stormwater drainage network modelled in DRAINS is summarised in Table 2-7 and was derived from the existing drainage plan (Appendix A) and estimates of drainage catchments.

Table 2-7 Summary of Pipe Network Assumptions

Catchment	Upstream Catchment (on site)		Downstream Catchment (off site)		
	Pipe Diameter (mm)	Pipe Length (m)	Catchment Area (ha)	Pipe Diameter (mm)	Pipe Length (m)
A	600	30	0.6	750	85
B	600	60	0.7	1200	160
C	600	30	0.6	600	100
D	600	30	0.5	750	135
E	600	30	1.0	600	85
F	600	30	0.6	600	125

Note: As the land use for Area G remains unchanged it has been excluded from the drainage analysis.

The downstream catchment in all models was considered to have an entirely impervious surface as they are largely composed of sealed carriageways.

2.6.2 Results

The peak flow rates were determined in the underground drainage network and overland in DRAINS. The flow rate at which overland flow initiated was identified for each catchment to estimate the capacity of each catchment to freely drain. The storms with the following Average Recurrence Intervals (ARI) were modelled:

- 1 year ARI;
- 5 year ARI;
- 10 year ARI; and
- 20 year ARI.

Existing Conditions

The ARI of stormwater that each outfall can pass under existing site conditions, without surcharging to the point of contributing to overland flow, was estimated and is provided in Table 2-7. Further results are provided in Appendix C. Table 2-8 identifies the highest ARI at which overland flow is not anticipated under existing conditions.

Table 2-8 Pre-development Drainage Level of Service

Catchment	ARI (Years)
A	<5
B	~20
C	<5
D	>20
E	<20
F	>20

Note: Land use for Catchment G will remain unchanged. It has therefore been excluded from the drainage analysis.

The more densely developed western catchments (A, C & E) result in overland flow initiating at a lower ARI compared to the less developed eastern catchments. This is due to the higher proportion of impervious surfaces having a greater contribution to runoff.

Based on this preliminary analysis, the outfalls in Catchments A and C appear to have less capacity than required to pass a 5 year ARI rainfall event. However, IPL advises no significant impacts on operations due to any ponding.

Developed Conditions

The ARI of stormwater that each outfall can pass post-development, without surcharging to the point of contributing to overland flow was estimated (Table 2-9). Further results are provided in Appendix C.

Table 2-9 identifies the highest ARI at which no overland flow is initiated for the proposed development scenario. Note that no account has been taken of stormwater capture and reuse or firewater storage in this review. Consequently, the levels of service indicated in Table 2-9 are conservative. The intention is to establish a 20 year ARI level of service in the pipe network during detailed engineering design. This could be achieved by upgrading existing NPC infrastructure, use of water tanks, infiltration, etc.

Table 2-9 Post-Development Drainage Level of Service

Catchment	ARI (Years)
A	<5
B	~20
C	<20
D	~20
E	<5
F	>20

Note: Land use for Catchment G will remain unchanged. It has therefore been excluded from the drainage analysis.

The level of service for Catchment C appears to be increased compared with pre-development conditions due to diversion of some of its catchment to drain through Catchment E, and to bunded areas ultimately discharging to Catchment D if the quality is appropriate. However, the combination of an increase in areas draining to Catchment E and the small increase in impervious area conservatively appears to reduce the level of service for the drainage system in that catchment. This reduction in level of service is caused by a small volume of runoff that can adequately be managed by measures such as capture of roof runoff, use of permeable paving, etc. This would be more closely examined during detailed engineering design to ensure the design level of 20 year ARI level of service is maintained.

2.7 Estimation of First Flush

Consistent with the parameters adopted by JBS Environmental (2005), it was assumed that 40% of rainfall that falls on pervious areas results in runoff and 100% of rainfall that falls on impervious areas results in runoff. The proposed first flush will be captured from 'Moderate' (see Section 2.2) contaminated areas only. The areas that are proposed to be captured include the road from the weighbridge around the AN store and container store areas. Roof runoff will be separated and discharged to a separate outlet. These areas fall within Catchment D as shown in Figure 2.2. The equation below was used to estimate the first flush volume.

$$Q = I*(A_I + 0.4*A_P)$$

where:

Q = Volume of runoff

I = Depth of rainfall to be captured as first flush

A_I = Area of impervious surface

A_P = Area of pervious surface

The first 10mm of rainfall is proposed to be intercepted and treated as first flush. There is 0.72 ha contributing to the first flush and it is assumed that 100% of the runoff will be collected. The required storage volume was calculated to be 72 m^3.

2.8 Contaminated Water in Emergency

Should a loss of containment or fire occur on site it will most likely be restricted to bunded plant areas.

Fire water will be sourced from the firewater ring main.

Contaminated water would be captured within the bund and sump system from which it would be pumped out and removed from site. To ensure the capture of contaminated water spills, a Contaminated Water Storage is proposed to be built parallel with the stormwater drainage system. The proposed storage would have a conservative capacity (250 m^3) and capture any other contaminated water flow resulting from a fire or loss of containment incident. The stormwater system would include emergency isolation valves preventing discharge to the Hunter River.

No significant environmental impacts are expected from contaminated water management during emergencies due to these design mitigation measures.

2.9 Annual Stormwater Runoff

The annual stormwater runoff from the site under existing conditions that is expected to outfall to the Hunter River was estimated, assuming no evaporation and the runoff relationships adopted in this report.

Mean annual rainfall at Newcastle Nobbys Signal Station AWS of 1134.1 mm (Bureau of Meteorology website, 2011) has been used in the following calculations.

Annual stormwater runoff volume under existing conditions is estimated in Table 2-10.

Table 2-10 Estimated Annual Site Runoff - Existing Conditions

Catchment	% Catchment Contributing to Runoff	Catchment Area (ha)	Runoff (ML/yr)
A	87%	8.7	85.8
B	60%	3.5	23.8
C	68%	5.0	38.6
D	47%	6.1	32.5
E	51%	6.5	37.7
F	47%	4.3	23.0
G	54%	2.7	16.6
TOTAL		**36.8**	**258.0**

Note: The stormwater runoff has been estimated for the catchment areas within the site boundary. The downstream catchment areas, such as road pavements, have not been considered as part of the site to be managed and therefore not included in the runoff volumes.

Annual stormwater runoff from the site under post-development conditions is estimated in Table 2-11.

Table 2-11 Estimated Annual Site Runoff - Proposed Conditions

Catchment	% Catchment Contributing to Runoff	Catchment Area (ha)	Runoff (ML/yr) (rounded)	Change (ML/yr)
A	87%	8.1	79.9	-5.9
B	60%	4.2	28.6	4.8
C	68%	3.0	23.1	-15.4
D	47%	7.9	40.7	8.1
E	51%	6.9	40.0	2.3
F	47%	4.1	21.9	-1.1
G	54%	2.7	1.6	0.0
TOTAL		**36.8**	**248.8**	**-7.2**

Note: The full catchment D area including bunded areas and the first flush catchment are included to be conservative.

With the proposed site development, the runoff is expected to decrease slightly by approximately 7.2ML per year.

2.10 Conclusions

The stormwater analysis completed in this Chapter is conservative as no account has been taken of the use of infiltration for some areas, or clean water capture and reuse.

The existing drainage system will generally be sufficient to manage water within current levels of service since the drainage will be separate from the proposed development. Buildings and other infrastructure on the western side of the site will have essentially the same impervious area as current conditions.

Final design of stormwater systems for the proposed development on the east of the site will take into account any earthworks, increase in impervious area, and separation of water into each of the three quality streams. Water sensitive urban design features, infiltration, and water capture and reuse may be needed to ensure an appropriate level of service for the site is maintained.

Flood Risk

3.1 Introduction

This Chapter assesses the flood risks associated with the site and the potential of the proposed development to impact on these flood levels.

3.2 Overview

Flooding is a natural process and can happen at any time in a wide variety of locations. It constitutes a temporary covering of land not normally covered by water and presents a risk when people, human and environmental assets are present in the flooded area. Assets at risk from flooding can include housing, transport and public service infrastructure, commercial and industrial enterprises, agricultural land and environmental and cultural heritage.

Flooding can occur from one or more of the following sources:

- Fluvial (riverine) - inundation of floodplains from rivers and watercourses; inundation of areas outside the floodplain due to influence of bridges, embankments and other features that artificially raise water levels; overtopping or breaching of defences; blockages of culverts; blockages of flood channels/corridors;
- Tidal - sea; estuary; overtopping of defences; breaching of defences; other flows (e.g. fluvial surface water) that could pond due to tide locking; wave action;
- Surface water - surface water flooding covers two main source including sheet runoff from adjacent land (pluvial) and surcharging of sewer (foul and surface water);
- Groundwater - water table rising after prolonged rainfall to emerge above ground level remote from a watercourse; most likely to occur in low-lying areas underlain by permeable rock (aquifers); groundwater recovery after pumping for mining or industry has ceased;
- Infrastructure failure - reservoirs; canals; industrial processes; burst water mains; blocked sewers or failed pumping stations; and
- Tsunami – impact on the east coast of Australia from a tsunami arising from subduction zone earthquakes in the Pacific.

Different types and forms of flooding present a range of different risks. Flood hazards from the speed of inundation, flood depth, flow velocity and duration of flooding can vary greatly. With climate change, the frequency, pattern and severity of flooding are expected to change and become more damaging.

3.3 Potential Flood Mechanisms Relevant to the Site

Potential flood mechanisms relevant to the Kooragang Island development site include:

- Fluvial (Riverine);
- Sea/Tidal;
- Tsunami; and
- Surface water.

3.3.1 Fluvial (Riverine) Flooding

The Hunter River has a large catchment area (~21,500 km^2) that delivers significant flood volumes to the Hunter River Estuary. Whilst some headwater dams have reduced flood volumes to some extent, significant flood volumes continue to pass into the estuarine environment.

Flood hazards at Kooragang Island and on low lying areas adjacent to the estuary are determined not only by the volume of catchment runoff, but also the timing and magnitude of tides during these events.

3.3.2 Sea/Tidal Flooding

Rivers flow into the sea through estuaries in which the tidal cycle heavily influences flow. Flooding that occurs in estuaries can be complex and difficult to predict. It is influenced not just by the volume of water travelling down the catchment through the river system, but also by the height and timing of tides and tidal surges. Tidal surges are caused by regional weather conditions such as pressure systems, wind direction and speed and local bathymetry (depth of the sea and estuary). The way the sea and river interact within the estuary not only causes a flood risk within the estuary itself, but the effects can also extend well beyond the immediate area. This is because of the effects of tide locking.

Tides follow a range of daily and seasonal patterns, which aids in their prediction. However, severe storms or extreme high tides during storm surges are more difficult to predict. The risk associated with coastal flooding depends on a number of factors, often in combination including: the height of tides, weather systems, wind and wave conditions, topography, and the effectiveness of drainage systems.

Flooding from the sea and tidal waters is often more severe than flooding from watercourses due to the hazards associated with potential flood velocities and resulting depths. Salt water flooding also causes greater damage to properties than fresh water.

3.3.3 Tsunamis

Tsunamis are another flood risk factor arising from the sea. Tsunamis arise from subduction zone earthquakes. The risk to Australian waters arises from such zones in the Indian and Pacific Oceans, which are known to have produced major tsunamigenic events in recorded history and are the most likely sources of future events.

3.3.4 Surface Water/Flash Flooding

Flooding of land from surface water runoff is usually caused by intense rainfall that may only last a few hours. The resulting water follows natural valley lines, creating flow paths along roads, through, and around developments and ponding in low spots, which often coincide with fluvial floodplains in low-lying areas. All surface water flooding on the proposed development site at Kooragang Island would be attributed to exceedance of the design capacity of the stormwater system which is to be designed for a 20 year ARI level of service. The stormwater system is discussed in Chapter 2. Any stormwater flooding would be small by comparison with tidal or riverine flooding at this site.

3.4 Existing Flood Risk

A number of sources of information were used to assess the existing and future flood risk to the site. These sources include:

- Newcastle Flood Planning Stage 1: Concept Planning, City of Newcastle Council, NSW, 2009; and
- Flood Risk Management Guide: Incorporating Sea Level Rise Benchmarks in Flood Risk Assessments, Department of Environment and Climate Change NSW, 2009.

3 Flood Risk

The Flood Planning Stage 1 study completed hydraulic modelling of tidal, fluvial and surface water/flash flooding of the Newcastle City Council administrative area, including Kooragang Island.

The Floodplain Development Manual (NSW Government, 2005) does not provide any methods to identify which parts of the floodplain act as:

- Floodway;
- Flood storage; or
- Flood fringe.

City of Newcastle (2009) adopted the following criteria consistent with the broad guidelines in the Floodplain Development Manual, Newcastle Council's Flood Policy and Development Control Plan, and taking into consideration the following factors:

- Peak flood velocity;
- Peak flood depth;
- Peak velocity*depth;
- Peak energy head ($v^2/2g+d$);
- Cumulative volume conveyed during the flood event; and
- Combinations of the above.

Table 3-1 Criteria for Floodway, Flood Storage and Flood Fringe (NCC, 2009)

Defined Area	Criteria	Explanation
Floodway	Velocity*Depth>1.0 m^2/s	Areas and flow paths where a significant proportion of flood waters are conveyed.
Flood Storage	Velocity*Depth<1.0 m^2/s and Depth >1.0m	Areas where floodwaters accumulate before being conveyed downstream. These areas are important for detention and attenuation of flood peaks.
Flood Fringe	Velocity*Depth < 1.0 m^2/s and Depth <1.0m	Areas that are low velocity backwaters within flood plain. Filling of these areas generally has little consequence to overall flood behaviour.

The majority of the site has been classified as flood fringe, with the exception of the most westerly strip of the site, which has been classified as flood storage.

The hydraulic modelling outputs were used to define a set of hydraulic behaviour thresholds, which represent a combination of flood depth and flood velocity. These thresholds are provided in Table 3-2.

Table 3-2 Hazard Categories and Indicative Suitability of Land (NCC, 2009)

Hazard Category	Velocity and Depth Characteristics	Indicative Use Suitability
H1	v<0.5m/s & d<0.3m	Hydraulically suitable for parked or moving cars.
H2	v<2m/s, d<0.8m & v<3.2 – 4*d	Hydraulically suitable for parked or moving heavy vehicles only, and for wading by able-bodied adults.
H3	v<2m/s, d<2m, v*d<1	Hydraulically suitable for light construction (e.g. timber frame and brick veneer), but not for vehicles or wading.
H4	v<2m/s, d<2.5m, $v*d<2.5m^2/1$	Hydraulically suitable for heavy construction only (e.g. steel frame and reinforced concrete).
H5	Remainder	Generally unsuitable for any construction type.

Other ratings on potential loss of life and property damage were also developed. Kooragang Island was rated L1 - low risk to life due to sufficient time being available to remove people from the risk to their lives under Probable Maximum Flood (PMF), riverine or sea flooding.

3.4.1 Fluvial and Tidal Flooding

Based on an analysis of data from City of Newcastle (2009) and local survey information for the proposed development site it was found that:

- Extreme riverine flooding (PMF) of the Hunter River would expose some areas on the eastern part of the site to category H1, H2 and H3 hazards, and some areas on the west of the site to hazard categories H2 and H3.
- Sea/tidal flooding would expose some areas to the east of the site to inundation in hazard categories H1 and H2, and some areas on the western side of the site to category H2 and H3 flooding.

The City of Newcastle issued a Flood Information Certificate to IPL dated October 2011 which identifies the expected flood levels for the 100 year ARI and the PMF levels from both flooding of the Hunter River and Tidal Flooding. The certificate shows that the entire site is at risk of flooding. The hydraulic behaviour categories are explained in Table 3-2 above. For this site, the hazard categories are H4 for Hunter River flooding and H3 for Tidal flooding.

3.4.2 Surface Water/Flash Flooding

If the amount of impermeable area on the site is increased with no compensatory or mitigation measures such as water sensitive urban design surface water management techniques, then it is possible the risk of flooding from surface water could increase. See Chapter 2.

3.5 Tsunami Hazard

Tsunami risk profiles around the Australian coastline are represented by offshore tsunami hazard maps that have been prepared by Geosciences Australia, under its Probabilistic Tsunami Hazard Assessment (PTHA)[2] program. This provides the likelihood and relative tsunami amplitude at the 100m depth contour around the coastline. This work focuses on the hazard arising from the main source of tsunami risk; subduction zone earthquakes, but does not consider other lower probability and less predictable tsunamis risk factors such as volcanoes, asteroids, submarine landslides or non-subduction zone earthquakes, While the tsunami hazard maps provide a relative offshore tsunami hazard around Australia, the maps are not intended to determine the inundation extent, run-up, damage or other onshore phenomena that may result from a tsunami event (but could be used as the basis to derive this).

In order to more quantitatively assess the risk to the site and potential impact arising from tsunamis, a detailed inundation model would be required for the Hunter River Estuary, taking into account the detailed local bathymetry and topography. A detailed inundation model such as this would normally be prepared to consider the regional risk, rather than specifically focussed on an individual site, and such a model has not been prepared for the Hunter River Estuary. The Tsunami hazard for the offshore area adjacent to Newcastle, derived from the PTHA data, is presented in Table 3-3. Table 3-3, derived from the PTHA maps, indicates the maximum tsunami amplitude which could be expected at an adjacent offshore location (100m depth) in any given year for a stated probability or chance. As discussed previously, the extent to which the approximate tsunami amplitudes provided in Table 3-3 may influence the site has not specifically been assessed.

Table 3-3 Tsunami Hazard for the Offshore Region Adjacent to Newcastle

Percent Chance of Occurring in Any Year	Average Recurrence Interval (Years)	Maximum Tsunami Amplitude (Metres)*
1%	100	0.2
0.20%	500	0.6
0.10%	1000	0.8
0.05%	2000	1.1
0.02%	5000	1.6

*measured at 100m depth contour

3.6 Potential Impacts of Climate Change Induced Sea Level Rise to 2100

3.6.1 Flood Risk

The Newcastle Flood Planning Stage 1: Concept Planning document (July 2009), adopts a design sea level of 3.4m AHD to represent the PMF including an allowance for climate change. The plan states that the 100 year ARI sea level condition of RL 2.3m AHD, is approximated as the current peak recorded level within Newcastle Harbour (RL 1.4m AHD), plus a sea level rise projection of 0.9m - the recommended allowance for climate change to 2100 from the NSW Flood Risk Management Guide (August 2010).

[2] http://www.ga.gov.au/hazards/tsunami/offshore-tsunami-hazard-for-australia.html

3 Flood Risk

DHI Group carried out numerical ocean modelling in 2008 and estimated the extreme ocean flood level within Newcastle Harbour of 3.4m AHD, including a sea level rise allowance for climate change of 0.9m. This level is based on the coincidence of several extreme meteorological conditions, which has been assumed to have a very low likelihood of occurrence, approximately equivalent to a PMF event. URS have mapped this PMF level of 3.4m AHD, and the 100 year ARI plus climate change level of 2.3m AHD onto a Digital Elevation Model (DEM) created from contours obtained from existing site survey. Figures 3-1 show the potential flood depths for the PMF in 2100 on the site under pre-development site conditions. The areas indicated as being inundated are based purely on the ground survey data, a PMF level of 3.4m AHD and assumed hydraulic connectivity between ponded areas. The higher ground to the east of the site is clearly shown, and is appropriate for the secure location of Ammonium Nitrate storage.

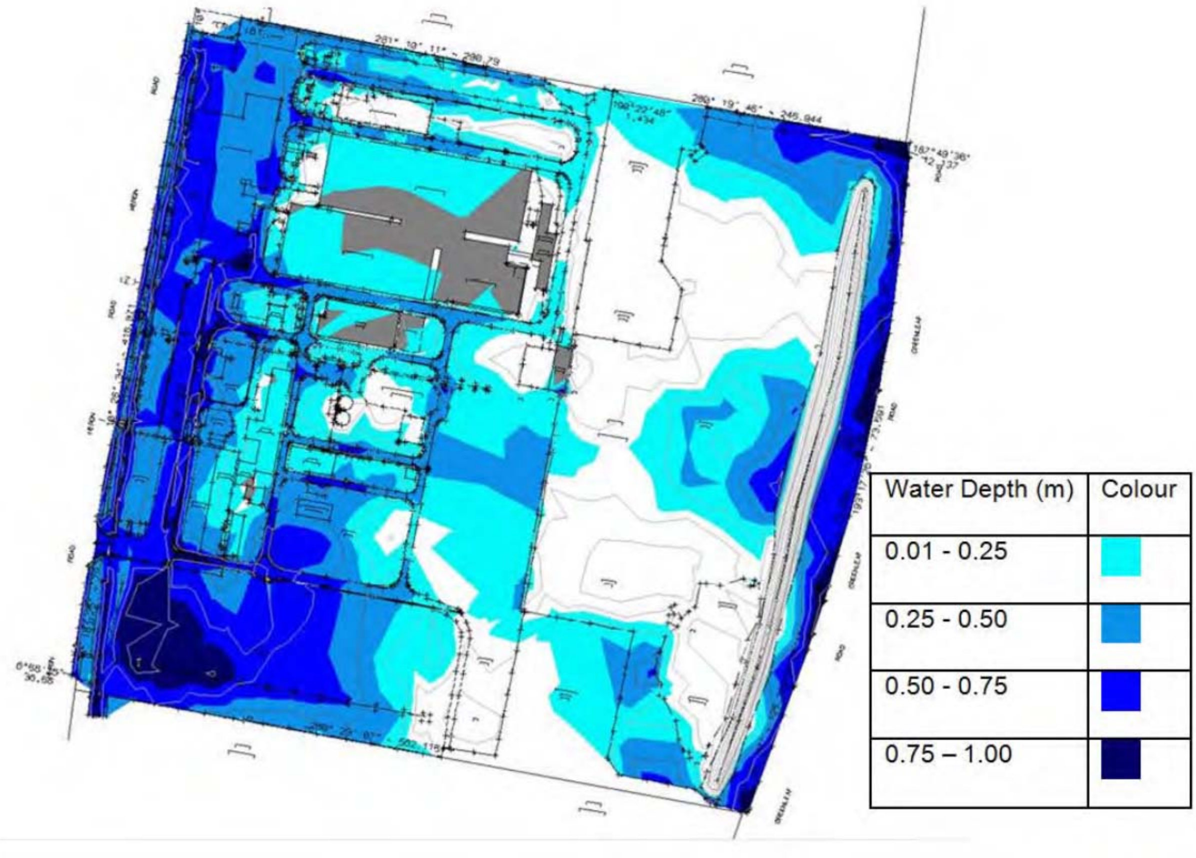

Figure 3-1 Potential Year 2100 Flood Depth Based on a 3.4m AHD tide level (Approximate PMF)

Figure 3-1 shows that there are some low spots on the development site with potential water depths of up to 1m for the PMF event only, but these are outside the footprint of the proposed major infrastructure. Some land forming as part of the construction could be introduced to minimise the extent of flooding of these areas. There will not be any flooding due to sea level in the 100 year ARI event (2.3m AHD) except that expected due to localised flooding caused by stormwater runoff.

3.6.2 Stormwater System Operability

Consideration has also been given to the impact of potential sea level rise, caused by climate change, on the operability of the site stormwater system.

The impact that rising sea level will have on the stormwater drainage capacity was assessed by modelling the drainage network in DRAINS with a constant tail water level. The tail water level adopted, as discussed in Section 3.6.1, was 1.4m AHD. It is predicted that by 2100 almost every high tide will exceed this level (BMT WBM, July, 2009). Under these conditions the level of service of the outfall drains for all catchments will reduce to less than a 1 year ARI and the drainage capacity becomes increasingly dependent on the driving head through the pipe network. If there was a requirement for the drainage outfall to drain freely and account for rising sea levels, then the entire drainage network of the Kooragang Island, including that of the site, would ultimately need to be redesigned to provide an acceptable level of service.

3.7 Conclusions

The majority of the site has been classified as flood fringe, that is: "*land that is affected by flooding after floodway and flood storage has been defined. Development within the flood fringe would not have any significant impact on the pattern of flood flows, and/or flood levels*." Floodplain Development Manual, NSW Government, 2002.

Based on an approximate analysis using spot survey data and flood information from City of Newcastle (2011) it has been shown that the site is at risk of flooding during the extreme flood events from both tidal and fluvial flooding. The adopted design criterion for paving of 3.5m AHD for the proposed development on Kooragang Island provides at least 1.2m freeboard above the 100 year ARI level (2.3m AHD) allowing for climate change, and 100mm freeboard above approximate PMF level (3.4m AHD). Furthermore much of the plant to be installed on site is modular and will sit in frames above the paving level. The risk of flooding is considered minimal. Some modifications to the stormwater drainage systems may be required on the site and more broadly on Kooragang Island in the longer term to accommodate potential climate induced sea level rise.

Water Supply and Usage

4.1 Introduction

This Chapter provides an assessment of water supply, water demand and water use efficiency measures considered as part of the proposed development based on an independent review of project documentation and process flow diagrams.

4.2 Water Supply

Water supply for the site is proposed to be sourced from the Hunter Water Corporation (HWC) mains water supply. HWC has indicated in preliminary discussions that water can be supplied to the site and the preliminary nominated connection point is the existing water main in Heron Road.

HWC potable quality water, generally meeting the requirements of the Australian Drinking Water Guidelines 2011 (endorsed by the National Health and Medical Research Council [NHMRC]), is supplied to the battery limit of the site. The HWC water supply is generally a high quality, low Total Dissolved Solids (TDS) potable water. TDS is generally less than 200 mg/L.

The water supply demand arising from existing infrastructure on the site (primary distribution centre) is approximately 150 litres per hour (L/hr) when the site is operating (47.5 hrs per week plus extended hours to meet variable demand). Existing demand is therefore typically less than 2 kilolitres per day (kL/d) and 0.5 mega litres per year (ML/yr).

The predicted water supply requirement for the new facility is about 180 kL/hr under normal operation, peaking to 220 kL/hr during start-up/shutdown operations, corresponding to an annual supply requirement of approximately 1,600 ML.

The only other supplementary water supply that would be reasonably potentially available to the site, other than HWC mains water, would be:

- Harvested roof runoff and stormwater; and
- Recycled water from HWC.

IPL is considering the potential use of harvested stormwater and has held discussions with HWC about the potential availability of recycled water. At this stage, the capacity of the Phase 1 HWC Kooragang Industrial Water Scheme is fully committed to other customers. Final decisions on the use of harvested/recycled water in the new facility will be made during the further design stages.

4.3 Water Demands

The water demand for the proposed development includes domestic potable water supply for employees, but is primarily for industrial demands. Water will also be required for the construction phase of the project.

4.3.1 Domestic Water Demand

Potable water will be utilised for domestic type uses on the site, including:

- Toilet flushing;
- Hand basins;
- Change-room showers;
- Lunchroom sinks;
- Drinking water; and

- Supply to eye washes and safety showers.

An estimate of 15 kL/d demand (5.5 ML/yr) has been made assuming a per capita consumption of 150 L/cap/d for 100 persons typically present at the site in any 24 hour period.

4.3.2 Industrial Water Demands

Industrial water demands comprise the significant majority of the overall site water demand. The main use is for make-up water to the site cooling towers, with feed water to demineralised water supply systems a significant secondary use. The main industrial water demands are presented in Table 4-1.

The cooling water is recirculated around the site for process cooling applications (via heat exchangers, etc). The water is cooled in forced draft evaporative cooling towers. A portion of the water is lost from the towers to the atmosphere by evaporation and a bleed, referred to as blow-down, is required to control the build-up of dissolved solids in the cooling water. Make-up water is required to maintain the system volume.

The demineralised water plant produces high (HP) and low (LP) pressure demineralised water supplies, which are then used for a range of specific process demands including boiler feed water, chilled water system and closed cooling water system make-up, and in the nitric acid plant absorption tower. The demineralised water plant removes dissolved solids (TDS) from the feed water utilising a mixed bed ion-exchange process. The dissolved solids in the water are adsorbed onto the ion-exchange resins and the product water is effectively “mineral” free. This prevents scaling (deposition) in steam system (boiler) applications and product quality impacts in the nitric acid plant.

Table 4-1 Summary of Industrial Water Demands

Water Demand	Specific Uses	Quantity	Comment
Cooling Towers Make-up	• Recirculated evaporative cooling water system	170 kL/hr continuous (~1460 ML/yr). 160 kL/hr will be provided by HWC water and the balance by recycled water (refer to Section 4.4).	Corresponds to about ~91% of the total site water demand. About 83% of this water is lost by evaporation, based on Cooling Tower operation at 6 cycles of concentration.
Demineralised Water Plant Feed	• Nitric Acid Plant Absorption Tower injection (HP). • Process Gas Compressor Inlet Pipe injection [for deposition control] (HP). • Nitric Acid Plant Steam System make-up (LP). • Auxiliary Boiler make-up (LP). • Chilled water circuit make-up and at filling lines (HP & LP). • Closed (secondary) cooling water circuit make-up and at filling lines (HP & LP).	Demineralised Water Plant capacity up to 20 kL/hr. High (HP) and Low (LP) demineralised water supplies produced.	
Other	• Pumps Gland Water. • Laboratory Water Supply. • Utility Water. • Fire Water.		

4.4 Water Usage Efficiency Measures

IPL has recognised that the water demand for the proposed development is significant and has considered ways of minimising water consumption in the design process. Measures that are proposed to be implemented include:

- The majority of the steam condensate would be recovered and returned to the boilers as feedwater.
- Process condensate from the AN Plant will be recovered and used in the NA Plant absorber and AN plant scrubbers.
- A number of process wastewater streams would be recycled as cooling water system make-up, namely:
  - Nitric acid plant steam drum blowdown;
  - Auxiliary boiler blowdown;
  - Clean process condensate from AN Liquor plant; and
  - Stormwater first flush.
- Closed circuit cooling water systems for some applications (chilled water circuit and secondary cooling water circuit).

As discussed in Section 4.3.2, the main water demand is the make-up to the recirculated evaporative cooling water system. The recycled water streams will provide about 10 kL/hr of the 170 kL/hr make-up to the cooling towers. The current cooling water demand assumes six (6) cycles of concentrations (the ratio of make-up to blow-down). After plant commissioning, IPL would investigate opportunities to further reduce water consumption in the cooling towers by operating them at higher cycles of concentration (e.g. operating at 9 cycles of concentration rather than 6 would reduce the make-up water demand from about 170 kL/d to 160 kL/d). This essentially involves operating the system at higher dissolved solids concentrations, which has potentially significant implications for equipment longevity, maintenance (corrosion, scaling, etc.), operating costs (additional cleaning, modified treatment chemicals regime) and wastewater quality; factors, which need to weighed against the potential water saving.

IPL has also considered other options that would potentially more significantly reduce the cooling water requirement, namely use of once-through cooling water from the Hunter River, and a fully closed circuit cooling system utilising air (fin-fan) coolers rather than evaporative cooling towers. These were identified as not viable based on economic and environmental impact considerations (in the case of once through cooling).

IPL will continue to investigate other opportunities to reduce water consumption throughout the detailed design process.

4.5 Construction Phase

Water supply would be required during the construction phase for construction use and general workforce amenities. This water would be potable water supplied by HWC.

During normal periods there would be demand of approximately 12 kL/d. Peak demand would be for hydrotesting of constructed tanks and may be in excess of 5,000 kL per day for short periods, with a total demand estimated at approximately 60 ML. There is also a provision for 1,700 kL of demineralised water to be utilised if required.

Wastewater Generation and Management

5.1 Introduction

This Chapter provides an assessment of wastewater generation and management for the proposed development based on an independent review of the process flow diagrams and designs.

5.2 Wastewater Sources

Similar to the water demands, the main wastewater sources at the site are those arising from industrial sources, and primarily the blow-down from the cooling water systems. The process design has been optimised to eliminate process wastewater from the NA and AN plants where possible.

5.2.1 Domestic Wastewater

Wastewater arising from ablution facilities and other domestic type uses, described in Section 4.3.1, will be discharged to existing and new septic systems. Sludge will be periodically collected and tankered offsite by a licensed waste contractor. The southern end of Kooragang Island is not sewered and Hunter Water has no plans to install sewers.

The existing operations on the site are serviced by septic systems that are designed with a capacity to service up to 250 equivalent persons (e.p.).

It is proposed to supplement the existing septic systems with new systems as part of the new development. Although the existing septic systems have sufficient capacity to service the whole site, they are not conveniently located relative to the new development.

Newcastle City Council allows pump-out systems for industrial and construction sites and will accept septic systems to current design standards. It has no standards of its own and an application to it for approval of on-site disposal requires a site and soil evaluation to support the proposed installation. This will be completed as part of the further design process of the development. The septic systems will be designed, including siting and soil assessments, in accordance with the requirements of *Australian/New Zealand Standard AS/NZS 1547:2012, On-site Domestic Wastewater Management* and will address the principals and requirements of NSW *Environment & Health Protection Guidelines - Onsite Sewage Management for Single Households* (referred to as the 'Silver Book'), as applicable.

In the event that a suitable location(s) for additional septic systems cannot be identified in the vicinity of the new development, then the existing septic systems, which have more than sufficient capacity, would be utilised. A pumped reticulation system will be required to transfer sewerage from the area of the new development to existing systems.

5.2.2 Industrial Wastewater

The main process related wastewater streams produced by the proposed development are summarised in Table 5-1.

Table 5-1 Summary of Industrial Wastewater Streams

Wastewater Source	Specific Source	Destination	Characteristics
Cooling Towers	Blowdown from recirculated cooling water system – bleed to control water quality.	Wastewater system.	~28.3 kL/hr continuous discharge (based on ~6 cycles of concentration in the cooling towers). Main wastewater stream arising from the site. Source of nitrate (typically ~30 mg/L, as NO_3),) and phosphate (~10 mg/L, as PO_4).
	Cooling tower filter backwash.	Wastewater system.	Intermittent. Minor Elevated total suspended solids (TSS).
Demineralised Water Systems Regeneration	Ion-exchange beds (mixed) regeneration wastewater.	Wastewater system.	Intermittent. Average flow ~0.1 kL/hr. Requires pH adjustment (neutralisation) prior to discharge to the wastewater system. Elevated TDS.
Process Condensate	Ammonium Nitrate Liquor Plant (Pumps 5A/B).	Recycled as cooling tower make-up.	Continuous. Approximately 8.6 kL/hr. Ammonium nitrate traces.
	Ammonium Nitrate Liquor Plant (Pumps 6A/B).	Reused in AN Prill Plant and Nitric Acid Plant.	Continuous. Approximately 15.6 kL/hr. Ammonium nitrate traces.
	AN Prill Plant Chiller.	Recycled as cooling tower make-up.	Intermittent. Approximately 1.9 kL/hr.
Ammonia Strippers	Nitric Acid Plant.	Collected in IBC or tank for off-site disposal by licensed waste contractor.	Intermittent. Minor. May contain traces of oil and ammonia. Average of 10 L/hr.
	Ammonium Nitrate Liquor Plant.	Collected in IBC or tank for off-site disposal by licensed waste contractor.	Intermittent. Minor May contain traces of oil and ammonia. Average of 5 L/hr.
Boiler Blowdown	Nitric Acid Plant Steam Drum.	Recycled as cooling tower make-up.	Continuous discharge of approximately 1 kL/hr. Source of phosphate (~10 mg/L, as PO_4).
	Auxiliary Boiler	Recycled as cooling tower make-up.	Continuous discharge of approximately 0.2 kL/hr. Source of phosphate (~10 mg/L, as PO_4).
Instrument Air and Plant Air systems condensate		Water condensate from intercooler/ aftercooler recycled to cooling tower.	Minor volume ~10 L/hr.
Laboratory Wastewater		Wastewater system	Minor.

Wastewater Source	Specific Source	Destination	Characteristics
Process Area Bund Sumps	Sumps with potentially concentrated wastewater.	Collected in sump or tank for recovery (waste ANSOL handling) or for off-site disposal by licensed waste contractor. May be discharged to wastewater system after confirmation of suitable quality.	Intermittent.
	Oily or potentially oily areas.	Wastewater system, via local oil water separators	Intermittent.
	Potential for moderate nitrogen impacts in wastewater.	Wastewater system.	Intermittent.
	Normally 'clean' areas.	Stormwater system after confirmation of quality.	Intermittent.
First flush stormwater	Stormwater runoff from areas with potential for moderate nitrogen quality impacts (refer to Chapter 2).	If quality is suitable first preference would be to utilise first flush water for cooling tower make-up. Alternative destinations based on quality, would be the Wastewater System, and potentially to stormwater if water quality meets specification.	Intermittent. Minor Average load of 260 L/d.

The typical total daily wastewater volume is predicted to be about 750 kL/d, (~270 ML/yr), about 680 kL/d of which (i.e. ~91%) will derive from the cooling water system blowdown.

The wastewater discharge streams only represent about 17% of the water that is supplied to the site from HWC mains. The balance is lost as moisture / water in product, but primarily to the atmosphere as evaporation from the cooling tower systems.

5.2.3 Stormwater

Stormwater management on the site generally is discussed separately in Chapter 2. It is noted in Table 5-1 that stormwater collected in the proposed first flush system (described in Chapter 2) would normally be directed to the cooling water system as make-up, but alternatively to the wastewater system, as it may contain ammonium nitrate. On average, this would constitute an insignificant volume to the cooling water or wastewater system.

5.3 Wastewater Management

Wastewater that is discharged to the plant wastewater system will be collected and treated in the wastewater treatment plant, prior to being discharged either via the proposed Hunter River outfall, (as described in Chapter 6) or disposed of offsite by a licenced waste contractor.

5.3.1 Sanitary Wastewater

It is proposed to collect and treat sanitary wastewater in onsite septic systems. These will be periodically desludged, with the waste disposed offsite by a licensed waste contractor.

5.3.2 Process Wastewater Treatment

The plant process wastewater treatment plant comprises the following processes:

- Equalisation;
- Oil/sediment separation; and
- pH correction (neutralisation)
  - acid dosing; and/or
  - alkali dosing.

The quality will be monitored prior to discharge in accordance with licence conditions. Provision will be made to store off-specification wastewater prior to disposal.

5.4 Wastewater Quality

The wastewater from the site, to be discharged to the Hunter River via an outfall, as discussed in Chapter 6, is predicted to typically meet the characteristics presented in Table 5 2. The predicted quality of the wastewater has been derived by IPL from mass balance, with consideration of the expected performance of the wastewater treatment plant. The quantity and characteristics of the individual source wastewater streams has been indicated in Table 5-1.

Table 5-2 Proposed Wastewater Quality

Parameter	Unit	Indicative Quality	Worst Case Quality
Temperature	deg C	28	35
pH	pH units	7.0 – 8.5	6.5 – 8.5
Total Nitrogen (as N)	mg/L	<75	150
Ammonia (as N)	mg/L	<37.5	75
Nitrate (as N)	mg/L	<37.5	75
Total Phosphate (as PO_4)	mg/L	<10	25
Total Suspended Solids	mg/L	<15	30
Oil & Grease	mg/L	<5	10
Salinity	MicroSiemens/cm	1,750	2,600

The normal volume of wastewater would be 750 kL per day with intermittent peaks up to 1500 kL per day from rainfall events.

The predicted quality of the wastewater has been derived by IPL from mass balance, with consideration of the expected performance of the wastewater treatment plant. The quantity and characteristics of the individual source wastewater streams has been indicated in Table 5-1. The quality is largely dictated by the cooling tower blowdown quality, as this comprises in excess of 90% of the load. The quality of the water in the cooling tower blowdown, is a function of the following factors:

- concentration effect of evaporation (variable depending on climatic conditions and selected operating cycles of concentration);
- primary feed water quality (HWC mains water);
- cooling water treatment additives (corrosion and scale inhibitors, biocide);
- quality of the wastewater streams (boiler blowdown and process condensates) recycled as cooling tower make-up;

- entrained solids (from atmospheric dust); and
- dissolved ammonium nitrate (from the air around the plant).

The concentration of total nitrogen from the cooling tower blowdown is predicted to be typically around 10 to 15 mg/L (as N), but potentially up to 35 mg/L associated with nitrogen species present in the cooling tower make-up water (HWC and recycled process wastewater streams), treatment additives, and entrainment and dissolution of ammonium nitrate for the atmosphere via the cooling tower inlet air. An additional nitrogen load allowance has been assumed to account for expected variable contributions from first flush stormwater and process area sumps wastewater, in particular.

The cooling tower blowdown will also be the main source of phosphate in wastewater. The concentration is typically predicted to be around 10 mg/L (as ortho-phosphate), which would be diluted slightly in final wastewater by other wastewater streams.

Oily water will be released to the wastewater system, as indicated in Table 5-1. Generally, these streams will be treated by local oil water separation, with an additional overall oil water separator in the wastewater treatment plant. The oil water separators to be employed will rely on gravity separation. Emulsification behaviour of oil is not expected as emulsifiers will not typically be used on the site.

Wastewater discharge will be continuously monitored with an automatic sampler and on-line for pH, temperature, volume and electrical conductivity.

5.5 Management of Wastewater Arising from Incidents (Firewater and Spills)

All process areas and material storage at the site would be adequately and appropriately sealed and bunded to prevent potential onsite and offsite impacts, including via the stormwater and wastewater systems. Some process areas are also roofed to prevent rainfall ingress. IPL has developed a bund drainage procedure that governs the direction of any liquids collected in a bunded area to ensure that they are appropriately managed based on their quality.

There are fixed fire protection systems present at the site. The bunding and containment systems have been designed to ensure that contaminated firewater that might be generated in an incident would be contained on site, to allow assessment of quality prior to appropriate disposal. The capacity of the first flush retention system and contaminated water pond has been designed in part to provide firewater retention, in addition to bunded process areas. The drainage systems are fitted with emergency isolation valves for use in an emergency to prevent offsite discharge.

5.6 Construction Phase

As discussed in Section 4.4, water supply will be required at the site for domestic purposes and particularly for hydrotesting of tanks and other equipment during commissioning. Potable quality water will be used for this purpose. The quantity will be relatively significant (~60ML) but the quality is not expected to be impacted by the testing, and it is proposed that this water, following quality testing, would be discharged to the Hunter River via the stormwater system.

Existing site facilities, augmented by portable utilities will be utilised by construction personnel for amenities, and wastewater will be ultimately be removed from site by licensed contractors.

Any wash water or other wastewater generated during the construction will be collected and disposed offsite by licensed contractors.

5.7 Nutrient Loads

Nutrients; nitrogen (N) and phosphorus (P), are discharged into the Hunter River Estuary from a very wide range of sources within the overall Hunter River Catchment, including from site activities; current and proposed. Site sources include stormwater, wastewater discharge, groundwater and atmospheric emissions. A high level mass balance assessment has been completed to consider the relative contribution to nutrient loads on the estuary, arising from current and proposed site activities.

5.7.1 Hunter River Catchment

Indicative nutrient contributions to the Hunter River arising from diffuse and point sources within the Hunter River Catchment were acquired from the National Pollution Inventory (NPI). Diffuse source, comprising the majority of the catchment contributions in this dataset derive from estimates made for the 1999/00 reporting period. These have not since been updated. The point source emissions data is for the 2010/11 reporting period. The National Pollution Database (http://www.npi.gov.au) was queried and the data is summarised in Table 5-3.

Table 5-3 Nutrient Loads on the Hunter River from Diffuse and Point Sources (NPI)

Nutrient Sources 2010/11 NPI	Nitrogen (kg)	Phosphorus (kg)
Grazing: steep, basalt and high rainfall-Hunter River [*]	669,719	82,425
Private Forests: low rainfall-Hunter River [*]	634,455	31,724
Cropping: basalt-Hunter River [*]	590,747	90,885
Grazing: basalt and high rainfall-Hunter River [*]	422,055	52,757
Grazing: low rainfall-Hunter River [*]	400,016	50,001
Grazing: steep and low rainfall-Hunter River [*]	373,308	46,663
Cropping-Hunter River [*]	361,904	55,676
Southern National Parks-Hunter River [*]	290,770	14,538
Grazing: steep and high rainfall-Hunter River [*]	217,390	27,173
Fertiliser and Pesticide Manufacturing [183]	210,057	-
Cropping on floodplains-Hunter River [*]	208,516	32,079
Urban-Hunter River [*]	170,526	22,002
Private Forests: steep and high rainfall-Hunter River [*]	168,035	8,402
Southern State Forests-Hunter River [*]	122,127	6,104
Grazing: high rainfall-Hunter River [*]	116,522	14,567
Northern State Forests-Hunter River [*]	63,012	3,150
Water Supply, Sewerage and Drainage Services [281]	54,291	13,643
Northern National Parks-Hunter River [*]	44,969	2,248
Mining and rehabilitation-Hunter River [*]	34,569	4,502
Wentworth Swamp STW-Hunter River [*]	14,353	-
Vineyards-Hunter River [*]	11,917	2,234
Singleton Shire Council STW-Hunter River [*]	10,490	
Raymond Terrace WWTW-Hunter River [*]	7,081	1,512

5 Wastewater Generation and Management

Nutrient Sources 2010/11 NPI	Nitrogen (kg)	Phosphorus (kg)
Shortland STW-Hunter River [*]	5,801	-
Dungog Shire Council STW-Hunter River [*]	4,226	1,798
Bolwarra STW-Hunter River [*]	3,053	2,485
Branxton STW-Hunter River [*]	1,716	147
Minmi STW-Hunter River [*]	951	2,801
Aberdeen STW - Scone Shire Council-Hunter River [*]	946	445
Merriwa Shire Council STW-Hunter River [*]	230	302
Murrurundi Shire Council STW-Hunter River [*]	164	88
Kearsley STW-Hunter River [*]	20	59
Cessnock STW-Hunter River [*]	-	1,821
Kurri Kurri STW-Hunter River [*]	-	953
Total (kg)	**5,213,936**	**573,184**
Total (t)	5,214	573

* Diffuse source estimates for the 1999/00 NPI reporting period.

Overall, the indicative load presented in Table 5-3 is likely to be an under-representation of the overall N contributions to the catchment load, in particular, as there will be significant contribution from atmospheric nitrogen e.g. oxides of nitrogen (NO_x), that is not fully accounted for in the estimate. Nevertheless, considering this, and also that the diffuse source nutrient load in the catchment may have varied somewhat since 1999, the table provides a reasonable order-of magnitude context whereby the significance of individual source contributions can be considered. The 210 tonnes per annum (tpa) reported nitrogen load derived from “Fertiliser and Pesticide Manufacture” is, for example, according to the NPI database, the reported emissions from the Orica Kooragang Island facility in the 2010/11 reporting period, representing about 4% of the total nitrogen load.

The overall annual Hunter River discharge is of the order of 1,900 GL/a. On this basis, the overall average N and P concentrations in the Hunter River, from the indicated loads, would be about 2.74 mg/L and 0.30 mg/L, respectively. Comparing these estimates with the water quality data presented in Table 6-1, it is apparent that the predicted P concentration is of the right order, and, as expected, the predicted N concentration from the load estimate is somewhat lower than that indicated by actual data.

5.7.2 Wastewater Discharge

There is no wastewater discharge outfall associated with the current site operations.

Wastewater from the proposed AN plant, as discussed in Section 5.4, will be discharged to the Hunter River via an outfall. The indicative water quality is provided in Table 5-2. In order to provide a reasonable load assessment, further consideration was given the potential concentration distribution of nutrients in the wastewater. This assessment is summarised in Table 5-4.

Table 5-4 Distribution of Nutrient Concentration in Proposed Wastewater Discharge

Nutrients	Unit	Percentile (%)*	Predicted Maximum Concentration
Total Nitrogen (as N)	mg/L	75	50
		90	100
		100	150
		Average	67.5
Total Phosphate (as P)	mg/L	90	3.3
		100	8.2
		Average	3.8

Indicative nutrient loads from proposed wastewater discharge have been derived assuming the typical daily discharge of 750 kL/d and estimated average concentrations of N and P (based on the percentiles indicated in Table 5-4), namely 67.5 mg/L N and 3.8 mg/L P. The corresponding estimated annual loads are as follows:

- Total Nitrogen: 18.5 tpa
- Total Phosphorus: 1 tpa

These represent about 0.35% and 0.17%, respectively, of the overall catchment loads presented in Table 5-3.

5.7.3 Stormwater

Existing Development

An estimate of the N and P discharge loads from the existing operations on the site can be derived from the stormwater quality monitoring data presented in Table 2-5, and the catchment yields presented in Table 2-10. Utilising the annual catchment yield, and the average contaminant concentrations for the respective catchments, estimates of the N and P loads are presented in Table 5-5.

Table 5-5 Nutrient Loads on the Hunter River from Existing Site Stormwater

Catchment*	Nitrogen Load (tpa)	Phosphorus Load(tpa)
A (EPL Monitoring Point 1)	7.4	11.2
C (EPL Monitoring Point 7)	2.5	2.7
E (EPL Monitoring Point 2)	2.4	4.5
Total	**12.3**	**18.4**

*as shown on Figure 2-1

These estimates are considered to be conservatively high, as they are effectively first flush concentrations rather than representative of the overall average concentrations (as explained in Section 2.5.1), which would be somewhat lower. It is notable that the indicated P load is higher than the N load, which is because the predominant fertilisers handled on the existing site are P based products. It is also notable that the site is adjacent to an existing Orica AN facility and deposited fugitive emissions from this facility may also be contributing to a portion of the N load from the site.

Future Development

In future, it is proposed that, in addition to the proposed new AN plant, the existing site operations will continue, and the stormwater drainage system currently present in this part of the site will broadly be retained in its current arrangement. As discussed in Section 2.5.1, IPL is in the process of implementing a pollution reduction plan (PRP) related to site stormwater.

The improvement measures to be implemented at the site have not yet been fully defined and therefore predicted improvements associated with implementation of the PRP have not been quantified. Nevertheless, it is expected that the contaminant loads from the western site catchments would be reduced to a measurable extent. Any load reductions achieved from the existing operations would potentially partly offset additional loads derived from the proposed development.

As discussed in Section 2, areas of the redeveloped eastern part of the site with potential for moderate quality impact will be subject to first flush collection and diversion. This will be directed to the wastewater system, and therefore the load is addressed in Section 5.7.2. The residual contaminant load from the remaining stormwater in this part of the site is expected to be relatively low, and would be mostly derived from fugitive particulate emissions, which will be largely accounted for under the associated atmospheric emissions load estimate presented in Section 5.7.5.

5.7.4 Groundwater

Groundwater from the site discharges to the Hunter River, and this groundwater contains some nutrients from site process reagents and possibly from septic systems. A high level estimate of nutrient loads associated with potential groundwater migration has been made based on the data presented in the Phase II ESA provided as Appendix G of the EIS. The focus area for this assessment was the proposed development area, rather than the existing development area. The only nitrogen species measured in the groundwater was ammonia.

The nutrient loads entering the Hunter River Estuary were determined using flow tube analysis. The groundwater flow contour map was divided into a number of discrete segments, with each flow cell being defined on the basis of groundwater flow vectors. The flow vectors were drawn perpendicular to the groundwater contours and directed to the corresponding arms of the Hunter River which are assumed to be the discharge points for subsurface flow. Since the site is an island, the flow is directed to the River on the western, southern and eastern sides. The flow field was divided into 9 flow cells and the rate of flow for each cell was determined using application of Darcy's Law and using hydraulic conductivity and hydraulic gradient determined on the basis of measurements in the observation bores. The cross sectional area of each flow cell was determined on the basis of the geological logs. Once the flow rate was calculated for each flow cell, then the load of nutrients was determined for that cell by multiplying the rate of flow by the concentration (for both ammonia and phosphate).

This analysis indicated that, relative to other sources, contributions from groundwater are insignificant. The predicted annual loads contributing to the Hunter River Estuary from these sources are:

- Total Nitrogen: 0.02 tpa;
- Total Phosphorus: 0.01 tpa.

As indicated in the first paragraph of this section, the predicted loads may be an under representation of the actual current and future loads. Nevertheless, this initial analysis has shown that even if the groundwater concentrations of N and P were markedly higher than assumed in the analysis, the overall controlling groundwater flux is sufficiently low that the loads contributing to the Hunter River are unlikely to become significant relative to other sources.

5.7.5 Atmospheric Emissions

The potential of airborne emissions to influence nitrogen levels in the Hunter River Estuary has been considered using basic estimates of particulate emissions from the proposed AN plant processes involving ammonium nitrate. There are no significant atmospheric phosphorus emissions associated with the new plant, so these have not been considered. The air quality assessment on which this is based in presented in Chapter 10 and Appendix E of the EIS. This estimate has been performed in accordance with the emission estimation methodology presented Section 6.3.2 of the air quality impact assessment for the project (Appendix E), with the following exceptions:

- Particulate matter emissions from combustion sources have been excluded as these are likely to have a negligible nitrogen content;
- An emission factor of 0.01 kg/t has been used for bulk load out activities, given the need to calculate Total Suspended Particulate (TSP) emissions. This emission factor has been sourced from US EPA AP42 Emission Factor Database (http://www.epa.gov/ttn/chief/ap42/ch08/bgdocs/b08s03.pdf);
- A conversion factor of 0.35 t nitrogen per t ammonium nitrate has been applied;
- Emissions from ANSOL concentration and bagging operations have been omitted on the basis that they are estimated to constitute <0.1% of total AN emissions.

Table 5-6 and Table 5-7 provide a summary of Nitrogen emission estimates for the AN bulk load out and AN plant (respectively).

Table 5-6 Nitrogen Emissions from AN Bulk Load Out

Parameter	Units	Value
AN Throughput	kg/h	28,0000
TSP Emission Factor	kg/h	0.01
TSP Nitrogen Emissions	hr/year	2,800
	tpa (AN)	2.8
	tpa (N)	1.0

Table 5-7 Nitrogen Emissions from AN Plant

Parameter	Units	Value
TSP Emissions	kg/h (AN)	3.91
	kg/h (N)	1.37
Operation	hr/year	8,400
TSP Nitrogen Emissions	tpa (AN)	11.5
	tpa (N)	4.0
TSP Nitrogen Emissions assumed to be contributing to the Hunter River Estuary	tpa (N)	3.0

Given that emissions from the bulk load out will occur at ground level, it has been assumed that these emissions will be deposited within the Hunter River catchment. This is a conservative assumption, as a portion of that which is deposited on the site will be captured by the first flush and will report to the wastewater load.

Emissions from the AN plant are expected to be subject to a greater degree of atmospheric dispersion due to the nature of these emissions, and would therefore be deposited over a wider area. The estimate of nitrogen deposition potential from the AN plant has been refined to exclude east-south-easterly and south-easterly winds (which occur 24% of the time), as it has been assumed that under these conditions, particulate emissions will not be deposited within the estuary catchment, but rather in the ocean. On this basis AN plant deposition potential is estimated at approximately 3.0 tpa.

The overall estimate nitrogen load contribution to the Hunter River estuary from atmospheric emission associated with the new plant is estimate as 4.0 tpa nitrogen. Again, this is a conservatively high assumption, as a proportion of the deposited nitrogen will not report to the estuary e.g. due to plant uptake.

An emission estimate has not been completed for the existing site operations. These are largely fugitive emissions associated with material handling, a reasonable proportion of which would be expected to be deposited on site. These would therefore tend to report to stormwater and are therefore partly accounted for in the estimated stormwater loads based on the monitored discharge quality.

5.7.6 Summary

The estimated nutrient loads on the Hunter River Estuary arising from current and proposed future site operations are summarised in Table 5-8.

Table 5-8 Indicative Total Contributory Nutrient Loads to the Hunter River from the IPL Site.

Load Source	Existing Development		Proposed Development	
	Nitrogen Load (t/a)	Phosphorus Load (t/a)	Nitrogen Load (t/a)	Phosphorus Load (t/a)
Wastewater Discharge	-	-	18.5	1.0
Stormwater	12.3	18.4	9.8*	14.7*
Groundwater	0.02	0.01	<0.1	<0.05
Atmospheric Emissions	-^	-^	4.0^	-^
Total	**12.3**	**18.4**	**32.4**	**15.8**
Hunter River Catchment	5,214	573	5,214	573
Relative Contribution (%)	0.24	3.2	0.62	2.8

*assumes a modest 20% load reduction through implementation of the PRP for the existing operations, and that additional load in stormwater from the new AN plant is largely accounted for under the wastewater discharge and atmospheric emissions contributions.
^not assessed from existing operations but partially accounted for in existing operations estimated stormwater loads

These estimates are indicative only but are considered to give a reasonable indication of the relative contribution of the various sources and order of magnitude of the overall contribution.

It can be seen from the table that the relative contribution of phosphorus from the site to the overall catchment load is relatively much higher than for nitrogen, with an estimate of the order of three percent. The limited contribution of additional phosphorus arising from the new development, and the expected load reduction from implementation of the PRP however mean that there is no expected increase in P load from the site in future.

For nitrogen on the other hand, there would be a relatively significant increase in the predicted contributory load arising from the proposed development, from about 12 tpa to 32 tpa. However, the relative contribution of nitrogen to the overall catchment load now and in the future is much lower at significantly less than one percent. As shown in Table 5-3, the relative contribution is equivalent to wastewater from a small sewage treatment plant.

Wastewater Discharges to the Hunter River

6.1 Introduction

This Section provides an assessment of the impacts of the proposed wastewater discharge (Table 5-2) to the South Arm of the Hunter River based on modelling by the Water Research Laboratory (WRL) at the University of New South Wales (UNSW) (refer to Appendix E). It outlines how the location of the discharge point was chosen and discusses potential issues.

6.2 Existing Water Quality in the Hunter River Estuary

After extensive searches and discussions with agencies it was concluded that the best currently available water quality data set for the Hunter River around Kooragang Island is that compiled by Sanderson and Redden (2001). The data consists of a variety of samples from sites in estuarine and freshwater areas. Sanderson and Redden classified sites into water quality zones as shown in Figure 6-1.

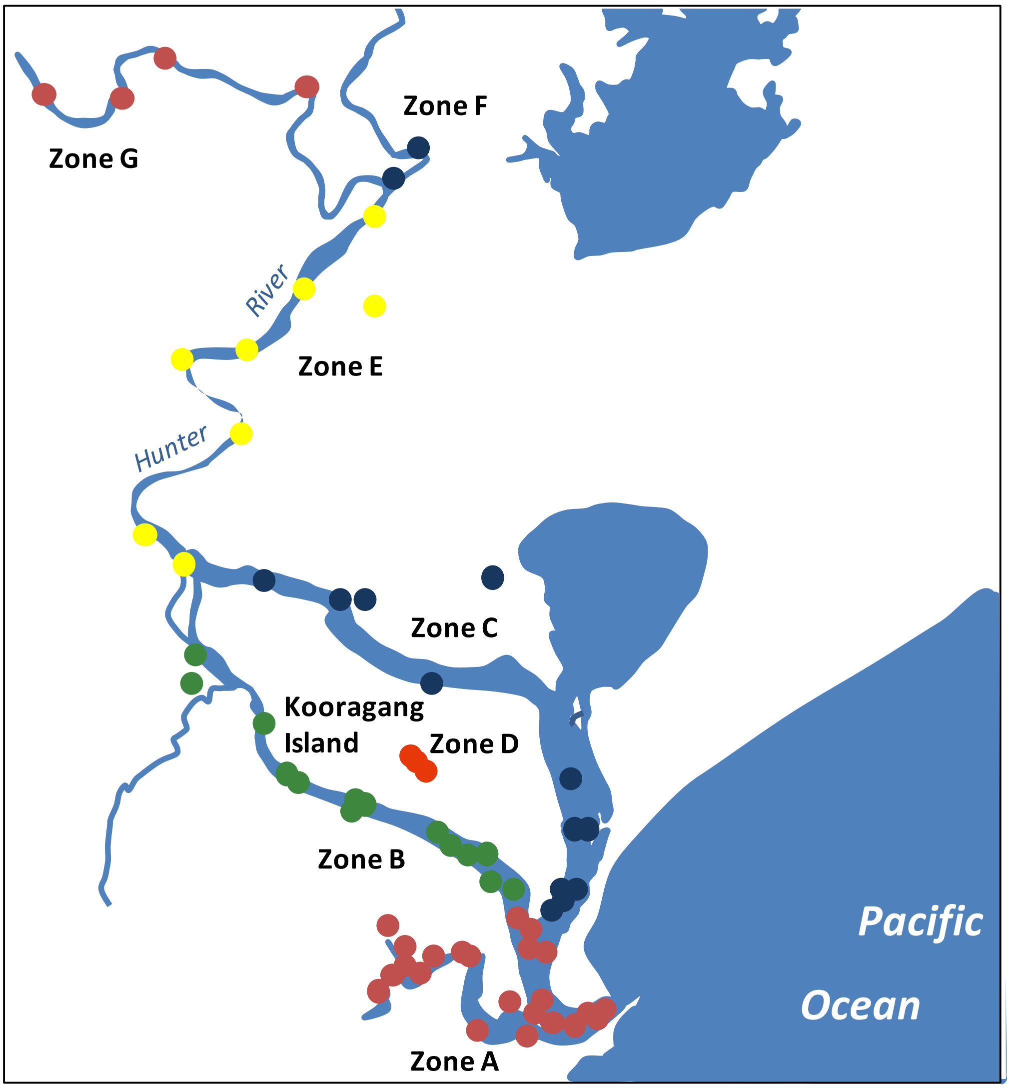

Figure 6-1 Water Quality Zones in the Hunter Estuary (Sanderson & Redden, 2001)

Three of the water quality zones are relevant to the proposed wastewater discharge:

- Zone A – This zone is found predominantly at the mouth of the Hunter River and includes the area around Walsh Point.
- Zone B – This zone includes most of the South Arm of the Hunter River.
- Zone C – This zone includes most of the North Arm of the Hunter River.

Data from each zone were amalgamated and statistics produced. Table 6-1 provides 90th percentile water quality (nutrients and non-filterable residue data only) for each of the relevant zones. In addition, the 90th percentile water quality statistics for the four monitoring locations closest to the preferred discharge location into the South Arm of the Hunter River adjacent to the IPL site are listed.

Table 6-1 90th Percentile Nutrients and Non-Filterable Residue (Sanderson & Redden, 2001)

Parameter	Units	90th Percentile Water Quality			
		Zone A	Zone B	P.D.L*	Zone C
Total Phosphorus	mg/L	0.53	0.15	0.64	0.27
Non Filterable Residual	mg/L	190	76	39	104
Nitrate	mg/L	0.63	0.39	1.96	0.33
Nitrite	mg/L	0.03	0.07	0.07	0.04
Total Kjeldahl Nitrogen	mg/L	14.50	7.00	9.42	10.30
Total Nitrogen**	mg/L	15.16	7.46	11.45	10.67

Notes:
* P.D.L. refers to the preferred discharge location adjacent to IPL's plant. Data shown reflects the 90th percentile water quality from four sites close to this location.
** Total Nitrogen was calculated as the sum of the other observed forms of nitrogen.

The information from Sanderson and Redden (2001) represents the best available dataset and has therefore been selected as a starting point for the Project assessment. IPL are in the process of conducting sampling in the Hunter Estuary to validate this dataset. This work is ongoing and will be reported whist responding to submissions following the exhibition period.

6.3 Potential Discharge Locations

The area in and around Kooragang Island falls within the Newcastle Port Site as defined by the State Environmental Planning Policy (Major Projects) Amendment (Three Ports) 2009 Newcastle Port Site Land Application Map. Thus, the predominant use of the estuary in this location is for shipping movements from a working port.

Discharge of wastewater was considered into either the North Arm or to the South Arm. Wastewater discharges adjacent to the IPL site are considered preferable to minimise infrastructure and costs.

South Arm immediately adjacent to the IPL site

The South Arm (Zone B) is heavily disturbed with poor water quality, and is subject to dredging and regular ship movements. It contains no significant aquatic habitat close to the site.

The South Arm only has approximately twenty percent of the river water volume passing through it. Consequently the ability for the wastewater stream to mix with river/estuarine water is more limited than in the North Arm.

North Arm of the Hunter River adjacent to IPL site

The North Arm of the Hunter River (Zone C) contains areas of SEPP 14 wetlands, and the Hunter Estuary Ramsar site is situated on the north side of Stockton Bridge. Ecological investigations have shown the presence of other wetland habitats in the area. Currently large ship movements into the North Arm are limited. On the whole, the condition of the North Arm of the Hunter River is less affected by anthropogenic impacts than the South Arm.

Whilst flows in the North Arm are greater than in the South Arm and rapid mixing of wastewater and river water would be likely to occur, discharges to the east of the site into the North Arm were not considered initially to be preferred given the location of potentially sensitive receptors.

6.4 Potential Impacts from Wastewater Discharges to the Hunter River

The wastewater stream that would be discharged into the Hunter River would be treated, stripped of oils and pH balanced. However it would still be relatively high in both nutrients and suspended solids. Daily composite sampling of nutrients would be undertaken. It is notable, based on the assessment present in Section 5.7, that the nutrient load contribution to the Hunter River Estuary system from the proposed discharge would be relatively minor in comparison with the total system inputs.

6.4.1 Environmental Values and Water Quality Objectives

Environmental Values for the Hunter River Estuary were identified by NSW government in 2006 (www.environment.nsw.gov.au) as:

- Aquatic ecosystems protection (fauna and flora);
- Primary recreation;
- Secondary recreation;
- Visual amenity; and
- Aquatic foods to be cooked before eating.

The primary goal for each environmental value is to maintain or improve water quality where possible.

It is noted that the definition of the estuary in this instance includes all waters from the mouth to Seaham Weir with no discrimination between the port and other areas. Supporting information accompanying the NSW government's assessment notes that – "*The limited data available indicate that the Hunter River does not have good enough water quality to protect aquatic ecosystems, or for fish and shellfish to be eaten raw. This is mainly due to high levels of pathogens (disease-causing organisms). Swimming is possible for some of the time, especially near the entrance of the estuary. Boating is usually safe*".

Newcastle City Council's Development Control Plan for Kooragang Island (NCC, 2005) aims to ensure that industrial activities do not adversely affect existing water quality on Kooragang Island or in the Hunter River.

Discussions with OEH in February 2012 suggested that the South Arm (Zone B) and lower estuary (Zone A) should both be considered condition 3 (highly disturbed) ecosystems as defined in the National Water Quality Management Guidelines (ANZECC, 2000).

The appropriate water quality objective for a condition 3 ecosystem is to maintain or improve (i.e. not worsen) existing water quality. It is therefore appropriate to compare the impacts of the proposed discharge with local water quality data (i.e. Table 6-1) rather than default ANZECC Guideline values.

6.4.2 Modelling Results

The Water Research Laboratory (WRL) at the University of New South Wales were contracted to assist in understanding the potential impacts of wastewater discharge. WRL assessed the potential for changes in background water quality due to the discharge of the wastewater stream, based on the wastewater quality indicated in Table 5-2 (noting that the distribution of specific nitrogen species was not considered). The modelling considered worst cast water quality and from this perspective is therefore conservative. Modelled changes in water quality were derived by desktop methods using existing detailed hydrodynamic model outputs to establish boundary conditions (see Appendix E).

Range of discharge scenarios were considered by WRL, in addition to two discharge locations, namely:

- South Arm immediately adjacent to the IPL site; and
- South Arm adjacent to Walsh Point Reserve.

The hydrodynamics of the north and south arm of the Hunter River are currently being further investigated by IPL, in consultation with the EPA, to confirm the results of the initial round of modelling. This work is ongoing and will be reported whist responding to submissions following the exhibition period.

The high salinity of the Hunter River estuary will ensure that a high proportion of Hydrogen ions (H^+) are available and reaction of bicarbonate atoms (HCO_3) in the wastewater stream with the estuary water will be almost instantaneous. Consequently pH is unlikely to cause any significant impacts regardless of discharge location.

Preferred Discharge Location: South Arm immediately adjacent to the IPL site

Various options for discharge were considered including locating the discharge point under existing wharves, discharge at surface, discharge at depth and potential use of diffusers.

Preliminary modelling of near surface discharges showed that the freshness of the wastewater compared with the estuarine environment meant that the wastewater stream would effectively float on top of the estuary.

Consideration was also given to discharge at depth. Modelling of discharge at depth (~11m) was found to provide the greatest dilution potential for the wastewater stream. Though the dilution achieved was likely to exceed the 90th percentile water quality for Zone B (Table 6-1), it was observed that there was a close match to the 90th percentile water quality for the four sites closest to the preferred discharge location (Table 6-1). Although the modelling did not account for mixing from regular ship movements and wind, some uncertainty remained as to whether the background water quality could be maintained in the longer term if there was an aggregation of nutrients due to low flow and at times infrequent tidal flushing, particularly during extended dry periods.

In summary, identified issues associated with potential discharge to the South Arm adjacent to the proposed development site included:

- poor flushing of tidal and river waters from South Arm (Zone B) to the lower estuary (Zone A);

- potential for cumulative build-up of nutrients particularly during extended dry periods; and
- a possibility that background water quality might be exceeded due to the build-up of nutrients in the poorly flushed environment.

This prompted consideration of an alternate location.

Alternate Discharge Location: South Arm adjacent to Walsh Point Reserve

An alternative discharge location was considered to the south of the preferred location, closer to Walsh Point. This location has similar physical characteristics to the area adjacent to IPL's site allowing discharge at depth (~11m).The Walsh Point Reserve location has a number of potential advantages as an alternative, as indicated by WRL (Appendix E), in that wastewater discharged at this location:

- would potentially have a high probability of moving into the lower estuary during typical tidal cycles;
- would be directly into Zone A which is a higher energy environment aiding mixing;
- Zone A also has a higher background concentration of nutrients (see Table 6-1) providing a higher level of confidence that existing concentrations would not be exceeded; and
- Zone A also has a high tidal exchange with the ocean under normal conditions suggesting that significant exchange of material could occur with the ocean.

It also has a number of potential drawbacks, however, including:

- the background concentrations of nutrients in this area (Zone A) are higher than in the alternate location (Zone B), indicting poorer water quality; and
- because of the more complex hydrodynamic interactions in this area, the simple modelling approach used at the preferred location was less appropriately applied at this location.

Although WRL suggested in its assessment that the potential discharge site closer to Walsh Point Reserve might be recommended based on the outlined advantages, no additional modelling was undertaken, and a high level assessment was made based on the existing modelling. In follow-up correspondence to IPL in relation to this matter, EPA suggested that it considered that an assessment of potential impact of discharge at the Walsh Point Reserve location utilising the existing modelling approach (which was not done) would likely be inadequate due to the complex hydrodynamics, but was potentially a more adequate approach when applied to the preferred discharge location originally assessed, due to lower water velocities and less potential wind effect.

Following the assessment by WRL of the initially nominated discharge and the alternate locations, a particular potential constraint was identified, following further discussion with NPC, relating to potential damage or destruction of an outfall by regular maintenance dredging conducted in the shipping channel in this part of the port. NPC advised that a discharge under K2 berth would be required to achieve a discharge at depth (i.e. approximately 10 m below low tide level) without interference from dredging.

Proposed Discharge Location

Following further consideration, based on the factors outlined above, IPL identified that its preferred discharge location was into the South Arm, immediately adjacent to the site, towards the southern boundary of the site, as shown in Figure B-1, presented in Appendix B of this report. This is effectively the preferred discharge location identified and discussed in Section 6.4.2, and originally assessed by WRL.

A bottom mounted diffuser would be installed to a depth of 10m beneath wharf K2. The wastewater would be discharged into and via a diffuser head with 4 nozzles (directed upward at 60 degrees to the horizontal, in a direction perpendicular to the river flow) and discharging at a minimum velocity of 4.4 m/s. The diffuser head would run in parallel to the wharf, set back 2m from the face of the berth. In this location it would not be impacted by dredging. Survey information obtained for this location has indicated that the nominated 10m depth can be achieved.

6.4.3 Conclusions

The conclusions of this assessment in relation to the wastewater discharge location are summarised as follows:

- The wastewater discharge is proposed to be located in the south arm of the Hunter River, due to the lack of ecological habitat (when compared to the north arm) in this location and its classification as a highly disturbed ecosystem;
- At a depth of approximately 11 m to ensure dilution of the wastewater stream in the receiving waters. Preliminary modelling of near surface wastewater discharges showed that the freshness of the wastewater compared with the estuarine environment meant that the wastewater stream effectively floated on top of the estuarine waters. Unfortunately these depths are only located in the shipping channels which are regularly dredged. The only suitable location identified with a depth close to this depth, was at 10m depth, beneath the K2 wharf.
- There were benefits and disadvantages associated with both of the potential south arm discharge locations considered. The location closer to Wash Point Reserve might on a qualitative level seem a preferable location due to the more dynamic environment, for this same reason, the assessment of this discharge to predict the impact quantitatively would be much more difficult and is therefore problematic.
- The overall increase in nutrient addition to the Hunter River Estuary arising from the proposed development will be, in the context of the overall inputs to the system, modest. There will be no increase in phosphorus additions, and whist the increase in nitrogen load arising from the site will be relatively significant, the overall load to be contributed by the site is relatively low at about 0.6% of the identified catchment inputs.

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Limitations

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Appendix A Existing Site Layout Information

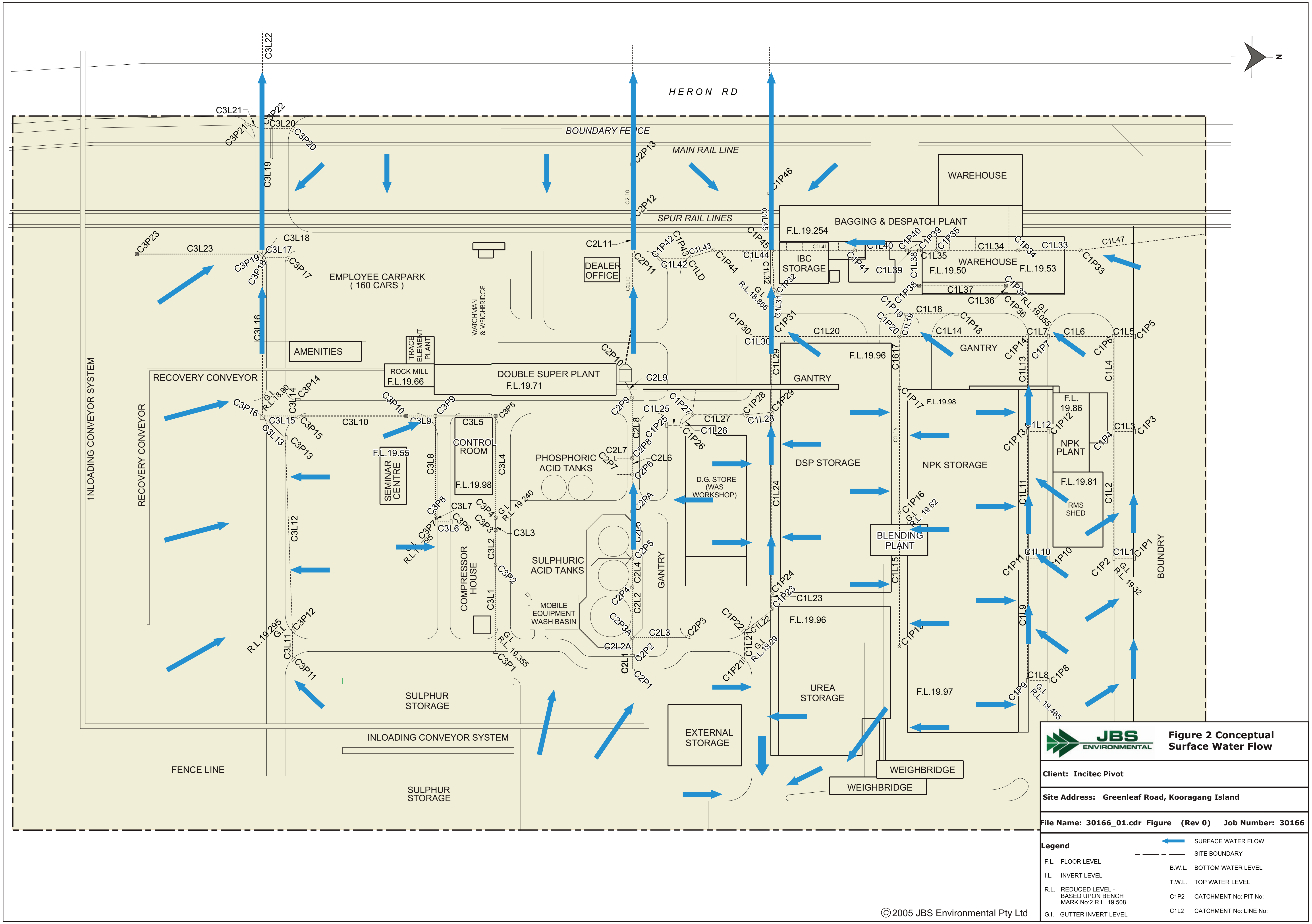


Figure 2 Conceptual Surface Water Flow

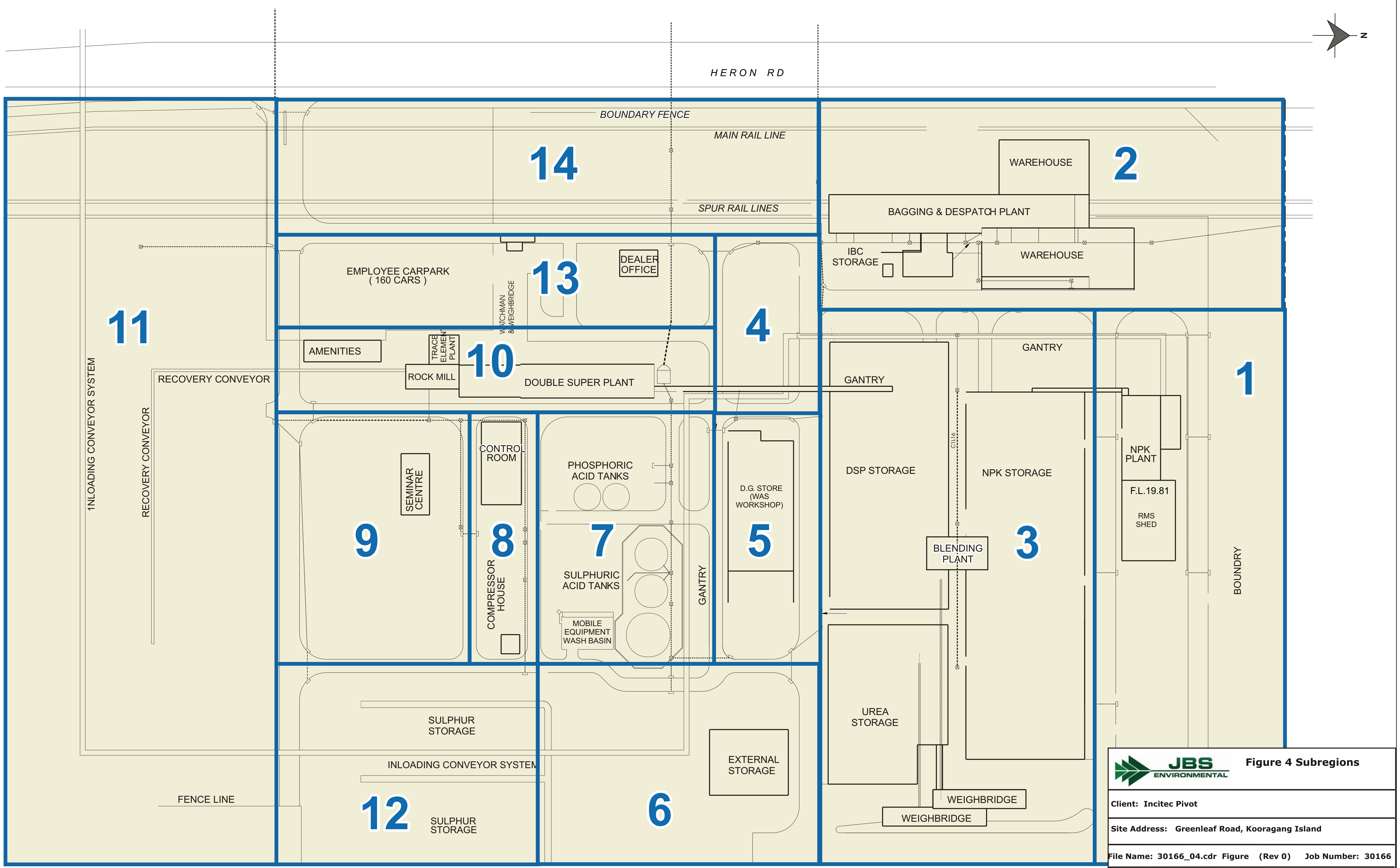
Client: Incitec Pivot

Site Address: Greenleaf Road, Kooragang Island

File Name: 30166_01.cdr Figure (Rev 0) **Job Number:** 30166

Legend

	SURFACE WATER FLOW		SITE BOUNDARY
F.L.	FLOOR LEVEL	B.W.L.	BOTTOM WATER LEVEL
I.L.	INVERT LEVEL	T.W.L.	TOP WATER LEVEL
R.L.	REDUCED LEVEL - BASED UPON BENCH MARK No:2 R.L. 19.508	C1P2	CATCHMENT No: PIT No:
G.I.	GUTTER INVERT LEVEL	C1L2	CATCHMENT No: LINE No:



JBS ENVIRONMENTAL

Figure 4 Subregions

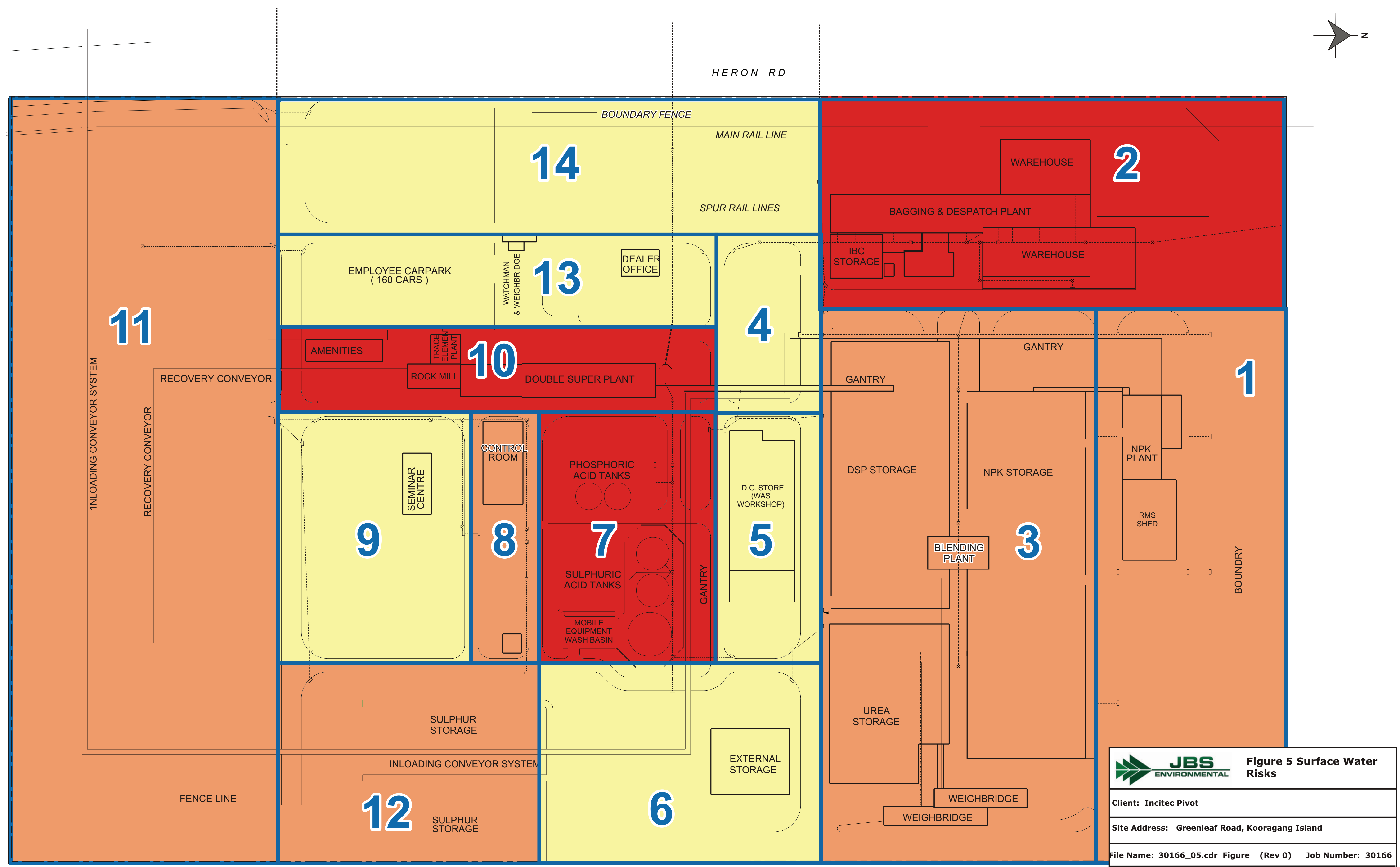
Client: Incitec Pivot

Site Address: Greenleaf Road, Kooragang Island

File Name: 30166_04.cdr Figure (Rev 0) Job Number: 30166

Legend

F.L.	FLOOR LEVEL	G.I.	GUTTER INVERT LEVEL
I.L.	INVERT LEVEL	B.W.L.	BOTTOM WATER LEVEL
R.L.	REDUCED LEVEL - BASED UPON BENCH MARK No:2 R.L. 19.508	T.W.L.	TOP WATER LEVEL
		C1P2	CATCHMENT No: PIT No:
		C1L2	CATCHMENT No: LINE No:



JBS ENVIRONMENTAL

Figure 5 Surface Water Risks

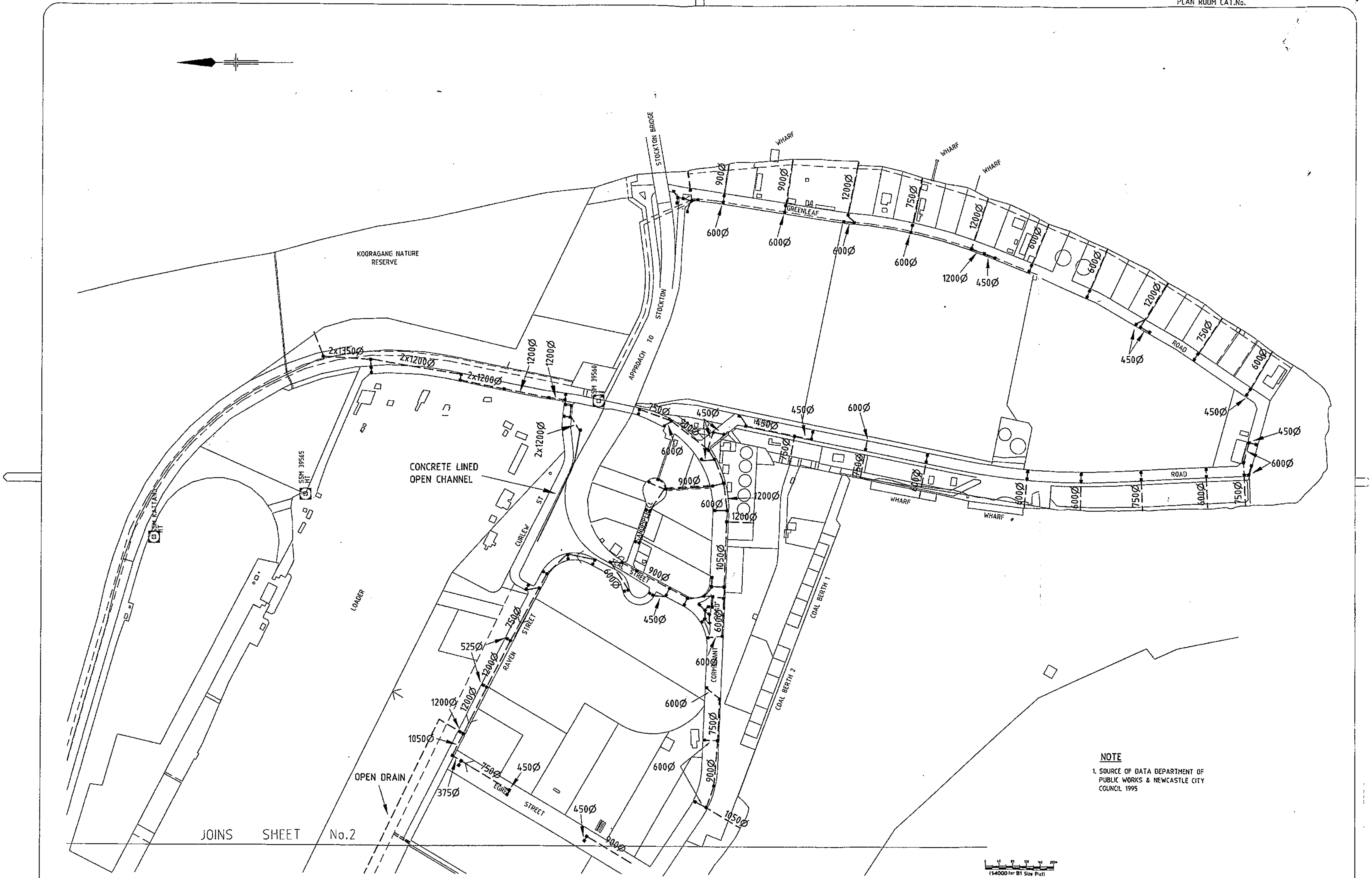
Client: Incitec Pivot

Site Address: Greenleaf Road, Kooragang Island

File Name: 30166_05.cdr Figure (Rev 0) **Job Number:** 30166

Legend

HIGH
MEDIUM
LOW



1	ISSUED WITH INFRASTRUCTURE REPORT		9.8.95
No	DETAILS OF AMENDMENTS	APPROVED	DATE

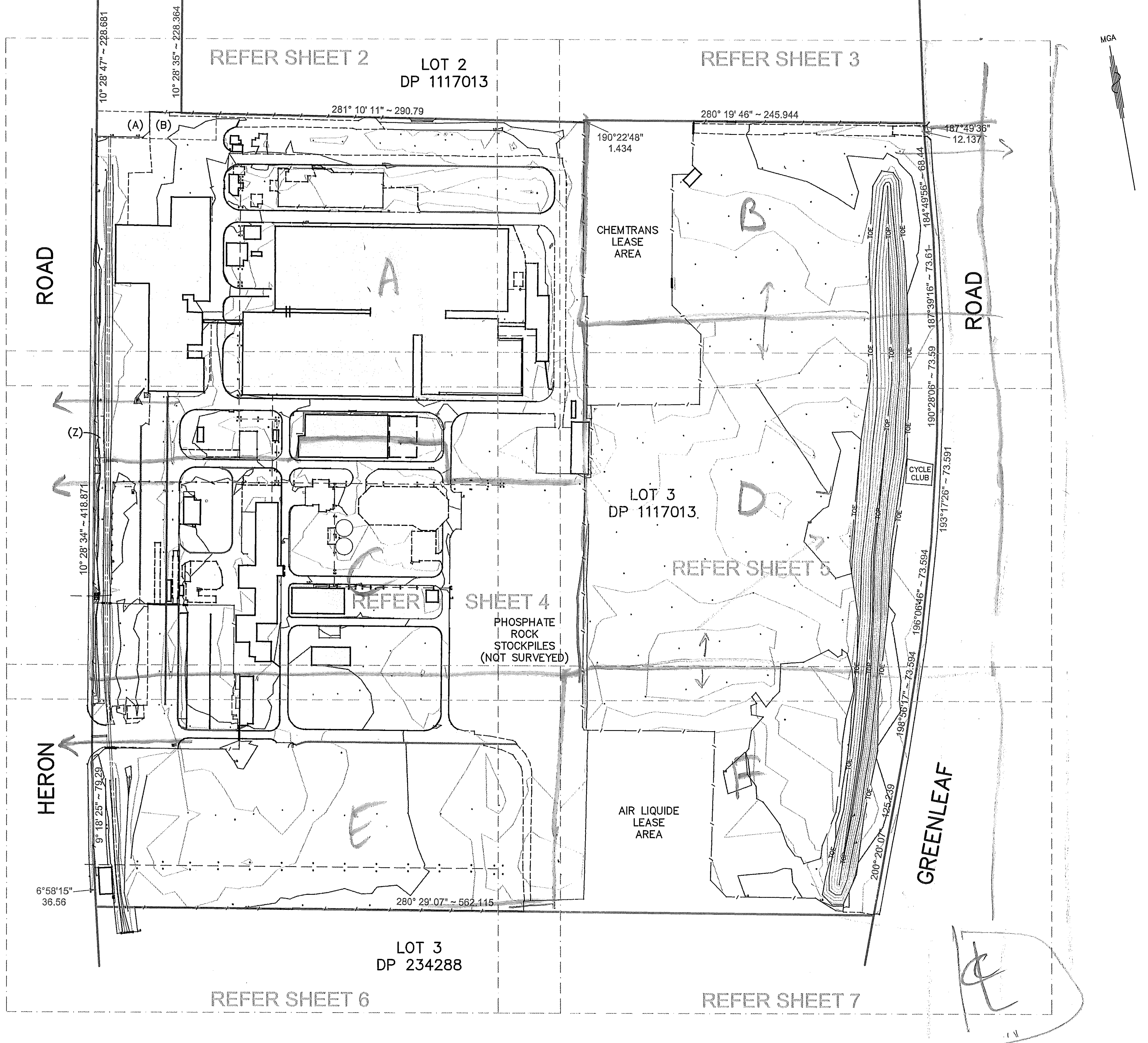
GHD Gutteridge Haskins & Davey Pty Ltd
956 Hunter Street Newcastle West, NSW 2302
P.O. BOX 5403 Newcastle West, NSW 2302
Telephone (049) 69 6655 Facsimile (049) 69 6259
221/3743/ 4.15

PREPARED NAME — SURVEYOR
TRACED NAME — PRINCIPAL SURVEYOR & MANAGER
EXAMINER NAME

PUBLIC WORKS
R.D.Christie
Director General of Public Works
M.N.Clarke
Chief Engineer

FIGURE 4.15
DRAINAGE DETAILS
SHEET 1

FILE



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DIAL 1100 BEFORE YOU DIG

SURVEY INFORMATION

1. THE SURVEY IS ON GROUND MAP GRID OF AUSTRALIA (MGA). BASED ON SSM 35953 – E385372.879 N6359696.633
2. ALL REDUCED LEVELS ARE BASED ON AUSTRALIAN HEIGHT DATUM (A.H.D)
3. ORIGIN OF LEVELS SSM 35953 RL2.664 (A.H.D)
4. CONTOUR INTERVAL IS 0.25m.

LINE TYPES

DENOTES STORMWATER PIPE
DENOTES DISH DRAIN
DENOTES UNDERGROUND ELECTRICITY CABLE
DENOTES WATER MAIN
DENOTES FENCING
DENOTES RETAINING WALL
DENOTES EDGE OF CONCRETE
DENOTES TOP OF BANK
DENOTES TOE OF BANK
DENOTES RAILWAY LINES
DENOTES EDGE OF BITUMEN
DENOTES EDGE OF GRAVEL
DENOTES OVERHEAD SERVICES

LEGEND

KIP	KERB INLET PIT
INV	INVERT LEVEL
SIP	SURFACE INLET PIT
LP	LIGHT POLE
PP	POWER POLE
ESS	ELECTRICITY SUB STATION
EPI	ELECTRICITY PILLAR
TPT	TELSTRA PIT
TPI	TELSTRA PILLAR
IPS	SEWER INSPECTION POINT
SMH	SEWER MANHOLE
WMT	WATER METER
HYD	HYDRANT
SV	STOP VALVE
GV	GAS VALVE
GMT	GAS METER
SGN	SIGN POST
CONC	CONCRETE
MW	MONITORING WELL
o	COLUMN FOR OVERHEAD SERVICES

(A) – EASEMENT FOR ACCESS 19 WIDE
(B) – EASEMENT FOR SERVICES 19 WIDE
(Z) – EASEMENT FOR ELECTRICITY PURPOSES 7 WIDE (VIDE Z456699)

IMPORTANT NOTES

1. INDEPENDENT INQUIRIES FOR UP TO DATE SERVICE LOCATIONS THROUGH THE RELEVANT AUTHORITIES MUST BE UNDERTAKEN PRIOR TO COMMENCEMENT OF ANY WORKS/EXCAVATION. EXACT SERVICE POSITIONS SHOULD BE ESTABLISHED BY APPROPRIATE MEANS. WE RECOMMEND PROFESSIONAL SERVICE LOCATORS.
2. THE BOUNDARIES SHOWN ON THIS PLAN ARE BASED ON OUR FIELD SURVEY. TO FORMALISE THESE DIMENSIONS WE WOULD RECOMMEND THE PREPARATION OF A REDEFINITION PLAN, SUITABLE FOR LODGEMENT AND REGISTRATION AT THE DEPARTMENT OF LANDS.
3. CRITICAL LEVELS (EG FLOOR LEVELS) AND CRITICAL LOCATIONS (EG STRUCTURES) MUST BE VERIFIED BY FURTHER SURVEY PRIOR TO FINAL DESIGN.
4. NO EXCAVATIONS HAVE BEEN MADE TO DETERMINE THE EXTENT TO WHICH ANY SUBJECT WALLS, FOUNDATIONS OR FOOTINGS MAY ENCROACH UPON ADJOINING LAND
5. NO EXCAVATIONS HAVE BEEN MADE TO DETERMINE THE EXTENT TO WHICH ANY ADJOINING WALLS, FOUNDATIONS OR FOOTINGS MAY ENCROACH UPON SUBJECT LAND
6. CONTOURS SHOWN DEPICT THE TOPOGRAPHY. CONTOURS DO NOT REPRESENT THE EXACT LEVEL AT ANY PARTICULAR POINT, EXCEPT AT SPOT LEVELS SHOWN.
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8. THESE NOTES ARE AN INTEGRAL PART OF THIS PLAN. REPRODUCTION OF THIS PLAN OR OF ANY PART OF THIS PLAN WITHOUT THESE NOTES BEING INCLUDED IN FULL WILL RENDER THE INFORMATION SHOWN ON SUCH REPRODUCTION INVALID AND NOT SUITABLE FOR USE.

REVISIONS

No	REVISION	SVY	DFT	CHK	DATE

Monteath & Powys Pty Ltd
ACN 000 861 110
SURVEYING
PLANNING
ENGINEERING DESIGN
PROJECT MANAGEMENT
125 Bull Street Newcastle NSW 2300
PO Box 726 Newcastle NSW 2300
T 61 2 4926 1388 F 61 2 4929 3475
E survey@monteathpowys.com.au
W www.monteathpowys.com.au

Surveyed	Drafted	Checked
SW	MAK	PJS

REGISTERED SURVEYOR

Scale @A1 : 1:1500 @A3 : 1:3000 | Original Size A1
DO NOT SCALE

Client: INCITEC PIVOT

Title: DETAIL SURVEY OF INCITEC PIVOT PLANT HERON ROAD, KOORAGANG ISLAND

CAD File: 11058B | Ref No: 11/058 | Date: 19/05/2011

Sheet No. 1/7
Revision .

Last Edit: 24/05/2011

Appendix B Proposed Development Layout Information

Source: Base maps from IPL, drawing AUG-00-131-PP-0000.dwg, received 20 April 2012. Aerial Imagery supplied by NearMaps.

Existing Structures			
1	Shed 1	12	Existing Weighbridge
2	Shed 2	13	Sulphuric Acid Inland Pipe
3	Shed 3	14	Existing Amenities Building
4	Shed 4 - DG Store		
5	Office/Seminar Center		
6	Bagging/Storage Warehouse		
7	Redundant Wax Tanks		
8	Sulphuric Acid Loadout		
9	Sulphuric Acid Storage Tanks		
10	Existing Entrance		
11	Operations Office		

Project Components			
1	Ammonia Tank	12	Substations
2	NA Plant	13	Contaminated Water Retention Pond
3	AN Plant	14	Waste Water System
4	TGAN Prill Store & Bagging	15	Proposed Entrance
5	NH_3 Flare	16	Proposed Weighbridge & Security
6	Cooling Tower Package	17	TGAN Container Store
7	NA Storage	18	Emergency Exit
8	ANSOL Storage and Handling	19	Ammonia Ship Berth
9	Administration Building	20	Modified Truck Parking
10	Workshop/Warehouse	21	Waste Water Discharge
11	Control Room		

N W E S

0 40 80 160 240 320 Meters

Datum: GDA94
Grid: MGA Zone 56

Legend

Site_Boundary
Proposed Waste Water Discharge
Existing Drainage
Proposed Drainage
Lot Boundaries
Proposed Internal Roads
Proposed Structure
Proposed Pipelines

INCITEC PIVOT LIMITED

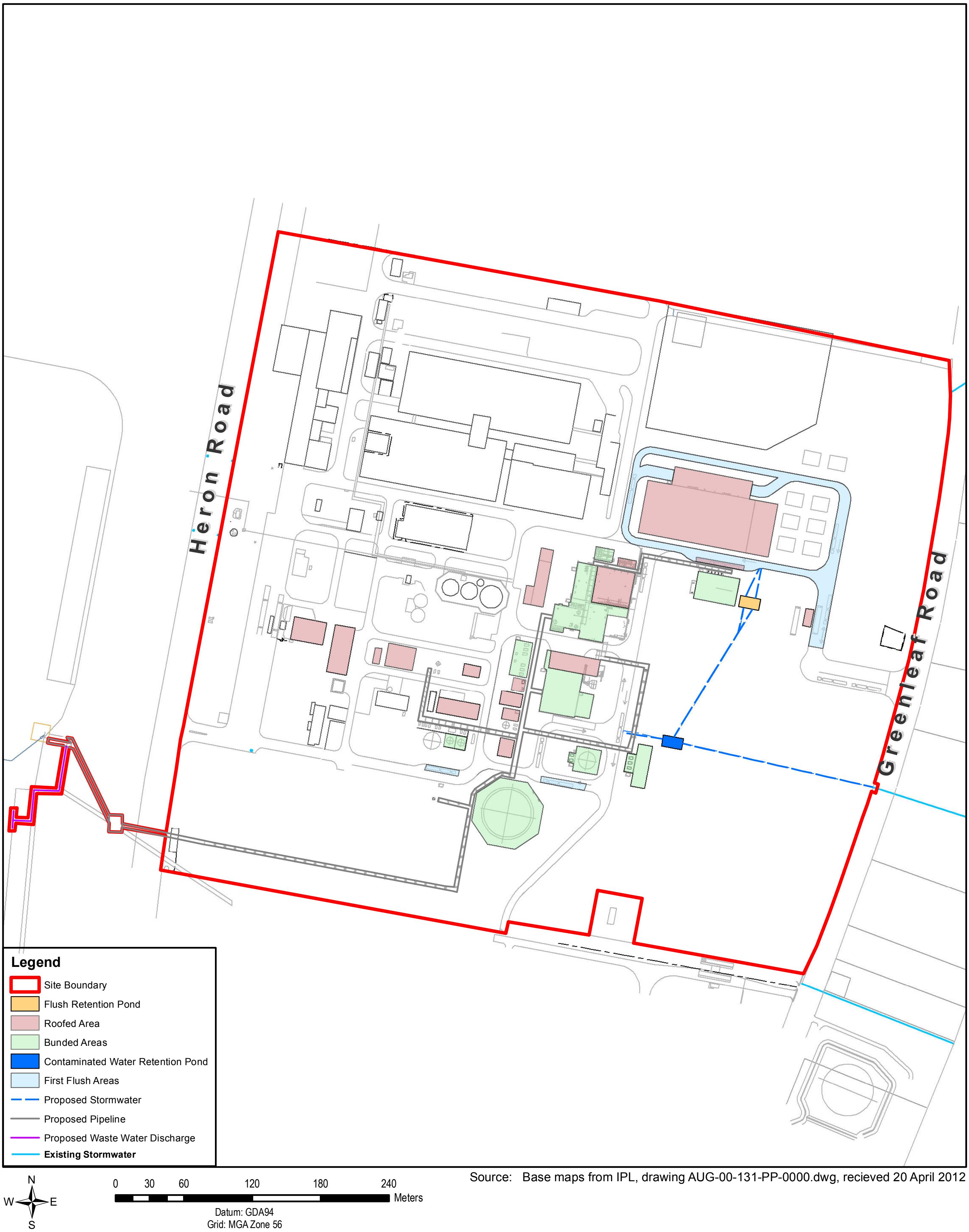
PROPOSED AMMONIUM NITRATE FACILITY

PROJECT LAYOUT

URS

File No: 43177771.051b.mxd Drawn: STB Approved: HQ Date: 27/08/2012

Figure: B-1
Rev. A A4



Source: Base maps from IPL, drawing AUG-00-131-PP-0000.dwg, recieved 20 April 2012

INCITEC PIVOT LIMITED

PROPOSED AMMONIUM NITRATE FACILITY

FIRST FLUSH AND BUNDED AREAS

URS

File No: 43177771.055b.mxd Drawn: STB Approved: WM Date: 27/08/2012

Figure: **B-2**

Rev. **A** A4

C

Appendix C Hydraulic Modelling Results

Appendix C - Hydraulic Modelling Results

This Appendix provides the peak flow rates (piped and overland flow) based on hydraulic modelling of the site under current levels of development and proposed future levels of development.

C.1 Current Development

Pipe and overland flow estimates for the site under current development levels are provided in Table C-1.

Red shaded cells indicate that overland flow occurred when modelled for the respective scenario. Green shaded cells indicate that overland flow was not initiated for that scenario.

Table C-8-1 Peak flow rates - Existing catchment

Catchment A

ARI	Pipe Flow (m3/s)	Overland Flow (m3/s)
1	0.8	0.0
5	0.8	0.7
10	0.8	0.9
20	0.8	1.3

Catchment B

ARI	Pipe Flow (m3/s)	Overland Flow (m3/s)
1	0.3	0.0
5	0.6	0.0
10	0.7	0.0
20	0.8	0.1

Catchment C

ARI	Pipe Flow (m3/s)	Overland Flow (m3/s)
1	0.3	0.0
5	0.5	0.2
10	0.4	0.3
20	0.4	0.5

Catchment D

ARI	Pipe Flow (m3/s)	Overland Flow (m3/s)
1	0.1	0.0
5	0.2	0.0
10	0.2	0.0
20	0.3	0.0

Catchment E

ARI	Pipe Flow (m3/s)	Overland Flow (m3/s)
1	0.2	0.0
5	0.3	0.0
10	0.4	0.0
20	0.4	0.1

Catchment F

ARI	Pipe Flow (m3/s)	Overland Flow (m3/s)
1	0.1	0.0
5	0.2	0.0
10	0.3	0.0
20	0.3	0.0

C.2 Future Development

Pipe and overland flow estimates for the site under current development levels are provided in Table C-2.

Red shaded cells indicate that overland flow occurred when modelled for the respective scenario. Green shaded cells indicate that overland flow was not initiated for that scenario.

Table C-8-2 Peak flow rates - Developed catchment

Catchment A

ARI	Pipe Flow (m3/s)	Overland Flow (m3/s)
1	0.7	0.0
5	0.8	0.5
10	0.8	0.7
20	0.8	1.1

Catchment B

ARI	Pipe Flow (m3/s)	Overland Flow (m3/s)
1	0.3	0.0
5	0.6	0.0
10	0.7	0.0
20	0.8	0.1

Catchment C

ARI	Pipe Flow (m3/s)	Overland Flow (m3/s)
1	0.2	0.0
5	0.4	0.0
10	0.5	0.0
20	0.5	0.2

Catchment D

ARI	Pipe Flow (m3/s)	Overland Flow (m3/s)
1	0.3	0.0
5	0.6	0.0
10	0.7	0.0
20	0.7	0.1

Catchment E

ARI	Pipe Flow (m3/s)	Overland Flow (m3/s)
1	0.3	0.0
5	0.4	0.1
10	0.4	0.2
20	0.4	0.3

Catchment F

ARI	Pipe Flow (m3/s)	Overland Flow (m3/s)
1	0.1	0.0
5	0.2	0.0
10	0.3	0.0
20	0.3	0.0

Appendix D Bureau of Meteorology IFD Information

Appendix D - Bureau of Meteorology IFD Information

Rainfall intensity (mm/hr) for **E385200 N6359800**

	ARI						
Duration	**1 Year**	**2 years**	**5 years**	**10 years**	**20 years**	**50 years**	**100 years**
5Mins	87	111	141	158	181	210	233
6Mins	81.5	104	132	148	169	197	218
10Mins	66.6	85.4	108	121	139	162	179
20Mins	48.7	62.4	79	88.5	101	118	131
30Mins	39.6	50.7	64.4	72.1	82.6	96.2	107
1Hr	26.8	34.3	43.7	49	56.2	65.5	72.6
2Hrs	17.4	22.3	28.5	32	36.8	43	47.7
3Hrs	13.3	17.1	21.9	24.7	28.4	33.2	36.8
6Hrs	8.44	10.9	14	15.8	18.1	21.3	23.7
12Hrs	5.42	6.99	9.04	10.2	11.8	13.9	15.5
24Hrs	3.56	4.61	6.01	6.86	7.95	9.39	10.5
48Hrs	2.32	3.02	4	4.59	5.35	6.37	7.15
72Hrs	1.74	2.28	3.04	3.5	4.09	4.89	5.51

E

Appendix E Impacts of Proposed Hunter River Discharge - WRL Report

URS

THE UNIVERSITY OF
NEW SOUTH WALES

Water Research Laboratory

School of Civil and Environmental Engineering

28 March 2012

WRL Ref: WRL2012019 DSR:BMM L20120328

Mr David Fuller
Senior Principal
URS Australia Pty Ltd
Level 6, 1 Southbank Boulevard
Southbank VIC 3006

By Email: david.fuller@urs.com

Dear David,

Kooragang Island Thermal Discharge Assessment

This letter summarises analysis of dilution, dispersion and subsequent constituent concentrations of a thermal, buoyant effluent once discharged to the South Arm of the Hunter River. This analysis has been undertaken by the Water Research Laboratory (WRL) at the University of New South Wales (UNSW) for URS Corp on behalf of Incitec Pivot Ltd.

Key findings from this study are:

- Effluent discharged on the surface remains at elevated concentrations in the receiving water due to low velocities in the South Arm and density differences (buoyancy effects).
- Receiving water velocities during dry conditions have limited tidal flushing, bounding the effluent extent to within the South Arm.
- Once released, effluent dispersion is relatively low resulting in effluent mixing with previously discharged effluent. This cycle of plume interaction results in increased concentrations in the South Arm.
- Advection of the effluent during average tidal cycle does not result in transport to the North Arm.
- High dilutions and lower concentrations can be achieved by a sub-surface discharge which improves mixing and entrainment throughout the water column.

Enclosed is the methodology and analysis undertaken by WRL to assess the proposed effluent under different discharge scenarios and the subsequent impact on receiving water concentration.

1. Background

Incitec Pivot are assessing an upgrade to an existing ammonium nitrate plant located at Kooragang Island in Newcastle, NSW (Figure 1.1). The upgraded plant will produce a thermal effluent to be discharged into the South Arm of the Hunter River via an existing open channel. The effluent characteristics are detailed in Table 1.1.

Table 1.1: Effluent Stream Characteristics

Parameter	Unit	Scenario 1	Scenario 2
Temperature	deg C	35	28
pH	pH units	6.5 – 8.5	7.0 – 8.5
Total Nitrogen	mg/L	150	75
Total Phosphate	mg/L	25	<10
Total Suspended Solids	mg/L	30	<15
Volume	kL/day	750	750
Conductivity	MicroSiemens/cm	2600	1750

2. Establishing Water Quality Reference Condition

To quantify the required dilution, the background water quality levels must be established. The South Arm receiving environment is classified as a highly disturbed ecosystem, or a Condition 3 Ecosystem as defined in ANZECC (2000). Typical Condition 3 systems are shipping ports and sections of harbours serving coastal cities, urban streams receiving road and stormwater runoff, or rural streams receiving runoff from intensive horticulture. The aim for a Condition 3 water body is to maintain current water quality.

The trigger values detailed in the ANZECC (2000) guidelines (Table 2.1) only apply to an undisturbed ecosystem, or a Condition 1 Ecosystem. Consequently, a reference condition must be set for this Condition 3 system based on available water quality data. The ANZECC guidelines recommends using an 80th percentile to improve water quality or 90th percentile value to maintain water quality.

Sanderson and Redden (2001) undertook analysis of water quality data throughout the Hunter River estuary. Monitoring locations were divided into geographic sub-sets to define various sections of the estuary (Figure 2.1). The discharge location borders on data sets for Zones A and B that define water quality for the entrance/harbour and the South Arm. North Arm water quality is assessed as Zone C. Water quality data statistics for Zone A, B and C are presented in Tables 2.2, 2.3 and 2.4 respectively. To WRL's knowledge, the Sanderson and Redden (2001) dataset is the best available at the time of undertaking this study.

To determine which of the datasets is most applicable to the effluent, the net transport of the South Arm was investigated. Using the historical freshwater discharges for the Hunter, Patterson and Williams Rivers, a dry year was selected. The 2005 dataset containing freshwater inflows of up to approximately a 1 in 1 year event with long periods of reduced inflow to assess conservative conditions as well as the impact of flushing.

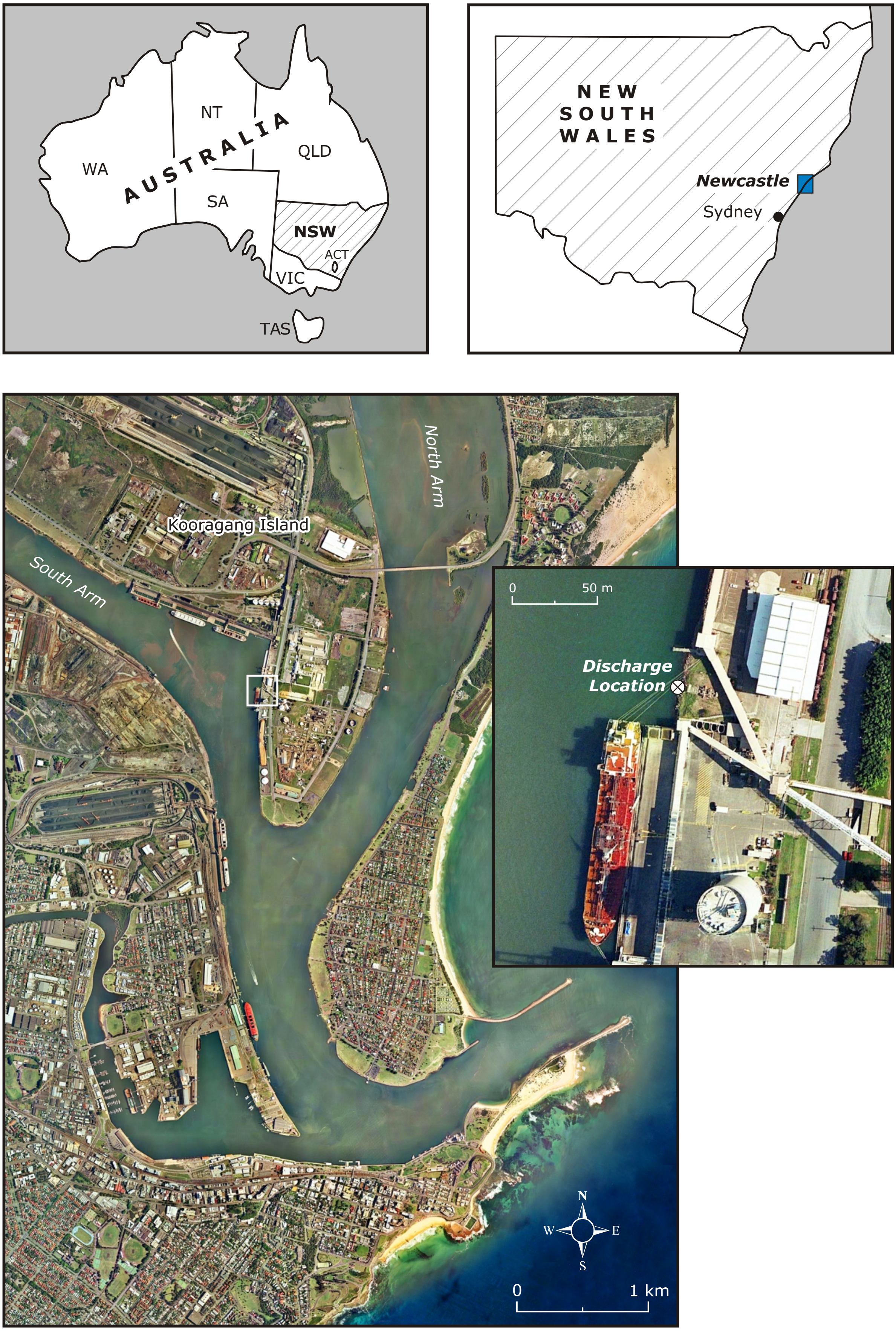


Figure 1.1: Location

Flows in the South Arm were assessed based on gauged flows and WRL's previous modelling experiences of the Hunter River. Net South Arm discharges were considered, with positive flow indicating discharge to the ocean, and negative flow indicating ingress up the South Arm. Cumulative South Arm flow and total Hunter River inflow were plotted against each other to assess correlation of South Arm flow direction to freshwater events (Figure 2.2). Note that the river discharge in Figure 2.2 are for both the North and South Arms, not just inflow to the South Arm.

A positive gradient for the cumulative South Arm flow indicates net discharge to the ocean, with a negative gradient indicating net inflow up the South Arm. As seen in Figure 2.2, a freshwater event induces net discharge from the South Arm with a positive gradient on the cumulative discharge curve. During low inflow conditions, or dry conditions, the gradient of the line is negative indicating net transport up the South Arm. As this is the predominant condition at the effluent discharge location, the water quality data for the South Arm (or Zone B) should be used to set the reference condition.

Flow conditions in the Hunter River and South Arm do not promote advection of effluent into the North Arm. Advection during average conditions does not result in transport of effluent downstream of the confluence of the North and South Arms of the Hunter River. If freshwater discharge from the South Arm is significant enough to transport effluent downstream of the confluence, it can be assumed that North Arm freshwater discharge is substantially higher, resulting in flushing of the effluent from the estuary entrance. Subsequently, North Arm water quality was not considered an appropriate reference condition for this study.

Based on the ANZECC (2000) guidelines, the 90^{th} percentile data in Table 2.3 was used as the background water quality and reference condition. Note that Total Nitrogen concentrations were not part of the Sanderson and Redden (2001) dataset but were calculated based on the sum of Total Kjeldahl Nitrogen, Nitrate and Nitrite for this study. Furthermore, the Sanderson and Redden (2001) analysis contains Total Phosphorus whereas the effluent stream data specifies Total Phosphate. Phosphate is a component of Total Phosphorus. It should be noted that the Total Phosphorus was used as the reference condition of Total Phosphate in the absence of other available data. The background concentration of Phosphate is likely to be less than the background Total Phosphorus concentration, however this should be investigated further prior to implementing the effluent discharge.

Sanderson and Redden (2001) did not investigate Total Suspended Solid (TSS) concentrations specifically, but do suggest that Non Filterable Residual (NFR) is similar to suspended particular matter (SPM) or TSS. For this investigation, the statistics for NFR is Zone B is assumed to be the concentration to Total Suspended Solids. The 90^{th} percentile NFR concentrations reported by Sanderson and Redden (2001) are significantly higher than the discharge stream, however background TSS concentrations in the receiving water should be investigated further.

It is also worth noting that of the effluent constituents outlined in Table 1.1, only pH, Total Nitrogen and Total Phosphorus (Total Phosphates) are specified as chemical stressors in estuarine environments. Suspended Solids are not specified as an indicator for estuarine/marine ecosystems, except where seagrasses are present. Extensive seagrass beds have not been present in the Hunter River for at least the past three decades though they are present in every other analogous river in NSW (Williams, et al., 2000). However, there is some recent evidence of sea grass along the North Arm (J. Murray, URS, pers. comm.).

Due to the high salinity of the receiving water during dry conditions, hydrogen protons (H^+) are expected to react with freely available bicarbonate (HCO_3^-) ions in the receiving water. This reaction is noted to be virtually instantaneous once mixing with receiving waters has been achieved (Schulz et al., 2006). As such, WRL does not consider pH to be of concern.

Table 2.1: Default Trigger Values for a Slightly Disturbed Estuary in South-Eastern Australia (ANZECC, 2000)

Table 3.3.2 Default trigger values for physical and chemical stressors for south-east Australia for slightly disturbed ecosystems. Trigger values are used to assess risk of adverse effects due to nutrients, biodegradable organic matter and pH in various ecosystem types. Data derived from trigger values supplied by Australian states and territories. Chl *a* = chlorophyll *a*, TP = total phosphorus, FRP = filterable reactive phosphate, TN = total nitrogen, NO_x = oxides of nitrogen, NH_4^+ = ammonium, DO = dissolved oxygen.

Ecosystem type	Chl *a* ($\mu g\,L^{-1}$)	TP ($\mu g\,P\,L^{-1}$)	FRP ($\mu g\,P\,L^{-1}$)	TN ($\mu g\,N\,L^{-1}$)	NO_x ($\mu g\,N\,L^{-1}$)	NH_4^+ ($\mu g\,N\,L^{-1}$)	DO (% saturation)[l]		pH	
							Lower limit	Upper limit	Lower limit	Upper limit
Upland river	na[a]	20[b]	15[g]	250[c]	15[h]	13[i]	90	110	6.5	7.5[m]
Lowland river[d]	5	50	20	500	40[o]	20	85	110	6.5	8.0
Freshwater lakes & Reservoirs	5[e]	10	5	350	10	10	90	110	6.5	8.0[m]
Wetlands	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data
Estuaries[p]	4[f]	30	5[j]	300	15	15	80	110	7.0	8.5
Marine[p]	1[n]	25[n]	10	120	5[k]	15[k]	90	110	8.0	8.4

na = not applicable;
a = monitoring of periphyton and not phytoplankton biomass is recommended in upland rivers — values for periphyton biomass (mg Chl *a* m^{-2}) to be developed;
b = values are 30 μgL^{-1} for Qld rivers, 10 μgL^{-1} for Vic. alpine streams and 13 μgL^{-1} for Tas. rivers;
c = values are 100 μgL^{-1} for Vic. alpine streams and 480 μgL^{-1} for Tas. rivers;
d = values are 3 μgL^{-1} for Chl *a*, 25 μgL^{-1} for TP and 350 μgL^{-1} for TN for NSW & Vic. east flowing coastal rivers;
e = values are 3 μgL^{-1} for Tas. lakes;
f = value is 5 μgL^{-1} for Qld estuaries;
g = value is 5 μgL^{-1} for Vic. alpine streams and Tas. rivers;
h = value is 190 μgL^{-1} for Tas. rivers;
i = value is 10 μgL^{-1} for Qld. rivers;
j = value is 15 μgL^{-1} for Qld. estuaries;
k = values of 25 μgL^{-1} for NO_x and 20 μgL^{-1} for NH_4^+ for NSW are elevated due to frequent upwelling events;
l = dissolved oxygen values were derived from daytime measurements. Dissolved oxygen concentrations may vary diurnally and with depth. Monitoring programs should assess this potential variability (see Section 3.3.3.2);
m = values for NSW upland rivers are 6.5–8.0, for NSW lowland rivers 6.5–8.5, for humic rich Tas. lakes and rivers 4.0-6.5;
n = values are 20 μgL^{-1} for TP for offshore waters and 1.5 μgL^{-1} for Chl *a* for Qld inshore waters;
o = value is 60 μgL^{-1} for Qld rivers;
p = no data available for Tasmanian estuarine and marine waters. A precautionary approach should be adopted when applying default trigger values to these systems.

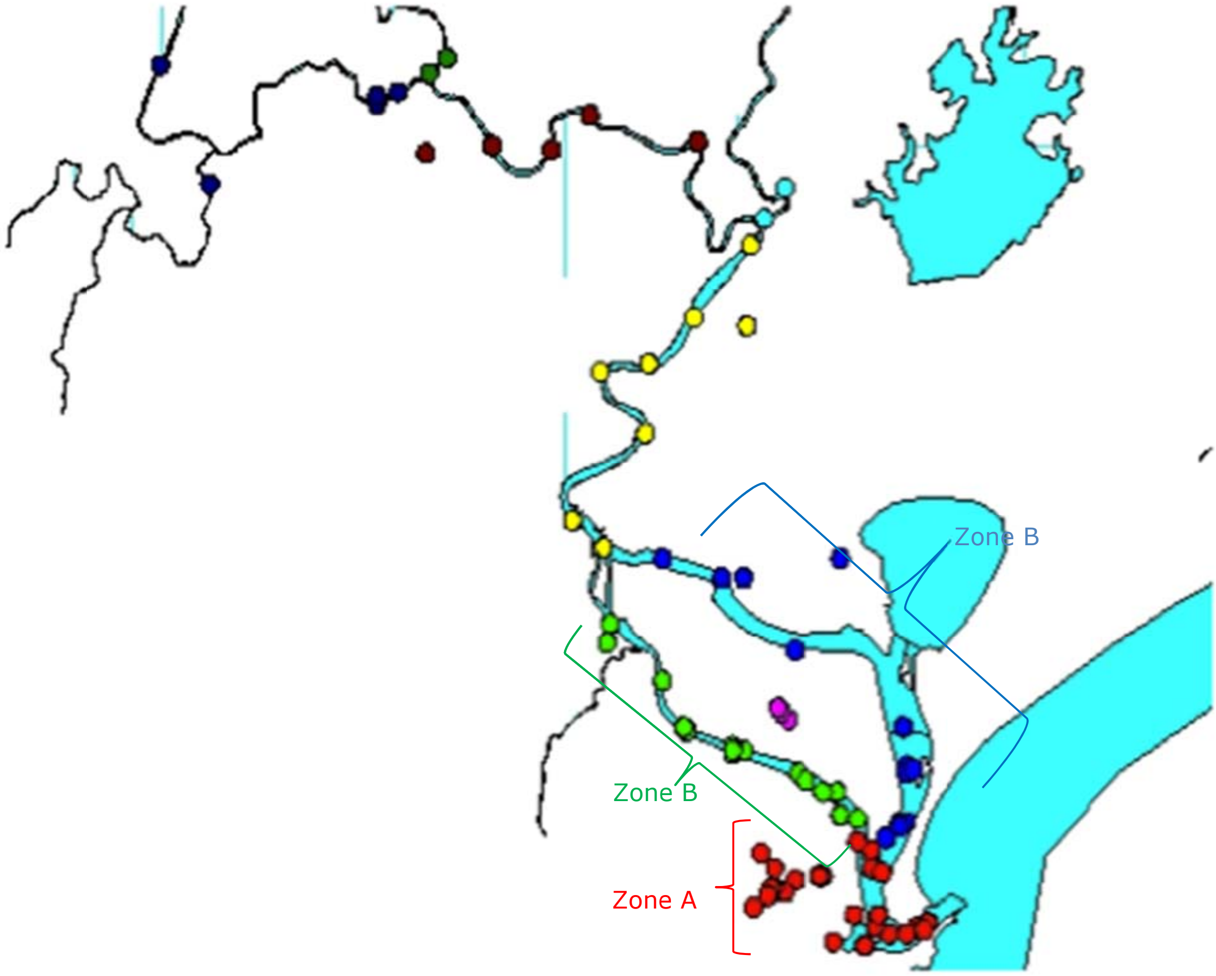


Figure 2.1: Water Quality Monitoring Locations/Zones.
Zone A = Red, Zone B = Green and Zone C = Dark Blue. (Sanderson and Redden, 2001).

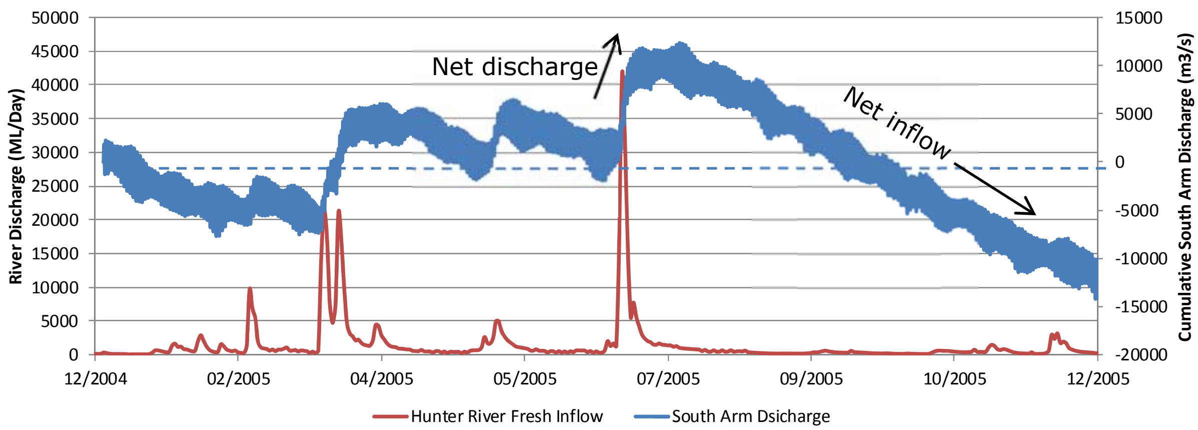


Figure 2.2: Freshwater Inflow Compared to Cumulative South Arm discharge.

Table 2.2: Water Quality Statistics for Zone A (Sanderson and Redden, 2001)

Parameter	Units	Mean	STD	90th Percentile	Median	10th Percentile	Max	Min	Geometric Mean
Conductivity	µs/cm	38888	13727	49800	44300	5070	53200	4490	33423
Chl-a	µg/L	4.36	3.43	9.30	3.00	1.00	13.00	1.00	3.30
BOD	mg/L	2.05	2.72	2.50	2.00	0.50	18.00	0.50	1.42
DO	mg/L	7.00	1.51	8.65	7.20	5.30	11.40	2.72	6.81
Enterococi	cfu/100ml	1522	4579	3000	192	10	44000	2	208
E.Coli	cfu/100ml	3520	10285	24000	2	0	38000	0	143
Non Filterable Residual	mg/L	93.22	160.49	190.00	43.00	6.00	1000	0.40	39.89
Faecal Colifrom	cfu/100ml	1738	4975	4050	100	8	38000	0	147
NO3	mg/L	0.32	0.72	0.63	0.15	0.02	6.13	0.01	0.14
NO2	mg/L	0.02	0.04	0.03	0.01	0.01	0.27	0.01	0.01
NH3	mg/L	0.48	1.79	0.60	0.14	0.03	20.00	0.00	0.14
Oxidizing Potential	mg/L	0.44	0.39	1.00	0.30	0.01	2.00	0.01	0.22
NOx	mg/L	0.30	0.59	0.47	0.16	0.04	6.40	0.00	0.15
Salinity	g/kg	21.96	11.76	35.97	25.55	2.50	40.16	0.40	15.50
pH		8.12	0.25	8.40	8.20	7.80	8.70	7.30	8.11
Soluble Reactive Phosphorus	mg/L	0.02	0.02	0.05	0.01	0.01	0.11	0.01	0.14
Secchi Depth	mg/L	0.40	0.32	1.00	0.30	0.10	1.40	0.00	0.33
Total Zooplankton Count	no./mL	44.65	231.24	90.00	1.50	0.00	2880	0.00	25.69
Total Phytoplankton Count	no./mL	185.16	525.12	396.00	30.00	0.00	3684	0.00	62.38
Total Phosphorus	mg/L	0.22	0.38	0.53	0.09	0.04	2.80	0.00	0.11
Temperature	C	18.96	2.69	23.00	19.00	15.50	25.60	12.00	18.77
Turbidity	NTU	10.18	13.64	21.00	5.70	2.48	90.00	0.70	6.52
Total Kjeldahl Nitrogen	mg/L	5.24	7.32	14.50	1.51	0.54	65.00	0.20	2.29
Total Nitrogen*	mg/L	5.58	8.08	15.16	1.67	0.57	71.40	0.22	2.44

* Sum of TKN, nitrate and nitrite

Table 2.3: Water Quality Statistics for Zone B (Sanderson and Redden, 2001)

Parameter	Units	Mean	STD	90th Percentile	Median	10th Percentile	Max	Min	Geometric Mean
Conductivity	µs/cm	35338	14676	48000	42150	4980	52400	435	28871
Chl-a	µg/L	6.67	6.93	15.40	4.34	1.74	42.16	0.51	4.52
BOD	mg/L	1.48	1.00	3.00	1.00	0.50	6.00	0.50	1.20
DO	mg/L	6.29	1.63	8.20	6.80	4.00	10.40	2.67	6.06
Enterococi	cfu/100ml	677	888	2300	255	10	2650	8	193
E.Coli	cfu/100ml	352	1449	140	4	0	6500	0	25
Non Filterable Residual	mg/L	36.6	41.2	76.0	24.0	8.0	335	1.0	24.3
Faecal Colifrom	cfu/100ml	227.3	999.4	400.0	20.0	2.0	13000	0.0	34.5
NO3	mg/L	0.19	0.12	0.39	0.18	0.07	0.46	0.02	0.15
NO2	mg/L	0.04	0.08	0.07	0.01	0.01	0.40	0.01	0.02
NH3	mg/L	0.45	0.67	1.00	0.23	0.05	4.27	0.00	0.22
Oxidizing Potential	mg/L	0.31	0.27	0.70	0.30	1.00	1.30	0.01	0.17
NOx	mg/L	0.19	0.23	0.30	0.14	0.05	2.10	0.05	0.13
Salinity	g/kg	20.93	12.41	35.28	21.35	0.90	37.85	0.20	12.94
pH		8.02	0.35	8.30	8.10	7.60	9.00	6.20	8.02
Soluble Reactive Phosphorus	mg/L	0.02	0.01	0.03	0.02	0.01	0.05	0.01	0.01
Secchi Depth	mg/L	0.33	0.25	0.80	0.20	0.10	1.00	0.00	0.27
Total Zooplankton Count	no./mL	26.89	48.76	118.00	0.00	0.00	180	0.00	57.12
Total Phytoplankton Count	no./mL	235.14	496.56	540.00	50.00	0.00	2160	0.00	72.33
Total Phosphorus	mg/L	0.09	0.07	0.15	0.07	0.04	0.51	0.01	0.07
Temperature	C	18.79	3.07	22.00	18.85	13.50	24.0	11.50	18.53
Turbidity	NTU	10.98	11.44	35.00	6.70	28.00	44.0	2.20	7.58
Total Kjeldahl Nitrogen	mg/L	2.52	2.94	7.00	1.10	0.60	16.0	0.20	1.50
NH4	mg/L	0.23	0.17	0.48	0.20	0.07	0.95	0.05	0.18
Chloroform	µg/L	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00
Total Nitrogen*	mg/L	2.75	3.14	7.46	1.29	0.68	16.86	0.23	1.68

* Sum of TKN, nitrate and nitrite

Table 2.4: Water Quality Statistics for Zone C (Sanderson and Redden, 2001)

Parameter	Units	Mean	STD	90th Percentile	Median	10th Percentile	Max	Min	Geometric Mean
Conductivity	µs/cm	35640	14453.7	49000	41200	4950	53000	3860	29961.391
Chl-a	µg/L	6.46	5.06	13.00	5.00	2.00	23.00	2.00	5.09
BOD	mg/L	2.03	1.84	3.00	2.00	0.50	11.8	0.5	1.51
DO	mg/L	6.57	1.46	8.20	6.91	4.60	8.80	2.87	6.38
Enterococi	cfu/100ml	667	958	1800	280	20	4600	4	224
E.Coli	cfu/100ml	13.47	33.37	70.00	0.00	0.00	130.00	0.00	18.97
Non Filterable Residual	mg/L	48.6	82.3	104.0	23.0	7.0	604.0	3.8	24.9
Faecal Colifrom	cfu/100ml	391.2	1157.5	900.0	40.0	4.0	7800.0	0.0	62.6
NO3	mg/L	0.19	0.12	0.33	0.18	0.04	0.51	0.01	0.14
NO2	mg/L	0.02	0.01	0.04	0.01	0.01	0.06	0.01	0.01
NH3	mg/L	0.45	1.34	0.70	0.13	0.04	20.00	0.00	0.17
Oxidizing Potential	mg/L	0.30	0.24	0.60	0.30	0.01	1.20	0.01	0.17
NOx	mg/L	0.17	0.10	0.30	0.15	0.05	0.52	0.01	0.13
Salinity	g/kg	19.18	12.68	34.94	20.95	1.20	40.51	0.40	11.59
pH		8.09	0.21	8.30	8.20	7.90	8.80	7.10	8.09
Soluble Reactive Phosphorus	mg/L	0.03	0.04	0.09	0.02	0.01	0.19	0.01	0.02
Secchi Depth	mg/L	0.29	0.25	0.60	0.23	0.00	1.00	0.00	0.28
Total Zooplankton Count	no./mL	18.48	27.03	72.00	2.00	0.00	82.00	0.00	19.31
Total Phytoplankton Count	no./mL	231.88	654.95	508.00	32.00	0.00	3564	0.00	54.73
Total Phosphorus	mg/L	0.15	0.22	0.27	0.10	0.04	2.00	0.01	0.10
Temperature	C	18.48	2.81	22.00	18.50	14.00	24.00	12.80	18.27
Turbidity	NTU	11.12	10.12	23.00	7.80	3.40	55.00	2.10	8.40
Total Kjeldahl Nitrogen	mg/L	3.91	4.40	10.30	1.40	0.56	21.20	0.20	2.06
Total Nitrogen*	mg/L	4.11	4.54	10.67	1.59	0.61	21.77	0.22	2.22

* Sum of TKN, nitrate and nitrite

3. Receiving Water Currents

A timeseries of receiving water velocity adjacent to the effluent discharge location was extracted from previous WRL 1-dimensional hydrodynamic modelling. No new modelling was undertaken for this study, with data being extracted from existing model result files.

The dry and wet weather statistics from 2005 are presented in Table 3.1. Due to channel choking at the northern extent of the South Arm, velocities during dry and wet periods are very slow. This is due to the majority of freshwater and tidal flows being conveyed by the North Arm. North Arm velocities are significantly greater, with 50th percentile velocities of 0.39 m/s during 2005. Slow velocities in the South Arm reduce the potential for mid and far-field mixing of the effluent.

Table 3.1: South Arm Receiving Water Current Statistics

Percentile (X)	Dry Weather (m/s) Velocity exceeded X percent of the time	Wet Weather (m/s) Velocity exceeded X percent of the time*
95th	0.002	0.002
90th	0.003	0.004
50th	0.016	0.017

*(1 year ARI freshwater flows)

4. Point Source Discharge

The velocity of the receiving water, the buoyancy and density of the effluent and the density of the receiving water determine the mixing potential. These make the dimensionless parameter:

$$\frac{B}{du^{*3}} \tag{4.1}$$

Where:
$B = (\Delta\rho/\rho)gQ_e$
$\Delta\rho$ = density difference between effluent and receiving water (kg/m3)
ρ = density of receiving water (kg/m3)
Q_e = effluent discharge (m3/s)
d = depth
u* = shear velocity estimated as 0.1 x velocity

Schiller and Sayre (1973) state that if $B/(du^{*3})$ is less than five then transverse spreading and dispersion is reasonably well described by neglecting density effects. Conversely, if B/du^{*3} is very large, an effluent will spread rapidly across the water surface to form a density driven current and will be likely to form a layer on the surface.

For the thermal discharge proposed at Kooragang Island, B/du^{*3} was calculated to be 3×10^7. This is based on the Scenario 1 effluent stream, 50th percentile velocities, and 50th percentile temperature and salinity of the receiving water as defined for zone B water quality statistics (Table 2.3). This result indicates that the effluent from the plant would form a buoyant layer on the surface.

To assess far-field dispersion a layer thickness is required. The near-field model JETLAG was used to determine the layer thickness. The near-field plume depth and radius resulting from discharge to the surface of the South Arm was assessed by inputing flow near the surface from a large pipe. This resulted in dilutions of 1 to 3 times at a depth of approximately 0.1 to 0.2 m depending on receiving water conditions. The larger layer thickness was used to determine whether the effluent could disperse under the maximum depth conditions.

Due to the conservative nature of this assessment, no mixing due to wind is considered. Subsequently, mixing of the less dense effluent layer with the surrounding water body is dependent on the turbulent mixing generated by the receiving water currents. The mixing potential of the receiving water can be estimated by the Richardson Number (Equation 4.2). A very small Richardson number indicates a well-mixed estuary, with a large Richardson number indicating poor mixing potential and a strongly stratified flow. The transition for a well-mixed to a strongly stratified estuary typically occurs in the 0.08 < R < 0.8 range.

$$R = \frac{(\Delta\rho/\rho)gQ_f}{WU_t^3} \tag{4.2}$$

Where:
ρ = density of receiving water = 1015 kg/m^3
$\Delta\rho$ = density difference between receiving water and effluent = 1015 – 995 = 20 kg/m^3
Q_f = freshwater inflow = effluent discharge (m3/s)
W = width of channel = 700 m
g = gravity = 9.81 m/s
U_t = rms tidal velocity (m/s) = 0.0162 m/s during normal conditions

The Richardson number for the South Arm receiving waters was calculated to be 0.554 indicating lower mixing potential and approaching strong stratification.

Due to the very slow ambient currents, turbulent mixing will not break down the stratification between the effluent and the receiving water during a single tidal cycle (~ 12 hours). Subsequently, the depth of the effluent layer was modelled at a constant thickness of 0.2 m.

4.1 Continuous Discharge

A continuous mass discharge was assessed based on the above 0.2 m effluent depth with all water quality components modelled as conservative constituents. For the discharge scenario to be environmentally feasible the concentration of effluent would need to be low by the time a full flood-ebb tidal cycle is complete. Once a full tidal cycle is complete, the effluent plume will then pass back over the discharge location and mix with freshly added effluent. This section focuses on determining the maximum concentration expected upon completion of a full tidal cycle.

The maximum concentration of a plume from a continuous point source can be calculated using equation 4.3. The width of the plume from the edge of the harbour is calculated from equation 4.4.

$$C_{max} = \frac{\dot{M}}{\bar{u}d}\frac{1}{\sqrt{4\pi D_t x/\bar{u}}} \tag{4.3}$$

$$b = 4\sigma = 4\sqrt{2D_t\, x/\bar{u}} \tag{4.4}$$

$$D_t = K.d.u^* \tag{4.5}$$

Where:
$\dot{M}$ = rate of input of mass = Q.c = Effluent discharge x concentration
$\bar{u}$ = Mean velocity (m/s)
d = depth (m)
D_t = transverse dispersion coefficient
u^* = shear velocity = 0.1 x u
x = distance from source
k = 5.93
b = plume width
σ = 25% of plume distribution extent

Equations 4.3 and 4.4 assume radial dispersion of the effluent in all directions. The discharge at Kooragang will be bound by shoreline resulting in a plume width of 2σ and a maximum concentration of 2 x C_{max}.

During dry conditions the plume is predicted to stay close to the shoreline, generally travelling between 400 and 700 m either side of the discharge location depending on tidal amplitude. Each time the tide reverses the plume is transported back over the discharge location, mixing with newly discharged effluent resulting in an increasingly concentrated buoyant layer.

Maximum likely concentration for a continuous effluent discharge with linear distance since release is shown in Figure 4.1 and 4.2 for Total Nitrogen and Total Phosphorus respectively for Scenario 1 effluent. Plume width from the edge of the harbour with distance travelled is presented in Figure 4.3.

The distance travelled by a particle during an ebb/flood tidal cycle varies between approximately 800 m and 1400 m depending on tidal amplitude and phasing. Therefore, the maximum distance the effluent will have to disperse/dilute prior to the addition of fresh effluent is conservatively 1500 m. As can be seen from Figures 4.1 and 4.2, Nitrogen and Phosphorus concentrations are in excess of double the background concentrations within this range.

Subsequently, during normal dry conditions, nutrient levels will be maintained above double the already elevated background concentrations. TSS was not assessed as the effluent discharge concentration is less than that of the background NFR concentration.

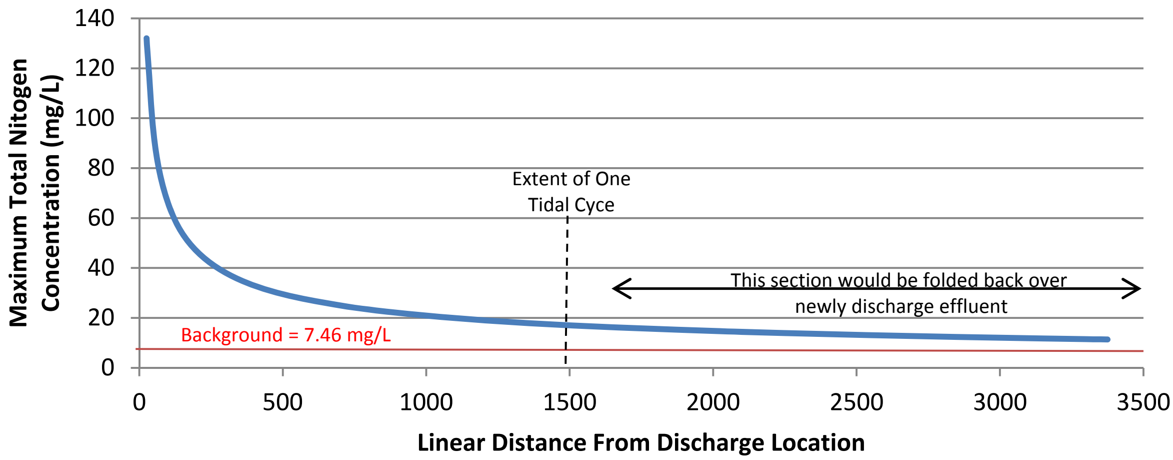


Figure 4.1: Maximum Total Nitrogen Concentration for Scenario 1 Effluent Discharge

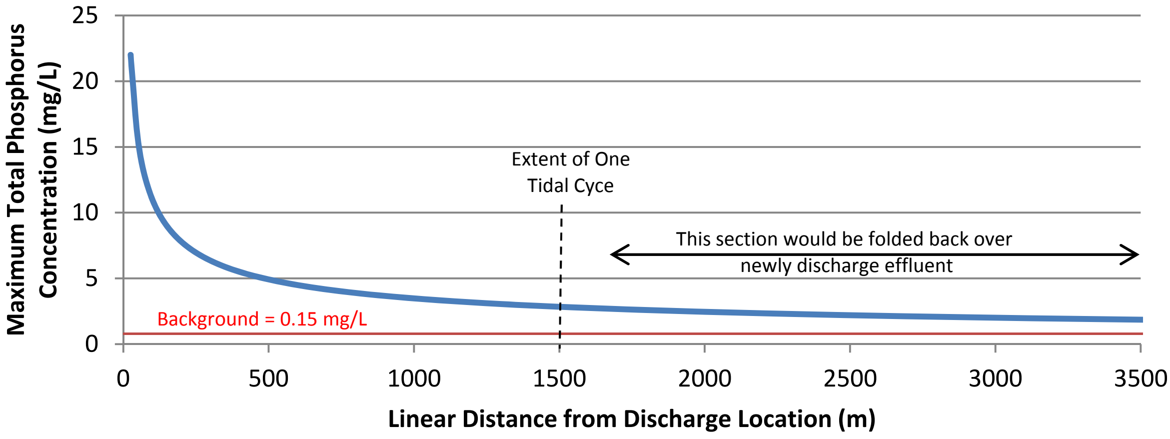


Figure 4.2: Maximum Total Phosphorus Concentration for Scenario 1 Effluent Discharge

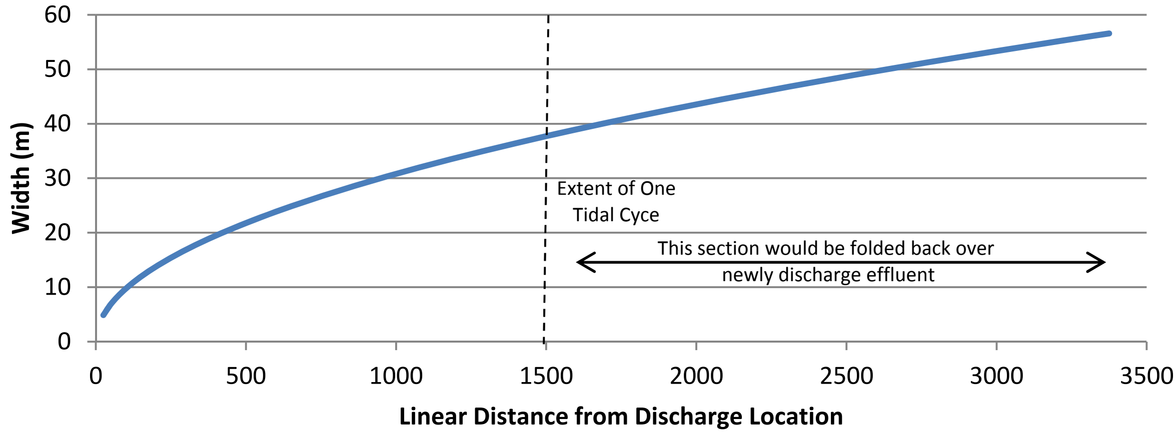


Figure 4.3: Plume Width

4.2 Effluent 'Puff' Discharge

Effluent discharge was then assessed as a series of discrete additions of pollutant mass creating dispersing 'puffs' of pollutant in the South Arm receiving water. This approach enables the interaction and extent of plume spreading to be visualised with reference to the site.

This approach considered a 'puff' of pollutant dispersion radially as described by Equation 4.4. The mass of pollutant was then considered to have a Gaussian distribution within the diameter of the plume and be constrained to a stratified depth of 0.2 m.

A 'puff' of arbitrary pollutant was added to the receiving water. The was repeated with a 'puff' of pollutant being released every three hours to show the oscillation of multiple plumes with the tide and subsequent confluence of effluent plumes (Figure 4.4). This visual interaction in conjunction with the analysis in Section 4.1 was used to assess the impact of reversing tides.

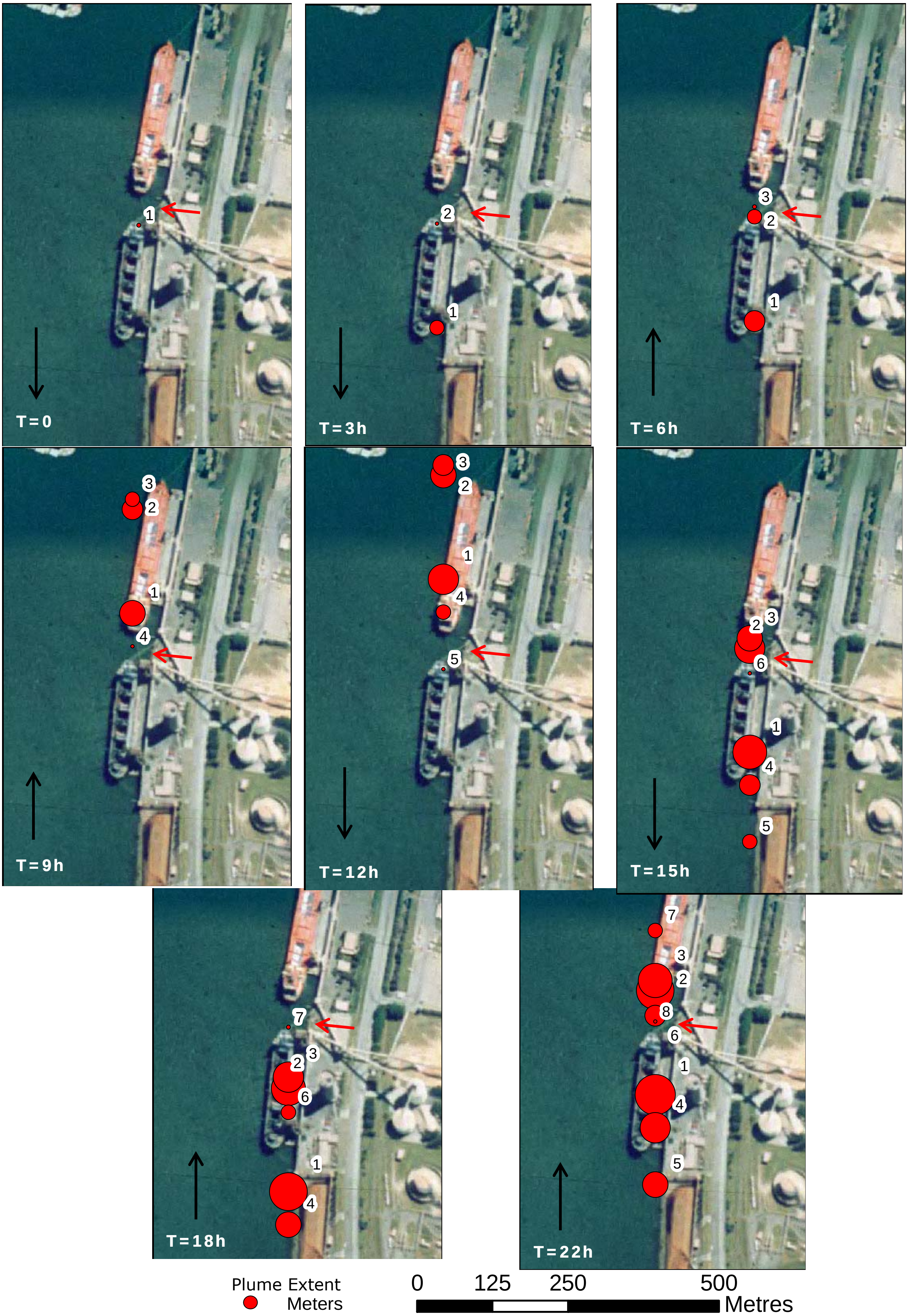


Figure 4.4: Discrete Plume Width and Interaction over Two Tidal Cycles

Figure 4.4 presents oscillating tidal velocities resulting in the interaction of multiple effluent 'puffs'. From assessment of the tidal velocities it is possible to determine the linear distance travelled over a tidal cycle. Again, the distance travelled varies based on tidal amplitude and phasing, generally carrying between 400 and 700m with each flood or ebb tide.

If effluent 'puffs' are to interact, then low concentrations are required before 'puff' interaction occurs. From Figures 4.1 to 4.2, the maximum concentration of a 'puff' along with position is known. These concentrations do not reach a low concentration (i.e. still have more than double background) before the plume is transported back over the discharge location and interacting with older/younger effluent 'puffs'. Figure 4.4 shows that interaction of plumes is likely to occur, resulting in addition of plume concentrations. Limited dispersion, combined with plume interaction will result in high concentrations of effluent in the receiving water.

In summary, analysis in Section 4.1 calculates that effluent concentrations will not disperse to low concentrations over a flood-ebb tidal cycle, with Section 4.2 confirming that plume interaction will occur resulting in elevated effluent concentrations. The outcome from this analysis is that a surface discharge, either continuous or discrete discharge, is unlikely to result in low effluent concentrations.

5. Consideration of Discharge from a Subsurface Diffuser

Since low ambient currents limit far-field mixing and dispersion, an alternative is to increase the initial near-field dilution via a subsurface outlet. This approach would result in significantly greater initial dilutions. The discharged effluent entrains large volumes of ambient water during the rise of the buoyant plume through the water column. Far-field dispersion and diffusion would then continue to reduce the effluent concentrations towards background 90th percentile levels. The far field dispersion following sub-surface discharge would be far greater than that of a surface discharged effluent, as the depth of the buoyant layer is significantly greater. The near-field model JETLAG was run for the Scenario 1 and 2 effluent characteristics (Table 1.1). Results for each discharge scenario are presented in Tables 5.1 and 5.2.

The following discharge configuration was used:

Outlet diameter = 50 mm
Depth of discharge = 11.5 m
Receiving water temperature = 19^{o}C with no stratification (Zone B mean)
Receiving water salinity = 22 ppt (Zone B mean)
Receiving water velocities = 50th, 90th and 95th percentile velocities during dry conditions (Table 3.1).

Table 5.1: Near-field Dilutions for Flow Discharged Normal to the Current Direction

Ambient Velocity	Dilution Under Scenario 1 Effluent Conditions	Dilution Under Scenario 1 Effluent Conditions
50th Percentile	164	162
90th Percentile	174	173
95th Percentile	175	174

This can be compared with dilutions of only 1 to 3 times from the open channel discharge configuration. Dilution is usually defined as:

$$S = \frac{total\ volume\ of\ sample}{volume\ of\ effluent\ contained\ in\ the\ sample} \quad (5.1)$$

Furthermore, the effect of dilution on the concentration of a constituent, C can be described by Equation 5.2.

$$S = \frac{C_d - C_s}{C - C_s} \quad (5.2)$$

Where:
C_s = background concentration of substance in ambient water
C_d = concentration of substance in effluent
C = concentration of substance

From equation 5.2 it is possible to obtain concentrations of the effluent constituents at the end of the near field (Table 5.2).

Table 5.2: Water Quality Constituent Concentrations at the End of the Near-Field Zone During 50th Percentile Ambient Currents for Scenario 1 Effluent Discharge

Water Quality Constituent	Effluent concentration (mg/L)	Background Concentration (90th Percentile – Zone B) (mg/L)	Concentration (mg/L)	Concentration above background (mg/L)
Total Nitrogen	150	7.46	8.33	0.87
Total Phosphorus	25	0.15	0.30	0.15
Total Suspended Solids	30	76	75.62	0

Table 5.3: Water Quality Constituent Concentrations at the End of the Near-Field Zone During 50th Percentile Ambient Currents for Scenario 2 Effluent Discharge

Water Quality Constituent	Effluent concentration (mg/L)	Background Concentration (90th Percentile – Zone B) (mg/L)	Concentration (mg/L)	Concentration above background (mg/L)
Total Nitrogen	75.0	7.46	7.88	0.42
Total Phosphorus	10.0	0.15	0.21	0.06
Total Suspended Solids	15.0	76	75.62	0

While the concentrations may increase over the long term in the low receiving water currents, the entrainment of lower water column waters will minimise this risk.

6. Summary

The impact of the proposed effluent discharge from the Incitec Pivot ammonium nitrate plant on Kooragang Island was assessed. Effluent concentrations in the receiving water were estimated based on an open channel surface discharge into the South Arm of the Hunter River. Far-field dispersion was assessed based on the effluent characteristics and the receiving water conditions. Dispersion due to wind and boat wake effects were not included.

Tidal velocities indicate that net transport during dry periods will be up the South Arm. Freshwater inflow into the South Arm was found to be limited as the majority of inflows are conveyed in the North Arm of the lower Hunter River. However, freshwater flow does result in a reversal of the net transport to discharge towards the ocean. This net discharge to the ocean was found to be short lived in comparison to longer periods of tidal ingress in the South Arm.

As net transport was found to be upstream from the discharge location, water quality data for the South Arm was used as the reference condition. Sanderson and Redden (2001) produced statistics of major water quality parameters, with the 90th percentile being used as the reference condition as per ANZECC (2000) recommendations.

The buoyant nature of the effluent, in conjunction with poor mixing potential in the receiving water resulted in low far field dispersion during worst case environmental conditions (i.e. no wind and no freshwater inflow). Over the course of a tidal cycle, effluent was found to be maintained at levels higher than double the background over an area approximately 1,000 m either side of the discharge. This is important as once a full tidal cycle has been complete, the effluent plume will pass back over the discharge location and be mixed with newly discharged high concentration effluent.

An alternative solution was subsequently investigated by WRL. A sub-surface discharge located near the harbour bed was investigated and near-field modelling was undertaken. It was found that high dilution (~160 times) was achievable based on the mean receiving water temperature and salinity. Resulting effluent concentrations were found to be significantly lower than that of a surface effluent discharge. Total Phosphorus was found to be approximately double the background, and Total Nitrogen was found to be within 15% of the background concentration by the end of the near field during Scenario 1 conditions.

If a surface discharge were to be further investigated, WRL would recommend detailed hydrodynamic modelling of the South Arm and harbour entrance be undertaken to determine percentile exceedence of effluent throughout the South Arm, under varying environmental and discharge conditions.

Furthermore, WRL recommends that a sub-surface discharge be investigated further to determine an optimal discharge configuration, maximising dilution and minimising risk to the receiving water. Concentrations are significantly lower with such a discharge configuration, but WRL makes no assessment whether these receiving water concentrations would be environmentally or regulatorily acceptable.

If you would like further information on either of these options, please do not hesitate to contact myself on (02) 8071 9800.

Yours sincerely,

G P Smith
Manager

7. References

ANZECC (2000), Australian Water Quality Guidelines for Fresh and Marine Waters, Australian and New Zealand Environment and Conservation Council.

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Schiller, E.J. and Sayre, W.W. (1973), Vertical mixing of heated effluents in open channel flow, Iowa Institute of Hydraulic Research rep. No. 148.

Schulz, K.G., Riebesell, U., Rost., Thoms, S. and Zeebe, R.E. (2006), Determination of the rate constants for the carbon dioxide to bicarbonate inter-conversion in pH buffered sea water systems, Marine Chemisty 100: 53-65

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THE UNIVERSITY OF NEW SOUTH WALES

Water Research Laboratory

School of Civil and Environmental Engineering

9 May 2012

WRL Ref: WRL2012019 DSR:GPS L20120509

Mr David Fuller
Senior Principal
URS Australia Pty Ltd
Level 6, 1 Southbank Boulevard
Southbank VIC 3006

By Email: david.fuller@urs.com

Dear David,

Kooragang Island Thermal Discharge – Background Water Quality and Recommendations

Following on from our teleconference on Friday 13th April 2012, this letter contains further investigation of water quality in the Hunter River with respect to establishing a local receiving water reference condition and WRL's recommendation on how to proceed.

1. Water Quality Reference Condition

As discussed in our letter issued 28th March, 2012 (WRL Ref: WRL2012019 DSR:BMM L20120328), water quality was initially investigated based on the Zones established by Sanderson and Redden (2001) as shown in Figure 1.

Zone B was initially selected as the reference condition for setting the background water quality level as dry weather conditions were found to result in net inflow up the South Arm. However, the proposed discharge location borders on the boundary of Zones A and B. Due to the difference in key nutrient concentrations in Zones A, B and C, the sites directly adjacent to the proposed discharge location were assessed (Figure 2). Key water quality parameters are detailed in Table 1. The number of samples is also included to demonstrate the basis of the water quality statistics.

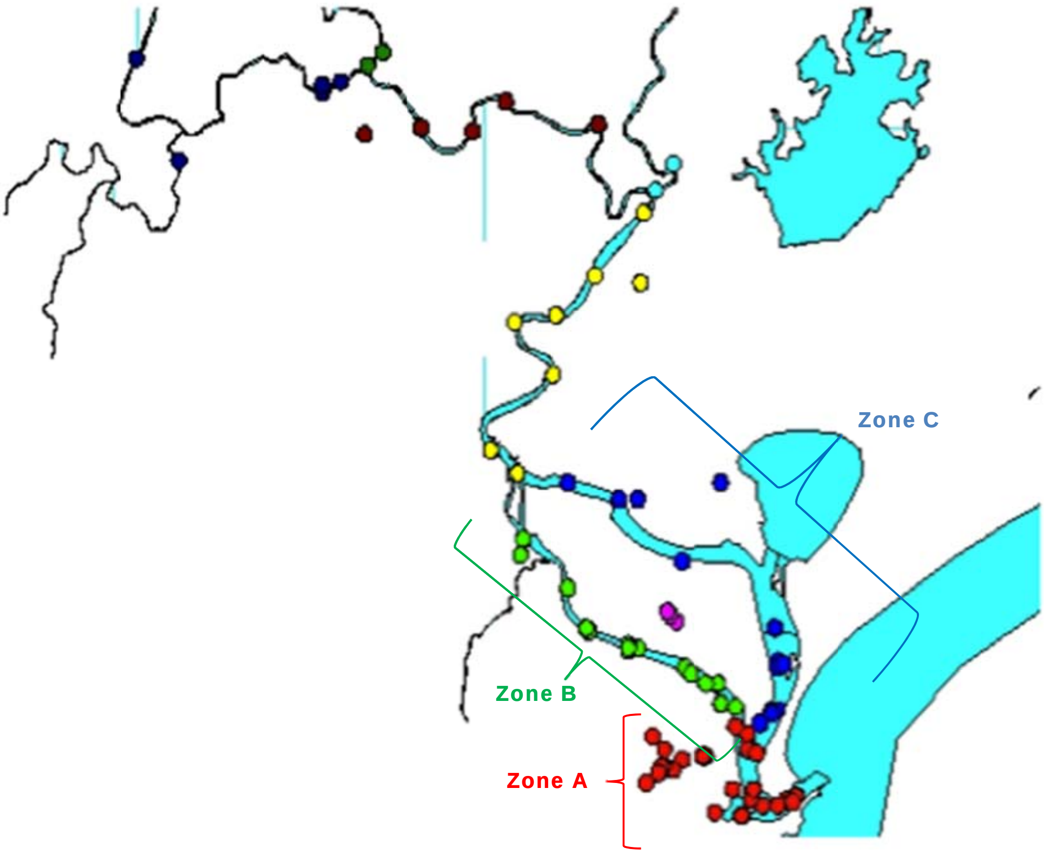

Figure 1: Water Quality Monitoring Locations / Zones.
Zone A = Red, Zone B = Green and Zone C = Dark Blue. (Sanderson and Redden, 2001).

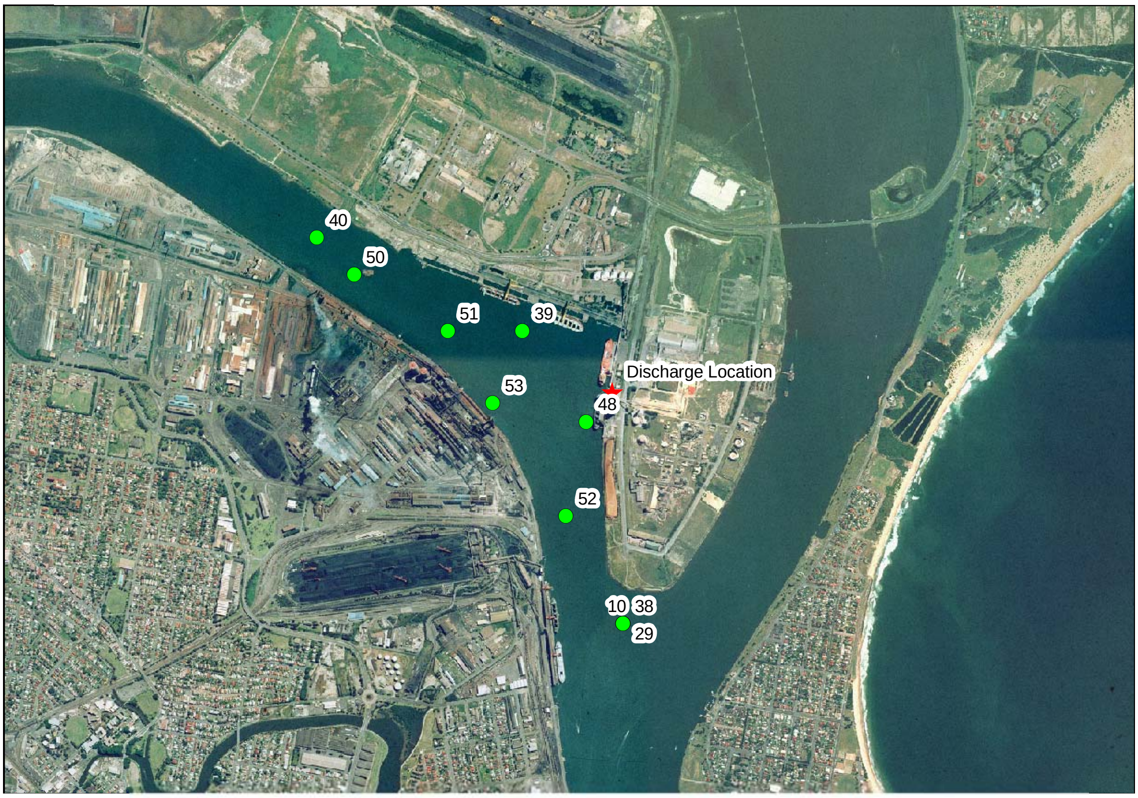

Figure 2: Water Quality Monitoring Locations Adjacent to Discharge Location

Table 1: 90th Percentile Concentrations for Key Water Quality Parameters at Sites near Discharge Location

Site ID	Zone	TP		NO3		NO2		TKN		NFR	
		mg/L	# Samples	mg/L	# Samples	mg/L	# Samples	mg/L	# Samples	mg/L	# Samples
10	A	1.07	94	1.955	40	0.07	40	1.7	94	42	40
29	A										
38	A	0.36	28					19.01	28	73	25
39	B	0.49	17					16	11	87.5	18
40	B	0.32	6					11.74	4	29.5	2
48	B	0.49	11					8.36	7	28	8
50	B							6.8	2	9	43
51	B							5.8	2	15	90
52	B	0.05	9					7.1	11	19	49
53	B	0.1	11.0					8.3	3	25	20

Key water quality parameters at the sites in Figure 2 (Table 1) were compared to the water quality statistics from Zones A, B and C (Figure 1). The 90th percentile concentrations for the sites near the discharge location were higher than 90th percentile concentration of Zone B, but less than 90th percentile concentration of Zone A.

Table 2: 90th Percentile Concentrations for Key Water Quality Parameters

Parameter	Units	ZONE A 90th Percentile	ZONE B 90th Percentile	ZONE C 90th Percentile	Nearby Sites 90th Percentile
Total Phosphorus	mg/L	0.53	0.15	0.27	0.64
Non Filterable Residual	mg/L	190.00	76.00	104.00	39.03
Nitrate	mg/L	0.63	0.39	0.33	1.96
Nitrite	mg/L	0.03	0.07	0.04	0.07
Total Kjeldahl Nitrogen	mg/L	14.50	7.00	10.30	9.42
Total Nitrogen*	mg/L	15.16	7.46	10.67	11.45

*Total Nitrogen = Sum of Nitrate, Nitrite and Total Kjeldahl Nitrogen

The 90th percentile concentrations of key water quality parameters are higher at sites near the discharge location when compared to previously presented Zone B statistics.

Table 3 presents the expected concentration above background for different dilutions at the end of the near-field for the Scenario 1 effluent concentrations.

Table 3: Key nutrient concentrations above background concentrations for various dilution at the end of the near-field zone.

Dilution	Initial Total Nitrogen Concentration (mg/L)	Total Nitrogen above background (mg/L)	Initial Total Phosphate Concentration (mg/L)	Total Phosphate above background (mg/L)
50	150	3.00	25	0.50
100	150	1.50	25	0.25
150	150	1.00	25	0.17
200	150	0.75	25	0.13
250	150	0.60	25	0.10

2. Revised Discharge Location

The previous preliminary assessment undertaken by WRL (WRL Ref: WRL2012019 DSR:BMM L20120328) showed that higher initial dilutions could be achieved via discharge through a sub-surface diffuser. Furthermore, relocation of the discharge outfall to a position that promotes flushing of effluent from the South Arm would increase the potential for far-field mixing. A revised discharge location was suggested by URS (Figure 3).

At the previous discharge location, released effluent was persistent in the South Arm due to slow receiving water velocities and subsequent poor flushing. A location closer to the end of Walsh Point would potentially promote flushing of effluent from the South Arm towards the confluence of the North and South Arms of the lower Hunter River estuary.

While discharge from a subsurface diffuser would increases initial mixing when compared to a surface discharge, effluent constituent concentrations are limited by the background constituent concentration. As stated in the previous letter report provided by WRL (WRL Ref: WRL2012019 DSR:BMM L20120328), discharged effluent is mixed with receiving water resulting in a constituent concentration at the end of the near field mixing zone that is elevated above the background concentration.

WRL's preliminary analysis of a sub-surface diffuser showed that this configuration could achieve dilutions in the order of 160 times. Higher dilutions may be achieved by optimising the design and location of the sub-surface diffuser.

Figure 3: Revised Discharge Location

3. Recommendations

Previous work undertaken by WRL found a surface discharge at the initial proposed location is not feasible due to the buoyant nature of the effluent and the poor mixing potential of the receiving water. Analysis by WRL showed that higher initial dilutions are achievable via a sub-surface diffuser discharge. Subsequent revision of the discharge location resulted in a second proposed discharge location located near Walsh Point which may potentially increase mixing and flushing of the effluent. Quantification of the likely improvement in mixing would require further detailed analysis.

It is WRL's recommendation that the regulatory authorities be consulted at this stage of the project to ensure that this approach is feasible. The NSW Office of Environment and Heritage (OEH) will need to approve the outlined water quality reference condition and set the required concentration of effluent parameters by the end of the near-field zone. This is typically set on a case-by-case basis.

The governing port authority will also require consultation regarding the location of a sub-surface diffuser in a busy shipping channel. The restrictions specified for the diffuser location and required concentrations will determine the necessary diffuser design.

4. Conclusion

The information detailed by WRL in this letter and the previous letter report (WRL Ref: WRL2012019 DSR:BMM L20120328) provides a preliminary assessment of the feasibility of an effluent discharge into the South Arm of the lower Hunter River estuary. If you have any questions or comments please do not hesitate to contact myself or Duncan Rayner on (02) 8071 9800.

Yours sincerely,

G P Smith
Manager

URS Australia Pty Ltd
Level 6, 1 Southbank Boulevard
Southbank VIC 3006Australia
T: 61 3 8699 7500F: 61 3 8699 7550
www.ap.urscorp.com