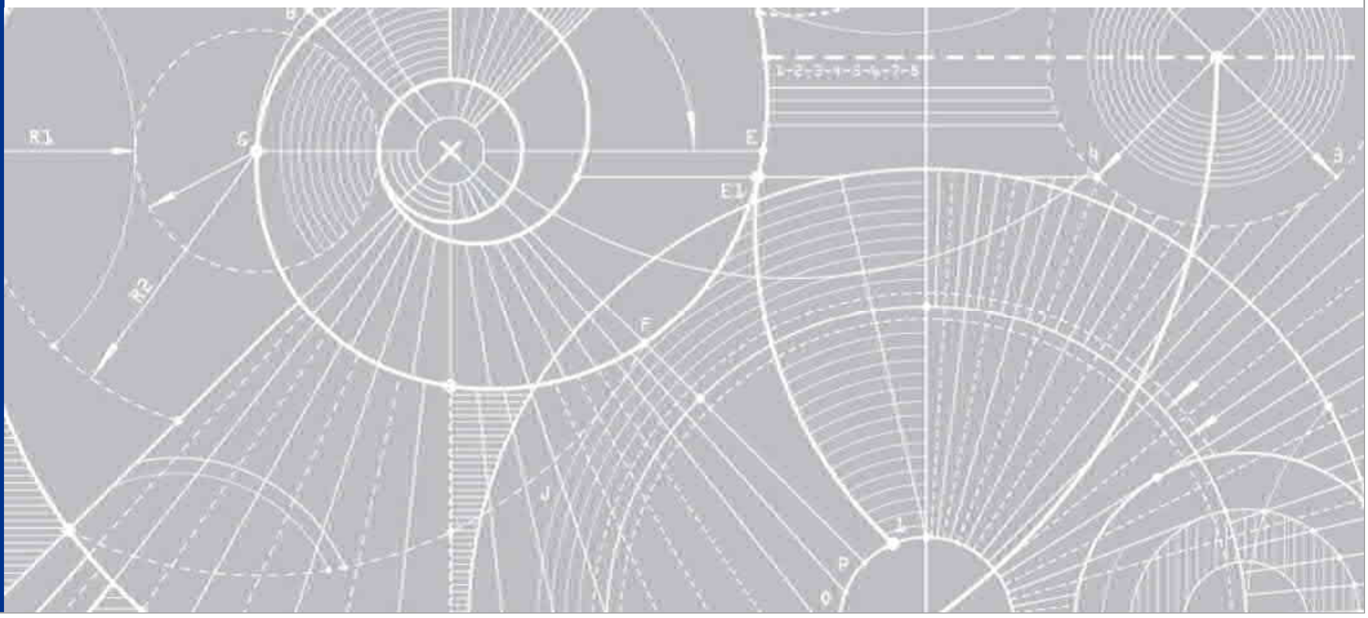
St Mary's Central Precinct

LEND LEASE

St Mary's Central Precinct Plan: Water, Soil and Infrastructure Report

20 September 2017



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St Mary's Central Precinct

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Document history and status

Revision	Date	Description	By	Review	Approved
Draft	02/03/15	Supersedes May 2009 report	MM	JC	JC
Draft	16/08/17	Supersedes March 2015 report	JC	ED	ED
Final	20/9/17	September 2017 updated report for rezoning submission	JC	ED	ED

Contents

Executive Summary

This report was originally produced and approved in May 2009. It has now been updated at the request of the NSW Planning & Environment department to support a request by St Marys Land Limited and Lendlease Development Ltd for an addendum to the *Sydney Regional Environmental Plan No 30 – St Marys.*

The majority of the 2009 approved report remains unchanged, and the main updates are limited to the revised layout plan Figures in chapter 1, the water quality assessment in Appendix B to make it consistent with the proposed regional basins rezoning, and the flood assessment in Appendix D to include the latest flood assessments that were completed in July 2015 and consequently approved by Penrith City Council (PCC).

Jacobs has prepared this report for Lend Lease to provide background information, describe existing and proposed conditions and provide water, soil and stormwater infrastructure management strategies for the Central Precinct of the site at St Marys. The report addresses the following aspects in relation to the Central Precinct of the site at St Marys:

- Introduction, background and proposed development
- The existing environment
- Performance objectives
- Management strategies for water quality; the stormwater trunk drainage system; and soil and water issues.
- Management strategies for groundwater and salinity
- Filling of land

The proposed **stormwater quality** management strategy for the Western Precinct is based on the principles of ecologically sustainable development and water sensitive urban design (WSUD). This strategy includes the use of water quality controls such as gross pollutant traps (GPT), stormwater basins and biofiltration basins.

The proposed development involves changes to the local catchments, including an increase in the amount of impervious area. A stormwater quantity assessment has demonstrated that runoff from the Precinct would not impact on peak flows in South Creek and therefore detention is not necessary. This is due to the location of the Central Precinct just upstream of the Creek and therefore is unique to this location within the St Marys development. Given that two basins are proposed for water quality purposes, however, these may be used for detention purposes.

Soil bore**, groundwater** and geophysical investigations in the Central Precinct indicate that shallow groundwater occurs at depths of 3 - 6 m and is of low salinity. It is concluded that the planned development is unlikely to result in surface salinisation and that the measures proposed in the report including raising the ground level by filling and consideration of limiting infiltration will further reduce this possibility.

The Central Precinct lies to the west of South Creek and the site is at risk **of flooding** from this watercourse. The proposed development involves filling the site to a level high enough so that it would be flood-free in a 100 year ARI event.

The Development Application (DA) for the adjoining Dunheved Precinct has been approved by PCC and this anticipates and reflects a filling scenario over the Central Precinct. The fill scenario for Central Precinct has been refined however the flood impacts are generally the same. Mitigation measures and detailed information are summarised in Appendix D.

A portion of the Central Precinct would be subject to the probable maximum flood (PMF) event and evacuation would be necessary. The flood evacuation strategy is provided in a separate report by Molino Stuart.

These measures proposed and detailed herein would achieve the SREP30, EPS and PCC requirements..

Important note about your report

The sole purpose of this report and the associated services performed by Jacobs is to provide a water, soil and stormwater infrastructure strategy for the Central Precinct of St Marys in accordance with the scope of services set out in the contract between Jacobs and the Client. That scope of services, as described in this report, was developed with the Client.

In preparing this report, Jacobs has relied upon, and presumed accurate, any information (or confirmation of the absence thereof) provided by the Client and/or from other sources. Except as otherwise stated in the report, Jacobs has not attempted to verify the accuracy or completeness of any such information. If the information is subsequently determined to be false, inaccurate or incomplete then it is possible that our observations and conclusions as expressed in this report may change.

Jacobs derived the data in this report from information sourced from the Client (if any) and/or available in the public domain at the time or times outlined in this report. The passage of time, manifestation of latent conditions or impacts of future events may require further examination of the project and subsequent data analysis, and re-evaluation of the data, findings, observations and conclusions expressed in this report. Jacobs has prepared this report in accordance with the usual care and thoroughness of the consulting profession, for the sole purpose described above and by reference to applicable standards, guidelines, procedures and practices at the date of issue of this report. For the reasons outlined above, however, no other warranty or guarantee, whether expressed or implied, is made as to the data, observations and findings expressed in this report, to the extent permitted by law.

This report should be read in full and no excerpts are to be taken as representative of the findings. No responsibility is accepted by Jacobs for use of any part of this report in any other context.

This report has been prepared on behalf of, and for the exclusive use of, Jacobs's Client, and is subject to, and issued in accordance with, the provisions of the contract between Jacobs and the Client. Jacobs accepts no liability or responsibility whatsoever for, or in respect of, any use of, or reliance upon, this report by any third party.

1. Introduction

1.1 Background

The St Marys Development site was endorsed by the NSW Government for inclusion on the Urban Development Program (UDP) in 1993. The site is owned by St Marys Land Limited and is being jointly developed by ComLand Limited and Lend Lease Development Pty Limited through their joint venture company, Maryland Development Company.

The site is located approximately 45km west of the Sydney CBD, 5km north-east of the Penrith City Centre and 12km west of the Blacktown City Centre. The main western railway line is located approximately 2.5km south of the site. The Great Western Highway is located another 1 km south and the M4 Motorway a further 1.5km south. The site has an area of 1,545 ha and stretches roughly 7km from west to east and 2km from north to south. It is bounded by Forrester Road and Palmyra Avenue in the east, The Northern Road in the west, Ninth Avenue and Palmyra Avenue in the north and the Dunheved Industrial Area, Dunheved Golf Clun and the suburbs of Cambridge Gardens, Werrington Gardens and Werrington County in the south.

The overall site, which has been rezoned for a variety of uses, comprises 6 developments “Precincts”, namely the Western Precinct, Central Precinct, North Dunheved Precinct, South Dunheved Precinct, Ropes Creek Precinct and Eastern Precinct. The boundaries of the Precincts within the St Marys site are shown in **Figure 1-1**.

Because the St Marys site straddles the boundary between two local government areas (i.e. Blacktown and Penrith), the State Government decided that a Regional Environmental Plan should be prepared to guide and control future development of the land.

Technical investigations into the environmental values and development capability of the land were commenced in 1994, and State Regional Environmental Plan 30 (SREP30) was subsequently gazetted in January 2001.

SREP30 is the main statutory planning framework document for the St Marys site. It contains planning principles, objectives and provisions to control development. The overarching aim of SREP30 is to provide a framework for the sustainable development and management of the St Marys site. The original precinct and zone boundaries of SREP30 were altered by the gazettal of Amendment No 1 in April 2006.

SREP 30 is accompanied by the St Marys Employment Planning Strategy (EPS) which identifies the aims for the future use and management of the site and sets out specific performance objectives and strategies to address key planning issues, including: conservation, cultural heritage, water and soils, transport, urban form, energy and waste, human services, employment, and remnant contamination risk.

The St Marys EPS identifies actions to be undertaken by local and State governments, as well as the obligations of developers. A Development Agreement was entered into in December 2002 between the joint venture developer and the NSW Government setting out the developer’s and State Government’s responsibilities in providing services and Infrastructure. A Development Agreement has also been entered into between PCC and the joint venture developer for the Dunheved Precinct and PCC wide transport contributions and will be updated for other contributions required as a result of the development of the Central and Western Precincts.

SREP30 requires the development control strategies contained within the St Marys EPS to be taken into account in any development proposals for the St Marys site. It also requires that a Precinct Plan be adopted by Council prior to any development taking place. Planning for any precinct is to address all of the relevant issues in SREP30 and the St Marys EPS, including preparation of management plans for a range of key issues.

On 29 September 2006 the Minister for Planning declared the Central Precinct to be a release area in accordance with the provisions of SREP30.

Figure 1-1 Precinct boundaries

1.2 Proposed Development

The Central Precinct is bounded by existing residential development in the suburbs of Werrington County and Werrington Downs to the south, land zoned for Regional Open Space to the east and land zoned for Regional Park to the north and west. There is also an area zoned for Drainage that adjoins the northern boundary of the precinct. The Precinct has a total area of approximately 133.1 ha. The land within the Precinct is proposed to be zoned Urban is intended to accommodate primarily residential uses, with limited non-residential uses such as local retail and commercial uses.

The proposed development of the Central Precinct, as shown in the Framework Plan at Figure 1-2, entails:

- A Village Centre zone, comprising a mix of community, open space and residential uses, in the central part of the Precinct
- Areas of open space
- Construction of roads, including connections to both the west and east, and stormwater infrastructure.
- Predominantly residential development in the remainder of the precinct

1.3 Purpose of this Report

This report has been prepared in accordance with the requirements of SREP30 and the EPS. It supports the draft Precinct Plan for Central Precinct and has been prepared to assist in determining the proposals for, and the planning principles, strategies and development controls that will guide the future development of all land within the Precinct in an integrated manner.

While the focus of the report is on the Central Precinct, the investigations carried out have taken into account the following:

- Relationship of the future development within the Precinct to the adjoining Regional Park,
- Future integration with the balance of the site and the existing surrounding neighbourhoods

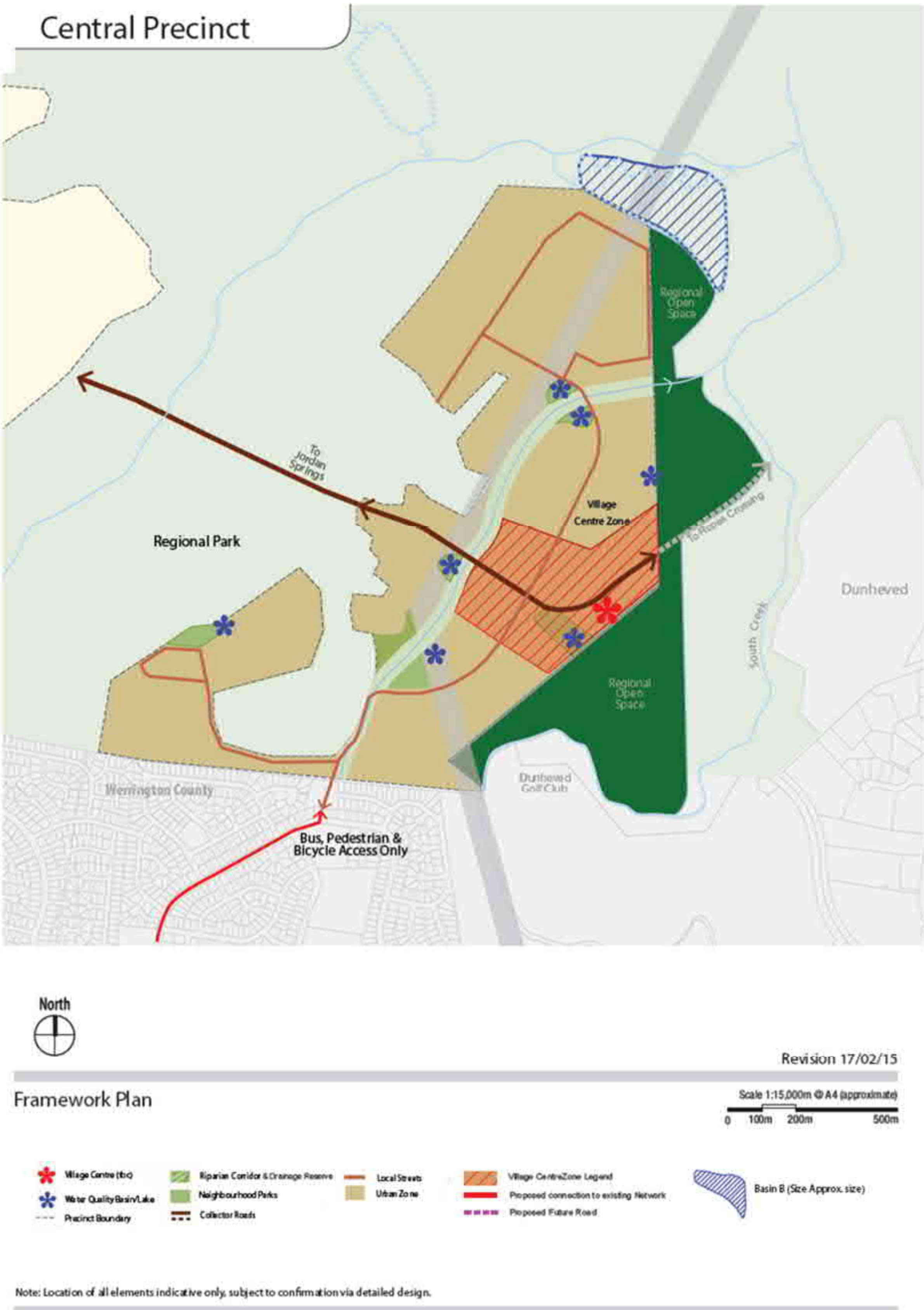


Figure 1-2 2015 Framework plan

2. Existing environment

2.1 Topography

The Central Precinct land surface is generally flat. Elevations vary from 29mAHD to 40mAHD within the Precinct area. The site generally drains via some minor drainage lines to South Creek which lies to the east of the Precinct.

2.2 Soils

Based on the Penrith 1:100,000 soil landscapes map (Bannerman and Hazelton, 1990) the two soil units within the site area include the Luddenham (lu) and South Creek (sc) soil landscapes (SL). The first is predominant within the southern and western third portion of the site, while the South Creek SL covers the remainder. A more detailed description is provided in **Section 5** of this report.

2.3 Groundwater & Salinity

Two groundwater-bearing systems are present within the St Marys site. These are referred here as the shallow and deep aquifers, but regolith (soil) and fractured shale bedrock aquifers would be more accurate titles. Neither would normally be regarded as true aquifers because of their low permeability, limited storage capacity, inhomogeneity and indefinite boundaries. A more detailed description is provided in **Section 5** of this report and **Appendix C.**.

2.4 Water quantity

There are two drainage lines in which runoff leaves the Central Precinct. The northern section of the Precinct drains in a north east direction towards South Creek, while the southern section drains south east to join South Creek just inside the site boundary.

An XP-RAFTS model was set up to predict existing peak flows from the site for a range of storm events. Details and results of the XP-RAFTS model are included in **Appendix A**.

A site specific *Stormwater Detention Strategy Report (2017)* has also been produced by Cardno in January 2017 that provides details on the hydrological assessments and the proposed detention strategy for Central Precinct. This report has been approved by PCC.

2.5 Water quality

The Central Precinct has been previously cleared and is currently fenced off to keep macro fauna (kangaroos and emus) within the site. The assessment of any potential impact on stormwater quality as a result of the proposed development needs to review existing water quality conditions and predict developed conditions (with water quality controls). In order to estimate the existing runoff pollutant loads and determine the effectiveness of the proposed stormwater treatment train, a water quality model was set up to estimate pollutant loads for existing and proposed (with controls) conditions. Details and results of the MUSIC water quality model are given in **Appendix B**.

2.6 Flooding

The Central Precinct lies to the west of South Creek and currently a portion of the site is below the 100 year ARI event in South Creek and a concurrent 20 year ARI flood in the Hawkesbury Nepean River. More details are included in Section 6 of this report. **Appendix D** has been updated to provide details of the latest flood assessments undertaken in 2015 that were approved by PCC..

3. Performance Objectives

The performance objectives for water, soils and infrastructure components are detailed in the SREP30 and the EPS, both prepared by the NSW Department of Urban Affairs and Planning, now the Department of Planning and Environment. The SREP and EPS objectives are summarised in this section along with an overview of the proposed management strategies. Sections of the report are referenced to identify where more information can be found.

Designs are also subject to the requirements of the Draft Penrith Development Control Plan (DCP) 2014, Section C3 Water Management; and PCC's Water Sensitive Urban Design (WSUD) Policy (2013) and Technical Guidelines (2014).

Table 3-1 SREP and EPS performance objectives for water, soil and infrastructure.

SREP 30 / EPS Clause No.	Requirement	Where addressed in this report
Content of draft precinct plans		
10.2.e	A draft precinct plan is to include proposals for, and information about, the following, for the land to which it applies: · drainage systems and flooding issues, including an assessment of the risk of flooding and damage likely to result	Flood evacuation
10.2.n	A draft precinct plan is to include proposals for, and information about, the following, for the land to which it applies: · any other major infrastructure, such as above or below ground trunk electrical systems, trunk sewerage or water supply lines	Services infrastructure
Conservation		
24.4 / 4.3.4	Infrastructure is to be designed and located to minimise potential adverse impacts on the conservation values of land.	Services infrastructure
EPS 4.4.11	Litter and pollution control measures designed to limit the entry of waste material into the creeks will be regularly maintained and monitored.	Catchment management strategy
Watercycle		
28.1 / 6.3.1	During and following construction, impacts upon water quality are to be minimised, through the utilisation of effective erosion and sediment control measures in accordance with industry standards.	Catchment management strategy
28.2 / 6.3.2	The use of the land to which this plan applies is to incorporate stormwater management measures that ensure there is no net adverse impact upon the water quality (nutrients & suspended solids) in South Creek and Hawkesbury-Nepean catchments.	Catchment management strategy
28.3 / 6.3.3	Water usage on and the importation of potable water onto the land to which this plan applies are to be minimised.	Catchment management strategy
28.4 / 6.3.4	Development is to be designed and carried out so as to ensure that there is no significant increase in the water table level and that adverse salinity impacts will not result.	Soils, groundwater & salinity

SREP 30 / EPS Clause No.	Requirement	Where addressed in this report
28.5 / 6.3.5	There is to be only minimal impact upon flood levels upstream or downstream of the land to which this plan applies as a consequence of its development.	Filling of land
28.6 / 6.3.6	Drainage lines are to be constructed and vegetated so that they approximate as natural a state as possible. Where it is necessary to modify existing drainage lines to accommodate increased stormwater runoff from urban areas, this should be done in a manner which maximises the conservation of indigenous flora in and around the drainage lines.	Catchment management strategy
28.7 / 6.3.7	Development is to be carried out in a manner that minimises flood risk to both people and property.	Filling of land
28.8 / 6.3.8	Changes in local flow regimes due to development are to be minimised for rainfall events up to the 50 percent AEP rainfall event.	Catchment management strategy
28.9 / 6.3.9	Gross pollutants are to be collected at, or as close as possible to, their source or at all stormwater outlets, or at both of those places, so that there is no increase in sediment/litter entering creeks as a result of development.	Catchment management strategy
Soils		
29 / 6.3.10	The development is to have regard to soil constraints to ensure that the risk of adverse environmental and economic impacts is minimised.	Soils, groundwater & salinity
Land below the PMF level		
49.5	Road systems on land which would be affected by the PMF are to be designed to facilitate safe evacuation during flood events.	Filling of Land
Services		
60	Development must not be carried out on any land to which this plan applies until arrangements have been made for the supply of water, sewerage drainage and underground power that are satisfactory to the consent authority.	Services infrastructure
EPS - Water & Soils		
6.4.3	There will be no formed trunk drainage channels on land zoned for the regional park.	Catchment management strategy
6.4.4	Water and drainage infrastructure through the regional park will be confined to existing established easements agreed with the National Parks Wildlife Service prior to transfer of the land with the exception of those drainage basins identified in the structure plan.	Catchment management strategy
6.4.5	A series of combined wetland/detention basins and wetlands will be provided on the site generally in locations outlined in the structure plan. The total wetland area on the site will be between 2% and 4.8% of the development catchment area.	Catchment management strategy
6.4.6	Additional investigations will be undertaken at the precinct plan stage to identify the exact boundaries and development capacity of the identified soil types.	Soils, groundwater & salinity

SREP 30 / EPS Clause No.	Requirement	Where addressed in this report
6.4.7	A precinct plan will include sufficient information on infrastructure design and management measures to demonstrate that water usage will be managed within the constraints of the Sydney Water Corporation service criteria and obligations.	Catchment management strategy
6.4.8	A watercycle management strategy will be prepared for each release area and submitted with each precinct plan. The strategy will identify the detailed actions, measure and design principles that will be implemented to meet the performance objectives relating to watercycle management. The strategy will: a) include infrastructure design and management measures which will minimise potable water usage on the site details will include: · incorporating best practice measure for the reuse of stormwater for irrigating open space areas · reducing demand on potable water · minimising adverse impacts on local groundwater regimes · incorporate measure in the infrastructure design, which ensure that changes in local flow regimes which result from the proposed development are minimised b) identify arrangements for the ongoing maintenance and monitoring of the watercycle management system c) ensure constructed trunk drainage channels are designed to convey the 100 year average recurrence interval (ARI) d) identify the relationship between staging of development within the precinct and the timing of provision of stormwater management measures.	Catchment management strategy
6.4.9	An electromagnetic induction (EM) survey of the site will be undertaken and submitted with the first precinct plan. The survey of all land will identify areas of high recharge as well as zones of concentration of salts in discharge areas.	Soils, groundwater & salinity

SREP 30 / EPS Clause No.	Requirement	Where addressed in this report
6.4.10	A groundwater management strategy will be prepared for each release area having regard to the findings of the EM survey, and be submitted with each precinct plan. The strategy will deal with: a) planning infrastructure such as subdivision layout and the location of dwellings, roads, wetlands and stormwater detention basins b) the cumulative impacts of development c) measures to be incorporated into the development to ensure the appropriate management of groundwater resources, such as: · adopting small garden/lawn areas to reduce irrigation requirements · planting low water requirements plants · using mulching cover – this shall not occur in drainage lines · including low flow watering facilities to avoid over watering by residents · introducing and implementing a tree planting program (especially in high recharge areas) plant species should be native, deep-rooted, large growing species, which will assist in retention of the groundwater at existing levels · retaining existing native tree cover wherever possible · not permitting drainage basins, infiltration pits or tanks to disperse surface water · promoting the use of drought resistant grasses within the development area.	Soils, groundwater & salinity
6.4.11	A flood evacuation plan must be prepared for each precinct and will be consistent with the regional flood evacuation plan prepared by the State Emergency Service. The plan will be submitted with the draft precinct plan. The plan will: a) demonstrate that continuously graded evacuation routes above the PMF for South Creek and the Hawkesbury- Nepean River are provided b) provide for progressive evacuations of developed areas within the site c) identify temporary evacuation centres on high ground.	Filling of Land
6.4.12	The information available on flooding and evacuation will be consistent with the education program in place for all lands similarly affected in the local government area.	Filling of Land

SREP 30 / EPS Clause No.	Requirement	Where addressed in this report
6.4.13	Precinct plans will incorporate the following trunk drainage system requirements: a) stormwater control facilities will be implemented on the site designed to prevent adverse impact on water quality as a result of development b) the stormwater management system for the site will be designed in accordance with the following requirements, unless alternative designs or specifications can meet the performance objectives outlined in section 6.3 above: · wetlands and detention basins will be designed to prevent thermal stratification: applicants will consider this objective in statements of environmental effects which accompany applications for such facilities · wetlands may need to be lined with an appropriate material to guard against water infiltration to the groundwater system · wetlands will be regularly cleared of noxious weeds · detention basins/wetlands will include native macrophytes and wetland species which will assist in erosion and sediment control and promote biodiversity · basins will meet the relevant Dam Safety Committee requirements · all basins and surrounding landscapes will be designed to allow machinery to undertake scheduled maintenance work every 1.5 years or less the design of basins and surrounding landscapes will facilitate access for machinery to undertake less frequent maintenance.	Catchment management strategy
6.4.14	On land subject to the PMF, precinct plans will ensure that services such as power, potable water, sewerage and drainage are located to minimise disruption during floods and will consider the need for flood proofing (consistent with the *NSW Floodplain Development Manual* or its successor) to guarantee supply.	Services infrastructure
6.4.15	The sewer system infrastructure for the site will: a) be designed to utilise best practice connections and construction techniques to result in a better ‘sealed’ or low infiltration system b) ensure pressure tests are carried out to ensure systems integrity c) ensure house connections are to be cut and welded as the system is built d) implement other best practice measures as appropriate at the time of development e) ensure that pumping station designs eliminate dry weather overflows and mitigate odour generation.	Services infrastructure
6.4.17	All trunk drainage infrastructure will provide appropriate safety measures to the consent authority’s satisfaction.	Catchment management
6.4.18	All trunk drainage infrastructure will be designed to reduce constraints on the flow of floodwaters, especially in relation to events above 1 percent AEP.	Catchment management strategy

SREP 30 / EPS Clause No.	Requirement	Where addressed in this report
6.4.19	Measures will be incorporated into infrastructure design to minimise demand for potable water. These will include: · specifying low water demand fixtures in all dwellings and other buildings where appropriate · limiting maximum pressure by managing system zonings (pressure zoning) having regard to critical water supply needs such as pressure for fire fighting · including above ground rainwater tanks for dwellings on lots greater than 400m^2 · using stormwater for irrigating open space areas · incorporating other best practice measures at the time of development.	Catchment management strategy

4. Catchment Management Strategy

4.1 Strategy objectives

The objectives of the total catchment management strategy are

- To safeguard the environment by improving the quality of stormwater runoff entering receiving waters.
- To protect geomorphic values of waterways
- To achieve efficient use of water and minimise demand for potable water
- Minimise changes to local flow regimes
- Minimise flood impacts upstream and downstream of the development
- Integrate stormwater management into the landscape
- Maximise source controls for runoff quantity and quality

These objectives would be achieved by employing current water management practice, which could incorporate the following water quality and quantity controls in the development:

- Rainwater tanks on residential lots for private irrigation reuse
- Water saving fixtures within the buildings
- Bioretention vegetated areas in open space areas
- Gross pollutant traps
- Constructed stormwater wetlands, dry infiltration bioretention basins or stormwater basins.
- Detention storage, where deemed necessary, integrated into basins.

This section outlines the proposed water quantity and quality controls that from the foundations of the catchment management strategy. It demonstrates that their performance meet these objectives.

4.2 WSUD Objectives

The objectives of the total catchment management strategy are in line with PCC's WSUD Policy (2013) and Technical Guidelines Ver. 3 June 2015.

4.3 Design principles

The water cycle management strategy for the Central Precinct development will be based on design principles and performance objectives detailed in the following documents:

- SREP No 30, 2001; and
- St Marys Environmental Planning Strategy (EPS), 2000

The adopted strategy will also consider additional local, state and federal government documents listed below:

PCC

- Development Control Plan 2014, Section C3 Water Management
- Water Sensitive Urban Design Policy, October 2013
- Water Sensitive Urban Design Technical Guidelines, Version 3 June 2015.
- Stormwater Drainage for Building Developments (Working Draft), 2013
- Engineering Construction Specification For Civil Works (Working Draft), 2013

State Government

- Department of Environment & Climate Change (DECC), Managing Urban Stormwater, Environmental Targets, Draft October 2007.
- South Creek Stormwater Management Plan, 1999-2000, Stormwater Trust
- Landcom, Soils and Construction, Vol. 1 2004, Vol. 2 2008

4.4 Background to Watercycle Management for the Project

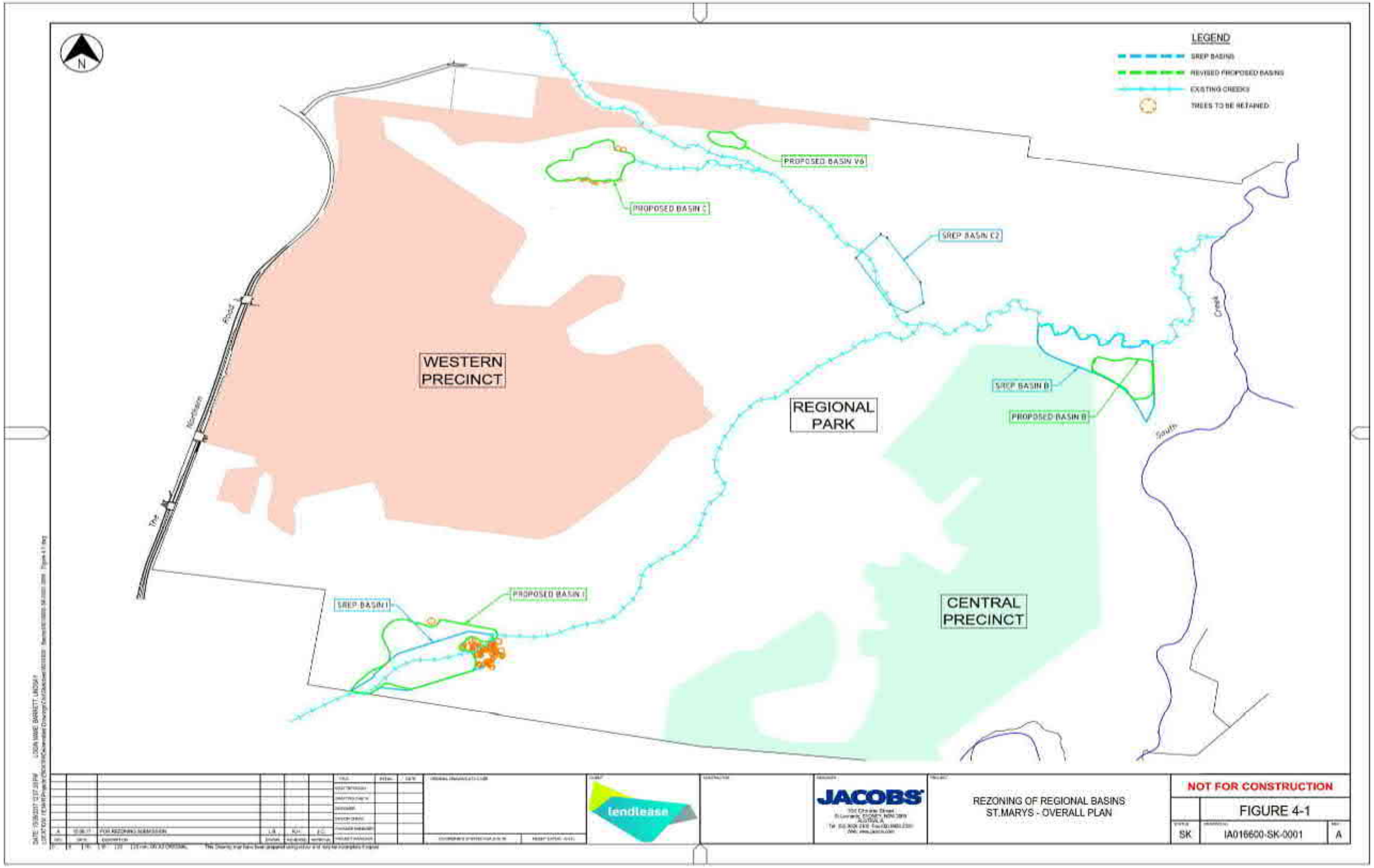
In 1998, a Watercycle Management Report was prepared by SKM, “ADI St Marys Watercycle and Soil Management Study, Final Study Report, August 1998”. The 1998 Study informed SREP30 and was published prior to the Federal Government (Australian Heritage Commission) announcement of lands at St Marys being listed on the Register of the National Estate (RNE). This resulted in a reduction of around 33% of the developable area within Precincts zoned under the original gazettal of SREP30. The SREP30 required amendment to reflect the RNE listing and the subsequent State Deed.

In 2005, SKM reviewed the previous assessment to identify the required number, size and location of stormwater management ponds within the Regional Park in accordance with the revised proposed SREP30 Land Use Plan to meet the water objectives. A history of pond sizes and what is currently proposed is shown in **Table 4-1**. Following the completion of the Central Precinct design of water quality controls, the regional basin locations and sizes were revisited to provide an optimum outcome for the Regional Park area and to also comply with the requirements SREP30 for the Western and Central Precincts. Refer to **Figure 4-1** for the previous SREP and proposed regional basins.

Table 4-1 Stormwater Management Pond History and Proposed for the Western and Central Precincts

Stormwater Management Pond ID	1998 Study (Basis of SREP30) Wetland land take (ha)	SREP30 Amendment (2005) Drainage zones within Regional Park land take (ha)	Precinct Plan (2009) Minimum land take (ha)	Current Precinct Plan (2017) Minimum land take (ha)
A1 – East Lake	2.2	-	2.5	5.4
A2 – Jordan Springs	3.7	-	2.8	3.5
B – Central Precinct	6	8	8	3.03
C1	3.4	-	2	-
C2	2.8	4.5	4.5	Refer to Basins C and V6 below
C	-	-	-	3.8
V6	-	-	-	0.7
C3	1.4	-	-	-
D – Central Precinct	0.6	-	2	0.39
E – Central Precinct	1.4	-	1	0.94
F	0.6	-	-	-
G	0.7	-	-	-
H	1.6	-	-	-
I	4	7.4	7.4	9.72

Figure 4-1 SREP and proposed re-zoned basin locations



4.5 Stormwater Quantity Management

4.5.1 Strategy objectives and design principles

The general objectives of the stormwater trunk drainage system are to:

- Safely convey runoff through the proposed development
- Integrate with the road and lot layout
- Integrate with the water cycle management system such that runoff quality and quantity are controlled efficiently.
- Account for the catchment management objectives listed in **Section 4.1**.

The design principles to achieve these objectives are based on the documents listed in **Section 4.3**.

4.5.2 Methodology

To achieve the management objectives, an assessment was undertaken using the XP-RAFTS hydrological model. Details of the assessment are included in **Appendix A**.

4.5.3 Proposed Drainage System

The following components would make up the drainage system. Design guidance is based on the PCC Stormwater Drainage for Building Developments (2013).

- Pit and pipe system able to carry flows up to the 5 year ARI storm
- Overland flow paths able to carry flows up to the 100 year ARI storm
- Open channels able to carry flows up to the 100 year ARI storm
- Combined detention / water quality basins able to provide the necessary quality and quantity controls, while also coping safely with the 100 year ARI flow.

4.5.4 Detention requirements

The 133 hectare site of Central Precinct is located immediately adjacent to South Creek and represents approximately 0.4% of the South Creek catchment area upstream of the site. Due to its location and small size relative to the South Creek catchment the proposed development has a negligible impact on flooding in South Creek. This has been demonstrated by hydrologic modelling which shows discharges from the site will be minimal compared to the discharges in South Creek even if stormwater detention is not provided. As the Central Precinct development will have a negligible impact on flooding in South Creek, stormwater detention is not considered necessary to mitigate against an increase in peak flow from the development. It is proposed that a merit based approach assessment be adopted for either zero or up to a maximum of two detention basins in the Central Precinct as supported by the hydrological/detention analysis in **Appendix A** which provides more details of the assessment.

4.5.5 Hydraulics

Channel top widths will be defined for the trunk drainage system during further consultation with the NSW Office of Water (NOW) regarding their requirements of channel makeup and riparian offsets under the Water Management Act, 2000. It is anticipated that the top widths will vary from 10m in the upstream catchments to 30m further downstream towards South Creek.

4.5.6 Classification of Watercourses

The Water Management Act, 2000 states a requirement to identify ‘rivers’ within the development site. Following a site inspection undertaken with the NOW (Department of Water and Energy at the time), the “rivers” for the Central Precinct as shown on **Figure 4-2** were identified. It was agreed with NOW that the ‘rivers’” will be refined during further consultation with NOW.

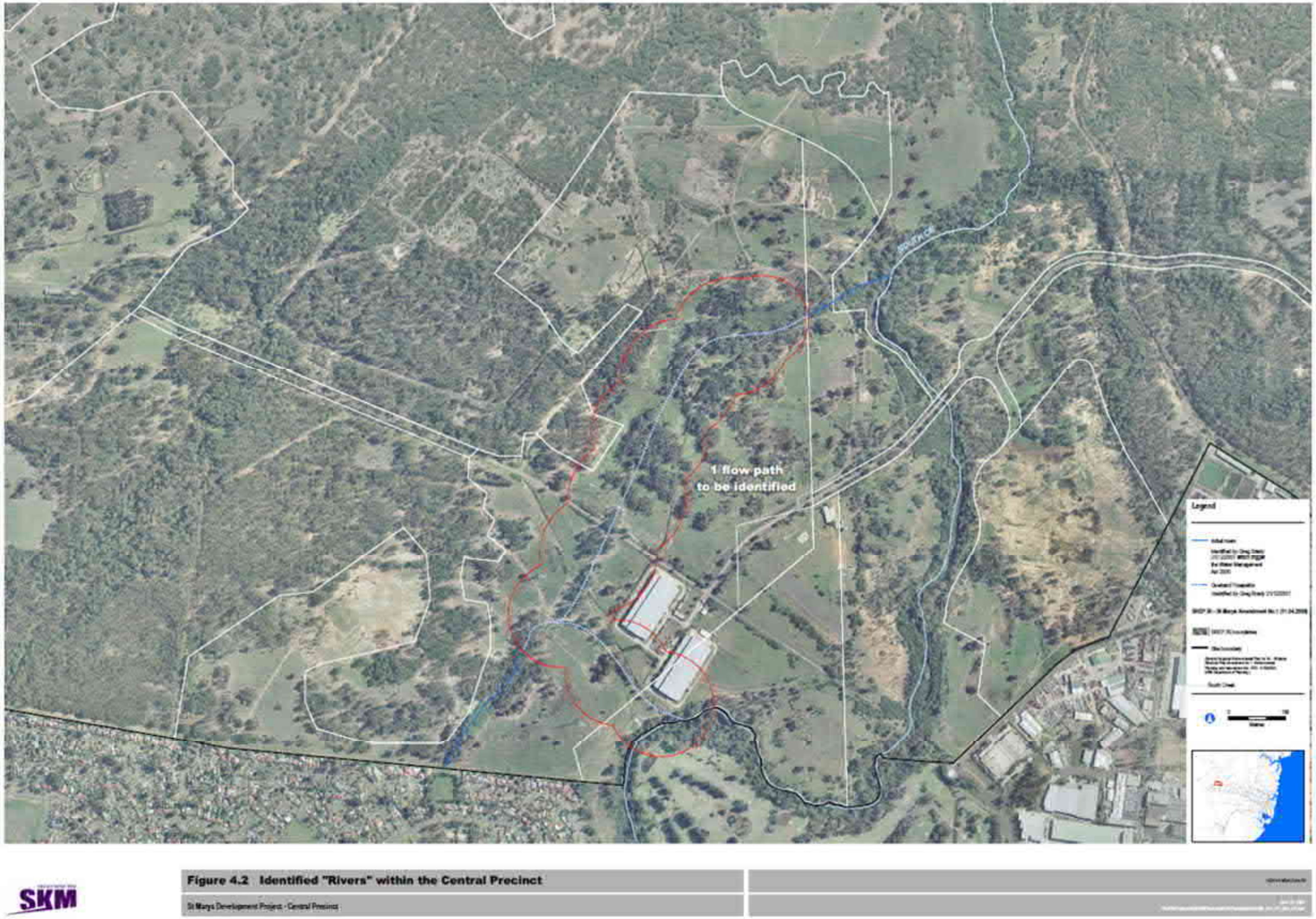


Figure 4-2 Identified ‘rivers’ within the Central Precinct

4.5.7 Stream forming flows

Channel form is a key aspect of stream health. The proposed development should not increase erosion in the downstream South Creek in comparison to pre-development levels. An assessment of stream forming flow has been undertaken using the stream erosion index (SEI) method, in accordance with PCC's WSUD Technical Guidelines (2014) and eWater's DRAFT MUSIC Guidelines (2010).This is a simplified method used in lieu of more complex 'erosion potential index' methods that require hydraulic modelling, particle size distribution analysis and quantification of shear stress. The SEI method instead uses flow as a surrogate to indicate erosion potential. It compares two scenarios – the post developed flow and existing flow regimes - calculating the annual average sum of flows that exceed a critical flow threshold. This critical flow threshold is the estimated flow above which erosion is expected to occur within the waterway. The SEI is ultimately calculated as a developed to existing flow conditions and should lie in the range of 2 to 5. An SEI in excess of 5 means that the development has a very high potential to cause erosion in the receiving stream.

The critical flow threshold is defined as x% of the 2 year ARI peak flow for existing conditions, where 10%< x < 50% depending on the substrate conditions of the receiving waterway. The 2 year ARI peak flow was calculated for the Western and Central Precincts using the probabilistic rational method described in Section 1.4.1a of the Australian Rainfall and Runoff Book 4. In line with studies undertaken in the past for Sydney, a value of 50% of the 2 year ARI peak flow was used, equalling 8.16m^3/s.

The MUSIC model was used to calculate the average annual flows in excess of this critical flow for existing and developed conditions. Routing was not included in the model and the effect of basins and rainwater tanks were removed. This generates a conservative result as it does not include factors that would contribute to reduced discharge from the development compared to the existing condition, including: lag times; attenuation; exfiltration from the base of basins; and reuse from rainwater tanks for garden irrigation. The results are shown in **Table 4-2**.

Table 4-2 Stream flow index

Parameter	Value
Q2, peak	16.32 m3/s
Q2, 50%	8.16 m3/s
Annual flows greater than critical flow – existing (a)	431 ML/yr
Annual flows greater than critical flow – developed (b)	919 ML/yr
Stream flow index (a/b)	2.1

The assessment demonstrates that flows above the existing critical flow approximately double as a result of the development, i.e. the SEI is equal to 2.1. This would be due to an increase in impervious area in the developed catchment. It is, however at the lower end of the safe range and also represents a conservative estimate. Thus it can be concluded that, according to the assessment methodology defined by PCC (2014) and eWater (2010), the channel form of South Creek will not be detrimentally affected by the proposed development.

4.6 Stormwater Quality Management

4.6.1 Strategy objectives

The water quality objective for the St Marys Project is to ensure that there is no net adverse impact upon the water quality in South Creek, as stated in the SREP30. Since the last Precinct Plan, PCC has issued the DCP (2015) and WSUD Policy (2013) and Technical Guidelines (2015) that include stormwater pollution retention criteria. For this assessment the 'No net adverse impact on South Creek' objectives have been applied as required by SREP30. The PCC WSUD (2013/2015) objectives have been applied for the individual precincts, these, are outlined in **Table 4-3**.

Table 4-3 Stormwater pollutant load reduction targets to mitigate development

Pollutant	WSUD Policy (2013) and Technical Guidelines (2015)
Total suspended solids (TSS)	85%
Total phosphorus (TP)	60%
Total nitrogen (TN)	45%

Three receiving water quality control points are located at South Creek as shown in **Figure 4-1**. These combined water quality points (S1, S2 and S3) receive all surface runoff generated from the Western and Central Precincts, and existing urbanised areas located further upstream of this catchment. The pollutant load assessments are undertaken at these locations, in accordance with the SREP30.

The design principles used for the strategy are based on the documents listed in **Section 4.3**.

4.6.2 Proposed Water Management System

To meet strategy objectives a number of stormwater management controls would be integrated into the overall drainage concept to manage stormwater quality and quantity where. These would be in line with PCC's WSUD Policy (2013) and Technical Guidelines (2015). The elements of the water management strategy are based on a hierarchy of stormwater management controls and create a stormwater treatment train. These controls could include:

Source controls

- At the residential lots, rainwater tanks maybe used to capture roof water for reuse. If recycled water is available, then rainwater tanks may be used depending on the demands on the lot.
- Bioretention systems will be provided where possible depending on the topography and gradients on site. These will be local neighbourhood type small open space areas that will act as large dry infiltration basin and will provide the start of treatment of stormwater runoff higher up in the sub-catchments. The treated runoff will be captured and conveyed in the drainage piping system and will not infiltrate into the natural soils.

Conveyance and treatment controls

- Stormwater that enters the piping system, would then pass through a gross pollutant trap (GPT) located immediately upstream of a larger dry infiltration basin, basin or a wetland. The GPTs would remove coarse sediment, litter and debris that are generated on the roads.
- Dry infiltration basins and or standard stormwater basins will be provided to supplement the treatment of stormwater provided by the source controls and GPTs. Runoff from a dry infiltration basin would be collected by perforated pipes located in the base of the infiltration system and discharged as polished stormwater into the downstream waterways. Stormwater basins are sized such that discharge has been treated through biochemical process within the basin.
- A variety of stormwater management basins have been investigated over the course of this project. This assessment recommends that two dry infiltration basins be included in the Central Precinct – basins D and E - in addition to basins C2, I and B that lie upstream within the wider Western and Central Precinct Plan area. These work within a wider stormwater strategy encompassing both the Western and Central Precincts. Basins I and B are required in particular to achieve the project water quality objectives of the Central Precinct and would be progressively constructed during the development
- Whilst dry infiltration basins and stormwater basins have been proposed in this Precinct Plan it should be noted that other WSUD water control measures such as wetlands may also be considered as an alternative during the detailed design stage, in accordance with PCC's WSUD Policy (2013).

Natural Systems Controls

In addition to the above water quality controls, natural system controls will also be adopted where possible. Natural system controls involve the management of areas within the catchment and creek systems that will remain unchanged. The use of natural system controls does not necessarily involve specific structural control measures, but rather a general planning approach. Natural systems controls recognises that natural waterways, floodplains and native vegetation perform essential hydrological and ecological functions that cannot easily be replicated by constructed stormwater control measures.

Therefore essential elements of the natural system will be retained in the development, and where degraded they will be rehabilitated and may include:

- Open space areas located near natural drainage lines
- Existing native vegetation maintained where possible
- Revegetation with native species to batters and open space areas will assist in reducing stormwater pollutant loads, and therefore assist in improving the long term water quality.

4.6.3 Water quality assessment methodology

A water quality assessment has been undertaken for the Western and Central Precincts to ensure that the measures detailed above meet the stormwater quality objectives. The assessment has taken into account the effects of stormwater basins, dry infiltration basins and household rainwater tanks. At this point in time it has not included the additional benefits of the source, conveyance and natural system control described previously. This would result in relatively conservative sizing for the proposed basins. The assessment accounted for the contribution of flows from all catchments entering South Creek from the West that impact on the three receiving water quality control points. The assessment also accounted for the influence of the Jordan Springs Lake (Basin A2) that was constructed in 2013, and the East Lake (Basin A1) that has already been sized for DA approval.

The MUSIC model (eWater CRC, Version 6.2) was used to assess the impact of the Jordan Springs, East Lake, and proposed Basins C2, B, D, E, and I on reducing pollutant loads into South Creek. The main purpose of the modelling was to demonstrate that proposed treatment measures would meet the water quality objectives. The MUSIC modelling was undertaken following the procedures and recommendations in PCC's *WSUD Technical Guidelines* (PCC, 2014). Full details of the modelling are provided in **Appendix B**.

4.6.4 Size of proposed water quality basins

The MUSIC modelling was used to size the proposed basins, which provided an estimate of the land area needed for the proposed water quality measures. The land take requirements of all the proposed stormwater basins in the Western and Central Precincts were shown previously in **Table 4-1**

4.6.5 Stormwater quality conclusion

Stormwater Quality modelling has demonstrated that the proposed basins in the Central and Western precincts achieve the overall water quality objectives of the SREP 30, EPS and Individual precincts achieve the more recent PCC Guidelines (2013/2015)..

4.7 Maintenance of stormwater controls

4.7.1 Water quality controls

The pollutant retention capability of any water quality control device is subject to it being maintained appropriately. The efficiency of a control reduces as the device fills with pollutants and maintenance must occur before the performance of the device falls below expected levels. Thus, a maintenance schedule must be prepared for each control. There will be regular maintenance and monitoring of all pollution control mechanisms. These tasks will be undertaken by the developer for a period of three years and then taken over by Council. The initial operation and maintenance regime of the water quality controls is summarised below in **Table 4-4** these would be refined at the detailed design stage.

Table 4-4 Operation and maintenance of water quality controls

Item	Maintenance Requirements
Gross pollutant traps (GPTs)	GPTs upstream of the basins should be maintained every three months or after each storm event, as required.
Dry Infiltration Basins	The bioretention basins should be inspected annually for trapped sediments. Excessive sediment should be removed and disposed of properly to maintain the extended detention depth and volume of the biofiltration area. Excessive dead plant debris should be removed to reduce the organic material and nutrient loads in the biofiltration area.
Stormwater basins	The basin area should be inspected annually for trapped sediments. Excessive sediment should be removed and disposed of properly to maintain the design volume of the basin. Excessive dead plant debris should be removed to reduce the organic material and nutrient loads in the basin.

Maintenance manuals will be prepared for the management of the various stormwater facilities as part of the development application. These manuals will identify the timing of and requirements for:

- maintenance of grass cover within formed channels to prevent erosion of channel bed and banks
- control of weeds
- removal of litter, debris and coarse sediments deposited during floods to formed channels as necessary particularly from detention storages that are located above basins
- the maintenance regime for heavy and light machinery for cleaning of sediments and organic
- material deposited within all parts of the basin
- litter and sediments trapped in gross pollutant traps
- monitoring of vegetation type and growth
- maintenance of conditions to ensure mosquito control
- appropriate safety measures.

4.7.2 Maintenance of water quantity controls

All water quantity controls will be maintained by Lend Lease for an initial three year period following construction. After this time, PCC will be responsible for ongoing maintenance.

4.7.3 Detailed operation and maintenance information

In line with the PCC WSUD Technical Guidelines (2014, p.4), a Draft Operation and Maintenance plan will be included in the Development Application (DA). In turn, the final Operation and Maintenance Plan will be provided at Construction Certificate Stage. The Plans will detail the following information:

- Site description (area, imperviousness, land use, annual rainfall, topography, etc.)
- Site access description
- Likely pollutant types, sources and estimated loads
- Locations, types and descriptions of measures proposed
- Operation and maintenance responsibility (council, developer or owner)
- Inspection methods
- Maintenance methods (frequency, equipment and personnel requirements including Work Health and Safety requirements)
- Landscape and weed control requirements

- Operation and maintenance costs
- Waste management and disposal options, and
- Reporting.

4.8 Cost estimates

In line with the PCC WSUD Technical Guidelines (2015) costs estimates for the capital, operation and maintenance costs for the water cycle management measures will be included in the Development Application (DA).

4.9 Soil and Water Management Strategy

This section describes the Soil and Water Management Strategy (SWMS) for the construction phase of the project and with respect to groundwater and salinity management measures should be read in conjunction with section 5.9 and Appendix C. It provides a preliminary strategy that, in accordance with PCC WSUD Technical Guidelines (p.4), would be developed into an Erosion and Sediment Control Plan at Construction Certificate Stage.

4.9.1 Overall Approach

A soil and water management plan would need to be prepared as part of the development application. Its purpose is to safeguard the environment during the construction stages of the development.

The objectives of the SWMS are to:

- Provide an overall erosion and sediment control concept for the proposed development
- Control the erosion of soil from disturbed areas on the site
- Limit the area of disturbance that is necessary
- Protect downstream water quality
- Prevent any sediment-laden water from entering South Creek.

In addition to the controls that have been identified in the SWMS, Erosion and Sediment Controls Plans (ESCP) for the site would need to be prepared at the development application stage in accordance with the requirements of Chapter 3 of PCC's Engineering Construction Specification for Civil Works (Working Draft) (2013) and the Landcom Soils and Construction Manual Volumes 1 and 2 (2004, 2008) known as the "Blue Book". The ESCP would describe the requirements for erosion and sediment controls, such as handling of excavation and filling, sediment fences, diversion drains, top soil stockpiles and reuse of soils, barrier fences, energy dissipaters, check dams, temporary culvert crossings and sedimentation basins.

4.9.2 Management Measures

The following soil and water management measures would be used during the construction phase of the development.

Land Disturbance Protection

Land disturbance during construction will be minimised to reduce the soil erosion hazard on site and may include the following

- Clearly visible barrier fencing will be installed at the discretion of the site superintendent to minimise unnecessary site disturbance and to ensure construction traffic is controlled. Vehicular access to the site will be limited to only those essential for construction work and they will enter and exit the site only through the stabilised access points

- Soil materials should be replaced in the same order that they are removed from the ground. It is particularly important that all subsoils are buried and topsoils are replaced on the surface at the completion of the works
- The duration of all works, and thus the potential for soil erosion and pollution, should be minimised
- Where practical, foot and vehicular traffic will be kept away from all recently stabilised areas
- Stockpiles should be seeded.

Erosion and Sediment Control Measures

The relevant measures listed below to address erosion and sedimentation should be used on site:

- Temporary construction access points
- Sediment filter fences
- Weed-free straw bales
- Barrier fences
- Diversion drain banks/channels
- Check dams
- Temporary sedimentation basins
- Top soil stockpiles.

These control structures are described in the following sections.

Temporary Construction access points

A tyre wash station and stabilised entry/exit structure should be installed at the access point to the site to reduce the likelihood of vehicles tracking soil materials onto public roads. A shaker ramp (cattle grid) will also be used in addition to the stabilised gravel access.

Sediment Filter Fences

Sediment filter fences should be installed where needed to confine the coarser sediment fraction (including aggregated fines) as near to their source as possible.

Barrier Fences

Barrier mesh fences should be installed to define those areas on site that should not be entered to avoid unnecessary soil/land disturbance.

Diversion Drain Banks/Channels

Diversion banks intended to remain effective for more than 2 weeks will be rehabilitated when possible. Hessian cloth can be used if tacked with an anionic bitumen emulsion (0.5L/m2). Foot and vehicular traffic will be kept away from these areas. Pipe culvert crossings that can withstand the maximum expected trucks loads will be installed where required. Concrete encasement for the pipe may be used if needed.

Check Dams

Check dams should be installed on diversion drains that are laid on longitudinal slopes greater than 2.5% to reduce runoff velocities. Check dams are to be located at intervals of approximately 100m.

Temporary Sedimentation Basins

Sediment basins will need to be constructed as the first phase of earthworks operations. These basins would be located at the furthest downstream point in their sub-catchment to maximise the capture and treatment of surface runoff during the construction phase. The sedimentation basins will need to be designed to suit type D (Dispersible) soils. Stored contents of the basins should be treated with gypsum or other approved flocculating agents where they contain more than 50mg/L of suspended solids. An energy dissipater rip rap may be installed at the weir outlet located at the downstream end of each sediment basin outlet to reduce runoff velocities where required.

Dust control

Measures will be in place to manage dust during construction. Where possible, existing vegetation will be retained or prompt revegetation and mulching of disturbed areas will take place. Traffic will also be controlled to minimise dust. If necessary, dust suppression wetting may be employed.

Top Soil Stockpiles

Stockpiles will be constructed away from hazardous areas, particularly areas that are likely to have concentrated water flows. Stockpiles may be seeded.

4.9.3 Main Principles of Erosion and Sediment Control during Construction

The main principles for erosion and sediment control are summarised below:

- Stockpile and reuse all topsoil
- Divert clean runoff water from the upstream drainage system around the disturbed open trench area
- Restrict vehicular access to stabilised entry and exit points with controls to reduce soil export attached to excavators and truck tyres exiting the site
- Restrict access to areas that do not require land disturbance
- Provide adequately designed sediment fences, barrier fences, catch drains, check dams, sediment fences and other required structures
- Ensure that the temporary top soil stockpiles are protected from erosion when works are unlikely to continue for long periods. Ensure that stockpiles are not placed in the flow path of upslope runoff
- Make provisions for emergency quick clean-up and removal of any accidental spills of soil on
- to public property and provide tanker with pump to cope with accidental runoff
- Provide wire mesh and gravel inlet filters at stormwater kerbs (if any) located downstream of the entrance to the site to trap any accidental spill of soil material
- Monitor and maintain all sediment and erosion control measures
- Minimise additional solid disturbance activities during wet weather
- Undertake water quality monitoring at the outlet of the sediment basins to ensure compliance with the DECC (formerly EPA) guidelines
- Stabilise rehabilitated surfaces as soon as possible
- Obtain additional information needed from the “Soils and Construction”, Landcom 2004 manual.

4.10 Conclusion

This sections has outlined the catchment management strategy for the proposed Central Precinct at St Marys. The strategy is in line with SREP30, EPA and PCC’s objectives for WSUD and construction.

Stormwater Quantity modelling has demonstrated that for the Central Precinct only, detention is not necessary. This is due to the proximity of the Precinct to South Creek and the fact that the peak runoff from the Precinct occurs earlier than the peak flows in the Creek.

Stormwater Quality modelling has demonstrated that a strategy that encompasses basins both in the Central and Western precinct achieves the overall water quality objectives of the SREP 30, EPS and more critical PCC Guidelines (2013/2014) in South Creek.

During construction sediment laden runoff poses a risk to downstream receiving environments. A management strategy has been outlined to mitigation this. Following construction, stormwater infrastructure will require ongoing maintenance to achieve designed performance and so a high level maintenance strategy has been proposed.

The strategies outlined in this section will be detailed further as the design progresses through to the DA and detailed design stages.

5. Soils, Groundwater & Salinity Management Strategy

5.1 Background to Soils, Groundwater and Salinity

5.1.1 Potential Salinity Concerns

Urban development has been identified as having the potential to increase the salt load in western Sydney landscapes that may already exhibit significant salinity. Although salinity has been identified as being natural to the western Sydney environment and not a consequence of previous industrial land uses, it poses a concern to developers of new subdivisions in the western Sydney region.

The main factors which lead to salinity in western Sydney have been identified as:

- The low rainfall and high evaporation potential with a considerable range in wet and dry years
- The input of salts from natural rainfall (cyclic salts)
- The extensive area of saline groundwater underlying much of the plain which is known to rise near to the surface at some geologic and topographic boundaries
- The common presence of duplex soils (of the Luddenham and South Creek soil landscapes) which are prone to water logging on lower slopes
- Subsoil layers in these soils which have a high susceptibility to sodicity and/or salinity.

Salinity can occur in one of the following ways:

- When brackish or saline groundwater rises near to the surface and where plant evapotranspiration or capillary rise encourages salts to concentrate over time.
- Where salts from the drainage water gradually accumulate at the top of impermeable clay subsoil. This can lead to surface salinity when a hydraulic link allows salts to rise through the profile. Alternatively the subsoil is exposed by excavation.
- Where cyclic salts in rainfall accumulate over time in areas with poor drainage and are concentrated by evaporation. This may occur when the sub-surface flow is blocked by building foundations.
- Where salt from deeply weathered soil landscapes is mobilised by perched water tables. These salts contain a high proportion of sulphates, which adds to the importance of this type of salinity because of the aggressive impact of sulphates on concrete and brickwork.

5.1.2 Development Requirements

The SREP30 and the EPS specify the following requirements with respect to groundwater and land salinity issues, which are applicable to the site:

- There should be no significant rise in the water table or in groundwater salinity as a result of this development
- An electromagnetic induction (EM) survey of the Precinct should be carried out,
- A Groundwater Management Strategy should be prepared for the site.

5.1.3 Objectives

The objectives of this investigation works were to:

- Satisfy the requirements of the SREP30 and the EPS with respect to groundwater and land salinity issues in the site
- Assess the existing salinity conditions in soil and groundwater at the site
- Predict the potential impact of urban development on the site’s landscape, especially the potential to increase surface runoff salt load and rising water table which might bring saline groundwater to the surface,

- Provide mitigation and management measures to ameliorate potential salinity impacts in the proposed urban development.

5.1.4 Scope of Works

In order to achieve the objectives described above, the following scope works was undertaken:

- Review of previous investigations, published technical literature, aerial photographs, and existing regional, data relating to geology, soil landscape, hydrogeology, topography and geochemistry relevant to the site and salinity in particular
- Evaluation of past and current soil and groundwater salinity data at the site to determine the potential source, transport, transformations and fate of geochemical species, including the potential for salt load increase due to rise in groundwater recharge
- Evaluation of past and current groundwater data to infer groundwater contours and potential groundwater flow at the site, including the potential extent of interaction between groundwater and the surface water
- Onsite walkover with cable locating contractor to confirm presence underground services prior to undertaking intrusive investigations works
- Drilling and logging of 26 soil boring locations across the site (to a maximum depth of 3 m),and installation of 3 piezometers (to a maximum depth of 10 m) in locations within the northern, eastern and south western portions of the site
- Field measurements of electrical conductivity (EC) and pH, collection of soil and groundwater samples from newly installed piezometers and existing piezometers
- Laboratory analysis of soil and groundwater samples quality assurance / quality control for the established field measured parameters (EC and pH)
- Mapping subsurface conductivity across the site and, by extension, soil salt content, using electromagnetic induction (EMI) methods,
- Development of a conceptual hydrogeologic model and groundwater management strategy for the site, incorporating past and current regional, local, and site specific data on geology, topography, groundwater, and geochemistry.

The scope of works undertaken for the salinity assessment of the Central Precinct is described in detail in this report, which also aims to respond appropriately to the requirements specified in the SREP30 and the EPS. This report includes recommendations towards the mitigation and management of potential salinity issues in urban development.

5.2 Review of Previous Investigations

Groundwater and salinity investigations have been carried out on the St Marys site in several phases since 1991. The earliest work was undertaken by Mackie Martin and Associates (MMA), and was primarily concerned with potential soil and groundwater contamination resulting from the use of the St Marys site over the preceding fifty years as an explosives production facility. The results from this investigation phase are reported by Mackie Martin (1991) in two report volumes. More detailed investigations and remedial work were later carried out by ADI Ltd and are described in their validation reports (including ADI Ltd, 1996). In addition to the contamination results, these reports reveal much about the natural groundwater system and about the salt cycle in the area.

Later studies, from 1998, were largely directed towards geotechnical and water cycle investigations for those portions of the site proposed for residential development. These comprised:

- Water cycle investigation at ADI St Marys site by SKM (Sinclair Knight Merz, 1998)
- Soils, salinity and groundwater in the Western Precinct, investigated by EIS and SKM (Sinclair Knight Merz, 2001)
- The Eastern Precinct, investigated by Jeffery and Katauskas (J&K) for Patterson Britton (Jeffery and Katauskas, 2003),

- Soils, salinity and groundwater investigation in the Dunheved Precinct (Sinclair Knight Merz, 2004).

5.3 Precinct Description

5.3.1 Topography

The site is occupied approximately by 108 ha of alluvial terrace lying on South Creek and 25 ha of residual clay/weathered shale terrain. The alluvial terrace land surface is nearly planar, rising generally southwards from RL 17 to 28 m AHD and the residual clay/weathered shale terrain is steeper, rising generally westwards between RL 29 to 40 m AHD.

A main gully, a tributary of South Creek, drains along the centre of the site towards the southwest. This gully has a cut down from 2 to 4 m below the terrace level. At the time of the investigation the more northern portion of the gully consisted of a train of shallow pools and swampy areas, and the southern portion was generally dry.

The surface of the alluvial terrace is nearly level to undulating, with a number of very shallow wet depressions (relief 0.2 to 0.4 m), resembling gilgais. They differ from gilgais in that the soil is not noticeably expansive, shrinkage cracks are relatively uncommon and generally less than 10 mm wide, with no significant ground heaving. It was evident that many of these gilgai-like wet patches were much diminished in area as a result of the drought and some have been reduced to bare earth.

5.3.2 Regional Geology

Based on the Penrith 1:100,000 geological map (Jones and Clark, 1991) shown in **Figure 5-1**, the site is underlain by Triassic Bringelly Shale (from the Wianamatta Group) and Pleistocene to Tertiary alluvial sediments.

The Bringelly Shale formation has a maximum thickness of about 300 m, although at the site this is expected to be about 90 m, when combined with the underlying Ashfield Shale. Both of these shales in turn overlie the Hawkesbury Sandstone. The Bringelly Shale is composed of shale, mudstone, claystone and some sandstone. The shale rocks are dark grey when fresh but weather brown. Fresh shale bedrock does not outcrop except in artificial excavations, although it is present at shallow depth on hill crests beneath 1 m or less of residual clay soil.

The Penrith geological map also shows a major geological structure, known as the Narellan Lineament, running in a north-south direction 500 m east of the site. This lineament could be a zone of either closely-spaced jointing or faulting, which defines the straight course of South Creek upstream from the St Marys area. Within the site area it may be responsible for the deep shale weathering noted in several subsurface investigations.

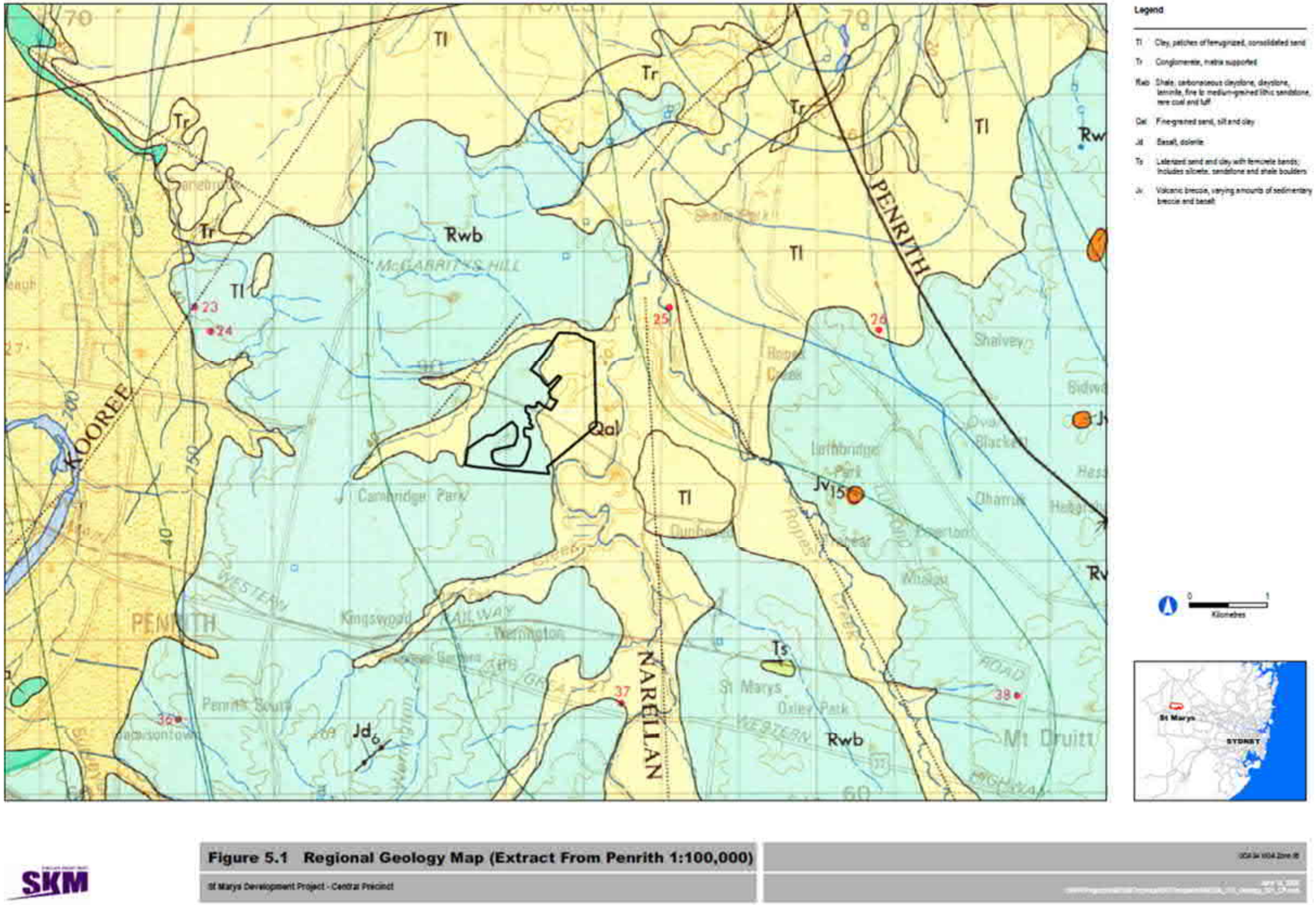


Figure 5-1 Regional geology map

Site Geology

The low level floodplain alluvium (from RL 17 to 28 m AHD) is of Quaternary age and the higher level weathered shale bedrock (from RL 29 to 40 m AHD) is of much older Triassic age. No surface outcrops of the fresh shale bedrock were observed during current investigation works and the predominant rock type encountered in soil bores drilled was weathered shale. The depth of weathered shale and residual clay cover in soil bores was everywhere greater than 3 m.

The lower slopes of the hills are generally mantled by 1 to 4 m of clay colluvium, which is being moved slowly downslope by soil creep and is merging with the floodplain alluvium that it closely resembles.

Soils

Based on the Penrith 1:100,000 soil landscapes map (Bannerman and Hazelton, 1990) an extract from which is shown in **Figure 5-2**, the two soil units within the site area include the Luddenham (lu) and South Creek (sc) soil landscapes (SLs). The first is predominant within the southern and western third portion of the site, while the South Creek SL covers the remainder. The Luddenham soil units are of residual origin are derived from weathered Bringelly Shale bedrock. The South Creek clay soil units of alluvial origin, derived from weathering, erosion and fluvial transport of the Bringelly Shale bedrock.

They differ in that the Luddenham SL is developed on older (Triassic age) higher level bedrock terrains, while the South Creek SL comprises those alluvial clay soils on the near-recent (Pleistocene) and present-day, active flood plain of watercourses such as South Creek.

Although these soils have many similarities, they differ in that the South Creek SL tends to have a shallower depth to the water table and hence to be more prone to waterlogging, more erodible and subject to more frequent flooding. The Luddenham SL is typically found on gently undulating rises on Bringelly shales. The typical Luddenham soil is a brown hardsetting silty clay loam overlying strongly pedal mottled brown clay, with texture increasing with depth. In the highest part of the landscape the clay extends only about 1 m before fresh shale bedrock is encountered. However, the heavy clay can extend for several metres in the lower parts of the landscape. Particularly on lower slopes, this soil type has poor drainage characteristics and is prone to salinity and sodicity. Shallow saline water tables also commonly occur beneath this landscape.

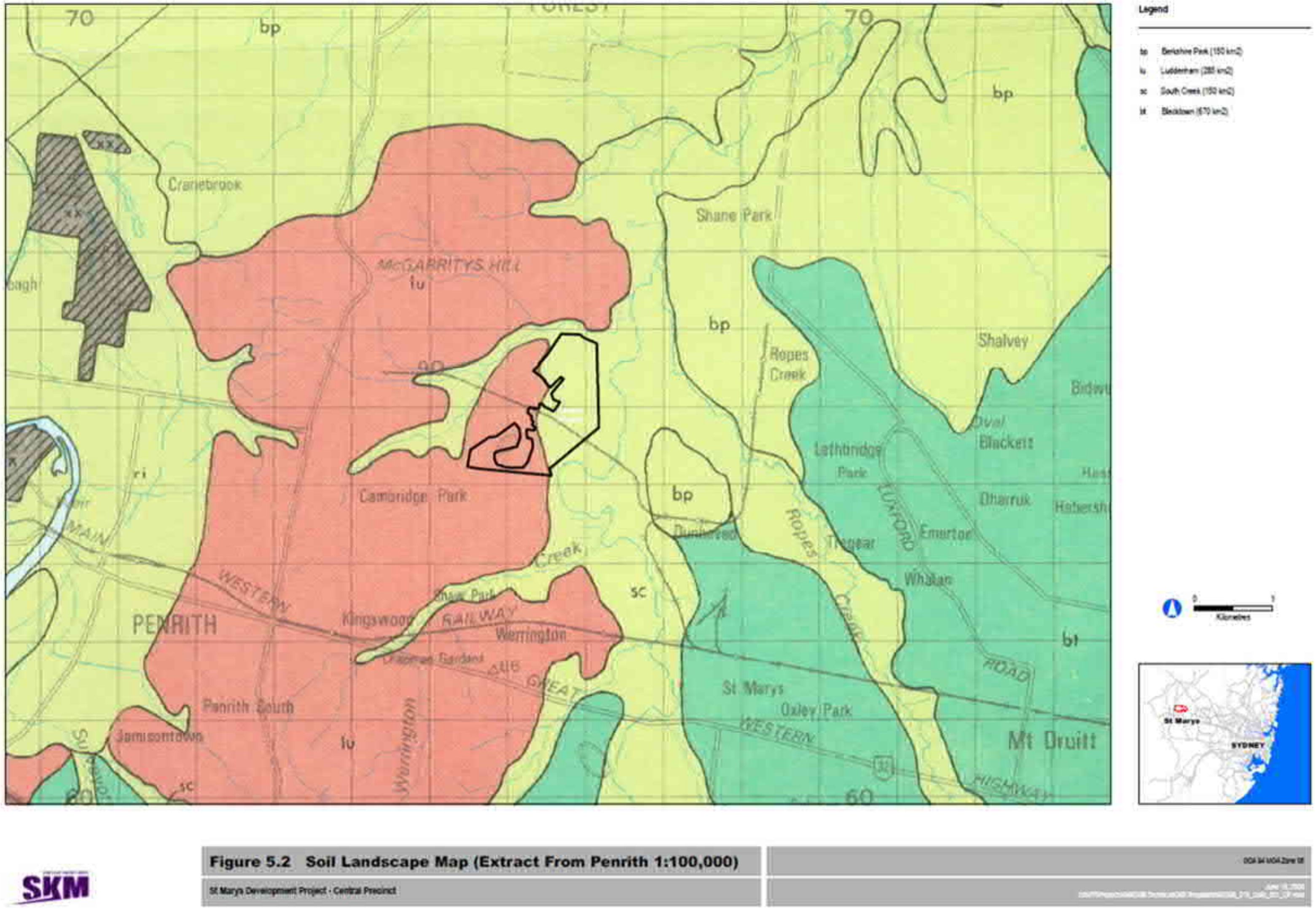


Figure 5-2 Soil landscape maps

For much of the western Sydney region, the Luddenham soil landscape lies above the South Creek soil landscape. The soil limitations are summarised in **Table 5-1**.

Table 5-1 Summary of Soil Limitations

Soil Landscape	Soil unit	Soil depth	Limitation
Luddenham (lu)	Lu2	up to 40cm	Very hard setting surface
			Low available water capacity
	Lu3	>50cm	Low wet strength
			Low permeability
			Low fertility
			High shrink-swell
			Low available water capacity
	Lu4	<90cm	Low wet strength
			Low permeability
			Low available water capacity
			High shrink-swell
South Creek (sc)	Sc2	15cm	High erodibility
			Hard setting surface
			Strongly Acid
			Low fertility
	Sc3	60-85ch	Shrink-swell potential
			Very high erodibility

Salinity potential maps released by the then Department of Land and Water Conservation (DLWC 2002) show the Luddenham, soil landscape as having a moderate salinity potential and the South Creek soil landscape as having a high salinity potential. Identified areas of existing salinity are usually found on the South Creek soil landscape and the boundary between the South Creek and Luddenham soil landscape.

5.4 Site Hydrogeology

Two groundwater-bearing systems are present within the St Marys site. These are referred here as the shallow and deep aquifers, but regolith (soil) and fractured shale bedrock aquifers would be more accurate titles. The relationship between them is illustrated by **Table 5-4**. Neither would normally be regarded as true aquifers because of their low permeability, limited storage capacity, inhomogeneity and indefinite boundaries. A true aquifer is a soil or rock layer able to store and transmit groundwater in sufficient quantity and adequate quality to sustain producing wells.

The main difference between these two 'aquifer systems' is that the shallow ones are more-or-less fresh, relatively permeable, but only ephemerally saturated while the deeper aquifers are tighter, permanently saturated and much more saline (with salt content approaching that of sea water in places). The use of the plural recognizes that both systems comprise a complex of scattered and discontinuous sub-aquifers of limited area and volume. The two systems are interconnected to varying degrees, such that in many places they cannot be distinguished. Many piezometers penetrate both aquifer systems, so their response (in terms of water level and salinity) is therefore a composite one.

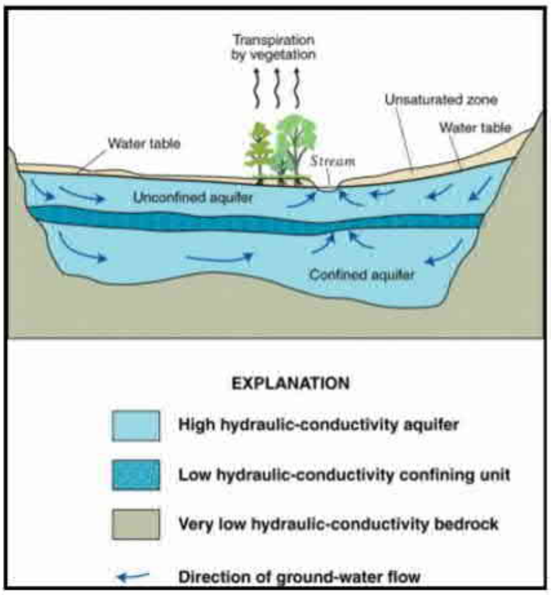


Figure 5-3 Relationship between shallow (unconfined) and deep (confined) aquifers

5.4.1 Shallow Aquifers

The shallow or soil aquifer system is composed of residual soil, colluvium (slope creep deposits), floodplain alluvium, lateritic ironstone and weathered shale bedrock. This heterogeneous mixture is referred to as the regolith aquifer in McNally (2004, 2005a) because it includes all those soil materials down to the unweathered shale rockhead (‘from fresh air to fresh rock’ being the colloquial definition of the regolith).

The shallow aquifer system at the site essentially comprises the deeper soils covering foot slopes and creek floodplains – the lower ground within the landscape. As well as having a much smaller area than the underlying shale bedrock aquifer, the shallow aquifers discharge into nearby streams rather than to the distant South Creek. The shallow aquifers are indicated by low ECa values on the EM conductivity map, which indicate low salinity groundwater at shallow depth. The Central Precinct EM map highlighted a conspicuous area of potential saline scalding within the southwestern portion of the site, which correlates with the Bringelly Shale bedrock and Luddenham soil landscape.

Although the materials making up the shallow aquifers are predominantly impervious clay, significant hydraulic conductivity can nevertheless develop along shrinkage fissures, root tubes, weathered rock joints, the A/B soil profile interface and the deeper soil/rock interface. The shallow aquifer permeability is anticipated to range from 0.12 m/d to 25 m/d and the almost instantaneous rise of the shallow water table following rainfall, which is characteristic of throughflow-dominated soil profiles and shallow unconfined aquifers provides an indication of this permeability.

Another distinguishing feature of the shallow aquifer systems is its low salinity. The Central Precinct EM map provided an indication of a low salinity shallow aquifer potentially occurring in the northern and eastern portions of the site. The salinity of shallow aquifer at the site less than 1,000 mg/L, which is consistent with the surface stream salinity of 100 to 2,510 mg/L (though generally <1,000 mg/L) and supports the hypothesis that discharge from this aquifer maintains stream baseflow.

Shallow aquifers are typically unconfined, whereas the deep bedrock aquifer system is generally confined or at least semi-confined. In other words, the upper surface of the shallow saturated zone is the water table, which is at atmospheric pressure the highest water cut in a borehole is close to the final standing water level. This contrasts with the deeper pressure aquifers, where the first water cut is usually several metres below the eventual SWL. Water can infiltrate from the surface and the water table may rise close to ground level in low-lying areas, possibly causing water-logging in especially wet years. However because this shallow groundwater has a salinity generally less than

1,000 mg/L, especially in wet years, its potential for salting is much less than the deep aquifer water, although concentration by evaporation is nonetheless possible in places.

5.4.2 Deep Aquifers

The deeper or fractured shale bedrock aquifer system at the site is expected to be much more extensive than the shallow one, and is likely to cover the entire area underlain by Bringelly Shale. The contours on the ‘piezometric surface’, defined by standing water levels in boreholes drilled into this confined aquifer indicate that the shale groundwater flows towards the northern end of South Creek and is not greatly affected by minor streams.

Given that its hydraulic conductivity is dependent on fracture intensity (m2 per m3), fracture continuity and aperture, the effective (as-tested) shale permeability at St Marys is relatively uniform. Rising head tests, based on SWL recovery after bailing (‘purging’), indicate an average permeability of 0.5 m/d, with a range from 0.05 to 1.90 m/d. This is at the high end of permeability ranges from 5 to 10 m/s (approximately 1 m/d to 0.00001 m/d) recorded in unweathered shales of the Sydney region (McNally, 2004). The reason for this relatively high permeability is considered to be the stress-relief fracturing in the fresh shale rock mass, which tightens with depth.

The deep aquifer system at the site is believed to have higher salinity properties, ranging from 500 to 8,000 mg/L TDS. The maximum salinity recorded at the site was 8,000 mg/L. Values less than

10,000 mg/L are indicative that mixing with fresh water from the upper aquifer may be occurring. At this stage it is not clear whether there are any mappable salinity trends across the site, as distinct from local salinity variations and the effects of local dilution.

Generally, piezometers screened within the deep shale aquifers elsewhere in western Sydney demonstrate a slow response after purging. Water levels in piezometers may take hours or days to reach equilibrium SWL. This piezometric response is likely to be a consequence of the generally low bulk permeability of the shale rock mass, the random distribution of fractures and the poor hydraulic connections within this fracture network. Water cuts are commonly not observed until the borehole has advanced some metres below what is the later recorded SWL. Because of this variable but usually poor fracture connectivity the shale aquifer may be unconfined (below hill crests), confined (especially below thick clay regolith on valley floors) or semi-confined.

The latter is probably the most common situation in the southwestern portion of Central Precinct site, for it describes a ‘leaky’ aquifer (or ‘aquitard’) in which water is stored in fractures or perched water tables. This water can move upward under pressure, but encounters frictional resistance along narrow and tortuous seepage paths. Hence a fresh aquifer can exist above a saline one, provided its water level (ie, its pressure ‘head’) is high enough to resist rising salt water.

5.4.3 Groundwater Conceptual Model

The understanding of the two aquifer systems provide a groundwater conceptual model which helps explain why groundwater in the shale is significantly more saline than in the alluvium. The two systems are likely to be connected, albeit via narrow conduits, through a leaky aquiclude. Groundwater flows by gravity from high to low levels, particularly from high to low pressure zones, and its movement is hindered by frictional resistance along the way. The longer its passage through the shale bedrock the more head pressure it loses and the more salt it gathers.

Rainfall is believed to infiltrate mainly on upper slopes or along watercourses, with extremely low uptake due to the tightness of the shale bedrock most precipitation runs off or is lost to vegetation. Windblown sea salt accompanies the rain and becomes stored within the soil B-horizon as moisture is lost by evapo-transpiration. It is presumed that some of this stored salt, at depths around 1m in the soil profile, is periodically dissolved and flushed downwards with the sinking groundwater or moves laterally with throughflow (McNally, 2005b). Were it not for such a salt-depleting mechanism, western Sydney would become a desert. The proportion of salt removed by throughflow to that infiltrating to groundwater is not known, though field evidence suggests the former is much the more effective salt-depleting mechanism.

Once within the shale, which may be present at depth of 1 to 2 m, the infiltrating water ‘steps’ slowly downwards through vertical joints and laterally along bedding planes. The groundwater distribution in the shale can be envisaged as a multitude of stacked and sporadically distributed perched water tables. Piezometers only 100 to 200 m apart may differ in SWL by 10 m or more, as they register different perched water tables. It appears that the water table in Bringelly Shale is not quite the smoothly inclined surface often portrayed in the literature.

5.4.4 Hydraulic Connection between Aquifers

Because water moves from higher to lower pressure, saline shale water tends to move downwards beneath hills and upwards to major watercourses such as South Creek, though the dominant source of the creek water remains the fresh upper aquifer. The processes controlling salinity in South Creek – and indeed in all permanent water courses in the shale terrain of western Sydney - appear to be as follows:

Following heavy or prolonged rain the upper aquifer is replenished, the water table rises and its salinity (never high) diminishes. Because of the much lower permeability of the shale, and despite its much larger outcrop area, little rainfall infiltrates to the bedrock aquifer. In fact most of the water penetrating below the plant root zone is directed down slope but within the soil profile by throughflow, without entering the groundwater cycle.

For most of the time between significant rainfall events, which may range from months to more than a year, the base flow to South Creek (and similar streams) is provided by the upper aquifers. High pressure in these layers normally inhibits salt entry from the lower aquifer, but this leakage increases as the water table subsides.

In drought years the discharge of South Creek and the level of the water table both fall, and salinity of the surface water increases. At the St Marys site we know that stream salinity may vary from about 100 mg/L to 2,500 mg/L, but this is probably not the full extent of its seasonal variability, due to the limited monitoring period.

In extreme droughts South Creek could dry up entirely, but salt can still be brought to the surface by capillary rise. This salt enrichment of the creek bed by evaporation would be apparent as a temporary conductivity spike following drought-breaking rains, as discharge from the replenished upper aquifer flushes out remnant salt.

5.5 Investigation Methodology and Results

5.5.1 Soil Bores

Twenty three soil bores (SKM 1-14, 16-23 and 25-28) were drilled to a maximum depth 3 m and three (SKM 4, 20 and 27) and to a maximum depth of 10 m between 28 and 30 May 2008, using a bobcat-mounted auger rig. Soil bore locations are shown in Figure 5-4. These soil bores were located to cover the maximum extent of the site possible, and were supervised and logged by qualified environmental scientists. Most soil bores were situated in order to provide detailed information on the shallow soil profiles and materials encountered.

Drilling was advanced through soil materials using 125 mm diameter continuous flight augers equipped with V-bits or tungsten carbide (TC) bits. The auger string was withdrawn at intervals for soil logging. Auger drilling was terminated when the rate of advance became very slow in weathered shale, at depths of 3 m. In some cases this slow drilling approached refusal, but definite V-bit or TC bit refusal on strong rock did not occur.

The three soil bores (SKM 4, 20 and 27) that were drilled to a maximum depth of 10 m to install PVC casing and screened intervals as groundwater observation wells (piezometers).

All soil bores were backfilled immediately after drilling and logging, with the exception of the piezometers. Soil bore locations are shown in Figure 5-4 and drilling logs are presented in **Appendix C**

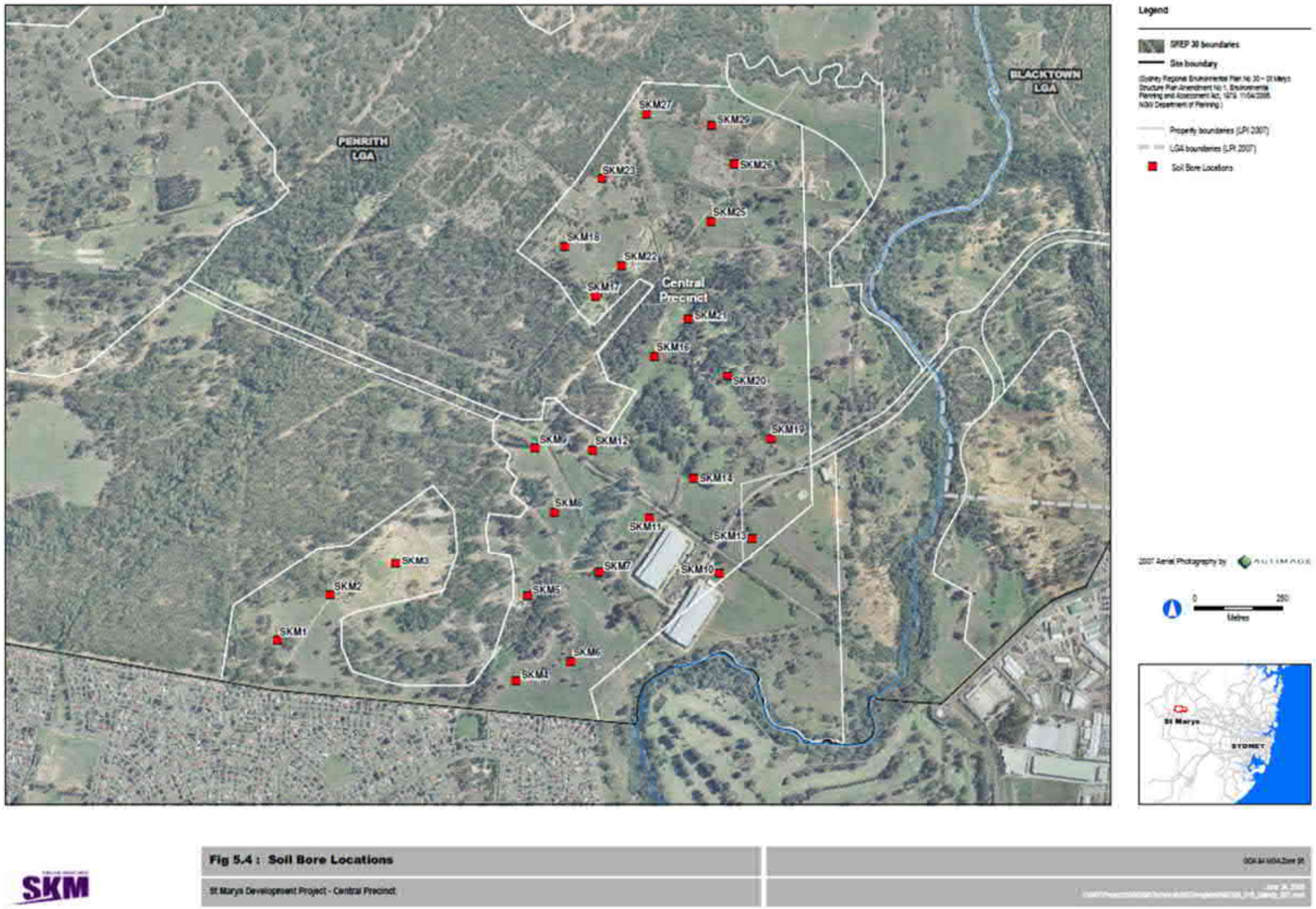


Figure 5-4 Soil bore locations

5.5.2 Soil Bore Results

Soil bore logs indicate that the predominant soil observed to the depth 3 m is yellow to brown clayey and fine sandy silt, which grades to a silty clay in places and, rarely, to a clayey sand. Dry, grey brown silt topsoil was observed in most soil bores and is also noticeable in gully walls and erosion scars, with faint layering visible. At the time of the investigation clay and silt subsoil was dry to moist and of stiff to hard consistency.

The deeper soil bores which were converted to piezometers indicated that alluvial silty clays and clayey silts, of stiff to hard strength and low to medium plasticity, extend to depths ranging from 5 to 8 m. This revealed that the depth of the alluvial clay is generally deeper than about 3 m, which as the maximum depth of most soil bores during this investigation.

The alluvial clay appears to be underlain by 1-2 m of extremely weathered shale, described as shaly clay on the auger logs because it is thoroughly ground up by the auger bit. In the cored sections of the boreholes most of the core losses are likely to have been in layers of extremely weathered (XW) shale. This XW shale is presumed to be similar in engineering properties to a very stiff to hard fissured clay, though it might equally be described as a very low strength rock.

5.5.3 Soil Salinity Results

Soil salinity results were obtained from field tests conducted during soil bore sampling on 1:5 soil in water suspensions, using a TPS water quality and conductivity meter. Samples were also taken for laboratory tests, carried out in the Department of Lands soils laboratory at Scone NSW. Results from both sets of testing are summarised in **Table 5-3** and salinity contours for depths 0.25, 0.5, 1 and 3m are shown in **Figure 5-5**, **Figure 5-6**, **Figure 5-7** and **Figure 5-8** respectively.

Soil salinity results have been compared against the EC_e values of soil salinity classes specified by the DLWC 2002 booklet titled Site Investigations for Urban Salinity. These values are summarised in **Table 5-2**.

Table 5-2 EC_e Values of Soil Salinity Classes (DLWC 2002)

Class	EC_e (dS/m)	Comments
Non saline	<2	Salinity effects mostly negligible
Slightly saline	2-4	Yields of very sensitive crops may be affected
Moderately saline	4-8	Yield so many crops affected
Very saline	8-16	Only tolerant crops yield satisfactorily
Highly saline	>16	Only a few very tolerant crops yield satisfactorily

Based on DLWC 2002 criteria the SKM field results correspond, by depth intervals, to:

- Depth 0.25 m (in topsoil or A-horizon), with EC_e ranging from 1.4 dS/m to 6.8 dS/m, equating to 19 % non-saline, 54 % slightly saline and 27 % moderately saline
- Depth 0.5 m (in subsoil or B-horizon), with EC_e ranging from 1.4 dS/m to 5.6 dS/m, equating to 27 % non-saline, 50 % slightly saline and 23 % moderately saline,
- Depth 1 m (in lower B-horizon), with EC_e ranging from 1.4 dS/m to 6.5 dS/m, equating to 23 % non-saline, 50% slightly saline and 27% moderately saline.
- Depth 3 m (in weathered shale), with EC_e ranging from 1.4 dS/m to 6.4 dS/m, equating to 30 % non-saline, 26 % slightly saline and 43 % moderately saline.

These results indicate that though salt accumulates with depth, the soil profile in the Central Precinct is generally of low salinity.

Table 5-3 Summary of Soil Salinity EC_e (dS/m) Results

Soil Bore	Depth (m bgl)	EC_e (dS/m)	Soil Bore	Depth (m bgl)	EC_e (dS/m)	Soil Bore	Depth (m bgl)	EC_e (dS/m)	Soil Bore	Depth (m bgl)	EC_e (dS/m)
SKM1	0.25	2.3	SKM7	0.25	5.2	SKM16	0.25	1.5	SKM23	0.25	2.5
	0.5	1.7		0.5	3.9		0.5	1.5		0.5	2.2
	0.75	2.2		1	4.6		1	1.5		0.75	2.2
	1	5.0		1.5	3.7		1.5	1.5		1	2.4
	1.25	2.6		2	4.4		2	1.5		1.25	2.4
	1.5	2.1		2.5	3.7		2.5	1.5		1.5	2.1
	1.75	3.5		3	4.3		3	1.6		1.75	2.4
	2	2.3	SKM8	0.25	1.8	SKM17	0.25	2.5		2	2.5
	2.25	2.8		0.5	2.0		0.5	2.3		2.25	4.1
	2.5	3.1		0.75	1.9		1	2.6		2.5	4.4
	2.75	6.6		1	2.2		1.5	2.7		2.75	4.7
	3	5.9		1.25	2.1		2	2.3		3	4.4
SKM2	0.25	2.4		1.5	2.4		2.5	1.8	SKM25	0.25	1.4
	0.5	1.7		2	2.5		3	1.8		0.5	1.4
	1	2.8		2.25	2.3	SKM18	0.25	2.6		0.75	1.4
	1.5	2.9		2.5	2.1		0.5	3.2		1	1.4
	2	4.6		2.75	2.2		0.75	3.4		1.25	1.4
SKM3	0.25	3.6		3	3.0		1	4.3		1.5	1.4
	0.5	3.9	SKM9	0.25	5.0		1.25	4.1		1.75	1.4
	0.75	4.1		0.5	4.2		1.5	3.6		2	1.4
	1	3.6		0.75	4.4		1.75	4.3		2.25	1.4
	1.25	3.9		1	3.9		2	2.6		2.5	1.4
	1.5	4.1		1.25	3.7		2.25	4.8		2.75	1.4
	1.75	3.8		1.5	4.3		2.5	7.0		3	1.4
	2	3.8	SKM10	0.25	1.5		2.75	5.4	SKM26	0.25	3.2
	2.25	3.6		0.5	1.5		3	5.0		0.5	1.9
	2.5	3.8		1	1.6	SKM19	0.25	2.0		1	1.8
	2.75	3.9		1.5	1.6		0.5	2.1		1.5	1.9
	3	3.6		2	1.6		0.75	1.9		2	1.6
SKM4	0.25	4.6		2.5	1.6		1	1.9		2.5	1.6
	0.5	4.4		3	1.7		1.25	2.0		3	1.7
	0.75	3.9	SKM11	0.25	5.5		1.5	2.0	SKM27	0.25	2.6
	1	4.6		0.5	5.6		1.75	1.9		0.5	2.0
	1.25	5.9		0.75	7.2		2	1.7		0.75	2.7
	1.5	3.6		1	6.5		2.25	2.6		1	2.9
	1.75	4.7		1.25	5.7		2.5	2.5		1.25	3.3
	2	4.0		1.5	5.9		2.75	2.6		1.5	4.0
	2.25	3.1		1.75	5.3		3	3.1		1.75	3.6
	2.5	3.9		2	6.1	SKM20	0.25	6.8		2	3.4
	2.75	4.2		2.25	5.4		0.5	4.6		2.25	6.6
	3	3.1		2.5	5.2		0.75	4.4		2.5	6.2
SKM5	0.25	4.2		2.75	6.2		1	5.1		2.75	6.2
	0.5	4.7		3	5.3		1.25	4.2		3	6.4
	0.75	4.6	SKM12	0.25	4.3		1.5	8.4	SKM29	0.25	2.5
	1	4.7		0.5	3.9		1.75	9.3		0.5	2.4
	1.25	4.8		1	3.4		2	7.1		0.75	2.4
	1.5	8.9		1.5	2.9		2.25	7.3		1	2.9
	1.75	7.5		2	4.9		2.5	6.4		1.25	2.6
	2	6.0		2.5	5.1		2.75	6.5		1.5	4.0
	2.25	6.4		3	4.3		3	5.5		1.75	3.4
	2.5	6.1	SKM13	0.25	2.3	SKM21	0.25	3.4		2	3.4
	2.75	6.6		0.5	2.2		0.5	4.0		2.25	8.3
	3	5.9		1	2.3		1	3.9		2.5	6.3
SKM6	0.25	2.2		1.5	1.4		1.5	4.7		2.75	6.0
	0.5	2.9		2	1.6		2	5.0		3	4.0
	0.75	3.3		2.5	1.6		2.5	4.5			
	1	3.7		3	1.5		3	3.5			
	1.25	5.3	SKM14	0.25	1.4	SKM22	0.25	2.8			
SKM7	0.25	5.2		0.5	1.5		0.5	2.2			
	0.5	3.9		0.75	1.5		0.75	2.6			
	1	4.6		1	1.5		1	2.7			
	1.5	3.7		1.25	1.5		1.25	2.3			
	2	4.4		1.5	1.5		1.5	2.2			
	2.5	3.7		1.75	1.5		1.75	1.9			
	3	4.3		2	1.5		2	2.1			
				2.25	1.6		2.25	2.1			
				2.5	1.7		2.5	3.1			
				2.75	1.7		2.75	3.2			
				3	1.8		3	3.0			

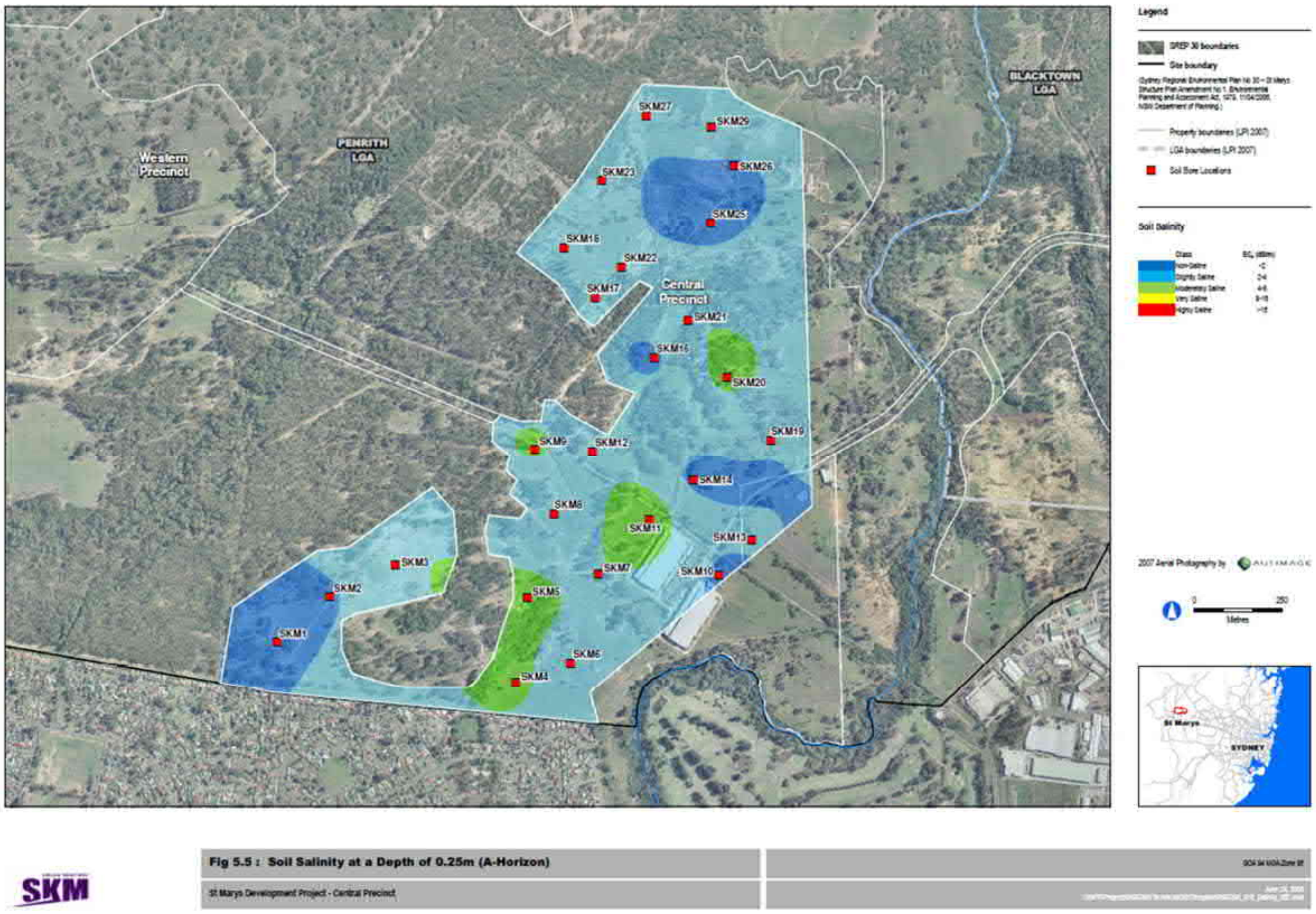


Figure 5-5 Soil salinity at depth 0.25m

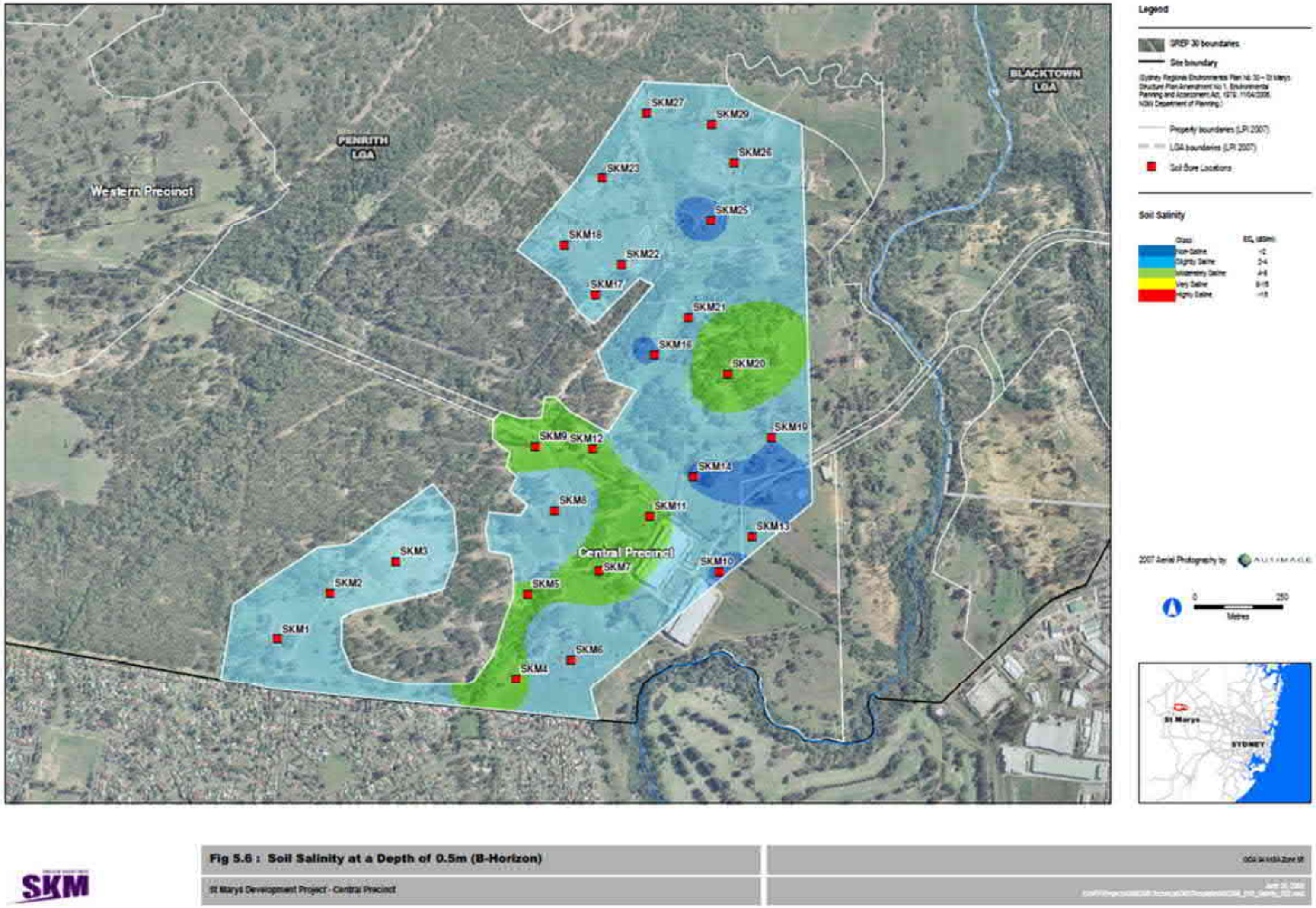


Figure 5-6 Soil salinity at depth 0.5m

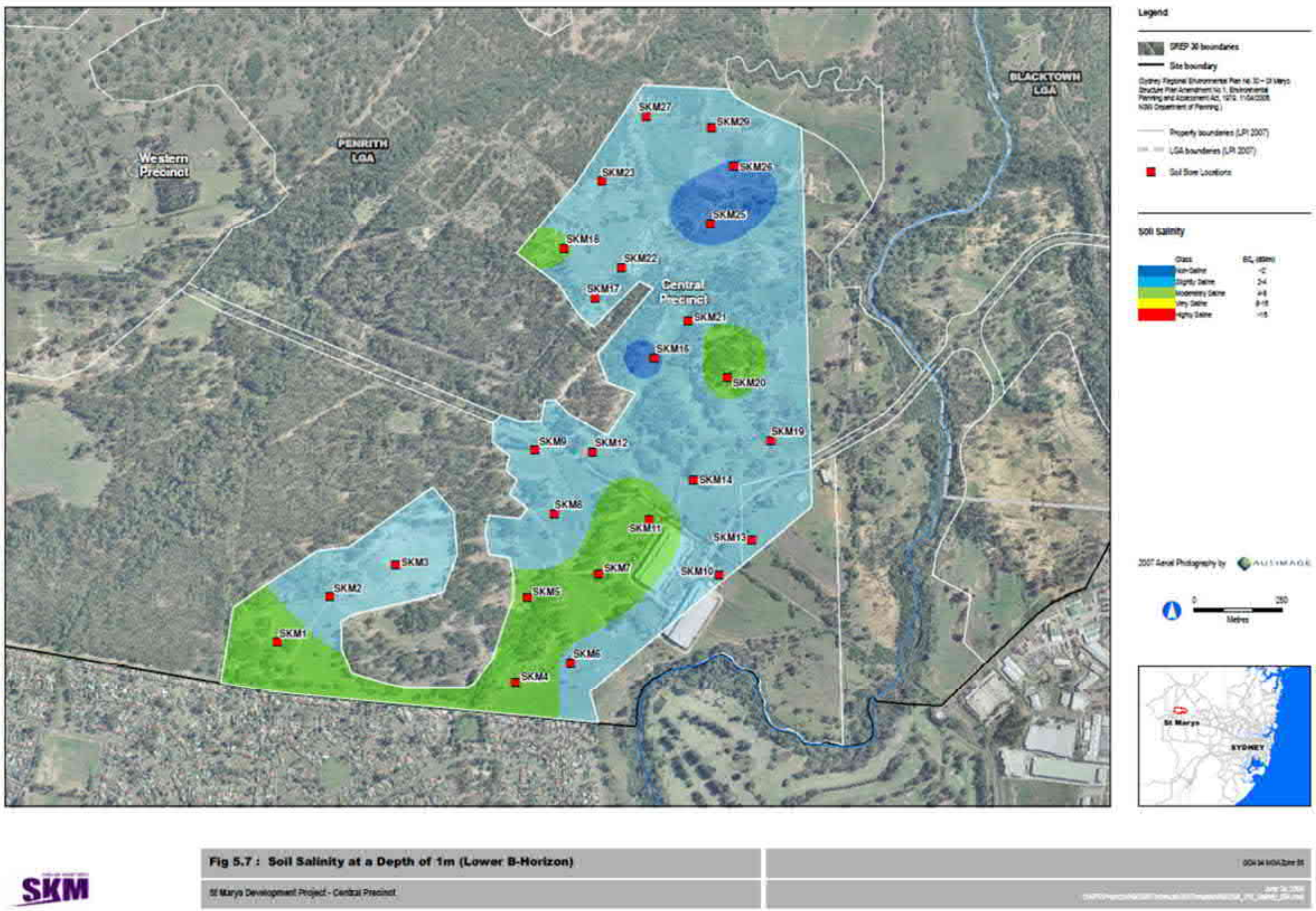


Figure 5-7 Soil salinity at depth 1m

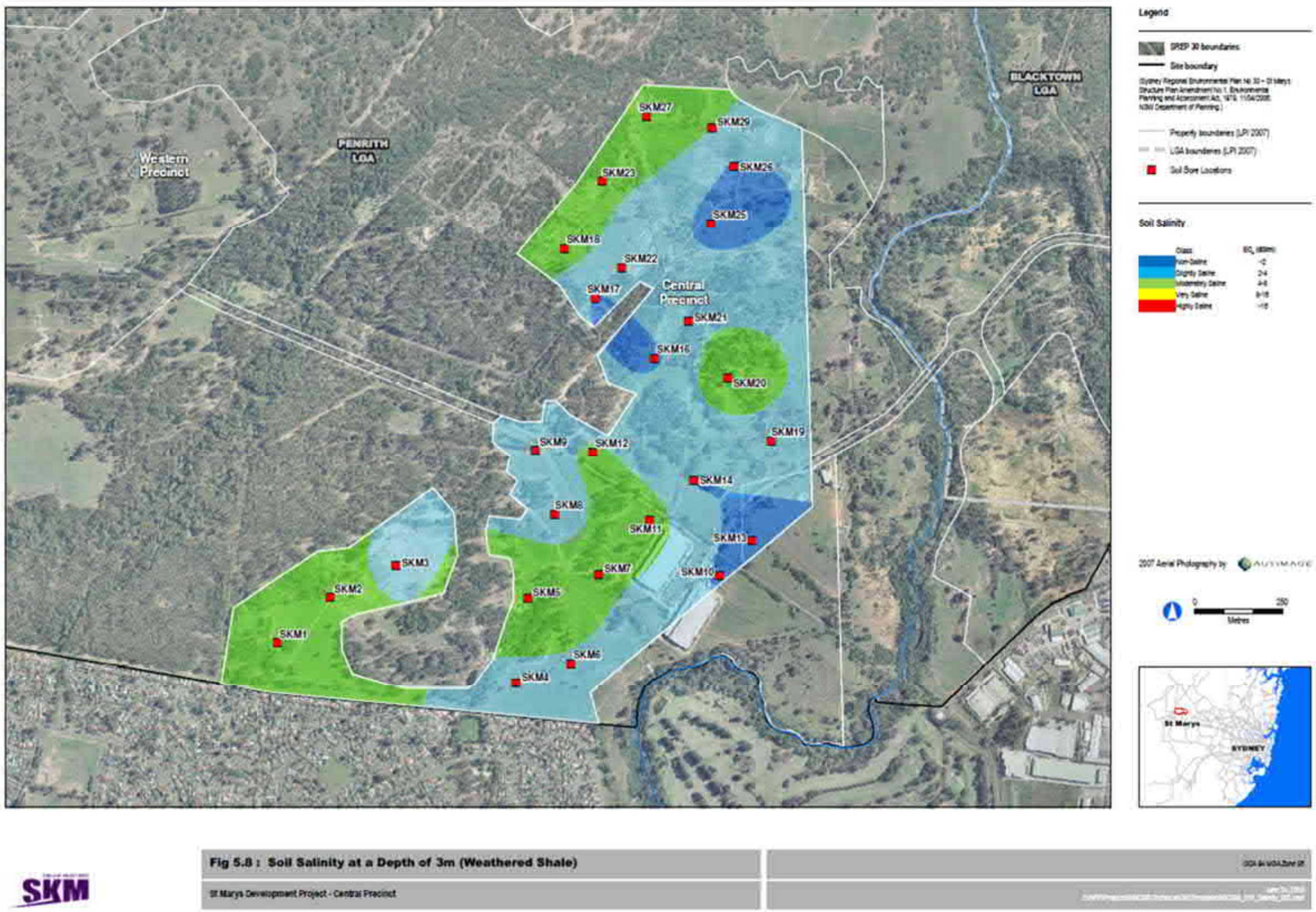


Figure 5-8 Soil salinity at depth 3m

5.6 Electromagnetic Soil Testing

An electromagnetic induction (EMI) survey was carried out across the site by Douglas Partners on 20 to 24 May 2008, with the primary aim of mapping variations in subsurface salinity, since this was assumed to be the main contributor to ground conductivity. The full results of this work are provided in their report (Douglas Partners, 2008), which is presented in Appendix C and summarised below.

The survey was carried out by means of a DualEM-4 conductivity meter mounted on a 4WD quad bike. The nominal 100 m by 100 m grid was distorted due to access limitations and obstacles, and the eventual traverse lines totalled 13 km, with readings at approximately 1 m intervals. Location control was provided by a differential GPS system mounted on the quad bike and linked to the DualEM-4.

The results indicate low apparent conductivities (ECa ranging from 60 to 100 mS/m) adjacent to the gully and in areas of shallow depressions on the alluvial terrace surface, and higher conductivities (ECa ranging from 100 to 200 mS/m) beneath more elevated ground. Overall, the EM results indicate that the subsurface is non-saline to slightly saline. However they also showed greater variability than the soil salinity measurements listed in Table 5-2, which were uniformly low. The reason for this discrepancy is expected to be soil bores being collected at a maximum depth of 3 m, whereas the DualEM-4 measures bulk conductivity to a depth of 6 m in this case.

The DualEM-4 results are believed to be a response to a number of factors affecting the overall ground conductivity:

- Variations in the clay mineral content and the depth of alluvial clay (and hence depth to shale bedrock)
- Variations in moisture content and degree of saturation within the clay blanket, and in the salinity of this pore water
- The presence or not of conductive lateritic ironstone in the subsurface.
- However the possibility of higher salinity at depths greater than 3m, probably due to saline groundwater below the water table, cannot be excluded.

5.7 Groundwater & Salinity Implications

5.7.1 Existing Groundwater Conditions

The hydrogeology of the St Marys property, including the Central Precinct site, is summarised in Mackie Martin (1991) and ADI Ltd (1996). The results of boreholes drilled between 1990 and 1996 in or close to the site suggest that both the unconfined shallow (soil) aquifer and the confined deep (shale bedrock) aquifer are present. Both aquifers have similar characteristics to those in other parts of the St Marys property – in that they are tight, with low to very low permeability and very limited storage capacity. Both probably consist of a series of stacked and sporadically distributed perched water tables – in effect, poorly interconnected lenses of saturated ground - rather than a single homogeneous water-bearing layer. The vertical connection between the soil and shale aquifers is poor, to judge by nearly dry soils observed in test pits, and they appear to have different recharge / discharge relations.

Recharge to the soil aquifer is by direct infiltration onto the surface of the alluvial terrace (from RL19 to 20 m), followed by throughflow across the A/B soil profile interface and temporary storage in shallow perched aquifers at depth ranging from 0.5 to 1 m. Discharge is by evaporation from puddles in shallow gilgai-like surface depressions, through transpiration by trees and by seepage to shallow pools in the unnamed western gully (at about RL 16 m). Limited information in the Mackie Martin (1991) report indicates that the shallow groundwater is of low salinity, ECe less than 2 dS/m, although both the surface puddles and the gully pools support halophyte vegetation including salt-tolerant reeds. No saline scalds were observed.

At present most infiltration to the shale aquifer is likely to be coming from the unlined effluent discharge channel in the eastern gully, at about RL 15 m. This is believed to have raised the water table by perhaps 1-2 m and reduced the salinity and to be moving slowly through the shale aquifer. It is presumed to ultimately discharge along South Creek at about RL 12 m.

5.7.2 Existing Salinity

Information on salinity at Central Precinct has been drawn from four sources:

- On-site conductivity testing carried out on 1:5 soil/water suspensions using a TPS water quality meter (results are listed on **Table 5-3**)
- Similar testing carried out independently by Department of Land under laboratory conditions on soil samples submitted by SKM (results provided in **Appendix C**)
- Previous piezometers from MM, 1991 shown in **Figure 5-9** (including SM1, SM5, SM6, SM7, SM8, SM30, SM51 and SM56) and groundwater results shown in **Figure 5-10**
- Electro-magnetic induction (EMI) surveys across the Precinct area to measure ground conductivity, carried out by Douglas Partners in 2008 and reported separately.
- The soil conductivity results on are consistently low and equivalent to less than 3.5 dS/m in the A and B horizons. Values of less than 3.5 dS/m in the top 1 m of the soil profile are unusually low for western Sydney, since salt is normally stored within the B-horizon and moved around in throughflow along the A/B horizon interface

The EMI survey results present a plot of relative ground conductivity averaged out over a depth of about 5-6 m. The EMI thus ‘sees’ to greater depth than the soil tests, which are limited to about 1m below the surface, but is influenced by several factors:

- Salt stored within the soil B-horizon and in saline groundwater below the water table
- Differences in clay content, and in moisture content between saturated and partly-saturated clays
- Differences in depth to the shale bedrock (and hence differences in the thickness of the overlying clay blanket),
- The presence or otherwise of lateritic ironstone gravel in the subsurface.

The B-horizon salinity at the site appears to be generally less than 3.5 dS/m, which is lower than elsewhere in the St Marys site. The salinity of the water in the shale aquifer, as noted above, is considerably higher, though still relatively low by the standards of the St Marys property and western Sydney.

5.7.3 Impact of Development

Salinity problems may arise when the existing stored salt is brought to the surface by a rising water table, or is washed laterally from the B-horizon by increased infiltration. We consider that though the EMI results show variations in the overall ground conductivity, the soil and groundwater test results indicate relatively low salinity overall.

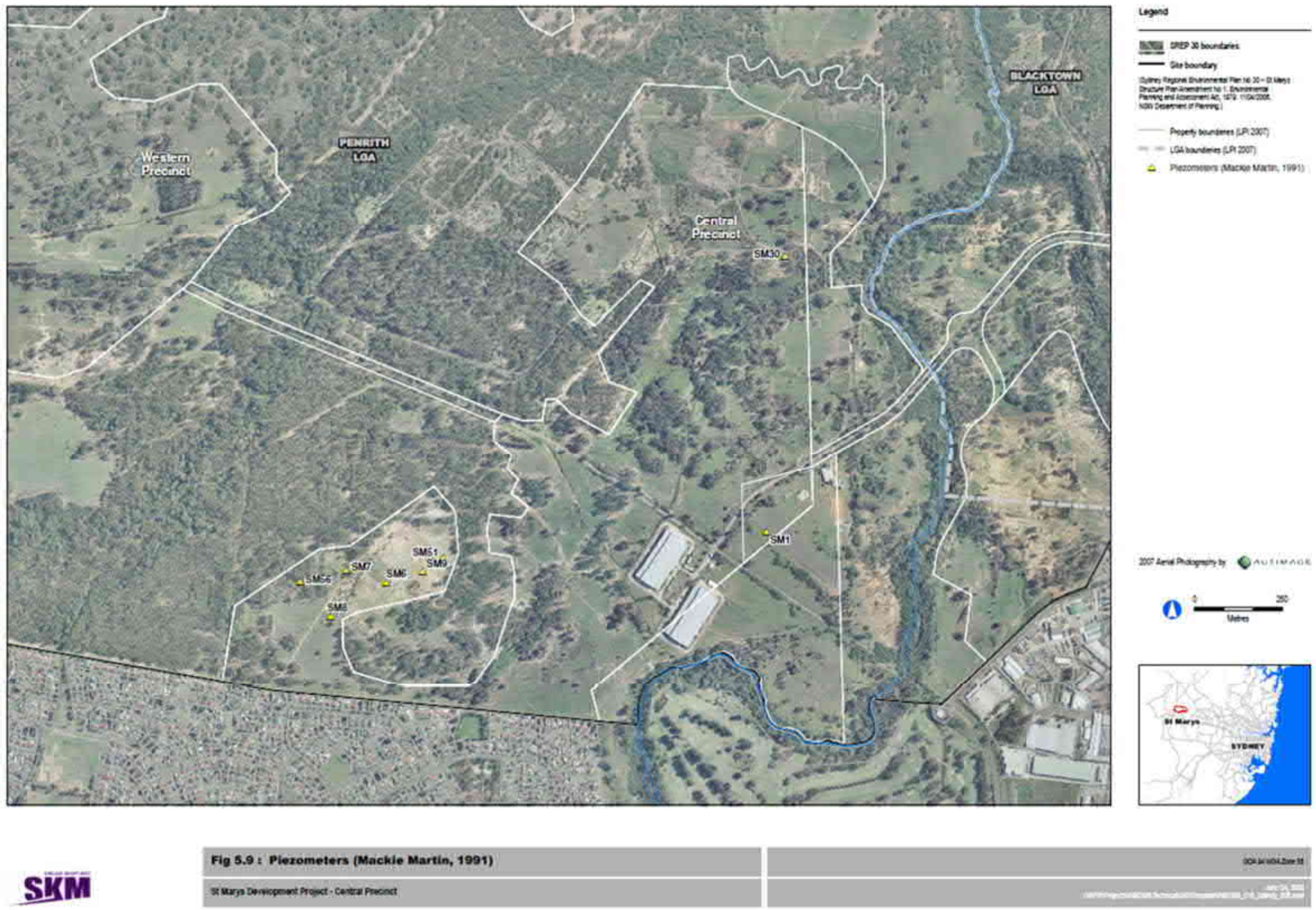


Figure 5-9 Piezometers (Mackie, Martin, 1991)

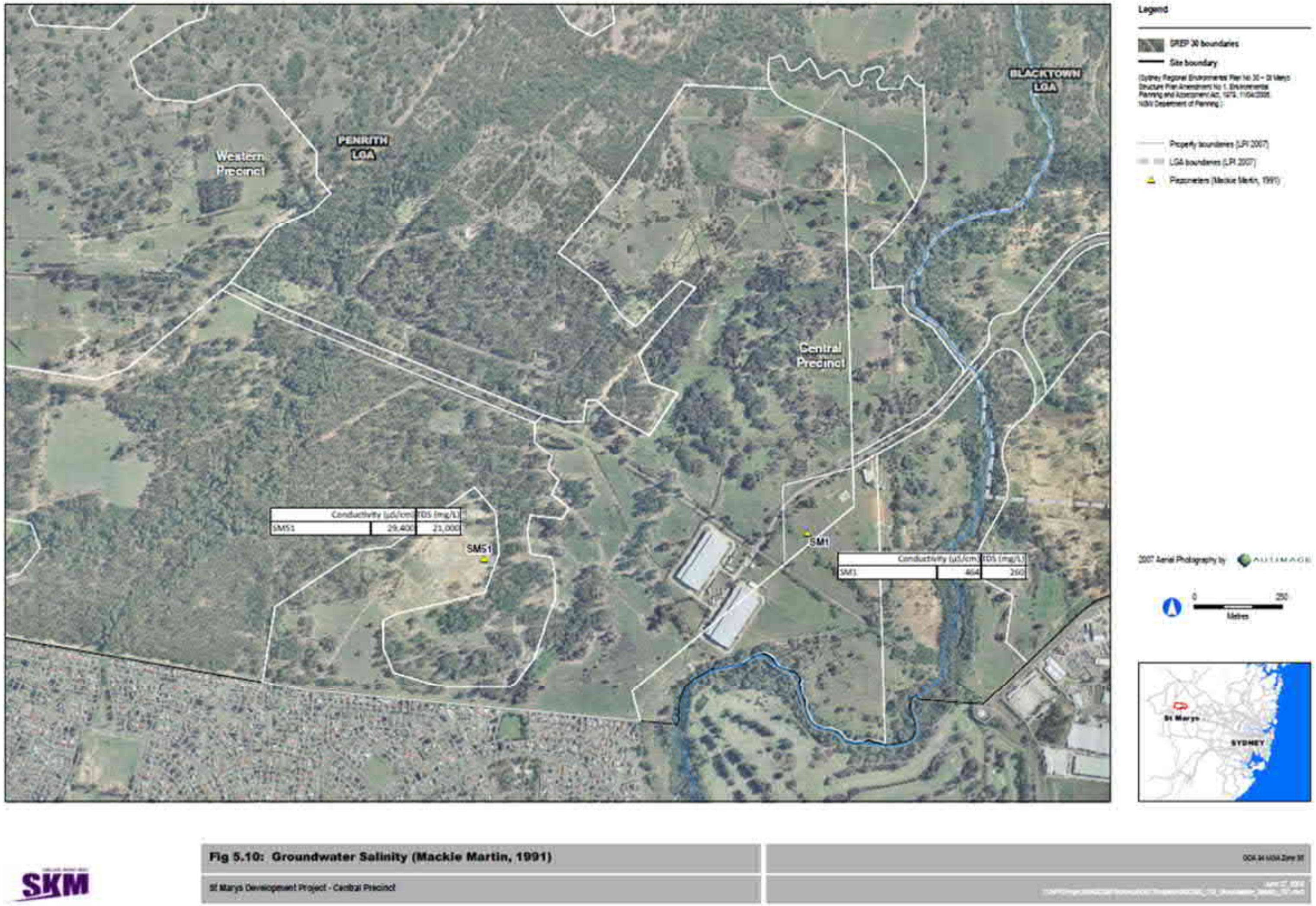


Figure 5-10 Groundwater salinity (Mackie, Martin, 1991)

5.8 Groundwater Management

Management of groundwater, and hence of salinity, to meet the requirements of SREP30 and the EPS implies that the water table will not rise significantly as a result of the proposed development. There should also be no increase in throughflow (lateral movement of water through the soil profile, but above the water table). In practice this means that infiltration to the soil profile and from there to the water table should be reduced if required. The proposed filled landform within the eastern portion of the Central Precinct and the management measures indicated below present opportunities for achieving these goals.

5.8.1 Key Issues

Key potential groundwater-related issues resulting from urban development in areas such as the Central Precinct are taken to include:

- Decreased rain interception and transpiration by trees, hence increased runoff and/or infiltration, as a consequence of land clearing (especially removal of deep-rooted trees) during subdivision construction
- Increased cumulative runoff (and probably more frequent peaks) from hard-surfaced areas such as roof tops, landscaped paving, roads and carparks
- Exposure of saline soils (especially saline and sodic/dispersive subsoils) as a result of cutting, filling and erosion
- Increased groundwater recharge due to garden watering, leaky pools, broken pipes, soakaways and parkland irrigation (especially with low salinity groundwater or recycled water)
- Increased groundwater recharge from stormwater water quality and detention basins, unlined drainage lines and ponded runoff generally.

5.9 Groundwater and salinity management measures

The specific measures that might be proposed for groundwater and salinity management at the site are in accordance with the DIPNR (2003) Western Sydney Salinity Code Practice, as follows:

- The design and installation of catchment wide ‘salt safe’ stormwater plans prior to the development of individual sub-divisions within the catchment. Such a system will have to demonstrably move salt emanating from home gardens, other irrigated areas and potentially existing saline hotspots to a safe discharge point- preferably the brackish waters of an existing creek system.
- Shaping the filled landform as a cambered embankment to shed water rapidly and directing this runoff into graded natural watercourses, while avoiding detention in natural and artificial ponds where possible.
- Constructing the base of the embankment of free-draining rock fill and providing subsoil drains(to South Creek) where necessary, to prevent water accumulating on the fill / former land surface interface.
- Making maximum use of paving, especially of car parks and storage areas, to reduce the ground area available for rainwater infiltration. It is assumed that most of the Precinct will be built over in any case.
- Collection of stormwater from paved areas and roofs and directing it through a drainage system to approved discharge points along natural drainage lines.
- Grassing, mulching and tree planting in unpaved areas, with preference given to native species with high water demand (but making allowance for the relatively dry St Marys climate). Preference should also be given to deep-rooted trees and shrubs over shallow rooted grasses.
- Minimisation as far as practicable of the site area to be irrigated.
- On individual house blocks ensure garden areas easily drain to any catchmentwide stormwater system to ensure that salt does not accumulate within the garden beds, adjacent to building foundations or other salt sensitive infrastructure.

- Prepare garden beds and building foundations to minimise the potential for long term impacts such as soil structure decline that in turn leads to drainage problems. This could involve application of gypsum to foundation clay materials and the installation of subsoil drainage.

The observations made in previous studies suggest that poor stormwater design leads to salinity outbreaks on poorly drained soils and hence ‘salt safe’ drainage and storm water plans are critical components of any western Sydney development irrespective of the source and quality of water.

5.9.1 Residences

The main priority for groundwater management in house construction and landscaping is preventing excessive infiltration, bearing in mind that the proposed residential areas are largely on land that has been cleared for over sixty years and where residents are likely to greatly increase rather than decrease the number of trees and shrubs within the first few years of occupation.

Remedial/compensatory measures might include:

- Encourage residents to use water and nitrogenous fertilisers sparingly in garden irrigation, especially where slightly saline (say 500 mg/L TDS) recycled water is being applied.
- Encourage planting of drought- and salt-tolerant native species and, where possible, deep- rooted trees.
- Ensure that buried pipes are fitted with leak-proof junctions to accommodate shrink and swell movements in clay soils.
- Ensure that all downpipes are linked to sealed stormwater drains or storage tanks, and that unlined surface ponding is minimised.
- In preparing the development application for the subdivision works individual lot measures would be identified and implemented through the development approval process and restrictions on the use of the land via section 88B instruments.

5.9.2 Stormwater Conduits

All paved areas such as roads and carparks should potentially be kerbed and guttered, and runoff directed into stormwater pipes. Where stormwater is directed along unlined natural gullies these should, so far as possible, be configured such that recharge to groundwater is minimised by:

- Clearing the bed of obstacles such as fallen trees and eliminating breaks in gradient
- Planting deep-rooted trees along the banks of the gully, but not in the channel
- Vegetating the channel floor and allowing for this vegetation to be periodically maintained. The aim of these measures should be to reduce infiltration into the groundwater.

5.9.3 Basins

The key groundwater management issue with respect to basin is to consider the provision of a liner on a case by cases basis to reduce interaction between groundwater and the water in the basin.

5.9.4 Recycled Water Irrigation

At this point in time, it is unknown whether recycled water will be available for the Central Precinct. Should recycled water be proposed for irrigation purposes a land capability assessment in conjunction with Sydney Water would need to be undertaken and submitted with future development applications.

5.9.5 Groundwater Monitoring

In order to evaluate the infiltration reduction strategy outlined above, it will be necessary to monitor fluctuations in groundwater level and changes in water quality. It is recommended to use the three piezometers installed by Jacobs during the 2009 investigation (refer **Figure 5-11**) and any other existing piezometers across the site.

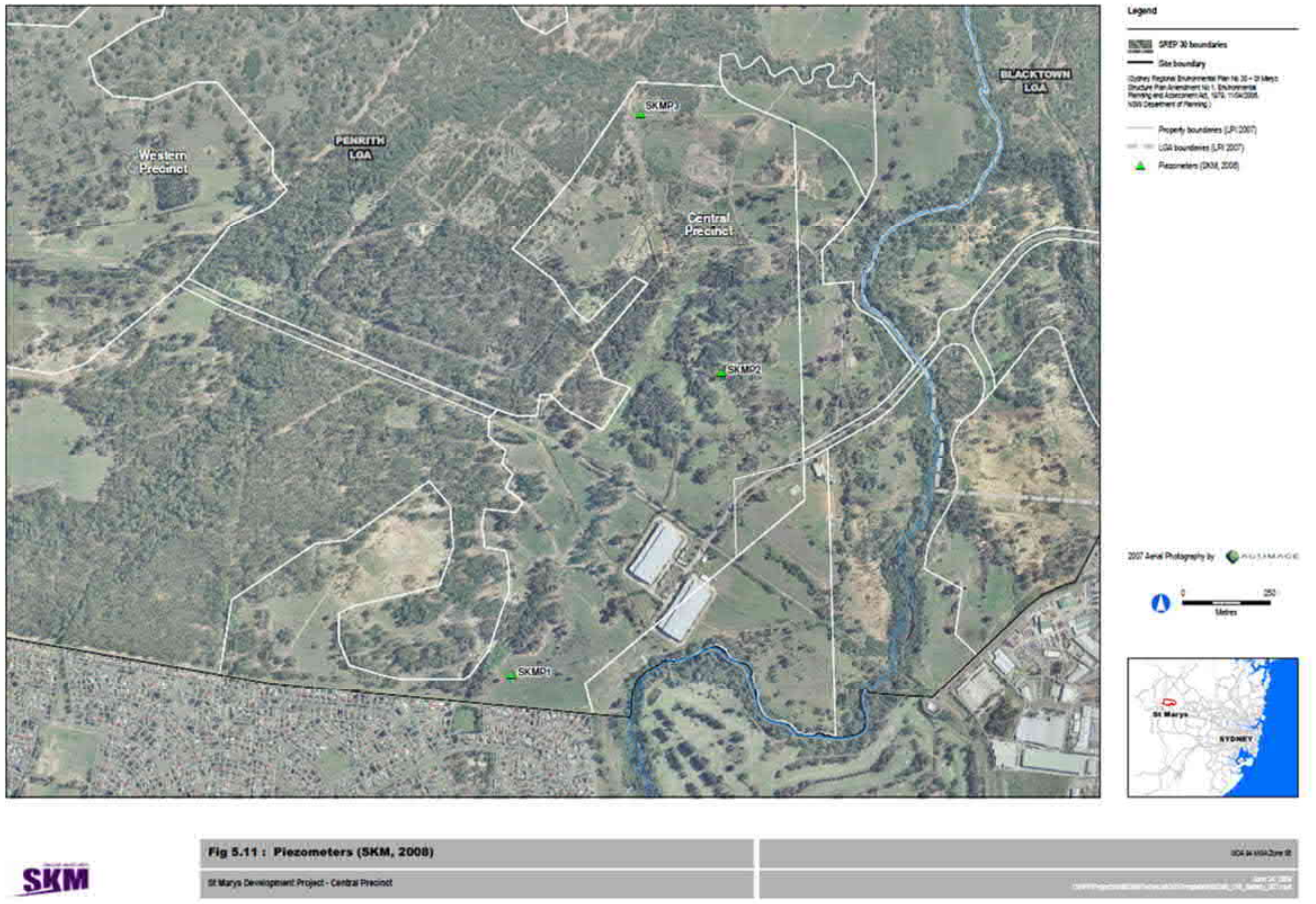


Figure 5-11 Piezometers (SKM, 2008)

5.9.6 Soil Salinity Management Measures

Erosion

- In the design phase of the study minimise the area of disturbance, in particular the extent of vegetation clearing.
- Optimise the route where possible to avoid steep slopes in order to reduce the potential for erosion of the natural landforms, cuttings and fill embankments.
- Carry out geomorphological and geotechnical investigations at waterway crossings to determine the stability of the streambed and banks and make recommendations on control measures required to minimise erosion impacts.

Excavation Methods

- Characterise the surface profile in respect to salinity (in accordance with the DLWC 2002 Site Investigations for Urban Salinity manual), depth to rock and associated excavation issues during construction planning and costing.
- Optimise the route to avoid areas of difficult excavation.

Soft Alluvial and Poor Drainage areas

- Carry out detailed investigation of stream crossings, alluvial and poorly drained areas.
- Optimise the route where possible to avoid those areas requiring significant trench support and dewatering, thus minimising dewatering and construction effort (construction methods, complexity, durations)
- Where possible select alignment based on land systems, groundwater and engineering geology overlays.

Quality Control

- Implement Management Strategies in accordance with Section 8.7 of the DIPNR (2003) Western Sydney Salinity Code of Practice and EPA Guidelines for construction and sediment control.
- Select appropriate salt resistant construction and piping materials, and select suitable temporary pavement and backfill materials

The salinity, erosion and sediment management strategy for the Central Precinct is summarised in **Table 5-4** and should also be read in conjunction with **Section 4.4** and **Appendix C** of this report.

Table 5-4 Overview of potential salinity, erosion and sediment management strategies

Objective	Benefit	Control	Details	Monitoring method	Management method
Salinity control: Minimise groundwater recharge	Prevent rising groundwater table level and development of saline soil problems	Minimise importation and use of potable water on the site	Reuse stormwater for irrigation of open areas. Minimise potable water demand	Install monitoring bore network	Monitor groundwater table levels. Perform regular random inspections of house sites and vegetation and general infrastructure areas.
		Reduce irrigation requirements	Adopt small gardens/ lawn areas. Establish low water requirement plants. Use mulch as much as possible. Use low flow water facilities.		

Objective	Benefit	Control	Details	Monitoring method	Management method
		Avoid use of infiltration pits to disperse surface water	Design stormwater system to negate need for home site stormwater storage disposal. Connect all downpipes directly to stormwater		
		Reduce leakage from basin and drainage facilities where possible	Line all permanent stormwater retention structures and basis if required		
Salinity control: Encourage use of groundwater as a resource	Maintain or lower groundwater table level	Encourage tree planting and retention, especially in areas of higher recharge	Use / retain native deep-rooted, large growing species.		

5.10 Soils Implication

Residual soils derived from weathered shale bedrock in western Sydney are typically of moderate to high reactivity (shrink-swell potential in response to drying and wetting cycles) and moderate dispersivity (the tendency of sodic soils to erode rapidly when in contact with fresh water). These characteristics are especially well developed where:

- There is a sharp texture contrast between a silty, low plasticity A-horizon and a high plasticity, sodic and saline B-horizon
- Where the soil profile, and especially the B-horizon is relatively thick, say 1-2m,
- On low gradient slopes and in low-lying ground, with grass rather than tree cover, where seasonal moisture changes within the soil profile are likely to be greatest.

Test results summarised on Table 5-2 indicate that the alluvial clays within the Central Precinct area are highly silty and of medium plasticity. The salinity results indicate that these clays are of low salinity, at least in the top 1m. The test pit logs demonstrate that the soil profiles, though deep (several metres), are poorly differentiated in terms of horizon development. These results suggest only moderate shrink-swell potential, by the standards of western Sydney clay soils.

Surface observations of widely spaced but narrow shrinkage cracks under the present drought conditions confirmed that these clays are of only moderate reactivity, despite the presence of shallow surface depressions resembling gilgais. In other parts of Australia gilgais are associated with the presence of high plasticity, highly reactive clay soils.

The relative absence of rill and gully erosion across the site, coupled with the low salinity of the soil B-horizon, suggest that these clays are of low dispersivity and hence comparatively non- erodible.

Filling of land within the project area, as proposed, will further reduce the impact of urban development on these soils. As well as protecting the natural soil profile from erosion by running water, the effect of a fill blanket will be to maintain relatively constant moisture content within the buried clay subgrade, thereby minimising the potential for both swelling and drying shrinkage.

5.11 Conclusion

Soil bore, groundwater and geophysical investigations in the Central Precinct indicate that shallow groundwater occurs at depths of 3 - 6 m and is of low salinity. Deeper water in the shale bedrock is moderately saline, in the range 3,500-8,000 mg/L, which is low by the standards of the St Marys property. It is concluded that the planned development is unlikely to result in surface salinisation and that the remedial measures proposed in the report – raising the ground level by filling such as at the Central Precinct – will further reduce this possibility.

5.12 References

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6. Filling of Land

6.1 Introduction

The proposed Lend Lease development at St Marys includes filling within the floodplain, modifications to road embankments and hydraulic structures and some channel works within South Creek.

Potential flooding impacts in South Creek and its tributaries due to the proposed development have been assessed by Jacobs using the 1D/2D hydraulic modelling program MIKEFLOOD, consistent with PCC's requirements.

This section provides a summary of the assessment, with full details included in **Appendix D**.

6.2 Background

Previous assessments were undertaken using a MIKE11 model produced by SKM (now Jacobs) in 2007. This MIKE11 model was previously considered to be appropriate by Blacktown City Council (BCC) and PCC. Recently, Worley Parsons has developed a new hydrodynamic model for South Creek and tributaries on behalf of PCC. This model has been developed using the RMA-2 software package, a two-dimensional hydrodynamic model. PCC advised Lend Lease that assessment of the potential flooding impacts should be consistent with PCC's new two-dimensional model.

6.3 Modelling the development scenario

This assessment has used the MIKEFLOOD hydrodynamic modelling package and was developed with consistent assumptions to produce results consistent with PCC's RMA-2 model. The MIKEFLOOD model was also developed to be consistent with the MIKE11 model (SKM 2007). The MIKEFLOOD model was used to assess the proposed developed scenario including proposed fill layout, upgrades to the East West Connector Road at South Creek, addition of Dunheved Link Road and removal of the abutments and embankment at Old Munitions Road.

6.3.1 Flood behaviour

Flood behaviour is modified in the developed case model due to placement of the fill areas. This has resulted in a constriction in the waterway width, particularly in the area around the East West Connector Road.

The fill in the Central Precinct causes modification to the flow patterns on the western bank of South Creek with existing overland flow paths interrupted by the proposed fill, including the filling of one drainage line that currently acts as an overflow path for South Creek in larger events. This results in localised flood impacts directly to the south of Central Precinct.

6.3.2 Flood impacts

Flood impacts have been assessed through producing afflux mapping. In the proposed scenario, both the East West Connector Road and Dunheved Link would raised and built respectively above the 5% AEP flood level. A large number of culverts would be required under Dunheved Link to convey floodwaters. A small landscaped berm (~30 m long and 0.5 m high) would be required in the design to prevent flood impacts to the upstream industrial area.

The proposed development produces 38 mm impact at the upstream site boundary and 16 mm at the downstream site boundary during a 1% AEP event. This is consistent with PCC's requirement of no more than 100 mm afflux at the site boundary and the level of impact adopted by PCC in the 2009 Central Precinct Plan and Development Control Strategy (DCS). Impacts upstream of the site are largely contained within the Dunheved Golf Course.

6.4 Conclusions

The following conclusions and recommendations are made from this investigation.

- A new MIKFLOOD hydraulic model was developed for the St Mary’s area. The purpose of this model was to meet PCC’s requirement to assess the proposed developments at Central Precinct and the Dunheved development in a manner consistent with Worley Parsons RMA-2 model.
- The MIKEFLOOD model produced design event peak flood levels consistent with those produced by the Worley Parsons RMA-2 model for existing flooding conditions. The MIKEFLOOD model was also deemed to be adequately consistent with the previous MIKE11 model (SKM 2007).
- The proposed development in its current form produces water level impacts limited to 38 mm at the upstream site boundary and 11 mm at the downstream site boundary in the 1% AEP regional tailwater event.
- The impacts of the proposed development meet PCC’s Development Control Plan (DCP) requirements of afflux not exceeding 100 mm at the upstream boundary (noting that the DCP is not explicitly applicable).
- The impact at the upstream site boundary is 38 mm, consistent with the level of impact adopted by PCC in the 2009 Central Precinct Plan and Development Control Strategy (DCS).
- The preferred developed scenario does not inundate any additional buildings in the 1% AEP event.
- Impacts at the downstream boundary are 11 mm and 16 mm in the regional and local tailwater scenarios. The characteristics of the floodplain result in relatively little attenuation, meaning these small impacts propagate for a significant distance downstream. However, these impacts do not cause a significant increase in flood extent, or result in inundation of additional properties. No material flood impact is therefore expected at downstream properties.
- An independent peer review by Worley Parsons using the South Creek Flood Study RMA-2 model predicted “impacts that are equal to or lesser than those documented in the FIA Report”. Furthermore, the review considered the impacts “minor for all areas outside of the Lend Lease site.”

The full flood assessment is provided in **Appendix D**.

7. Flood Evacuation Strategy

A flood evacuation strategy has been developed and is outlined in a separate report by Molino Stewart (2014).

Appendix A. Assessment of drainage controls

A.1 Introduction

A hydrologic assessment of the proposed St Mary's Central Precinct was undertaken to evaluate the proposed development's impact on flood discharges in South Creek. The proposed development is a very small area relative to the total upstream catchment of South Creek (0.4%) and is located immediately upstream of South Creek. Runoff from the catchment therefore contributes directly to the flows of South Creek without impacts on any intermediate area. The assessment therefore tested the hypothesis that:

The Central Precinct development has a negligible impact on flooding in South Creek and that stormwater detention is not necessary to mitigate against an increase in downstream peak flows and flood levels.

A.2 Existing South Creek flooding

A Flood Study of South Creek is currently being prepared by Worley Parsons (2014) for PCC. Flood modelling of South Creek was undertaken for the Flood Study and design flow hydrographs at a location immediately upstream of the Central Precinct site were provided to Jacobs for the 20 year ARI, 100 year ARI and PMF events.

The 20 and 100 year ARI flow hydrographs for the existing conditions in South Creek are shown in **Figure A 1**. The Draft Flood Study identified the 36-hour storm duration as critical and it is assumed the hydrographs provided relate to this critical duration**.** The hydrographs show the peak flow in South Creek is 825m³/s and 1140m³/s for the 20 and 100 year ARI events respectively, and both events peak at 27 hours following the start of the event.

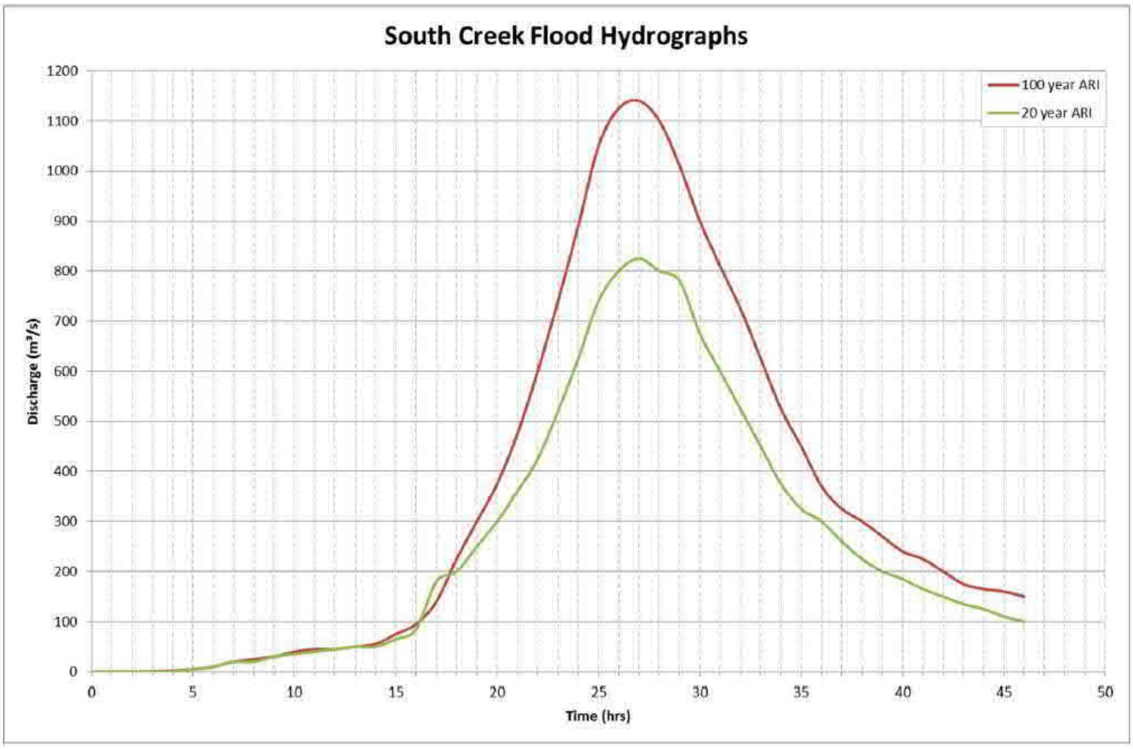

Figure A 1 South Creek hydrographs for the 20 yr and 100 yr ARI 36-hour storm from the Draft South Creek Flood Study (Worley Parsons, 2014)

A.3 Methodology

This assessment was undertaken using XP-RAFTS to represent the hydrological network of the South Creek catchment. The model simulates runoff hydrographs at defined points for a given set of catchment conditions and rainfall events. The generated runoff hydrograph is routed through the system to provide flow results at a number of node locations throughout the network.

The model was used to determine peak flow discharge from the Central Precinct into South Creek for the following conditions:

- Existing catchment conditions
- Proposed developed catchment conditions without peak flow mitigation
- Proposed developed catchment conditions with peak flow mitigation

The model results were then compared against discharges in South Creek obtained from the Draft South Creek Flood Study (Worley Parsons, 2014).

A.4 Central Precinct modelling

A.4.1 XP-RAFTS model set up

Design rainfall Intensity-Frequency-Duration (IFD) data was generated for the St Marys location following the procedure outlined in Australian Rainfall & Runoff (ARR, 1987). The design rainfall data was used in the RAFTS model and a suite of storm durations were input for each ARI rainfall event. The site IFD data is shown in **Table A. 2.** The site IFD data is very similar to IFD values provided in PCC's Stormwater Drainage for Building Developments (2013) guidelines. IFD data is shown in **Table A 2** below.

Table A 1 IFD Rainfall Data

Duration (min)	2yr ARI	20yr ARI	100yr ARI
20	52.82	91.89	121.9
30	42.83	74.46	98.75
60	29.05	50.44	66.86
90	23.04	39.89	52.81
120	19.48	33.65	44.51
180	15.33	26.41	34.89
360	10.16	17.42	22.97
720	6.75	11.51	15.15

The Central Precinct area is approximately 133 hectares in size (approximately 0.4% of the South Creek catchment area) and is currently rural in nature. The proposed development will convert the current rural area to residential development with associated road networks and stormwater drainage infrastructure. As no recent updates to parameter requirements were found in the recent PCC documentation, the parameters used in the XP-RAFTS modelling for the existing and developed conditions were selected according to previous investigations undertaken by Jacobs. They are shown in **Table A 2.**

Table A 2 Adopted XP-RAFTS model parameter values

Parameter	Existing	Developed
Imperviousness (%)	0	80%
Catchment equal slow area slope (%)	0.5	0.5
Initial & continuing loss (pervious)	10mm & 2.5 mm/hr	10mm and 2.5 mm/hr
Initial and continuing loss (impervious)	-	1mm and 0 mm/hr
PERN (pervious)	0.05	0.025
PERN (impervious)	-	0.015

Developed conditions were modelled with and without stormwater detention for peak flow mitigation. A number of detention volume and outflow configurations were tested to determine the volumes and configuration necessary ensure peak outflows from the Precinct did not exceed existing levels for the design events modelled.

A.4.2 Results

The XP-RAFTS models were run for the 2 year, 20 year, and 100 year ARI design events. Each design event was run for a number of storm durations ranging from 1 hour to 72 hours to ensure the critical event was identified. The results are given in **Table A 3, A 3** and **A 5**.

Table A 3 - 2 year ARI Results

Storm duration (hrs)	Existing		Developed –no detention		Developed – with detention	
	Peak flow (m^3/s)	Time to peak (hrs)	Peak flow (m^3/s)	Time to peak (hrs)	Peak flow (m^3/s)	Time to peak (hrs)
1	1.0	1	21.7	0.4	0.6	1.5
1.5	1.3	1.5	**23.4**	**0.5**	0.8	1.8
2	1.6	2	22.2	0.6	1.4	2.1
3	2.1	2.8	14	0.8	1.8	2.8
6	2.5	4.7	9.6	2	2.2	5.1
12	**3.0**	**9.6**	9.2	7	2.8	10
18	2.6	11	6.2	7	2.6	11.2
36	2.7	20	4.7	18	**2.8**	**20.2**
72	1.7	20.2	3.2	20	2	20.2

Table A 4 - 20 year ARI Results

Storm duration (hrs)	Existing		Developed –no detention		Developed – with detention	
	Peak flow (m^3/s)	Time to peak (hrs)	Peak flow (m^3/s)	Time to peak (hrs)	Peak flow (m^3/s)	Time to peak (hrs)
1	2.9	1	39.9	0.4	2.4	1.2
1.5	3.8	1.5	**43**	**0.5**	2.9	1.6
2	4.6	2	40.5	0.6	3.2	2.1

Storm duration (hrs)	Existing		Developed –no detention		Developed – with detention	
	Peak flow (m^3/s)	Time to peak (hrs)	Peak flow (m^3/s)	Time to peak (hrs)	Peak flow (m^3/s)	Time to peak (hrs)
3	5.5	2.5	24.7	0.8	3.6	2.7
6	**6.2**	**4**	17.3	2	3.9	5.1
12	6.2	9.2	16.1	7	4.5	10.1
18	5.7	9.1	11.3	7	4.4	11.2
36	6.1	19.1	8.8	18	**4.7**	**20.3**
72	4.9	20	6.4	19.7	3.8	20.2

Table A 5 - 100 year ARI Results

Storm duration (hrs)	Existing		Developed –no detention		Developed – with detention	
	Peak flow (m^3/s)	Time to peak (hrs)	Peak flow (m^3/s)	Time to peak (hrs)	Peak flow (m^3/s)	Time to peak (hrs)
1	4.9	1	50.4	0.4	3.4	1.2
1.5	6.3	1.5	**54**	**0.5**	3.9	1.6
2	7.4	2	50.9	0.6	4.2	2.1
3	8.4	2.5	30.8	0.8	4.5	2.8
6	**8.9**	**3.8**	21.7	2	5.6	5.1
12	8.3	7.1	19.9	7	**8.5**	**9.2**
18	7.8	9.1	14.1	7	7.3	11
36	8.3	18.8	11.2	18	7.9	19.2
72	6.9	20	8.3	19.3	4.7	20.2

A summary of peak flows and critical durations is shown in **Table A 6**.

Table A 6 Central Precinct peak flows, critical storm durations and time to peak flow

Design event (ARI)	Existing			Developed –no detention			Developed – with detention		
	Peak flow (m^3/s)	Critical storm duration (hrs)	Time to peak (hrs)	Peak flow (m^3/s)	Critical storm duration (hrs)	Time to peak (hrs)	Peak flow (m^3/s)	Critical storm duration (hrs)	Time to peak (hrs)
2 yr	3.0	12	9.6	23.4	1.5	0.5	2.8	36	20.2
20 yr	6.2	6	4	43	1.5	0.5	4.7	36	20.3
100 yr	8.9	6	3.8	54	1.5	0.5	8.5	12	9.2

These results demonstrate the critical storm duration and time to peak flow for the Precinct runoff is different from that of the South Creek.

The results also show that the proposed development will result in increased peak flows from the Precinct and a shortened critical storm duration and time to the peak flow if stormwater detention is not provided. The introduction of stormwater detention will prevent existing peak flows from the Precinct being exceeded and will result in a longer critical storm duration and time to the peak flow compared to existing.

However, the next section demonstrates that these changes are not significant to the peak flows in South Creek, and therefore have minimal impact upon flood levels downstream.

A.5 Impact on South Creek flooding

The impact of the development on flood discharges in South Creek have been assessed by looking at the flow coming from the development area during the peak flow in South Creek. As stated previously, the peak flow in South Creek occurs during the 36-hour storm, after a period of 27 hours.

Therefore the existing and developed hydrographs for runoff from the development during the 36-hour storm event, generated using XP-RAFTS, were compared with the South Creek hydrograph (shown previously in shown in **Figure A 1)**.

The 20 year and 100 year ARI events for the 36-hour storm are provided in **Figure A 2** and **Figure A 3** respectively. They show that peak flows from the development occur on average at around 19 hours into the storm. This 8 hours before the peak flow occurs in South Creek.

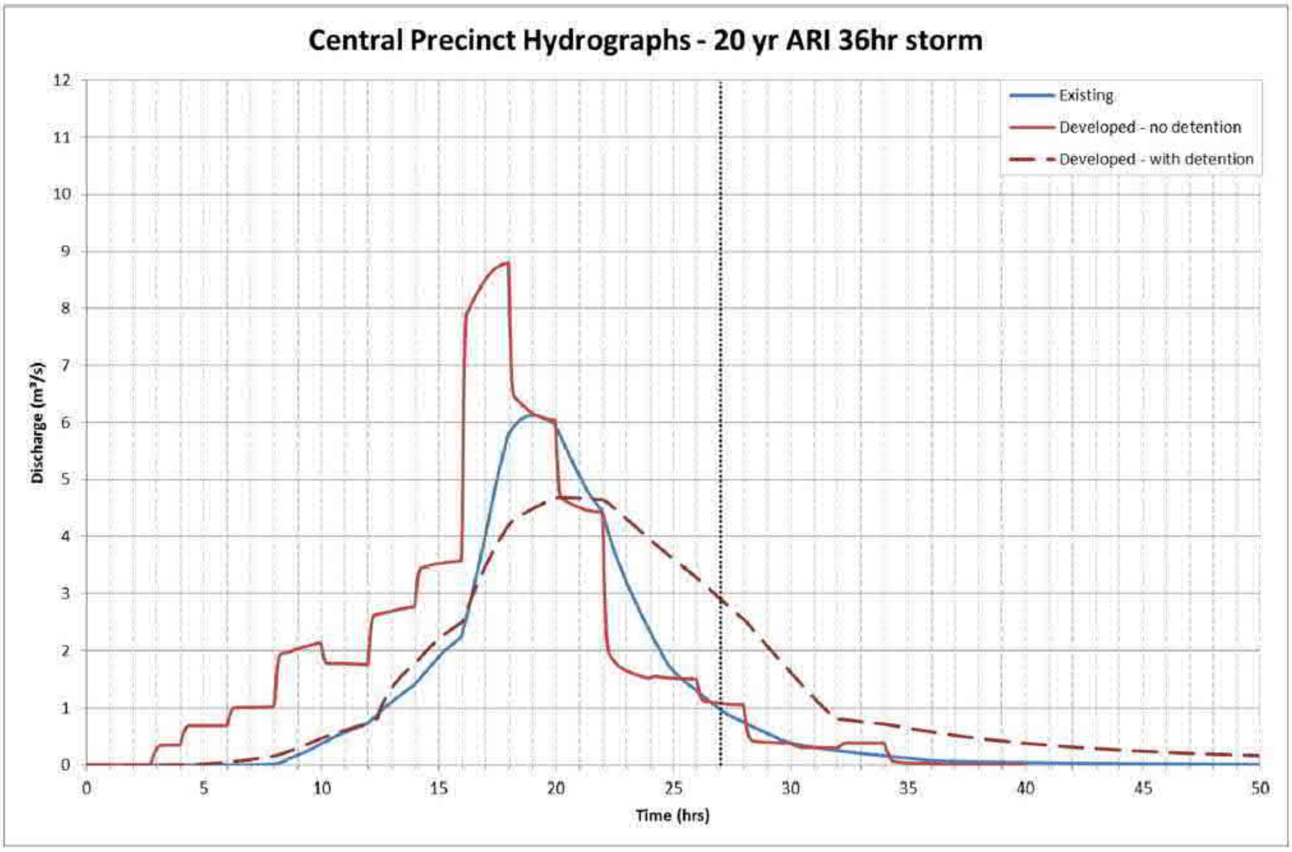

Figure A 2 Central Precinct hydrographs for the 20 year ARI, 36 hour storm duration. This shows the peak occurring prior to the time of the South Creek peak, depicted with the black dotted line.

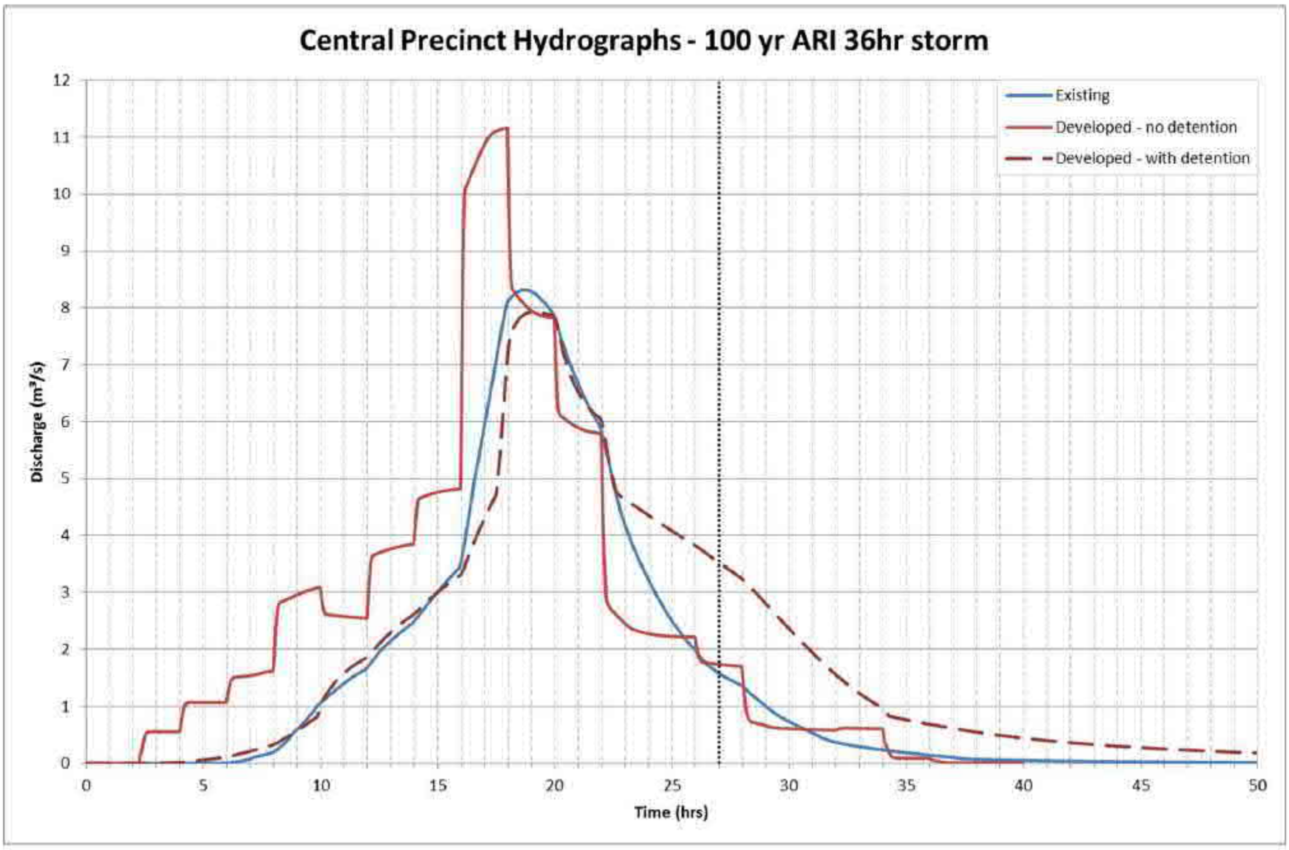

Figure A 3 Central Precinct hydrographs for the 100 year ARI, 36 hour storm duration. This shows the peak occurring prior to the time of the South Creek peak, depicted with the black dotted line.

A comparison between the hydrographs from the Precinct runoff and South Creek for the 100 yr ARI 36-hour storm is shown in **Figure A 4.** Here it can clearly be seen that peak flows of South Creek are significantly larger than the flow coming off the Precinct, both with and without detention. At this point in time, flow from the Precinct are insignificant.

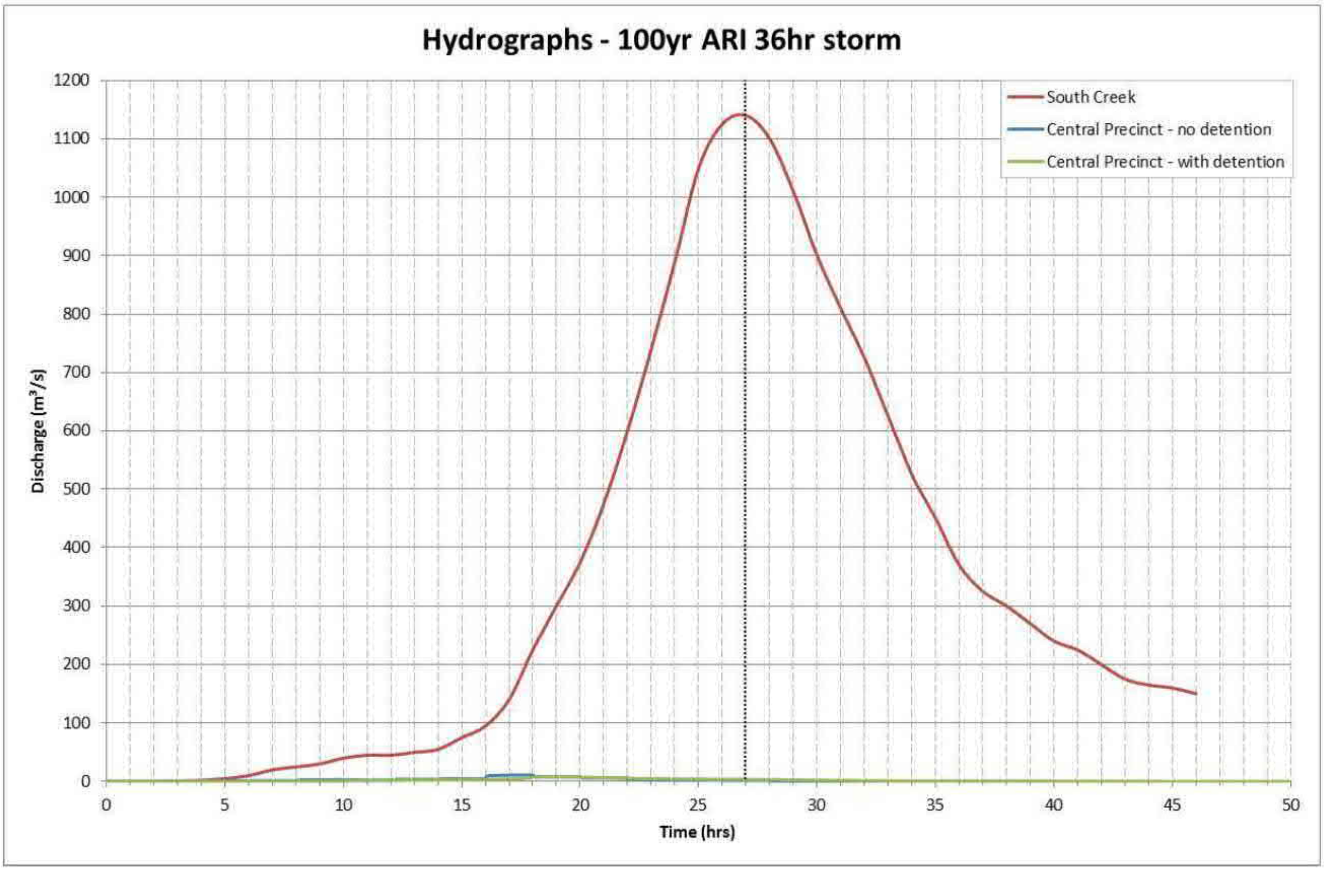

Figure A 4 Comparison of Central Precinct and South Creek hydrographs

These results are summarised and described numerically in **Table A 6.** It shows the estimated total flow in South Creek during the flood peak at 27 hours after the onset of rainfall, as well as the flow from Central Precinct at this time.

Table A 7 Flows for the 36-hour duration event at the peak of South Creek, 27 hours after the onset of rainfall

Design event (ARI)	Existing		Developed –no detention		Developed – with detention	
	Flow from Central Precinct (m^3/s)	Total South Creek flow (m^3/s)	Flow from Central Precinct (m^3/s)	Total South Creek flow (m^3/s)	Flow from Central Precinct (m^3/s)	Total South Creek flow (m^3/s)
20 yr	1	826	1.1	826.1	2.9	827.9
100 yr	1.6	1141.6	1.7	1141.7	3.5	1143.5

The hydrographs and results shown above in **Table A 6** above indicate that:

- Flows from the Central Precinct are minimal compared to the peak flow in South Creek, with ratios ranging from 0.1% to 0.4%.
- Providing detention delays the runoff from the Precinct so that it is released closer to the time of peak flow in South Creek. Therefore providing detention creates a paradox and does not benefit flood levels downstream.

A.6 Conclusion and recommendation

The 133 hectare site of Central Precinct is located immediately adjacent to South Creek and represents approximately 0.4% of the South Creek catchment area upstream of the site. Due to its location and small size relative to the South Creek catchment the proposed development has a negligible impact on flooding in South Creek. This has been demonstrated by hydrologic modelling which shows discharges from the site will be minimal compared to the discharges in South Creek even if stormwater detention is not provided. As the Central Precinct development will have a negligible impact on flooding in South Creek, stormwater detention is not considered necessary to mitigate against an increase in peak flow from the development. It is proposed that a merit based approach assessment be adopted for either zero or, as two basins are proposed for water quality mitigation, up to a maximum of two detention basins in the Central Precinct.

Further details on the PCC approved detention strategy for the Central Precinct have been provided in the January 2017 Cardno report : *Stormwater Detention Strategy Jan 2017.*

Appendix B. Assessment of Water Quality Controls

B.1 MUSIC modelling design objectives

A water quality assessment was undertaken using the MUSIC water quality model (eWater CRC, Version 6.2). The main purpose of the modelling was to determine the land take required for stormwater management basins to ensure that the water quality objectives of the SREP30 are met. These are:

- No net increase in annual pollutant load into the receiving waterways

The following section provides information on the MUSIC modelling inputs and process

B.2 Modelling methodology

The following methodology was adopted in the MUSIC model:

- The Western and Central Precincts have been considered together for water quality purposes. There are three discharge areas for these two Precincts: at S1, S2 and S3 as shown Figure 4-1. The combined annual pollutant load at the discharge points for the existing case was firstly compared to the combined annual pollutant load in the developed case. This is similar to the approach that was adopted in the 1998 SKM Watercycle Management Report and the 2009 Precinct Plan. Then mitigation controls were sized to ensure:
    - that mean annual loads would not exceed existing conditions
    - that the post development mean annual loads were reduced in comparison to a developed case without mitigation.
- It has been estimated that the actual stormwater management basin surface area is approximately 75% of the land take required. The remaining approximated area would be required for detention, pathways and benching purposes. The stormwater management ponds for the Western and Central Precinct have been modelled assuming an average 1.5m depth across the pond.
- There is an existing pond in the southern portion of the Western Precinct that not been included in the modelling for this assessment. For the future development case the function of this existing pond will not change compared to its existing function and can be therefore omitted from the modelling.

·

B.3 Data inputs

B.3.1 Rainfall data:

Pluviograph data for use in the model was obtained from the Bureau of Meteorology for station 67113 Penrith Lakes AWS for the period December 1 January 1999 to 31 December 2008. The ten year period was entered to the MUSIC model and the model was run at six minute time steps.

B.3.2 Catchment areas

The Central Precinct area is approximately 133 ha and the total catchment area upstream of the receiving point is approximately 1331 ha, incorporating existing urban, forested and rural areas as well as proposed development areas. Table B 1 provides all the subcatchment areas used in the MUSIC, as depicted in Figure B 1.

Table B 1 – MUSIC model catchment areas

Catchment name	Size (ha)
Cranebrook	7.9
College	2.4
Forested 1	1.7
Forested 2	1.8
Regional Park	33.1
Rural 1	62.1
Rural 2	8.2
Regional Park 1	81.6
Regional Park 2	306.6
V3c1	10.5
V3c2	7.9
V3c3	8.4
V6a	11.4
V6c	1.7
V6b	7.8
Llandilo 1	8.9
Llandilo 2	9.4
Llandilo 3	1.1
EL1	4.7
EL2	39.8
EL3	5.0
EL4a	22.0
EL5	16.8
EL6	14.0
EL7	18.4
EL8	8.2
EL9	37.7
EL10	43.2
EL11	74.4
Urban 1	27.1
Urban 2	20.8
Urban 3	84.7
Urban 4	169.3
CP1*	123.9
CP2	40.1

*Total catchment for CP1 is 133ha but in the MUSIC model the area of proposed basins and drainage stream have been removed

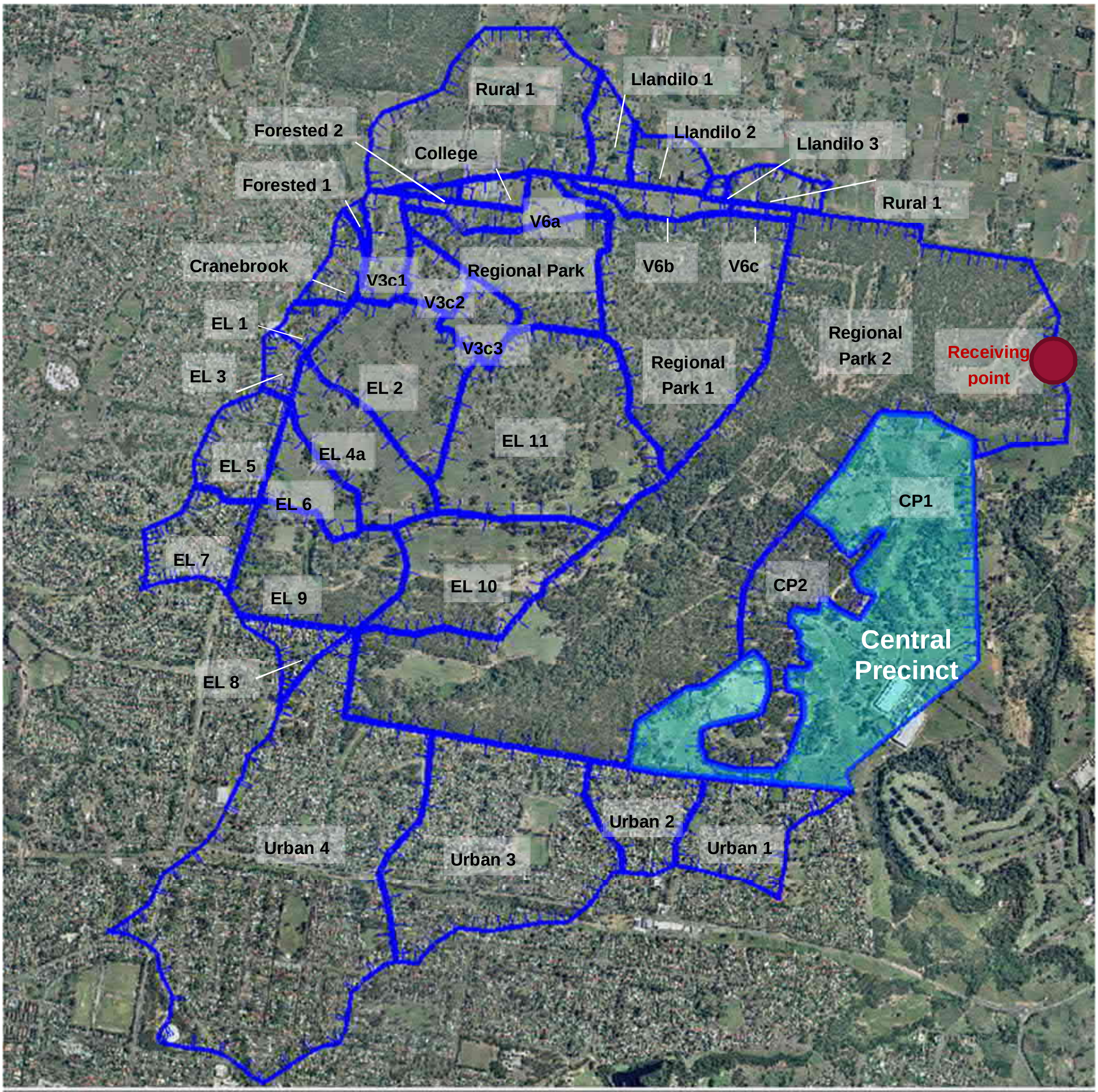

Figure B 1 Catchments used in the MUSIC model

B.3.3 Pollutants

Pollutant concentration parameters for each catchment were reviewed based on appropriate literature and known catchment conditions. The adopted typical values used for total suspended solids (TSS), total phosphorus (TP) and total nitrogen (TN) are given in **Table B 2**

Table B 2 : Event Mean Concentration (EMC) values adopted for the catchments

Catchment	TSS		TP		TN	
	Base flow	Storm flow	Base flow	Storm flow	Base flow	Storm flow
Urban[1]	15.8	141	0.141	0.251	1.29	2
Agricultural[2]	25.1	200	0.132	0.537	1.19	3.89
Forested[3]	7.94	120	0.0316	0.12	0.724	1.8

1-Values from Council's WSUD Technical Guidelines (PCC, 2015)

2-Default MUSIC values used

3-Default MUSIC values modified for pre-development conditions to take into account high levels of faecal matter from resident kangaroos.

B.3.4 Rainwater tanks

The effect of rainwater tanks on water quality has been included in the model. The number of tanks in use has been estimated based on the requirement for properties over 400m^2 requiring rainwater tanks. The rainwater tanks would be used for irrigation of landscaped areas only. The irrigation demand on tanks has been taken from Council's *WSUD Technical Guidelines* (PCC, 2015). Rainwater tanks would not be plumbed to dwellings for internal use. Also treated effluent reuse is not available for this development. The parameters used for modelling the rainwater tanks were are shown in **Table B 3**.

Table B 3 : Rainwater tanks in the Central Precinct

Parameter	Value
Estimated total number of tanks	1811
Size of each tank	2 kL
Estimated tank dimensions (length x width)	2210 mm x 700 mm
Average daily internal demand	-
Average annual irrigation demand	75 kL/yr/lot

B.3.5 Treatment nodes

B.3.5.1 Gross Pollutant Traps

Treatment would be provided by GPTs, dry infiltration basins and stormwater basins, all of which have been included in the MUSIC model. The GPTs included in the MUSIC model for the Central Precinct are Humeguard units. CDS units are also included in the model in a number of locations within the Western Precinct. The GPT removal rates used in the model were selected from Council's *WSUD Technical Guidelines* (PCC, 2015) and manufacturers' documentation, and are outlined in **Table B 4.**

Table B 4 GPT removal rates

Pollutant	CSD unit	Humeguard
Gross pollutants	95%	85%
TSS	70%	0%
TP	30%	0%
TN	0%	0%

B.3.5.2 Dry infiltration basins

Within the Central Precinct, five dry infiltration basins are proposed in accordance with the Cardno proposed basins for Central Precinct). The details of the proposed dry infiltration basin and the MUSIC modelling for the Central Precinct are provided in a separate Stormwater Management report prepared by Cardno. **Water quality Basins**

Outside the Central Precinct the Jordan Spring Basin, East Lake and basins C2, I and B also contribute to reducing pollutant loads in South Creek and have been included in the MUSIC model. The design of Jordan Springs Lake, East Lake and C2 are detailed in separate reports. The design parameters used in the model to represent Basins I and B are outlined in **Table B 5**. The exfiltration rates for Basins B and I has been taken from in situ soil tests.

The northern sub-catchment of 35ha in the Central Precinct discharges into Basin B to receive water quality treatment before discharging into an existing tributary of South Creek.

Table B 5 - Water quality basin design parameters

Pond parameter	Basin I	Basin B
Water surface area (m^2)	38,645	20,850
Water volume (m^3)	69,480	36,275
Extended detention depth (m)	0.3	0.3
Exfiltration rate	13.5 mm/hr	0.32mm/hr

B.4 MUSIC Model Results

The indicative locations of the proposed stormwater management basins that would meet the water quality objective for the Western and Central Precinct were shown in **Figure 4-1** and the land take for each is summarised in xx.

The MUSIC model can provide the annual pollutant load exported for Total Suspended Solids (TSS), Total Phosphorus (TP) and Total Nitrogen (TN). The results are shown in **Table B 6**.

Table B 6 MUSIC Results for the Western and Central Precincts

Pollutant	Existing	Developed without mitigation	Developed with mitigation	Reduction from existing	Complies with SREP30 / EPS criteria?
TSS (kg/yr)	155,000	310,000	79,500	-48%	Y
TP (kg/yr)	270	531	231	-13%	Y
TN (kg/yr)	2,360	4,140	2,230	-4%	Y

In conclusion, the water quality assessment demonstrates that the proposed strategy achieves the design objective. The SREP and EPS objectives of improving on existing conditions have been met for TSS, TP and TN.

Appendix C. Douglas Partners groundwater and soils report including borehole logs

Douglas Partners
Geotechnics • Environment • Groundwater

Integrated Practical Solutions

REPORT
ON
SALINITY INVESTIGATION

CENTRAL PRECINCT
ST MARYS

Prepared for
SINCLAIR KNIGHT MERZ

Project 45529
July 2008

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Geotechnics • Environment • Groundwater

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JL:mh

Project 45529

16 July 2008

REPORT ON SALINITY INVESTIGATION
CENTRAL PRECINCT, ST MARYS

1. INTRODUCTION

This report presents the results of a salinity investigation by Douglas Partners (DP) of approximately 170 ha of the Central Precinct of a proposed residential development west of South Creek at St Marys (Figure 1 below), in an area formerly occupied by Australian Defence Industries (ADI). The work was commissioned by Sinclair Knight Merz (SKM), who carried out a concurrent geotechnical investigation and provided field and laboratory test results for use by DP in the salinity assessment.

Figure 1 – Approximate site location

In accordance with our Revised Proposal Syd080035 dated 11 March 2008, the salinity investigation comprised:

- non-intrusive electromagnetic (EM) profiling by DP to acquire soil conductivity data;
- test bore drilling, soil sampling and testing by SKM (on which DP subsequently relied); and
- analysis and reporting by DP of soil salinities and related soil aggressivities, with no reference to other site conditions such as sodicity or groundwater.

This report describes the EM profiling carried out between 20 and 22 May 2008 and presents the results of the EM profiling, subsequent laboratory testing and correlation with the EM data. An assessment is presented of soil salinities within anticipated residential foundation depths and within likely services depths, together with a preliminary salinity management plan. Appendix A contains drawings showing field data, inferred salinities and salinity constraints maps.

2. SITE DESCRIPTION AND ACCESS

The centre of the site is approximately 500 m west of South Creek and comprises undulating, sometimes steep, grass covered fields, some fenced-off areas, dense stands of trees, spoil mounds in the north and a warehouse complex in the southeast corner (see Figure 1 on page 1 and Photos 1 and 2 below). Parts of the site were inaccessible for EM profiling or required significant variations to the planned grid of survey lines. Where the resulting survey line spacings were excessive, soil salinity could not be assessed. These areas are identified in the attached Drawings (Appendix A).

Photo 1 – Grassed field and dense trees

Photo 2 – Spoil mound in north of Precinct

3. REGIONAL GEOLOGY

Reference to the Penrith 1:100 000 Geological Series Sheet (Ref. 1) indicates that the site is underlain by Bringelly Shale of the Wianamatta Group of Triassic age. This formation typically comprises shale, carbonaceous claystone, laminite and some minor coaly bands. Bedrock may be mantled by alluvium (fine sand, silt and clay) of Quaternary age within the drainage systems of South Creek on the eastern side of the site and a tributary of South Creek on the western and northern sides of the site.

4. SALINITY POTENTIAL

The Department of Infrastructure, Planning and Natural Resources (DIPNR, now DNR), on their map entitled "Salinity Potential in Western Sydney 2002" (Ref. 2), indicates "high salinity potential" in the immediate vicinity of the tributary to South Creek, which flows northward beyond the western and northern site boundaries. Throughout the Central Precinct however, a "moderate salinity potential" is mapped, indicating scattered areas of scalding and indicator vegetation but no mapped salt concentrations. These DIPNR inferences are based on soil types, surface levels and general groundwater considerations but are not in general ground-truthed, hence it is not generally known if actual soil salinities are consistent with the mapped salinity potentials.

5. INVESTIGATION METHODS

5.1 Electromagnetic (EM) Profiling

EM profiling was undertaken as part of the examination of soil salinity potential, enabling rapid continuous measurement of apparent conductivity, to supplement the laboratory electrical conductivity testing of discrete soil samples.

Apparent conductivity is variously referred to as ground conductivity, terrain conductivity, bulk conductivity or bulk electrical conductivity and is generally designated as σ_a or ECa. Although measurement of apparent conductivities can include contributions from a variety of sources including groundwater, conductive soil and rock minerals and metals, it has been estimated (Baden Williams in Spies and Woodgate, 2004, Ref. 3) that in 75 - 90% of cases in Australia, apparent conductivity anomalies can be explained by the presence of soluble salts. Apparent conductivity can therefore be considered, in the majority of cases, a good indicator of soil salinity.

The survey was undertaken using a DualEM-4 ground conductivity meter mounted 1 m above the ground surface from the side of an all terrain vehicle (ATV), as indicated in Photo 3 (below).

Photo 3 – DualEM-4 mounted on ATV

The DualEM recorded data using the Horizontal Coplanar (HCP) and Perpendicular (PRP) coil configurations concurrently, for theoretical Depths of Exploration (DoE) of 4.6 m and 2.4 m respectively. The DualEM responds to ground conductors at depths up to approximately 6 m below the coils, however the DoE are defined as the theoretical depths at which 70% of the total response should be received. Allowing for the height of the coils above ground, it can be said that in the HCP and PRP configurations, the DualEM was responding largely to soils at depths up to 3.6 m and 1.4 m, respectively.

A Sokkia Crescent R130 Differential Global Positioning System (DGPS) receiver, antenna and TDS Recon hand-held computer were employed to digitally record grid coordinates at 1 second intervals as the ATV was navigated around the survey area. ECa data were acquired at a 1 second repetition rate and logged to a GeoScout digital data logger, which also recorded the DGPS data.

Data were obtained along approximately 22 km of linear traverse (28,000 data points) in all accessible parts of the site, with an average data point spacing of 1.5 m. A grid of primary survey lines 100 m apart was approximated in the accessible areas as shown by the ECa measurement points (track of the ATV) in Drawing 1 (Appendix A).

5.2 Horizontal Control

All field measurements and mapping for this project have been carried out using the Geodetic Datum of Australia 1994 (GDA94) and the Map Grid of Australia 1994 (MGA94), Zone 56. Digital mapping has been carried out in a Geographic Information System (GIS) environment using MapInfo software.

5.3 Test Bores and Soil Tests

As part of the salinity investigation, 26 test bores were drilled across the site by SKM. The locations of 16 of these test bores were recommended by DP after examination of the EM data, in order that laboratory tests could be made of salinities at the locations of ECa anomalies and background values. Some recommended locations were not accessible for drilling and the locations actually drilled were 9 m to 67 m (average 35 m) from recommended locations. Drilled locations are shown in Drawings 4 and 5 (Appendix A) and Table 1 (Appendix B).

At 23 of these locations, test bores were drilled to depths of 3 m. Remaining test bores were drilled to refusal at depths of 1.25 m to 2.0 m. Soil samples were taken at intervals of 0.25 m (to maximum depths) at 17 locations and at 0.5 m intervals below depths of 0.5 m at the remaining 9 locations. All samples were tested by SKM for pH (the primary indicator of soil aggressivity), for $EC_{1:5}$ (the conductivity of a 1:5 soil:water paste) and for soil texture (M) which allows computation of soil salinity ECe from the formula ECe = M x $EC_{1:5}$.

6. FIELD WORK RESULTS

6.1 EM Profiling

On completion of EM profiling, apparent conductivity (ECa) field data, from both HCP and PRP coil configurations, were added to the GIS database for interpolation onto regular grids throughout the area surveyed. Drawings 2 and 3 (Appendix A) present the apparent conductivities as colour images with continuous colour spectral scales in milliSiemens/metre (mS/m). Areas of most interest are those at the red end of the spectrum (up to 200 mS/m), representing the highest apparent conductivities and potentially the highest salinities, which are generally concentrated in the southern half of the site and the central north of the site. The value of EM profiling, with high along-line sampling density and appropriate line spacings, is the ability to identify local variations in the salinity distribution which are not visible in the broader-scale salinity potential map and not identifiable by spot tests such as drilling.

6.2 Soil Sampling and Testing

Details of the subsurface conditions encountered in the test bores are presented elsewhere by SKM, however SKM test results (Table 1, Appendix B) indicates the following textural groups:

LIGHT CLAY	25%;
CLAY LOAM	53%;
LOAM	2%;
SANDY LOAM	15%; and
SAND	5%.

Table 1 also lists the results of pH and $EC_{1:5}$ tests and ECe calculations for all samples.

7. SALINITY ASSESSMENT FROM TEST BORE RESULTS

The DLWC guideline for salinity investigations (Ref. 4) applies the method of Richards (1954, Ref. 5) and Hazelton and Murphy (1992, Ref. 6) to the classification of soil salinity on the basis of ECe. The implications of the resulting salinity classes on agriculture are described in Table 2 (below) and it is commonly considered that moderately saline to highly saline soils (as defined in Table 2) require management in the urban built environment.

Table 2 – Soil Salinity Classification

Class	ECe (dS/m)	Implication
Non Saline	<2	Salinity effects mostly negligible
Slightly Saline	2 – 4	Yields of sensitive crops affected
Moderately Saline	4 – 8	Yields of many crops affected
Very Saline	8 – 16	Only tolerant crops yield satisfactorily
Highly Saline	>16	Only a few very tolerant crops yield satisfactorily

dS/m = deciSiemens/metre

To assess the distribution of salinity within the depths of impact of the proposed residential development, vertical soil salinity profiles (Figures 2a to 2c, following pages) were constructed from the test data detailed in Table 1 (Appendix B).

Four of these profiles (at Test Bores SKM10, SKM14, SKM16 and SKM25) show unusually uniform, non-saline conditions from surface to depths of 3 m. Three profiles (at Test Bores SKM5, SKM20 and SKM29) show "intermittent" type profiles with peak salinities at depths of 1.5 m to 2.5 m, in the very saline range. The remaining profiles show very mixed distributions but are generally of "normal" or "intermittent" types indicating normal water balance between infiltration and discharge (increasing salinity with depth) or some fluctuation in water balance with residual salinity maxima at depths of 1 m to 2.75 m, in the moderately saline range.

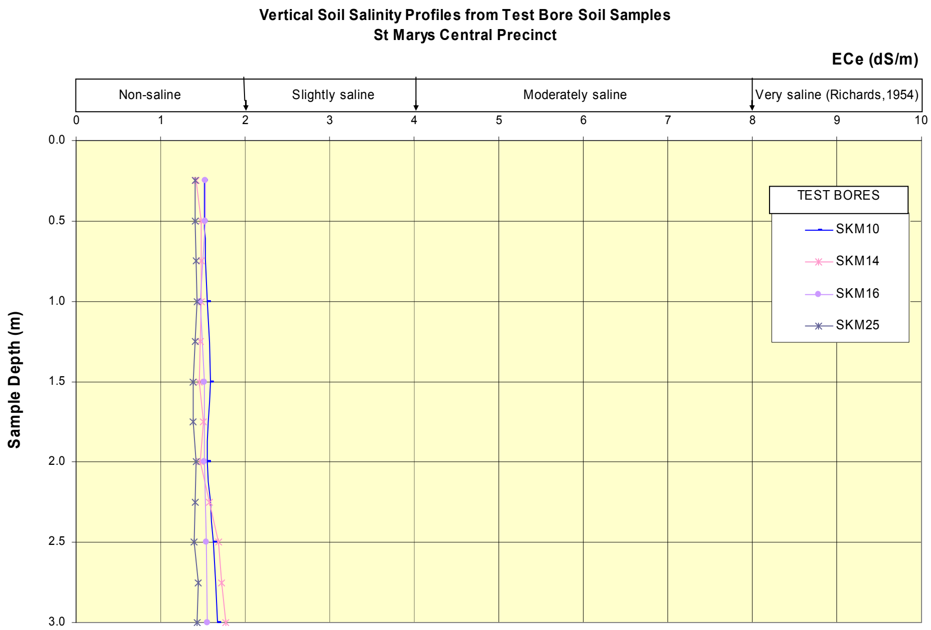


Figure 2a – "Uniform" Vertical Soil Salinity Profiles

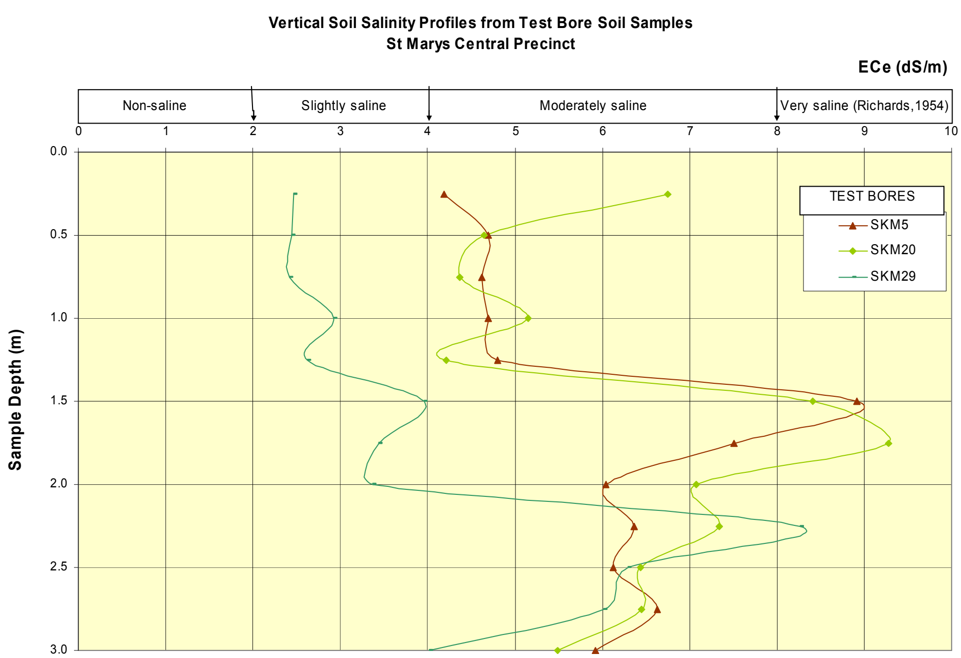


Figure 2b – "Intermittent" type Vertical Soil Salinity Profiles

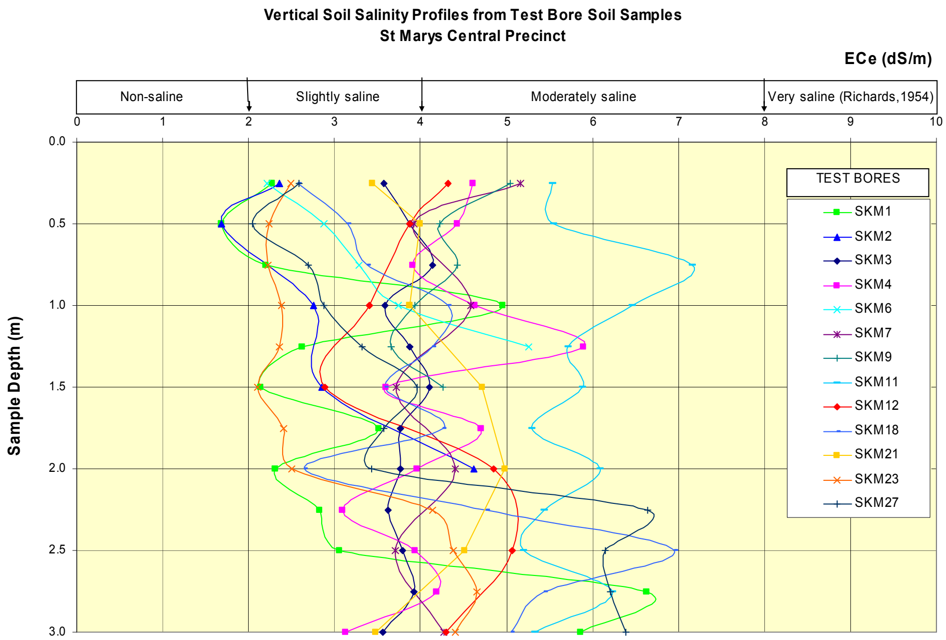


Figure 2c – "Mixed" Vertical Soil Salinity Profiles

Individual sample salinities are subject to lateral and vertical variability of soils and finite precision in determination of the textural classes used as $EC_{1:5}$ multipliers. This may lead to unrealistic salinity classifications of parts of the investigation area based on single (e.g. maximum) salinity results in those parts, particularly if the derived ECe value lies close to a class boundary. Classification of areas based on calculated "bulk salinities" are considered more practical. Bulk salinities are not derived by physically bulking or mixing together soil samples for single laboratory measurements but are "thickness-weighted averages" calculated from individual sample salinities ECe and the vertical extents (dZ) of those salinities (taken as midway between sample depths or at the upper or lower bounds of the bulking interval), using the formula:

Bulk ECe (over depth interval Z) = $\Sigma(ECe_i * dZ_i) / Z$, where $Z = \Sigma(dZ_i)$.

Bulk salinities above and below 0.8 m are used herein as the basis for the determination of salinity constraints throughout the site, since 0.8 m generally approximates the maximum depth of residential slabs and footings and bulk salinities can then represent soil conditions in the upper “foundation zone” and the lower “services zone”. Table 1 (Appendix B) lists all individual sample salinities and all calculated bulk salinities.

From the distribution of bulk salinities shown in Table 3 below, soils at the test bore locations within the “foundation zone” are predominantly slightly saline but are moderately saline in a significant percentage of locations. Although four individual samples (from depths of 1.5 m to 2.5 m at Test Bores SKM5, SKM20 and SKM29), were found to be very saline, the soils within the “services zone” at the test bore locations are predominantly moderately saline.

Table 3 – Distribution of Bulk Salinities at Test Bore Locations

Class	ECe (dS/m)	% of Locations	% of Locations
		Depths < 0.8 m	Depths > 0.8 m
Non Saline	<2	19	23
Slightly Saline	2 – 4	**46**	31
Moderately Saline	4 – 8	31	**46**
Very Saline	8 – 16	4	0
Highly Saline	>16	0	0

8. SALINITY ASSESSMENT INCORPORATING EM RESULTS

The DLWC salinity investigation guideline allows for a reduction in the density of test locations and the number of laboratory tests, when an EM investigation is carried out and the ECa results are correlated with the laboratory ECe results, enabling interpolation of data throughout the EM survey area at the high spatial density of that data.

To carry out the required correlations, the ECa values, obtained with PRP and HCP coil configurations at the closest points to the test bores, were plotted in scattergrams (Figures 3 and 4, following page) against bulk ECe values for the zones above and below depths of 0.8 m, respectively.

Reasonable linear trends between these parameters indicate that the EM system is responding primarily to soil salinity (not to other surface or subsurface conductors) and that the EM data obtained with the PRP and HCP configurations are reasonable measures of the salinity above and below 0.8 m, respectively.

Lines of best fit define these trends and provide scale factors of 3.10 and 3.73 by which to multiply apparent conductivities ECa (in dS/m), to estimate apparent salinities ECe (in dS/m) throughout the EM data set, above and below 0.8 m, respectively.

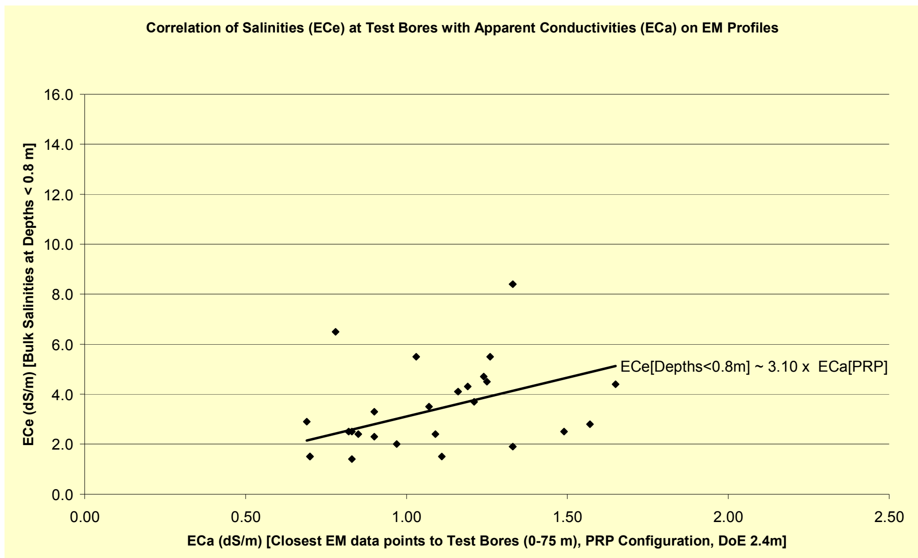


Figure 3 – Correlation of Bulk ECe (above 0.8m) and ECa (PRP) data

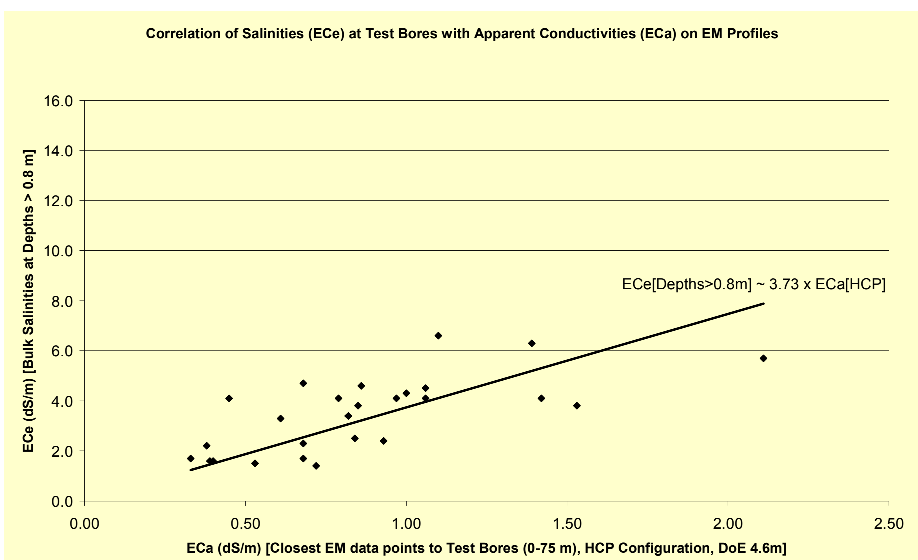


Figure 4 – Correlation of Bulk ECe (below 0.8m) and ECa (HCP) data

The scale factors were applied to all apparent conductivity grid data for presentation as apparent salinity images (Drawings 4 and 5, Appendix A) with continuous colour spectral scales in dS/m, based on the Richards classification scheme.

The 2-D surfaces (imaged in Drawings 4 and 5) were contoured at the 2 dS/m, 4 dS/m and 8 dS/m levels, corresponding to boundaries of the salinity classes of Richards, providing a direct subdivision of the study area into non-saline (<2 dS/m), slightly saline (2 - 4 dS/m), moderately saline (4 – 8 dS/m) and very saline (8 – 16 dS/m) classes.

Apparent salinities shown in Drawing 4 indicate non-saline to moderately saline conditions at depths less than 0.8 m, throughout the investigated site area. Small zones of moderately saline soil are inferred throughout the Precinct, but the largest and most saline zones are inferred in the southwest and southeast corners (around Test Bores SKM1 and SKM6) and in the south central area (150 m west and east of Test Bore SKM8).

Apparent salinities shown in Drawing 5 indicate non-saline to very saline conditions at depths greater than 0.8 m, throughout the investigated site area. A near-continuous zone of moderately saline soil is inferred from the southwestern corner through the central south to Test Bore SKM9, where a small very saline inlier is indicated. Significant zones of moderately to very saline soil are inferred in the north of the area (around Test Bores SKM22 and between Test Bores SKM27 and SKM29).

9. ASSESSMENT OF SOIL AGGRESSIVITY TO CONCRETE AND STEEL

Table 1 (Appendix A) presents the variations of pH with depth at the test bore locations, together with the corresponding concrete and steel aggressivity ranges indicated in Australian Standard AS2159:1995 (Piling – Design and Installation). AS2159 defines generally impermeable clay soils above the groundwater table to be in “Condition B” and permeable sands and all soils below the groundwater table to be in “Condition A”, leading to variations in the classifications of soil aggressivity. As indicated in Section 6.2 (above), 20% of sampled soils were found (from textural tests) to be either sandy loams or sands, and these samples have been classified as if in Condition A.

It should be noted that AS2159 was formulated to improve the longevity of deep piles where access (for inspection and remediation of salt damage) was expected to be minimal. This standard was not formulated for the protection of concrete and steel in slabs and shallow foundations or infrastructure and recommendations for concrete strength, based on AS2159 aggressivity classifications, represents a conservative approach to protection of these structures.

The pH measurements at test bore locations indicate that all tested soils are non-aggressive to steel. Tested soils are also generally non-aggressive to concrete, with only 3 samples mildly aggressive, at depths of 1.5 m to 2.5 m in Test Bores SKM2, SKM20 and SKM23.

10. CONSTRAINTS TO DEVELOPMENT

10.1 Salinity Constraints

Two primary data sources were employed for assessment of soil salinity:

- ECe estimates derived from 251 laboratory tests of soil samples from 26 test bores; and
- ECa (apparent conductivity) data obtained at 28,000 measurement stations.

These sources of data were correlated and combined in a joint interpretation, providing a practical means of assessing salinity and defining areas where there is a risk that urban development will be affected by soil salinity, or will adversely affect the salinity of the environment.

To better assess the constraints that saline soils may place on the proposed development, two data sets were employed to construct salinity constraints areas for two depth intervals (Drawings 6 and 7, Appendix A).

These data sets were:

- locations of test pits where calculated bulk salinities over the relevant depth interval, exceeded 4 dS/m (i.e. specific locations of moderately or more saline soil); and
- regions formed by the 4 dS/m and 8 dS/m apparent salinity contours, derived by correlation of apparent conductivities (ECa) from EM profiling, with the bulk salinities over the relevant depth interval.

For a conservative approach, salinity constraint areas were defined which encompassed and sometimes combined these mapped locations and regions.

Drawing 6 (Appendix A) shows multiple constraint areas due to inferred moderately saline soils at depths less than 0.8 m. These areas comprise approximately 20 ha in total, distributed throughout the site, with the largest individual area occupying 6 ha in the southwestern corner. An individual bulk salinity value in the very saline range, at Test Bore SKM11, was not supported by EM data and this location has been included in the moderately saline constraint region.

Drawing 7 shows multiple constraint areas due to inferred moderately saline soils at depths greater than 0.8 m. These areas comprise approximately 37 ha in total, with the largest individual area of 26 ha in the southern half of the site. Three small constraint areas (approximately 1 ha in total) are shown, where very saline soil is inferred at depths greater than 0.8 m.

Within the constraint areas described above, soils should be treated as moderately saline or very saline as indicated and these areas should be subject to appropriate levels of salinity management during development.

10.2 Aggressivity Constraints

As indicated in Section 9 (above), soils were assessed as non-aggressive to steel and generally non-aggressive to concrete, with only 3 samples mildly aggressive. To the extent that the 26 test bores are representative of the soils throughout the Central Precinct, aggressivity is not considered to impose any constraints on development.

11. PRELIMINARY SALINITY MANAGEMENT PLAN

Preliminary management strategies are recommended below, for implementation within the constraint areas having perceived risks due to moderately or more saline soils. Areas outside of these constraint areas are considered to have a diminished salinity risk, however since soil and groundwater conditions can change with time, some general management strategies are also listed for the areas of non-saline to slightly saline soils.

These strategies are aimed primarily at:

- Maintaining the natural water balance;
- Maintaining good drainage;
- Avoiding disturbance or exposure of sensitive soils;
- Retaining or increasing appropriate native vegetation in strategic areas; and
- Implementing building controls and engineering responses where appropriate.

11.1 Non-Saline and Slightly Saline Areas

Efforts should be made throughout the proposed development area to prevent or restrict changes to the water balance that will result in rises in groundwater levels, bringing more saline water closer to the ground surface. As a precaution, development must be planned to mitigate against the effects of any potential salinisation that could occur, even in the areas outside the inferred moderate salinity constraint zones of Drawings 6 and 7. In these non-saline and slightly saline areas, the soils and topography still render the site saline prone and such areas if poorly managed may, over time, become saline. As a result the following management strategies are recommended for all areas of the development:

- Avoid water collecting in low lying areas, along shallow creeks, floodways, in ponds, depressions, or behind fill embankments or near trenches on the uphill sides of roads. This can lead to water logging of the soils, evaporative concentration of salts, and eventual breakdown in soil structure resulting in accelerated erosion.

- Where stormwater retention ponds are required, these should not be created directly downslope of areas with a moderate level of salinity.
- Roads and the shoulder areas should be designed to be well drained, particularly with regard to drainage of surface water. There should not be excessive concentrations of runoff or ponding that would lead to waterlogging of the pavement or additional recharge to the groundwater. Road shoulders should be included in the sealing program should rural construction methods be used.
- Surface drains should generally be provided along the top of all batters to reduce the potential for concentrated flows of water down slopes possibly causing scour. Well-graded subsoil drainage should be provided at the base of all slopes where there are road pavements below the slope to reduce the risk of waterlogging.
- As an alternative to slab-on-ground construction, suspended slab or pier and beam construction should be considered, particularly on sloping sites as this will minimise exposure to saline or aggressive soils and reduce the potential cut and fill on site which could alter subsurface flows.
- It is essentially that in all masonry buildings a brick damp course be properly installed so that it cannot be bridged either internally or externally. This will prevent moisture moving into brickwork and up the wall.
- Consideration could be given to the use of to slotted drainage pipes to promote subsurface drainage in service trenches, with such pipes fitting into the stormwater pits in lower areas where pipe invert levels are within about 1 m of existing water levels in adjacent creek lines.
- Service connections and stormwater runoffs should be checked to avoid leaking pipes which may affect off site areas further down slope and increase groundwater recharge resulting in increases in groundwater levels.
- Landscaping and garden designs must not be placed against walls, as such placement may nullify the benefits of the damp course.

11.2 Moderately Saline and Very Saline Areas

In addition to the precautions listed above, the following recommendations are made for areas falling within the moderately saline and very saline constraint zones of Drawings 6 and 7 (Appendix A).

- It is preferable that stormwater retention ponds, if required, are created outside areas with a moderate level of salinity. In the event that such ponds are located within the areas of moderate salinity, consideration of the saline conditions should be taken into account by the designers. The most appropriate mitigation measures should be assessed on a site by site basis once the design of the basins has been completed and may include:
    - conditioning of the soil to be utilised within the embankment of the ponds, with gypsum, to minimise the risk of structural degradation/erosion
    - careful control of compaction and moisture control during earthworks to ensure creation of a low permeability embankment to retard migration of saline water into the pondage
    - lining of the stormwater ponds with an appropriate liner (such as HPDE) where the results of further analysis preclude other practical measures
    - development of a water quality monitoring plan and appropriate treatment, such as adjustment of pH levels prior to discharge to the surrounding environment.
- With regard to regrading within the development footprint, a minimum surface slope of 1V:40H (where achievable) is suggested in order to improve surface drainage and reduce ponding and waterlogging, which can lead to evaporation and salinisation.
- Where possible, materials and waters used in the construction of roads and fill embankments should be sourced from outside the shallow salinity constraint zones shown on Drawing 6, and/or from depths of less than 0.8 m within the footprints of the deeper salinity constraint zones of Drawing 7, or should be imported from outside the development area where the material has been classified in situ or in stockpiles as non saline to slightly saline.
- In areas of cut and fill within the shallow salinity constraint zones of Drawing 6 or where cutting impacts on the deep salinity constraint zones of Drawing 7, salinisation could be a

problem and a capping layer of either topsoil or sandy materials should be placed over the locally derived filling to reduce capillary rise, act as a drainage layer and also reduce the potential for dispersive behaviour in any sodic soils.

- Where concrete slabs are constructed within the moderately saline or very saline constraint zones, at depths after earthworks which impact on the moderately saline or very saline soils, use of a bedding layer of sand (100 mm thick), overlain by a membrane of thick plastic (damp proof as opposed to vapour proof) is recommended under concrete slabs to act as a moisture barrier and drainage layer and to restrict capillary rise under the slab. The sand will help protect the membrane from rupture and the Building Code of Australia (1990) does not require compaction of the recommended thickness of 100 mm. As an alternative method for protection of concrete slabs for non-residential construction (where membranes may not be a requirement of the Building Code), high strength (32 MPa) concrete may be placed directly on a layer of crushed rock. Such rock should be sourced locally from an area classified as non-saline or slightly saline or should be imported after stockpiling, testing and classification as non-saline or slightly saline.
- To the extent that the 26 test bores are representative of the soils throughout the Central Precinct, aggressivity is not considered to impose any constraints on development, hence no recommendation is made herein for the use of higher strength (32 MPa or higher) concrete in residential slabs and footings, based on the guidelines of AS2159. Furthermore, within the “foundation zone” below the present ground surface, concrete of greater strength than 25 MPa is not considered necessary within the guidelines of AS2870 (Residential slabs and footings), currently under revision. However, 32 MPa concrete is recommended by AS2870 within areas of very saline soil, and such strengths are recommended herein for any mass concrete required within the very saline constraint areas inferred within the “services zone” of the Central Precinct (Drawing 7).
- Salt tolerant grasses and trees should be considered if re-planting close to creeks and in areas of moderate and greater salinity to reduce soil erosion and maintain the existing evapotranspiration and groundwater levels. Reference should be made to an experienced landscape planner or agronomist.
- Other measures that can be considered to improve the durability of concrete in saline environments include reducing the water to cement ratio (hence increasing strength),

minimising cracks and joints in plumbing on or near the concrete, reducing turbulence of any water flowing over the concrete.

- There are various exposure classifications and durability ratings for the wide range of masonry available. Reference should be made to the supplier in choosing suitable bricks of at least exposure quality. Water proofing agents can also be added to mortar to further restrict potential water movement.
- Exposure class masonry must be used below damp proof courses.
- Appropriate subsoil drainage must be used for all slabs, footings, retaining walls and driveways.

12. ADDITIONAL RECOMMENDATIONS

Additional investigation should be undertaken in development areas which are to be excavated deeper than 3 m or into rock at shallower depth, where direct sampling and testing of salinity has not been carried out. Salinity management strategies herein may need to be modified or extended following additional investigations by deep test pitting and/or drilling, sampling and testing for soil and water pH, electrical conductivity, TDS, sodicity, sulphates and chlorides.

DOUGLAS PARTNERS PTY LTD

Reviewed by

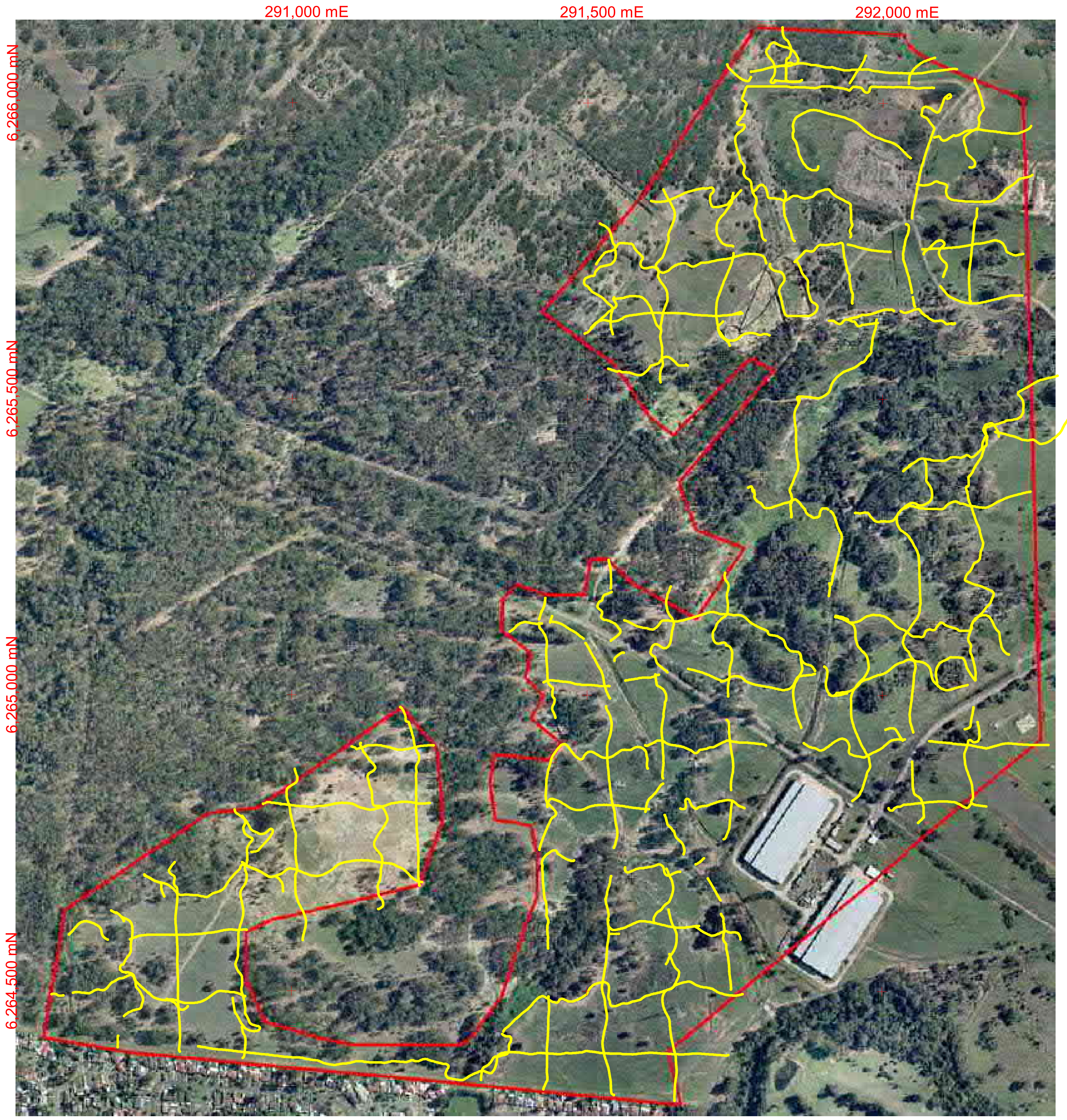
J Lean
Principal

T J Wiesner
Principal

References:

1. Geological Survey of New South Wales, 1991. *Geology of 1:100 000 Penrith Geological Series Sheet 9030 (Edition 1).*
2. Salinity Potential in Western Sydney 2002, Department of Infrastructure, Planning and Natural Resources, 2003.
3. Spies, B. and Woodgate, P. 2004. Salinity Mapping Methods in the Australian Context. Technical Report. Natural Resource Management Ministerial Council, January 2004.
4. NSW Department of Land and Water Conservation, 2002. Site Investigations for Urban Salinity
5. Richards, L. A. (ed.) 1954. Diagnosis and Improvement of Saline and Alkaline Soils. USDA Handbook No. 60, Washington D.C.
6. Hazelton, P. A. and Murphy B. W. 1992. A Guide to the Interpretation of Soil Test Results. Department of Conservation and Land Management.

APPENDIX A
Drawings 1 to 7



LEGEND

+ Grid: GDA94 / MGA94 (Zone 56)

Points of measurement of Apparent Conductivity with a DualEM-4 system, forming profiles on a grid with approximate dimensions 100m x 100m

Boundary of Central Precinct

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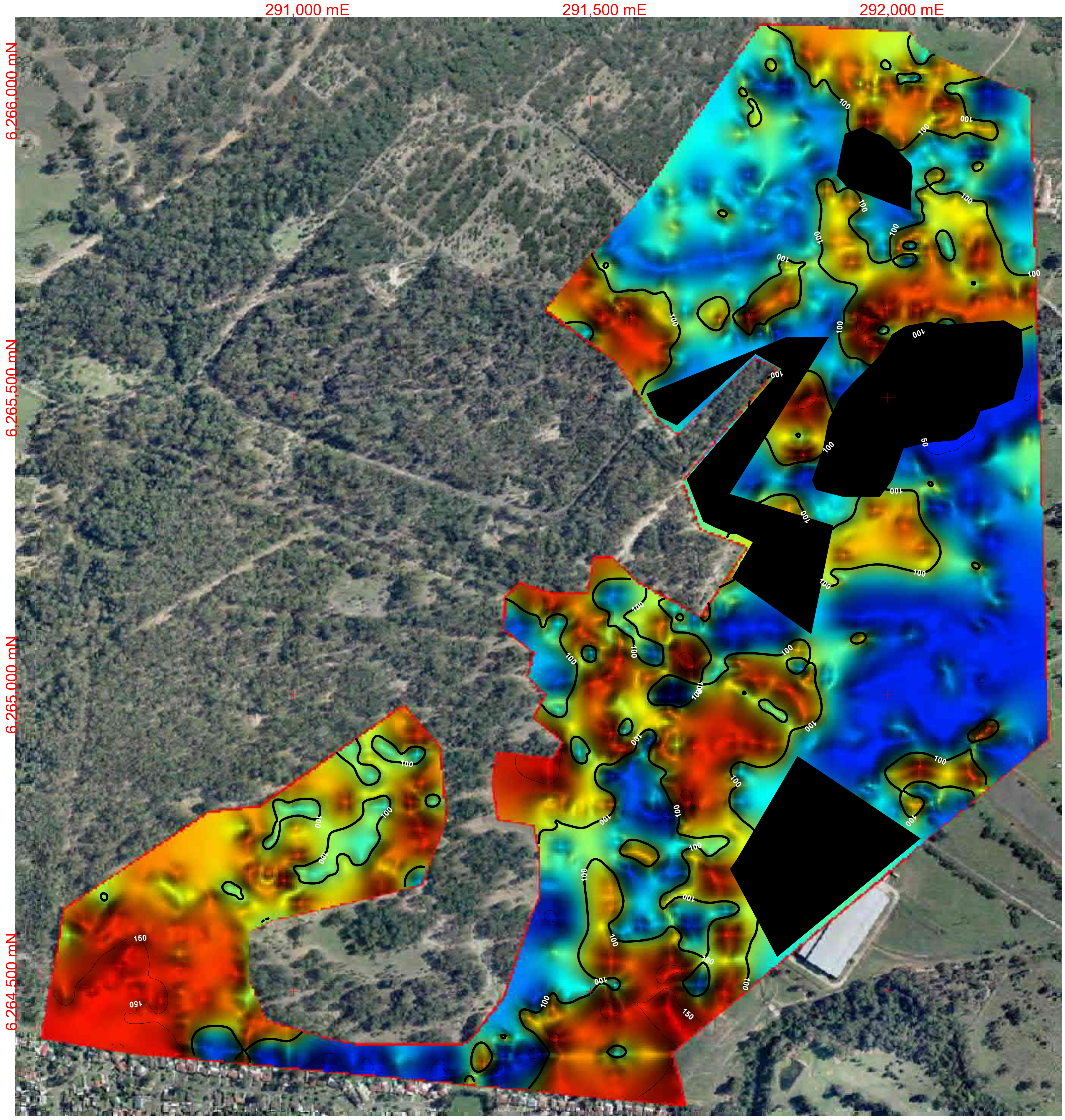
TITLE:

LOCATIONS OF ELECTROMAGNETIC (EM) PROFILES

SALINITY INVESTIGATION
CENTRAL PRECINCT
ST MARYS, NSW

CLIENT: Sinclair Knight Merz			
DRAWN BY: JL	SCALE: 1:7500 @ A3	PROJECT No: 45529	OFFICE: SYDNEY
APPROVED BY:		DATE: 18 JUNE 2008	DRAWING No: 1



Note: Apparent Conductivities were measured by EM profiling with a DualEM-4 system in PRP coil configuration, with a theoretical Depth of Exploration (DoE) of 2.4m.

Apparent Conductivities

mS/m

200
138
127
119
113
109
105
101
96.7
93.5
89.8
86.2
81.8
76.6
69.8
60.6
0

LEGEND

- Grid: GDA94 / MGA94 (Zone 56)
- Region inaccessible to EM profiling
- 100 mS/m contour on Apparent Conductivity grid
- 50/150 mS/m contours on Apparent Conductivity grid

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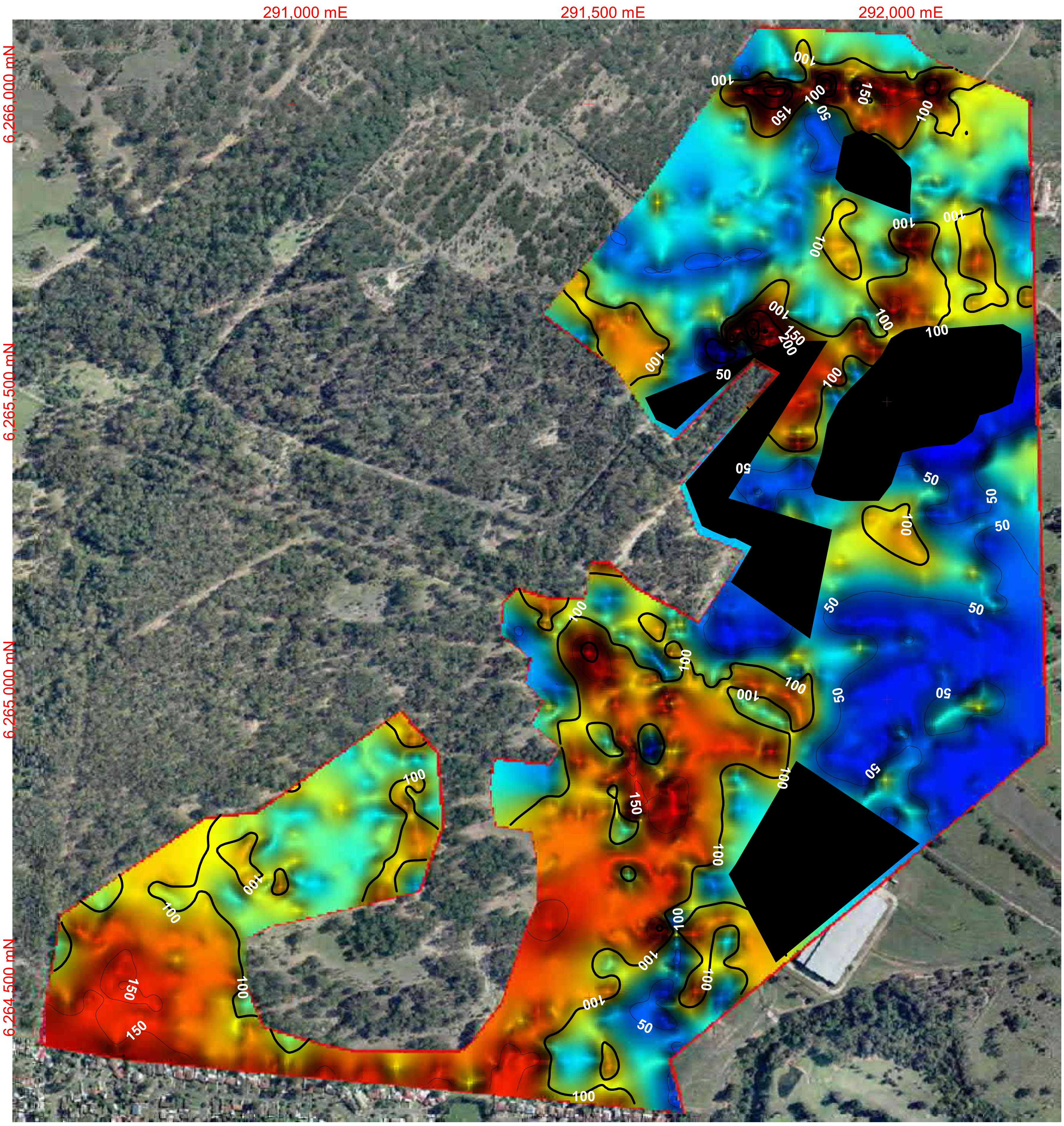
TITLE:

APPARENT CONDUCTIVITIES FROM EM PROFILING WITH A DUALEM-4 SYSTEM IN PRP COIL CONFIGURATION

SALINITY INVESTIGATION
CENTRAL PRECINCT
ST MARYS, NSW

CLIENT: Sinclair Knight Merz			
DRAWN BY: JL	SCALE: 1:7500 @ A3	PROJECT No: 45529	OFFICE: SYDNEY
APPROVED BY:		DATE: 18 JUNE 2008	DRAWING No: 2



Note: Apparent Conductivities were measured by EM profiling with a DualEM-4 system in HCP coil configuration, with a theoretical Depth of Exploratopn (DoE) of 4.6m.

LEGEND

- Grid: GDA94 / MGA94 (Zone 56)
- Region inaccessible for EM profiling
- 100 mS/m contour on Apparent Conductivity grid
- 50/150 mS/m contours on Apparent Conductivity grid

Apparent Conductivities

mS/m

200
141
128
118
109
102
96.6
90.9
85.2
80.5
76.3
70.5
64
56.2
45.9
31.6
0

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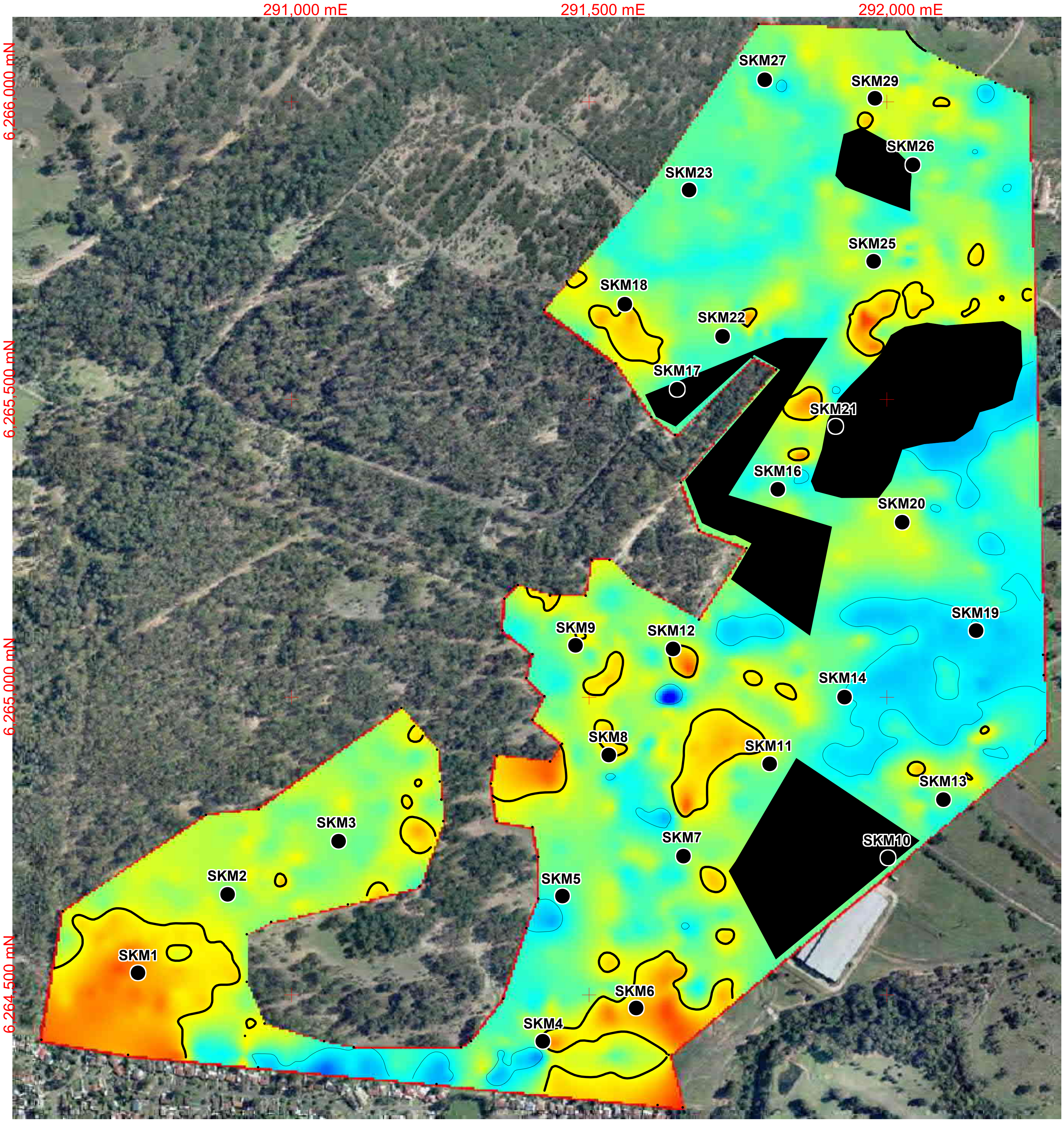
TITLE:

APPARENT CONDUCTIVITIES FROM EM PROFILING WITH A DUALEM-4 SYSTEM IN HCP COIL CONFIGURATION

SALINITY INVESTIGATION
CENTRAL PRECINCT
ST MARYS, NSW

CLIENT: Sinclair Knight Merz			
DRAWN BY: JL	SCALE: 1:7500 @ A3	PROJECT No: 45529	OFFICE: SYDNEY
APPROVED BY:		DATE: 18 JUNE 2008	DRAWING No: 3



Note: Apparent Salinities were derived from EM profiling with a DualEM-4 system in PRP coil configuration (DoE 2.4m), correlated with Bulk ECe values (for depths<0.8m) from laboratory tests on soil samples.

Apparent Salinities at Depths < 0.8 m (dS/m)

6		Moderately saline
4.5	4	
3		Slightly saline
1.5	2	
0		Non-saline

LEGEND

- Grid: GDA94 / MGA94 (Zone 56)
- SKM Soil Bore
- Region inaccessible for EM profiling
- 4 dS/m contour on Apparent Salinity grid
- 2 dS/m contour on Apparent Salinity grid

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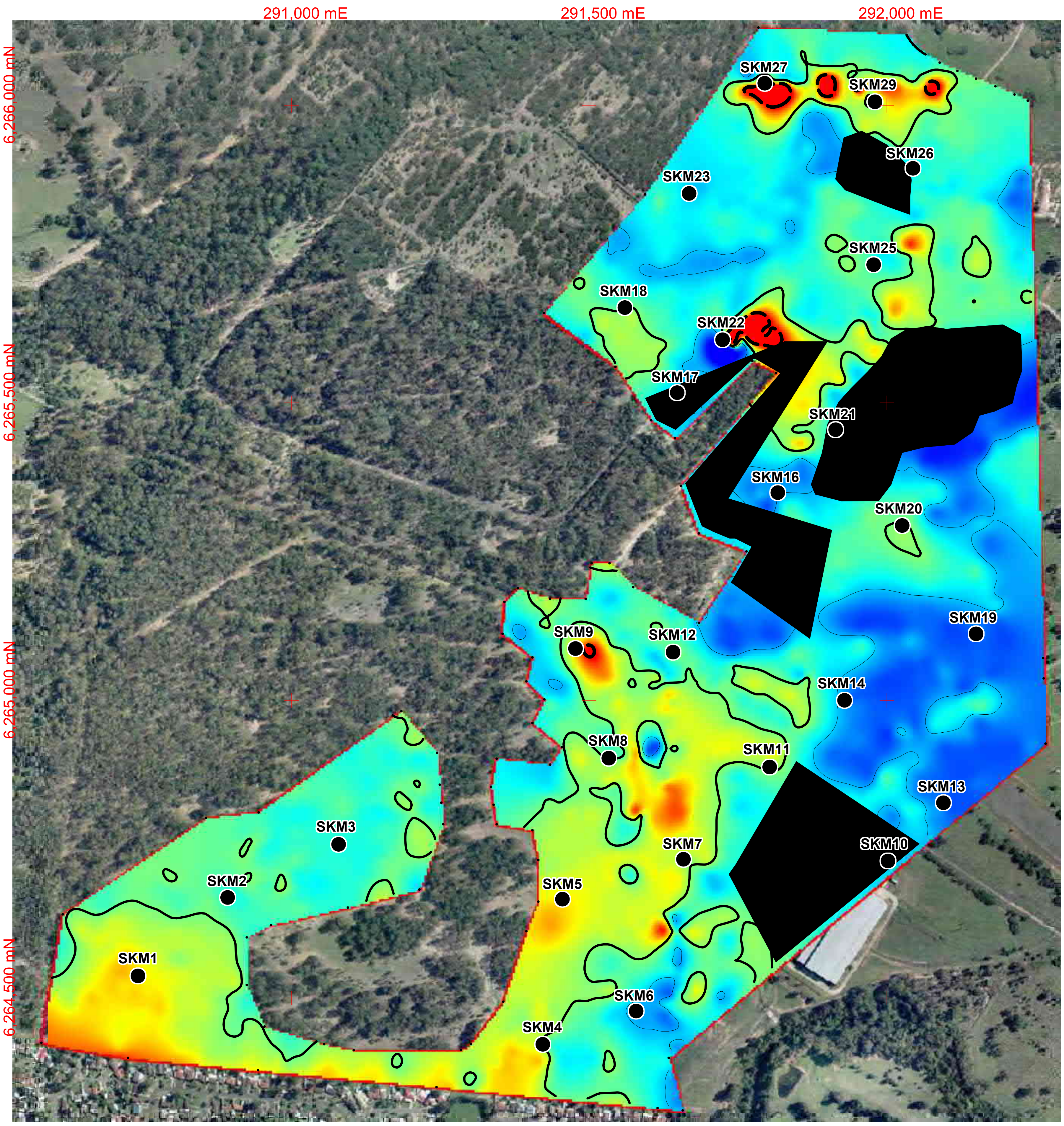
TITLE:
APPARENT SALINITIES AT DEPTHS < 0.8 m

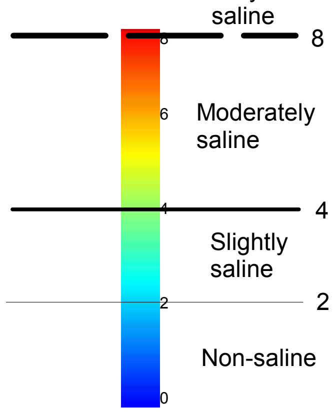
SALINITY INVESTIGATION
CENTRAL PRECINCT
ST MARYS, NSW

CLIENT: Sinclair Knight Merz			
DRAWN BY: JL	SCALE: 1:7500 @ A3	PROJECT No: 45529	OFFICE: SYDNEY
APPROVED BY:		DATE: 18 JUNE 2008	DRAWING No: 4



Note: Apparent Salinities were derived from EM profiling with a DualEM-4 system in HCP coil configuration (DoE 4.6m), correlated with Bulk ECe values (for depths>0.8m) from laboratory tests on soil samples.

Apparent Salinities at Depths > 0.8 m (dS/m)



LEGEND

- Grid: GDA94 / MGA94 (Zone 56)
- SKM Soil Bore
- Region inaccessible for EM profiling
- 2 dS/m contour on Apparent Salinity grid
- 4 dS/m contour on Apparent Salinity grid
- 8 dS/m contour on Apparent Salinity grid

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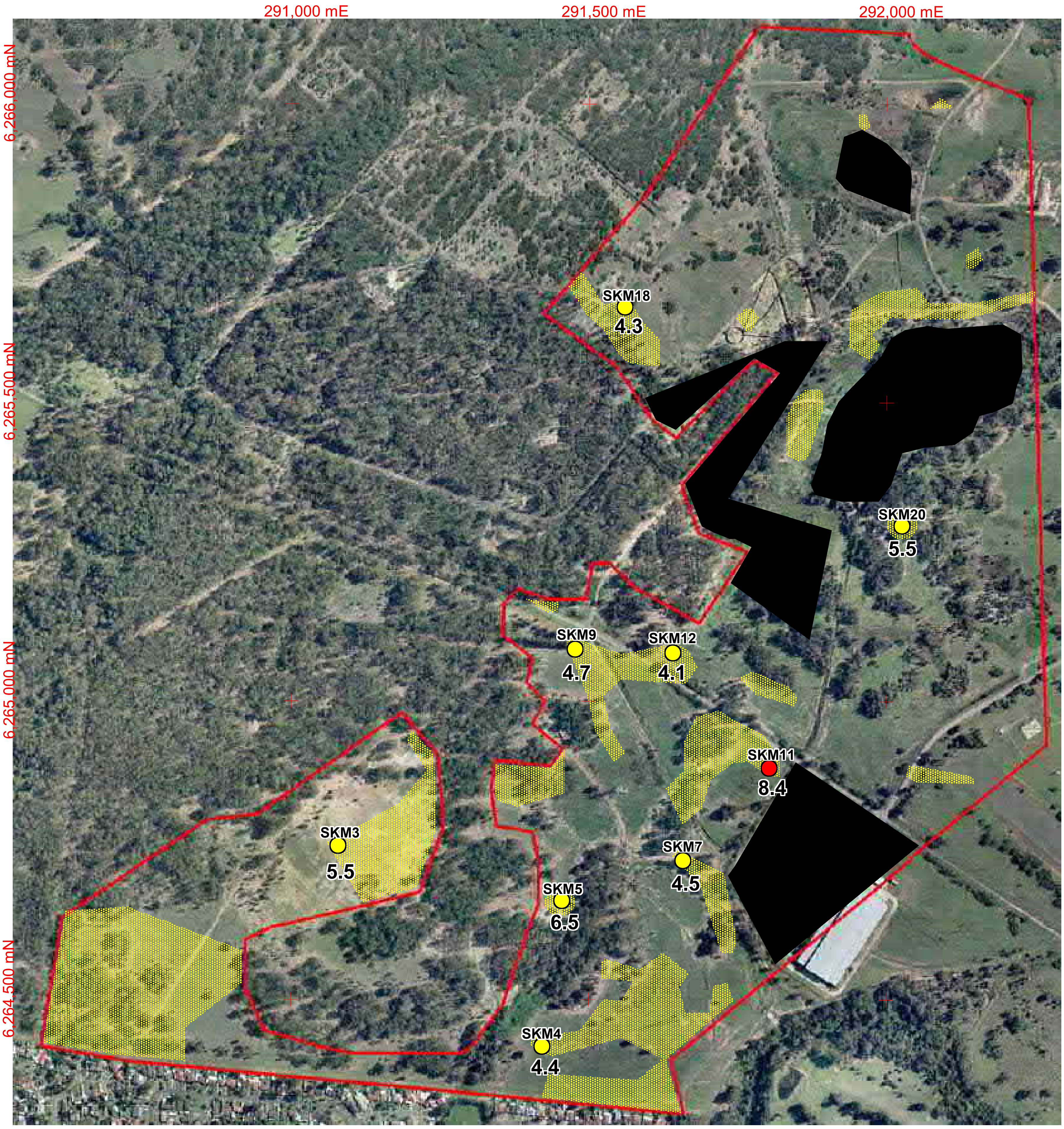
TITLE:

APPARENT SALINITIES AT DEPTHS > 0.8 m

SALINITY INVESTIGATION
CENTRAL PRECINCT
ST MARYS, NSW

CLIENT: Sinclair Knight Merz			
DRAWN BY: JL	SCALE: 1:7500 @ A3	PROJECT No: 45529	OFFICE: SYDNEY
APPROVED BY:		DATE: 18 JUNE 2008	DRAWING No: 5



LEGEND

- Grid: GDA94 / MGA94 (Zone 56)
- Region inaccessible for EM profiling
- SKM Test Bore showing bulk salinity in dS/m (moderately saline) at depths < 0.8m
- SKM Test Bore showing bulk salinity in dS/m (very saline) at depths < 0.8m
- Area of development constraint due to inferred moderate salinity at depths < 0.8 m

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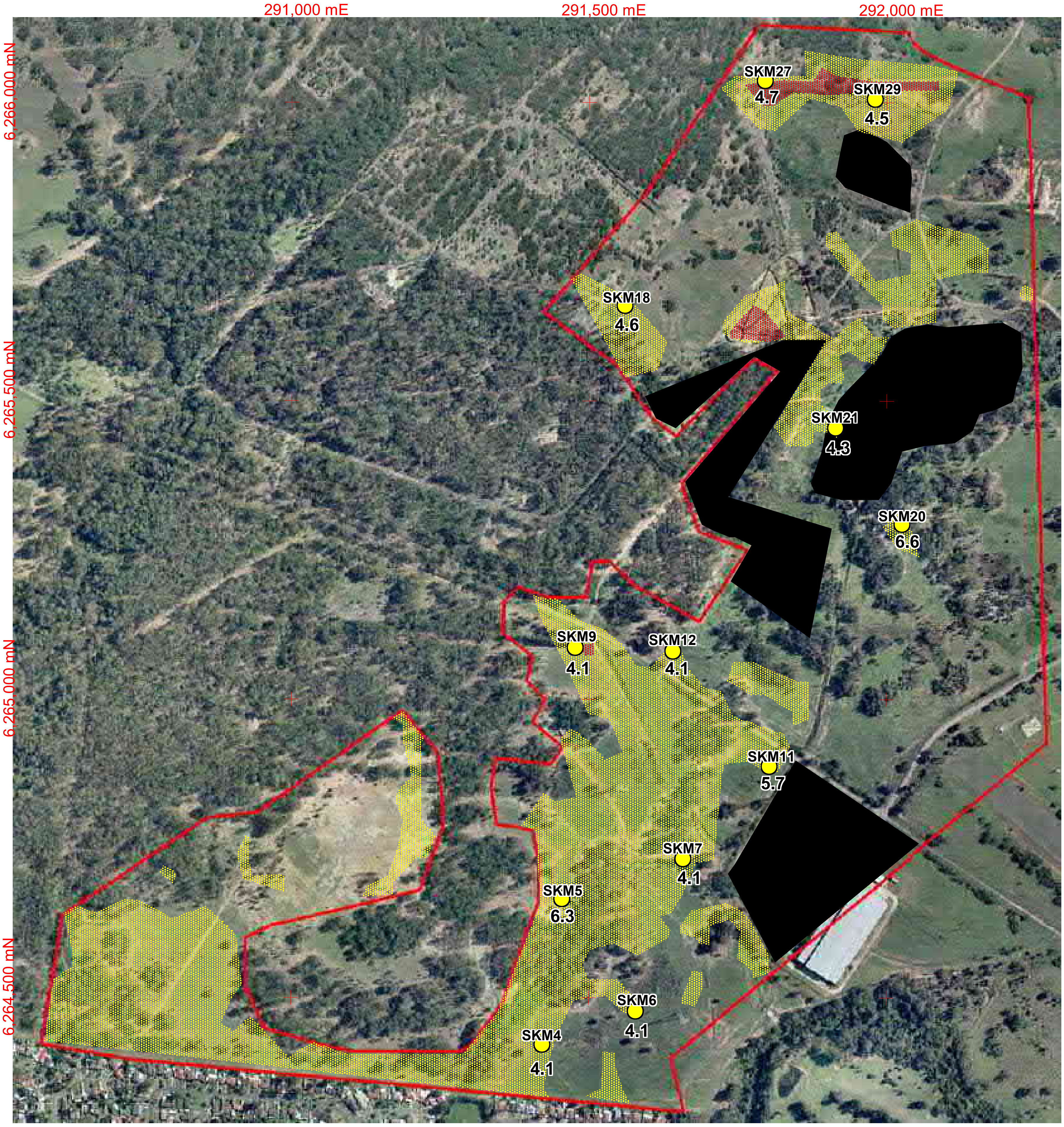
TITLE:

SALINITY CONSTRAINTS AT DEPTHS < 0.8 m

SALINITY INVESTIGATION
CENTRAL PRECINCT
ST MARYS, NSW

CLIENT: Sinclair Knight Merz			
DRAWN BY: JL	SCALE: 1:7500 @ A3	PROJECT No: 45529	OFFICE: SYDNEY
APPROVED BY:		DATE: 18 JUNE 2008	DRAWING No: 6



LEGEND

Grid: GDA94 / MGA94 (Zone 56)

Region inaccessible for EM profiling

SKM Test Bore showing bulk salinity in dS/m (moderately saline) at depths > 0.8 m

Area of development constraint due to inferred moderately saline soil at depths > 0.8 m

Area of development constraint due to infered very saline soil at depths > 0.8 m

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TITLE:

SALINITY CONSTRAINTS AT DEPTHS > 0.8 m

SALINITY INVESTIGATION
CENTRAL PRECINCT
ST MARYS, NSW

CLIENT: Sinclair Knight Merz			
DRAWN BY: JL	SCALE: 1:7500 @ A3	PROJECT No: 45529	OFFICE: SYDNEY
APPROVED BY:		DATE: 18 JUNE 2008	DRAWING No: 7

APPENDIX B
Table 1 – Salinity-Related Test Bore Data, Lab Tests and Assessments

TABLE 1: SALINITY-RELATED TEST BORE DATA, LAB TESTS AND ASSESSMENTS, PROJECT 45529, CENTRAL PRECINCT, ST MARYS

Test Bore	Coordinates East (m MGA94)	North	RL (m AHD)	Sample Depth (m)	pH	Soil Condition [AS2159]	Soil Aggressivity To Concrete [AS2159 pH criteria]	To Steel	Soil Texture Group [after DLWC]	Textural Factor [M] [after DLWC]	$EC_{1:5}$ [Lab.] (µS/cm)	EC_e [M x $EC_{1:5}$] (dS/m)	Salinity Class [Richards 1954]	ECe Bulk [depths<0.8m/depths>0.8m] (dS/m)	Salinity Class [Richards 1954]
SKM1	290742	6264539		0.25	6.9	B	Non-Aggressive	Non-Aggressive	Clay loam	9	253	2.3	Slightly Saline	2.8	Slightly Saline
				0.50	6.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	187	1.7	Non Saline		
				0.75	7.0	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	259	2.2	Slightly Saline		
				1.00	7.0	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	583	5.0	Moderately Saline	3.8	Slightly Saline
				1.25	6.4	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	308	2.6	Slightly Saline		
				1.50	6.8	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	252	2.1	Slightly Saline		
				1.75	6.9	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	413	3.5	Slightly Saline		
				2.00	7.2	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	272	2.3	Slightly Saline		
				2.25	6.4	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	332	2.8	Slightly Saline		
				2.50	6.1	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	360	3.1	Slightly Saline		
				2.75	6.5	A	Non-Aggressive	Non-Aggressive	Sand	17	390	6.6	Moderately Saline		
				3.00	6.6	A	Non-Aggressive	Non-Aggressive	Sand	17	345	5.9	Moderately Saline		
SKM2	290893	6264671		0.25	6.7	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	168	2.4	Slightly Saline	2.0	Slightly Saline
				0.50	6.7	B	Non-Aggressive	Non-Aggressive	Clay loam	9	187	1.7	Non Saline		
				1.00	5.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	306	2.8	Slightly Saline	3.4	Slightly Saline
				1.50	4.8	B	Mild	Non-Aggressive	Clay loam	9	317	2.9	Slightly Saline		
				2.00	7.5	A	Non-Aggressive	Non-Aggressive	Sand	17	272	4.6	Moderately Saline		
SKM3	291079	6264760		0.25	8.7	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	255	3.6	Slightly Saline	5.5	Moderately Saline
				0.50	8.9	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	277	3.9	Slightly Saline		
				0.75	8.8	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	296	4.1	Moderately Saline		
				1.00	8.9	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	256	3.6	Slightly Saline	3.8	Slightly Saline
				1.25	8.6	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	277	3.9	Slightly Saline		
				1.50	8.5	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	293	4.1	Moderately Saline		
				1.75	8.7	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	269	3.8	Slightly Saline		
				2.00	8.8	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	269	3.8	Slightly Saline		
				2.25	8.7	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	259	3.6	Slightly Saline		
				2.50	8.7	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	271	3.8	Slightly Saline		
				2.75	8.8	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	280	3.9	Slightly Saline		
				3.00	8.5	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	254	3.6	Slightly Saline		
SKM4	291422	6264424		0.25	7.6	B	Non-Aggressive	Non-Aggressive	Clay loam	9	512	4.6	Moderately Saline	4.4	Moderately Saline
				0.50	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	492	4.4	Moderately Saline		
				0.75	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	434	3.9	Slightly Saline		
				1.00	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	515	4.6	Moderately Saline	4.1	Moderately Saline
				1.25	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	655	5.9	Moderately Saline		
				1.50	7.0	B	Non-Aggressive	Non-Aggressive	Clay loam	9	400	3.6	Slightly Saline		
				1.75	7.6	B	Non-Aggressive	Non-Aggressive	Clay loam	9	523	4.7	Moderately Saline		
				2.00	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	440	4.0	Slightly Saline		
				2.25	7.6	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	364	3.1	Slightly Saline		
				2.50	7.6	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	463	3.9	Slightly Saline		
				2.75	7.7	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	493	4.2	Moderately Saline		
				3.00	8.4	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	368	3.1	Slightly Saline		
SKM5	291455	6264668		0.25	7.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	465	4.2	Moderately Saline	6.5	Moderately Saline
				0.50	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	522	4.7	Moderately Saline		
				0.75	7.1	B	Non-Aggressive	Non-Aggressive	Clay loam	9	513	4.6	Moderately Saline		
				1.00	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	521	4.7	Moderately Saline	6.3	Moderately Saline
				1.25	7.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	533	4.8	Moderately Saline		
				1.50	7.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	991	8.9	Very Saline		
				1.75	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	835	7.5	Moderately Saline		
				2.00	7.7	B	Non-Aggressive	Non-Aggressive	Clay loam	9	671	6.0	Moderately Saline		
				2.25	7.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	707	6.4	Moderately Saline		
				2.50	7.3	B	Non-Aggressive	Non-Aggressive	Loam	10	613	6.1	Moderately Saline		
				2.75	8.4	B	Non-Aggressive	Non-Aggressive	Loam	10	663	6.6	Moderately Saline		
				3.00	7.2	B	Non-Aggressive	Non-Aggressive	Loam	10	592	5.9	Moderately Saline		
SKM6	291579	6264480		0.25	8.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	246	2.2	Slightly Saline	2.5	Slightly Saline
				0.50	9.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	320	2.9	Slightly Saline		
				0.75	9.4	B	Non-Aggressive	Non-Aggressive	Loam	10	329	3.3	Slightly Saline	4.1	Moderately Saline
				1.00	9.4	B	Non-Aggressive	Non-Aggressive	Loam	10	374	3.7	Slightly Saline		
				1.25	7.1	B	Non-Aggressive	Non-Aggressive	Loam	10	526	5.3	Moderately Saline		
SKM7	291658	6264735		0.25	7.9	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	607	5.2	Moderately Saline	4.5	Moderately Saline
				0.50	7.8	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	462	3.9	Slightly Saline		
				1.00	8.2	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	540	4.6	Moderately Saline	4.1	Moderately Saline
				1.50	7.9	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	438	3.7	Slightly Saline		
				2.00	8.0	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	518	4.4	Moderately Saline		
				2.50	8.1	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	436	3.7	Slightly Saline		
				3.00	7.7	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	502	4.3	Moderately Saline		

Test Bore	Coordinates East (m MGA94)	North	RL (m AHD)	Sample Depth (m)	pH	Soil Condition [AS2159]	Soil Aggressivity To Concrete [AS2159 pH criteria]	To Steel	Soil Texture Group [after DLWC]	Textural Factor [M] [after DLWC]	$EC_{1:5}$ [Lab.] (µS/cm)	EC_e [M x $EC_{1:5}$] (dS/m)	Salinity Class [Richards 1954]	ECe Bulk [depths<0.8m/depths>0.8m] (dS/m)	Salinity Class [Richards 1954]
SKM8	291533	6264905		0.25	7.3	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	215	1.8	Non Saline	1.9	Non Saline
				0.50	7.6	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	232	2.0	Non Saline		
				0.75	7.3	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	228	1.9	Non Saline		
				1.00	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	249	2.2	Slightly Saline	2.4	Slightly Saline
				1.25	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	234	2.1	Slightly Saline		
				1.50	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	267	2.4	Slightly Saline		
				2.00	7.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	275	2.5	Slightly Saline		
				2.25	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	254	2.3	Slightly Saline		
				2.50	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	234	2.1	Slightly Saline		
				2.75	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	246	2.2	Slightly Saline		
				3.00	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	333	3.0	Slightly Saline		
SKM9	291477	6265089		0.25	9.1	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	360	5.0	Moderately Saline	4.7	Moderately Saline
				0.50	8.6	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	302	4.2	Moderately Saline		
				0.75	8.7	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	316	4.4	Moderately Saline	4.1	Moderately Saline
				1.00	8.4	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	281	3.9	Slightly Saline		
				1.25	8.7	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	261	3.7	Slightly Saline		
				1.50	8.9	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	304	4.3	Moderately Saline		
SKM10	292002	6264732		0.25	8.1	B	Non-Aggressive	Non-Aggressive	Clay loam	9	169	1.5	Non Saline	1.5	Non Saline
				0.50	7.6	B	Non-Aggressive	Non-Aggressive	Clay loam	9	169	1.5	Non Saline		
				1.00	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	173	1.6	Non Saline	1.6	Non Saline
				1.50	7.7	B	Non-Aggressive	Non-Aggressive	Clay loam	9	177	1.6	Non Saline		
				2.00	8.0	B	Non-Aggressive	Non-Aggressive	Clay loam	9	173	1.6	Non Saline		
				2.50	7.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	181	1.6	Non Saline		
				3.00	7.9	B	Non-Aggressive	Non-Aggressive	Clay loam	9	186	1.7	Non Saline		
SKM11	291803	6264890		0.25	7.1	B	Non-Aggressive	Non-Aggressive	Clay loam	9	615	5.5	Moderately Saline	8.4	Very Saline
				0.50	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	617	5.6	Moderately Saline		
				0.75	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	795	7.2	Moderately Saline		
				1.00	7.3	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	760	6.5	Moderately Saline	5.7	Moderately Saline
				1.25	6.4	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	673	5.7	Moderately Saline		
				1.50	7.3	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	692	5.9	Moderately Saline		
				1.75	7.3	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	623	5.3	Moderately Saline		
				2.00	7.1	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	717	6.1	Moderately Saline		
				2.25	7.2	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	640	5.4	Moderately Saline		
				2.50	7.1	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	612	5.2	Moderately Saline		
				2.75	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	693	6.2	Moderately Saline		
				3.00	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	592	5.3	Moderately Saline		
SKM12	291641	6265083		0.25	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	480	4.3	Moderately Saline	4.1	Moderately Saline
				0.50	7.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	430	3.9	Slightly Saline		
				1.00	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	378	3.4	Slightly Saline	4.1	Moderately Saline
				1.50	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	321	2.9	Slightly Saline		
				2.00	7.6	B	Non-Aggressive	Non-Aggressive	Clay loam	9	539	4.9	Moderately Saline		
				2.50	7.7	B	Non-Aggressive	Non-Aggressive	Clay loam	9	563	5.1	Moderately Saline		
				3.00	7.1	B	Non-Aggressive	Non-Aggressive	Clay loam	9	477	4.3	Moderately Saline		
SKM13	292095	6264830		0.25	7.1	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	166	2.3	Slightly Saline	2.3	Slightly Saline
				0.50	7.5	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	160	2.2	Slightly Saline		
				1.00	7.4	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	165	2.3	Slightly Saline	1.7	Non Saline
				1.50	6.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	161	1.4	Non Saline		
				2.00	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	175	1.6	Non Saline		
				2.50	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	176	1.6	Non Saline		
				3.00	7.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	165	1.5	Non Saline		
SKM14	291929	6265002		0.25	7.2	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	168	1.4	Non Saline	1.5	Non Saline
				0.50	7.2	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	174	1.5	Non Saline		
				0.75	7.5	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	175	1.5	Non Saline		
				1.00	7.6	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	175	1.5	Non Saline	1.6	Non Saline
				1.25	8.0	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	172	1.5	Non Saline		
				1.50	7.5	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	172	1.5	Non Saline		
				1.75	7.5	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	177	1.5	Non Saline		
				2.00	7.8	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	174	1.5	Non Saline		
				2.25	7.5	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	186	1.6	Non Saline		
				2.50	7.9	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	198	1.7	Non Saline		
				2.75	7.9	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	203	1.7	Non Saline		
				3.00	7.9	B	Non-Aggressive	Non-Aggressive	Clay loam	9	197	1.8	Non Saline		
SKM16	291817	6265351		0.25	7.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	170	1.5	Non Saline	1.5	Non Saline
				0.50	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	170	1.5	Non Saline		
				1.00	7.1	B	Non-Aggressive	Non-Aggressive	Clay loam	9	163	1.5	Non Saline	1.5	Non Saline
				1.50	7.1	B	Non-Aggressive	Non-Aggressive	Clay loam	9	169	1.5	Non Saline		
				2.00	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	169	1.5	Non Saline		
				2.50	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	172	1.5	Non Saline		
				3.00	6.6	B	Non-Aggressive	Non-Aggressive	Clay loam	9	173	1.6	Non Saline		

Test Bore	Coordinates East (m MGA94)	Coordinates North (m MGA94)	RL (m AHD)	Sample Depth (m)	pH	Soil Condition [AS2159]	Soil Aggressivity To Concrete [AS2159 pH criteria]	Soil Aggressivity To Steel [AS2159 pH criteria]	Soil Texture Group [after DLWC]	Textural Factor [M] [after DLWC]	$EC_{1:5}$ [Lab.] (µS/cm)	EC_e [M x $EC_{1:5}$] (dS/m)	Salinity Class [Richards 1954]	ECe Bulk [depths<0.8m/depths>0.8m] (dS/m)	Salinity Class [depths<0.8m/depths>0.8m] [Richards 1954]
SKM17	291648	6265519		0.25	7.6	B	Non-Aggressive	Non-Aggressive	Clay loam	9	282	2.5	Slightly Saline	2.4	Slightly Saline
				0.50	7.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	261	2.3	Slightly Saline		
				1.00	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	293	2.6	Slightly Saline	2.3	Slightly Saline
				1.50	7.6	B	Non-Aggressive	Non-Aggressive	Clay loam	9	296	2.7	Slightly Saline		
				2.00	7.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	261	2.3	Slightly Saline		
				2.50	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	204	1.8	Non Saline		
				3.00	6.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	199	1.8	Non Saline		
SKM18	291560	6265662		0.25	7.1	B	Non-Aggressive	Non-Aggressive	Clay loam	9	286	2.6	Slightly Saline	4.3	Moderately Saline
				0.50	6.9	B	Non-Aggressive	Non-Aggressive	Clay loam	9	350	3.2	Slightly Saline		
				0.75	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	376	3.4	Slightly Saline		
				1.00	6.6	B	Non-Aggressive	Non-Aggressive	Clay loam	9	480	4.3	Moderately Saline	4.6	Moderately Saline
				1.25	6.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	460	4.1	Moderately Saline		
				1.50	6.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	399	3.6	Slightly Saline		
				1.75	6.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	473	4.3	Moderately Saline		
				2.00	7.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	294	2.6	Slightly Saline		
				2.25	7.2	A	Non-Aggressive	Non-Aggressive	Sand	17	280	4.8	Moderately Saline		
				2.50	6.2	A	Non-Aggressive	Non-Aggressive	Sand	17	409	7.0	Moderately Saline		
				2.75	6.1	A	Non-Aggressive	Non-Aggressive	Sand	17	320	5.4	Moderately Saline		
				3.00	7.3	A	Non-Aggressive	Non-Aggressive	Sand	17	297	5.0	Moderately Saline		
SKM19	292150	6265114		0.25	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	217	2.0	Non Saline	2.9	Slightly Saline
				0.50	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	234	2.1	Slightly Saline		
				0.75	7.7	B	Non-Aggressive	Non-Aggressive	Clay loam	9	215	1.9	Non Saline		
				1.00	7.7	B	Non-Aggressive	Non-Aggressive	Clay loam	9	214	1.9	Non Saline	2.2	Slightly Saline
				1.25	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	225	2.0	Slightly Saline		
				1.50	7.8	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	232	2.0	Non Saline		
				1.75	7.4	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	219	1.9	Non Saline		
				2.00	7.7	B	Non-Aggressive	Non-Aggressive	Clay loam	9	193	1.7	Non Saline		
				2.25	7.2	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	184	2.6	Slightly Saline		
				2.50	7.2	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	176	2.5	Slightly Saline		
				2.75	7.5	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	184	2.6	Slightly Saline		
				3.00	7.4	A	Non-Aggressive	Non-Aggressive	Sand	17	184	3.1	Slightly Saline		
SKM20	292026	6265296		0.25	7.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	750	6.8	Moderately Saline	5.5	Moderately Saline
				0.50	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	516	4.6	Moderately Saline		
				0.75	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	486	4.4	Moderately Saline		
				1.00	7.6	B	Non-Aggressive	Non-Aggressive	Clay loam	9	572	5.1	Moderately Saline	6.6	Moderately Saline
				1.25	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	468	4.2	Moderately Saline		
				1.50	6.1	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	601	8.4	Very Saline		
				1.75	5.8	A	Mild	Non-Aggressive	Sandy loam	14	663	9.3	Very Saline		
				2.00	7.4	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	505	7.1	Moderately Saline		
				2.25	7.3	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	524	7.3	Moderately Saline		
				2.50	7.4	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	460	6.4	Moderately Saline		
				2.75	7.2	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	461	6.5	Moderately Saline		
				3.00	7.3	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	392	5.5	Moderately Saline		
SKM21	291914	6265457		0.25	7.6	B	Non-Aggressive	Non-Aggressive	Clay loam	9	383	3.4	Slightly Saline	3.7	Slightly Saline
				0.50	7.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	444	4.0	Slightly Saline		
				1.00	7.9	B	Non-Aggressive	Non-Aggressive	Clay loam	9	430	3.9	Slightly Saline	4.3	Moderately Saline
				1.50	7.9	B	Non-Aggressive	Non-Aggressive	Clay loam	9	524	4.7	Moderately Saline		
				2.00	7.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	553	5.0	Moderately Saline		
				2.50	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	502	4.5	Moderately Saline		
				3.00	7.9	B	Non-Aggressive	Non-Aggressive	Clay loam	9	386	3.5	Slightly Saline		
SKM22	291724	6265608		0.25	6.7	B	Non-Aggressive	Non-Aggressive	Clay loam	9	313	2.8	Slightly Saline	3.5	Slightly Saline
				0.50	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	239	2.2	Slightly Saline		
				0.75	6.6	B	Non-Aggressive	Non-Aggressive	Clay loam	9	288	2.6	Slightly Saline		
				1.00	6.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	300	2.7	Slightly Saline	2.5	Slightly Saline
				1.25	6.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	253	2.3	Slightly Saline		
				1.50	6.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	242	2.2	Slightly Saline		
				1.75	6.6	B	Non-Aggressive	Non-Aggressive	Clay loam	9	212	1.9	Non Saline		
				2.00	7.0	B	Non-Aggressive	Non-Aggressive	Clay loam	9	238	2.1	Slightly Saline		
				2.25	6.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	237	2.1	Slightly Saline		
				2.50	6.3	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	219	3.1	Slightly Saline		
				2.75	6.4	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	232	3.2	Slightly Saline		
				3.00	6.8	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	217	3.0	Slightly Saline		
SKM23	291668	6265854		0.25	6.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	277	2.5	Slightly Saline	3.3	Slightly Saline
				0.50	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	249	2.2	Slightly Saline		
				0.75	6.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	248	2.2	Slightly Saline		
				1.00	6.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	265	2.4	Slightly Saline	3.3	Slightly Saline
				1.25	6.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	262	2.4	Slightly Saline		
				1.50	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	234	2.1	Slightly Saline		
				1.75	6.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	268	2.4	Slightly Saline		
				2.00	6.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	278	2.5	Slightly Saline		
				2.25	6.1	B	Non-Aggressive	Non-Aggressive	Sandy loam	14	296	4.1	Moderately Saline		
				2.50	5.9	A	Mild	Non-Aggressive	Sandy loam	14	313	4.4	Moderately Saline		
				2.75	6.4	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	333	4.7	Moderately Saline		
				3.00	6.2	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	315	4.4	Moderately Saline		

Test Bore	Coordinates East (m MGA94)	North	RL (m AHD)	Sample Depth (m)	pH	Soil Condition [AS2159]	Soil Aggressivity To Concrete [AS2159 pH criteria]	To Steel	Soil Texture Group [after DLWC]	Textural Factor [M] [after DLWC]	$EC_{1:5}$ [Lab.] (µS/cm)	EC_e [M x $EC_{1:5}$] (dS/m)	Salinity Class [Richards 1954]	ECe Bulk [depths<0.8m/depths>0.8m] (dS/m)	Salinity Class [Richards 1954]
SKM25	291978	6265734		0.25	6.9	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	166	1.4	Non Saline		
				0.50	7.3	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	166	1.4	Non Saline	1.4	Non Saline
				0.75	7.4	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	167	1.4	Non Saline		
				1.00	6.7	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	168	1.4	Non Saline		
				1.25	7.3	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	165	1.4	Non Saline		
				1.50	6.9	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	163	1.4	Non Saline		
				1.75	6.7	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	164	1.4	Non Saline		
				2.00	7.2	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	167	1.4	Non Saline	1.4	Non Saline
				2.25	7.2	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	166	1.4	Non Saline		
				2.50	7.0	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	164	1.4	Non Saline		
				2.75	6.5	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	170	1.4	Non Saline		
				3.00	7.5	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	169	1.4	Non Saline		
SKM26	292044	6265896		0.25	7.0	B	Non-Aggressive	Non-Aggressive	Clay loam	9	354	3.2	Slightly Saline	2.5	Slightly Saline
				0.50	7.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	211	1.9	Non Saline		
				1.00	7.7	B	Non-Aggressive	Non-Aggressive	Clay loam	9	201	1.8	Non Saline		
				1.50	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	210	1.9	Non Saline		
				2.00	7.0	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	186	1.6	Non Saline	1.7	Non Saline
				2.50	7.5	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	184	1.6	Non Saline		
				3.00	6.9	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	202	1.7	Non Saline		
SKM27	291795	6266039		0.25	7.0	B	Non-Aggressive	Non-Aggressive	Clay loam	9	287	2.6	Slightly Saline		
				0.50	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	228	2.0	Slightly Saline	2.5	Slightly Saline
				0.75	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	299	2.7	Slightly Saline		
				1.00	7.4	B	Non-Aggressive	Non-Aggressive	Clay loam	9	319	2.9	Slightly Saline		
				1.25	7.0	B	Non-Aggressive	Non-Aggressive	Clay loam	9	369	3.3	Slightly Saline		
				1.50	7.0	B	Non-Aggressive	Non-Aggressive	Clay loam	9	440	4.0	Slightly Saline		
				1.75	7.0	B	Non-Aggressive	Non-Aggressive	Clay loam	9	397	3.6	Slightly Saline		
				2.00	7.0	B	Non-Aggressive	Non-Aggressive	Clay loam	9	381	3.4	Slightly Saline	4.7	Moderately Saline
				2.25	6.7	A	Non-Aggressive	Non-Aggressive	Sand	17	391	6.6	Moderately Saline		
				2.50	7.1	A	Non-Aggressive	Non-Aggressive	Sand	17	362	6.2	Moderately Saline		
				2.75	6.6	A	Non-Aggressive	Non-Aggressive	Sand	17	365	6.2	Moderately Saline		
				3.00	6.6	A	Non-Aggressive	Non-Aggressive	Sand	17	376	6.4	Moderately Saline		
SKM29	291980	6266008		0.25	7.0	B	Non-Aggressive	Non-Aggressive	Clay loam	9	274	2.5	Slightly Saline		
				0.50	7.3	B	Non-Aggressive	Non-Aggressive	Clay loam	9	272	2.4	Slightly Saline	2.4	Slightly Saline
				0.75	6.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	269	2.4	Slightly Saline		
				1.00	5.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	325	2.9	Slightly Saline		
				1.25	5.7	B	Non-Aggressive	Non-Aggressive	Clay loam	9	291	2.6	Slightly Saline		
				1.50	6.7	B	Non-Aggressive	Non-Aggressive	Clay loam	9	440	4.0	Slightly Saline		
				1.75	6.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	383	3.4	Slightly Saline		
				2.00	6.1	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	396	3.4	Slightly Saline	4.5	Moderately Saline
				2.25	6.5	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	972	8.3	Very Saline		
				2.50	7.1	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	741	6.3	Moderately Saline		
				2.75	7.0	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	708	6.0	Moderately Saline		
				3.00	7.1	B	Non-Aggressive	Non-Aggressive	Light clay	8.5	473	4.0	Moderately Saline		

Appendix D. Filling of Land

Lend Lease

Central Precinct of St Mary's Project Development

FLOOD ASSESSMENT REPORT

Final

20 July 2015

JACOBS

Central Precinct of St Mary's Project Development

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REV A	-	Practice Review	S. Watt		
REV B	13 Nov 2013	Delivery Review	S. Watt	J. Constandopoulos	J. Constandopoulos
REV C	18 Nov 2013	Delivery Review	S. Watt	J. Constandopoulos	J. Constandopoulos
REV D	8 Jul 2014	Update with revised developed scenario D117	S. Watt	J. Constandopoulos	J. Constandopoulos
REVF	10 Feb 2015	Update with revised developed scenario D153	C.Stephens	S.Watt	J. Constandopoulos
REVH	17 Feb 2015	Updated as per feedback received from Lend Lease	C.Stephens	S.Watt	J. Constandopoulos
REVI	13 Apr 2015	Minor changes based on feedback received from Council	C.Stephens	S.Watt	J. Constandopoulos
REVJ	5 May 2015	Update based on feedback from Council including revised existing scenario E056 and developed scenario D166	S.Watt	J.Constandopoulos	J.Constandopoulos
REVJA	6 May 2015	Minor update to FPA assessment	S.Watt	J.Constandopoulos	J.Constandopoulos
REVK	28 May 2015	Update to include revised existing scenario E061 and development scenario D209	S.Watt	S.Dooland	S.Watt
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REVL	20 July 2015	Update based on outcomes of WP Peer Review & inclusion of phasing plans	S.Watt	J.Constandopoulos	J.Constandopoulos

Executive Summary

Lend Lease is seeking to develop a series of villages as part of their St Mary's Project including the Dunheved and Central Precincts in the Penrith City Council area. These two development areas are located adjacent to the South Creek and Ropes Creek floodplains, near the confluence of the two creeks.

Potential flooding impacts in South Creek and its tributaries due to the proposed development, on the Dunheved and Central Precincts of the St Marys Project, have been assessed by Jacobs using the 1D/2D hydraulic modelling program MIKEFLOOD.

Previous assessments were undertaken using a MIKE11 model produced by SKM (now Jacobs) in 2007. This MIKE11 model was previously considered to be appropriate by Blacktown City Council (BCC) and Penrith City Council (PCC).

Recently, Worley Parsons has developed a new hydrodynamic model for South Creek and tributaries on behalf of PCC. This model has been developed using the RMA-2 software package, a two-dimensional hydrodynamic model. PCC advised Lend Lease that assessment of the potential flooding impacts of the Dunheved and Central Precinct areas should be consistent with PCC's new two-dimensional model.

This assessment has been completed using the MIKEFLOOD hydrodynamic modelling package and was developed with consistent assumptions to produce results consistent with PCC's RMA-2 model. The MIKEFLOOD model was also developed to be consistent with the MIKE11 model (SKM 2007).

The MIKEFLOOD model was used to assess a range of alternative developed options to arrive at the preferred developed option.

The preferred developed scenario includes the proposed fill layout for both the Dunheved and Central Precinct areas, upgrades to the East West Connector Road at South Creek, widening of the South Creek Bridge and additional high flow culverts on the west bank, addition of Dunheved Link Road, removal of the abutments and embankment at Old Munitions Road and removal of existing spoil piles within the floodplain. In the preferred option, both the East West Connector Road and Dunheved Link would be located above the 1% AEP flood level with no freeboard. The East West Connector Bridge over South Creek would be replaced with widening of the high flow path on the left bank by removal of the existing left bridge abutment. An additional bank of 9/ 4200 mm X 2700 mm RCBCs has been included under the section of the East West Connector Road to the west of the South Creek Bridge. An M-lock bridge structure with a waterway area of approximately 160m^2 would be required under the Dunheved Link to convey floodwaters. A small landscaped berm (~30 m long and 0.5 m high) was included in the design to prevent backwater flow into the industrial area on Links Road. However, it is unlikely that this will be required under the current preferred configuration.

The NSW Government's Floodplain Development Manual (2005) (the Manual) advocates a merit-based approach to assessment of filling within the floodplain which takes into consideration social, economic and ecological factors, as well as flooding characteristics.

A number of hydraulic improvements to the floodway are proposed as part of the planned development to maintain flood conveyance through this reach. These include:

- removal of Old Munitions Road embankment;
- removal of stockpiles on the western floodplain;
- increased waterway area through the South Creek bridge; and
- Additional culverts under the western section of the EWC road.

The channel conveyance within South Creek is essentially unchanged upstream and downstream of the site boundary. Additionally, there is no change in the timing of flows arriving at locations downstream of the site.

The preferred development scenario produces 38 mm impact at the upstream site boundary and 11 mm at the downstream site boundary during a 1% AEP regional tailwater event. This is consistent with PCC's Development Control Plan requirement of no more than 100 mm afflux at the site boundary, noting that the DCP is not explicitly applicable. Furthermore, both the upstream and downstream impacts are consistent with the level of impact adopted by PCC in the 2009 Central Precinct Plan and Development Control Strategy (DCS).

Upstream impacts affect a total of five (5) lots - the St Marys WWTP site, Sydney Water Recycled Water Scheme site, Dunheved Golf Course and Links Road (road reserve and footpath). No buildings are impacted on these lots inundated within the developed scenario.

A breakout from Ropes Creek flows through the St Marys WWTP site and re-joining the Ropes Creek floodplain to the north of the Dunheved Precinct via a formed channel likely used to discharge treated water to South Creek. Within the model, impacts on the St Marys WWTP site occur as the north-east corner of the Dunheved fill area encroaches on this formed (~10 m width) channel. The 10 m grid resolution within the model is coarse in comparison to the width of the formed channel and culvert, and therefore may over-estimate the impacts on this lot. Modelled impacts within this site are less than 90 mm and generally in the order of 20-40 mm in a localised low area. These impacts are not considered a material impact on this site. However, if required, these impacts could likely be mitigated by minor refinements to the Dunheved fill area during detailed design.

The remainder of the impacts are largely contained within the Dunheved Golf Course which is significantly flooded under existing conditions. There is no increase in the duration of flooding expected within the Golf Course under developed conditions.

The characteristics of the downstream floodplain result in relatively little attenuation of impacts, meaning small impacts of approximately 10-20 mm propagate for a significant distance downstream. These impacts do not cause a significant increase in flood extent, result in inundation of additional properties not currently inundated and are not expected to produce material flood impacts on downstream properties.

No additional properties will be affected by the Flood Planning Area in the developed scenario. Four (4) upstream industrial properties on Links Road will potentially be affected by an increase while one property will potentially be affected by a decrease in the area affected by the Flood Planning Area.

Impacts on velocities greater than 0.02 m/s in the 1% AEP event are confined within the site boundary.

It is important to consider the potential for cumulative filling within the floodplain as part of a merit-based assessment. The Lend Lease site encompasses the width of the South Creek floodplain through this reach. The Central Precinct and Dunheved fill areas form part of a larger regional plan which includes handover of the residual floodplain areas to the NSW Government to manage as National Park. Therefore, future filling and further encroachment within this section of South Creek is highly unlikely. Hence, the assessment for the proposed developed scenario can be considered to be a cumulative assessment of the total fill possible in this reach of the floodplain.

Worley Parsons undertook an independent peer review of REVKA of this Flood Assessment Report in July 2015 and found the approach and model "*to be a suitable tool for the assessment of flood behaviour and flood impacts for the Lend Lease site*". The review indicated that the South Creek Flood Study model predicted "*impacts that are equal to or lesser than those documented in the FIA Report*". Furthermore, the review considered the impacts "*minor for all areas outside of the Lend Lease site.*"

The preferred developed scenario presented here provides an appropriate balance of flood immunity, upstream and downstream flood impacts, constructability and environmental impact that is achievable. It has also been demonstrated that development of the preferred developed scenario can be phased such that impacts during development do not materially exceed the impacts of the preferred developed scenario. The exact structure configurations, final fill platforms and phasing will be determined and modelled in more detail during detailed design.

Contents

1. Introduction

1.1 Background

Lend Lease is seeking to develop a series of villages as part of their St Mary's Project including the Dunheved and Central Precincts in the Penrith City Council area. These two development areas are located adjacent to the South Creek and Ropes Creek floodplains, near the confluence of the two creeks.

Potential flooding impacts in South Creek and its tributaries due to the proposed development, on the Dunheved and Central Precincts of the St Marys Project, has been assessed by Jacobs (formerly SKM) over the last 10 years, on behalf of Lend Lease.

These assessments used a MIKE11 hydrodynamic model incorporating lateral linking (quasi two-dimensional), developed by SKM in 2007. The MIKE11 model was considered to be appropriate by Blacktown City Council (BCC) and Penrith City Council (PCC) for the previous flood impact assessments of the St Marys Project.

Recently, Worley Parsons has developed a new hydrodynamic model for South Creek and tributaries on behalf of PCC. This model has been developed using the RMA-2 software package, a two-dimensional hydrodynamic model.

PCC has advised Lend Lease that assessment of the potential flooding impacts of the Dunheved and Central Precinct areas should be consistent with PCC's new two-dimensional model. However, preliminary discussions with PCC indicate that the RMA-2 computer model is unlikely to be made available to third parties.

Lend Lease commissioned Jacobs to develop an updated two-dimensional hydrodynamic model for South Creek and tributaries, to provide an assessment of the potential impacts of the Central Precinct development, consistent with PCC's new requirements. This modelling has been completed using the MIKEFLOOD hydrodynamic modelling package.

PCC have provided information on assumptions and parameters that were used in their RMA-2 model. The MIKEFLOOD model developed for this assessment has been developed with consistent assumptions and aims to produce consistent results to PCC's RMA-2 model.

An independent peer review of REVKA of this Flood Assessment Report was undertaken by Worley Parsons in July 2015 entitled *Central Precinct of St Mary's Project Development – Flood Assessment Report: Peer Review*. Worley Parsons found the approach and model "*to be a suitable tool for the assessment of flood behaviour and flood impacts for the Lend Lease site*".

1.2 Purpose of the report

The purpose of this report is to detail the development and application of the new MIKEFLOOD hydraulic model built for the purposes of assessing impacts of the proposed St Mary's development consistent with PCC's requirements.

1.3 Report outline

This report details the hydraulic modelling and assessment of potential flood impacts, under the following headings:

- Section 2 - Review of available information
- Section 3 - Hydraulic model development
- Section 4 - Existing flood behaviour
- Section 5 - Flood assessment for development scenario
- Section 6 - Conclusions and recommendations

1.4 Reliance Statement

The sole purpose of this report and the associated services performed by Jacobs is to provide a model consistent with Penrith City Council's model to assess potential impacts of the Central Precinct development area in accordance with the scope of services set out in the contract between Jacobs and the Lend Lease.

In preparing this report, Jacobs has relied upon, and presumed accurate, certain information (or absence thereof) provided by Lend Lease, PCC, Worley Parsons and other sources. Except as otherwise stated in the report, Jacobs has not attempted to verify the accuracy or completeness of any such information. If the information is subsequently determined to be false, inaccurate or incomplete then it is possible that our observations and conclusions as expressed in this report may change.

Jacobs derived the data in this report from a variety of sources. The sources are identified at the time or times outlined in this report. The passage of time, manifestation of latent conditions or impacts of future events may require further examination of the project and subsequent data analysis, and re-evaluation of the data, findings, observations and conclusions expressed in this report. Jacobs has prepared this report in accordance with the usual care and thoroughness of the consulting profession, for the sole purpose of the project and by reference to applicable standards, procedures and practices at the date of issue of this report. For the reasons outlined above, however, no other warranty or guarantee, whether expressed or implied, is made as to the data, observations and findings expressed in this report.

This report should be read in full and no excerpts are to be taken as representative of the findings. No responsibility is accepted by Jacobs for use of any part of this report in any other context.

This report has been prepared on behalf of, and for the exclusive use of Lend Lease, and is subject to, and issued in connection with, the provisions of the agreement between Jacobs and Lend Lease. Jacobs accepts no liability or responsibility whatsoever for, or in respect of, any use of, or reliance upon, this report by any third party.

2. Review of available information

2.1 Previous investigations

2.1.1 SKM MIKE11 model

In 2007, SKM developed a MIKE11 hydrodynamic model incorporating lateral linking (quasi two-dimensional) to assess the potential flooding impacts in South Creek and its tributaries to the proposed development, on the Dunheved and Central Precincts of the St Marys Project. Reporting on this assessment can be found in, the *Dunheved Precinct Development Application* (SKM, March 2007) and *Addendum* (SKM, December 2007).

2.1.2 Worley Parsons RMA-2 model

In 2013, Worley Parsons was commissioned by PCC to undertake the *South Creek Flood Study*. As part of this study, Worley Parsons developed an RMA-2 hydrodynamic model.

While the RMA-2 model has not been made available, selected model outputs have been provided to Jacobs for the purpose of model verification in the vicinity of the Dunheved and Central Precinct sites (the Project site). This information was provided in a memo dated 5 July 2013 and entitled *South Creek Flood Study: Provision of DRAFT Results at ADI Site, St Marys* (Worley Parsons, 2013). Information provided within this memo included:

- inflow hydrographs for 5%, 1% and PMF flood events;
- tailwater conditions for these events for both regional and local flood events;
- resulting flood surface profiles for these events with both regional and local tailwater conditions;
- resulting flood extents for these six events; and
- classification of flood characteristics in the flood corridor.

The hydrographs provided were exported from the RMA-2 for both Ropes Creek and South Creek. The locations of the hydrograph exports were directly upstream of the Project site. Three hydrographs were provided at each location for the 5% AEP, 1% AEP and PMF events.

A downstream tailwater level was provided for each event based on two tailwater scenarios, representing a local flood event and a local event coincident with a regional flood event in the Hawkesbury River. The tailwater for each flood scenario was reported at a location in South Creek 2.6km downstream of Richmond Road.

Water surface level profiles were provided along both South and Ropes Creeks. The South Creek profile extended from Dunheved Road to Eighth Avenue while the flood profile in Ropes Creek extending from Ropes Crossing Boulevard to the confluence with South Creek.

Maps of flood inundation extents were provided for the 5% AEP, 1% AEP and PMF events with local tailwater conditions. Hydraulic category mapping was provided based on the 1% AEP flood under local tailwater conditions.

2.2 Available data

2.2.1 Terrain data

LIDAR point data from the previous SKM study was available for use in this study.

2.2.2 MIKE11 model for South Creek

SKM's 2007 MIKE11 model of South Creek and the lower section of Ropes Creek was available for use in this study. This model was developed as part of the *Dunheved Precinct Development Application* in 2007 and was developed to assess the impact of the development.

2.3 Additional data collected for this study

2.3.1 Terrain data

Additional LIDAR data was obtained from AAM hatch in the form of thinned xyz points to enable the model to be extended past the Richmond Road Bridge. This data was captured in 2008 and has a stated accuracy of ±0.15 m vertical and ±0.5 m horizontal.

2.3.2 Data on waterway crossings

Hydraulic structures within the model area were characterised using detailed ground survey. Additional survey was sourced for the following structures:

- East West Road Bridge at South Creek;
- East West Road Bridge at Ropes Creek;
- Christie Street Bridge at South Creek;
- Eighth Avenue Bridge at South Creek;
- Richmond Road Bridge at South Creek; and
- Stony Creek Road Culverts at South Creek.

2.3.3 Detailed Survey

Detailed survey of ground levels and building floor levels in the Links Road industrial area was completed by RPS in April 2015 on behalf of Lend Lease

2.4 Proposed fill layout

The fill layout was provided by Lend Lease for the proposed development of the Dunheved and Central precinct sites.

The layouts of these two sites are discussed in more detail in Section 5.

3. Hydraulic model development

3.1 Adopted hydraulic modelling approach

The MIKEFLOOD package developed by the Danish Hydraulics Institute (DHI) (version 2011) was adopted for the investigation. MIKEFLOOD links the two-dimensional (2-D) hydraulic modelling package MIKE21 to the one dimensional (1-D) hydraulic modelling package MIKE11. This allows for detailed 1-D modelling of specific hydraulic structures inside a 2-D flood model.

The MIKE21 model represents the investigation area topography as a terrain grid, with the following parameters input to the model to define flow behaviour:

- Design or historical inflow time series;
- Downstream boundary conditions;
- Terrain roughness (entered as Manning's roughness); and
- Eddy viscosity.

3.2 Model area

The hydraulic model area starts approximately 500 m downstream of Christie St and extends just past the Richmond Road Bridge over South Creek. The model extent extends significantly upstream and downstream of the Project site for use of the modelling results to assess flood impacts through the Project site, as well as upstream and downstream of the site.

The adopted hydraulic model extent is shown in Figure 3-1.

3.3 Terrain development

The terrain used in the hydraulic modelling was developed from a combination of data sources. Aerial laser scanning (ALS) survey data by AAM and point data from the previous SKM study was collected for the selected model area.

From this, a regular grid of 10 m x 10 m resolution was developed for input to the hydraulic modelling. The 10 m grid was selected in order to maximise runtime efficiency while maintaining model definition. The existing East West Connector road crest was stamped on the model manually to improve the representation of overtopping of the road.

Terrain levels for the Links Road industrial area were stamped on the model based on detailed survey to improve the resolution of the model in this area. Figure 3-2 presents the adopted terrain levels in this area.

3.4 Inflow hydrographs

No hydrologic modelling was undertaken for this assessment.

To meet the PCC requirement that the assessment provide consistent results to the RMA-2 model produced by Worley Parsons model, model inflow hydrographs were digitised and adopted as provided by Worley Parsons (Worley Parsons, 2013). It was assumed that the provided hydrographs were representative of flows for the critical duration storm at the Project site. No verification of the appropriateness of the adopted flows was undertaken.

The inflow hydrographs provided were digitised and applied to the model at an approximately equivalent location within the MIKEFLOOD model. The South Creek inflow was introduced to the model approximately 300 m upstream of the Christie St / Dunheved Rd. The Ropes Creek inflow was introduced immediately downstream of Rope Crossing Boulevard.

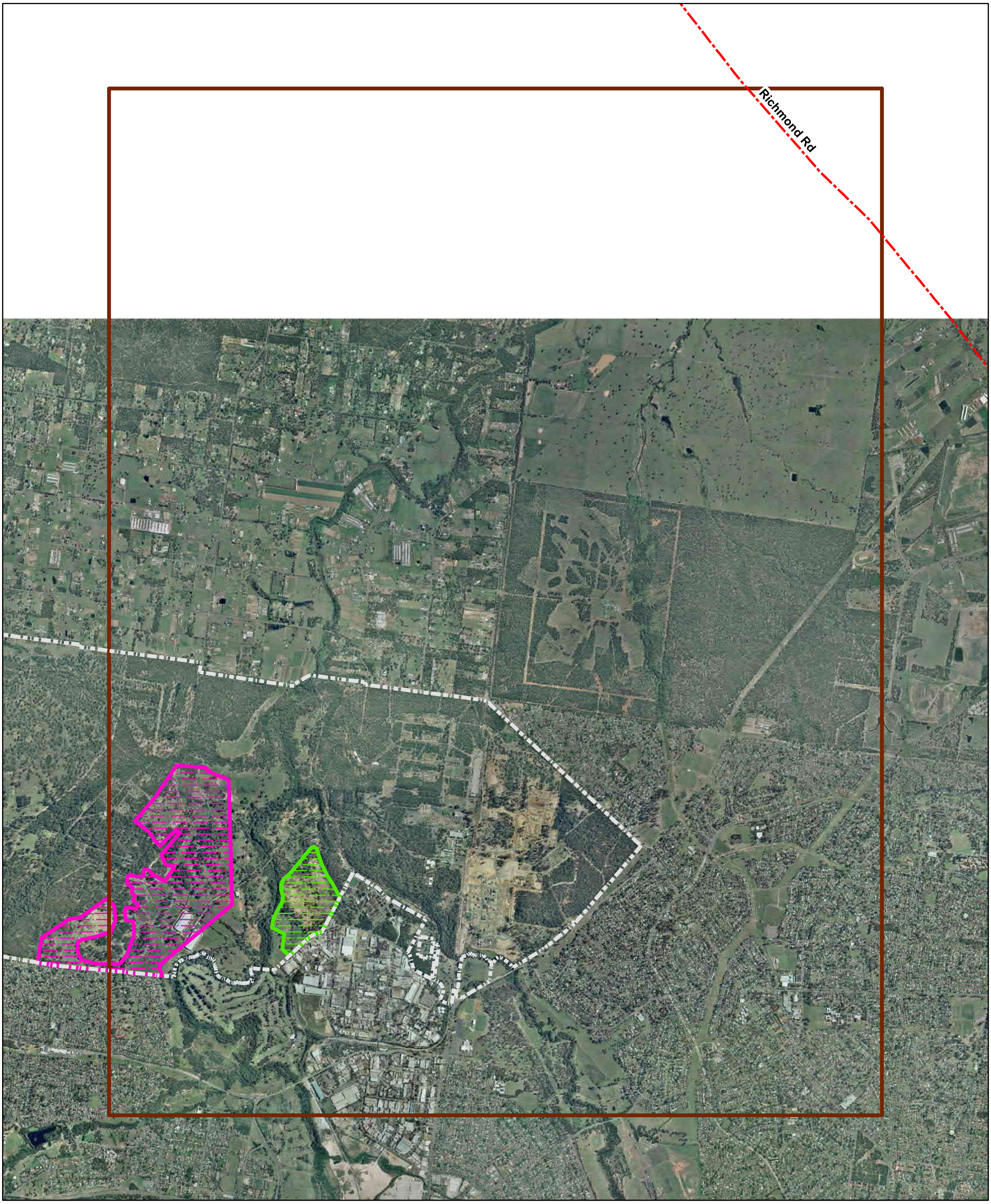

Figure 3-1: Hydraulic Model Extent

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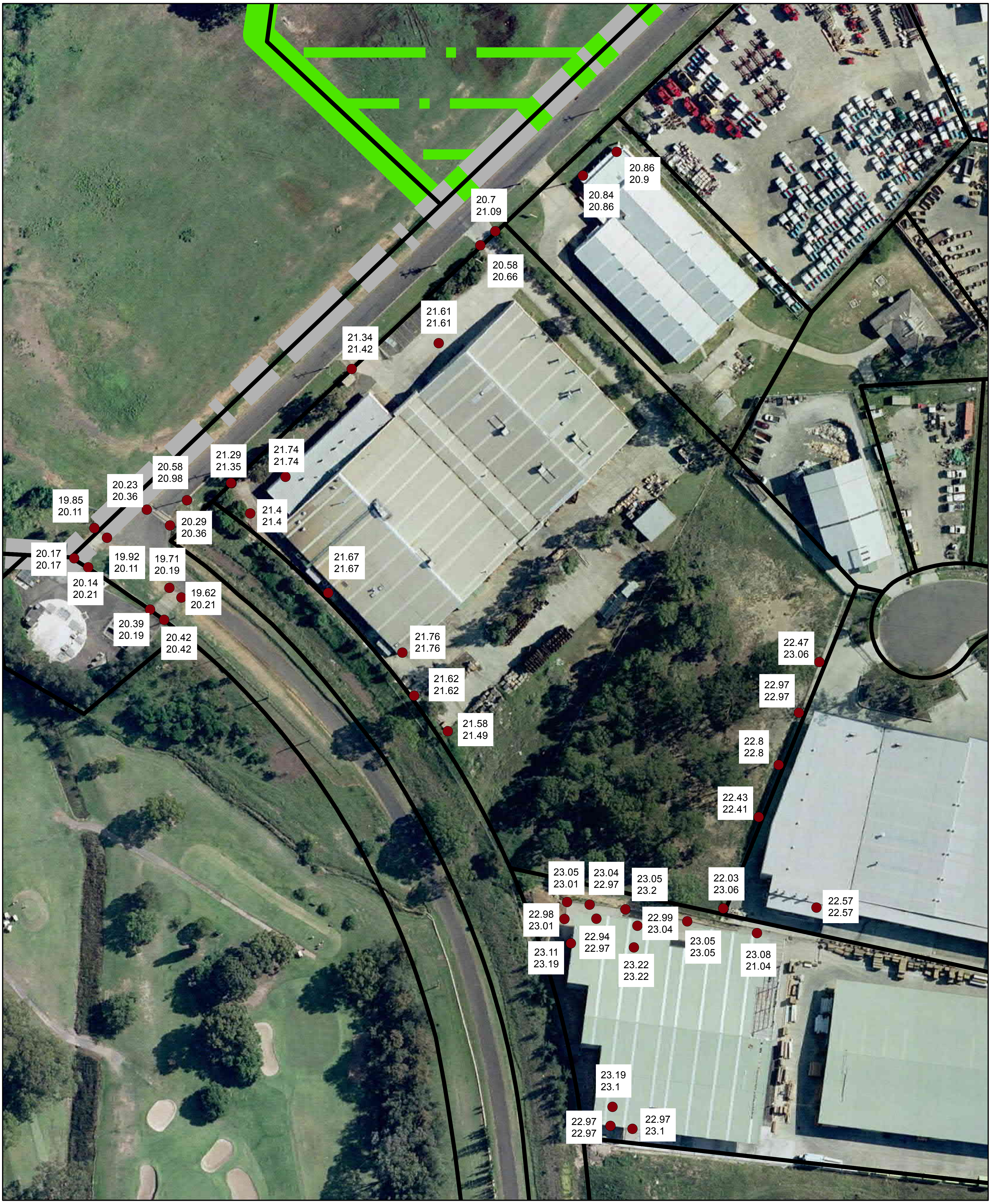

Legend

Site Boundary

Dunheved Precinct

Survey Points: Survey Level / Existing Model Terrain Level

Figure 3-2: Terrain Analysis
Existing Terrain
Incorporation of Detailed Survey

0 15 30 60 90 120 150 Meters

Scale in A3

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Last Modified: 20/07/2015
By: S.Watt

3.5 Downstream boundary conditions

Similarly to meet the PCC requirement that the assessment provide consistent results to the RMA-2 model produced by Worley Parsons model, downstream boundary conditions were adopted to replicate those nominated by Worley Parsons (2013).

Worley Parsons had modelled two downstream boundary conditions for each flood event, to simulate:

- local tailwater conditions for a storm centred over the local catchment; and
- regional tailwater conditions for a local flood coinciding with a flood in the Hawkesbury River.

Jacobs replicated the six (6) scenarios run by Worley Parsons and these two tailwater conditions were modelled for the 5% AEP, 1% AEP and PMF events.

The tailwater levels adopted by Worley Parsons were extracted from the RMA-2 model at its downstream boundary, located 2.6km downstream of Richmond Road. Peak water surface profile long sections were also provided along South Creek to Eighth Avenue.

The boundary of the MIKEFLOOD model is located immediately downstream of Richmond Road, between these two locations. To determine consistent boundary conditions to apply to the MIKEFLOOD model, these levels were interpolated based on an assumed linear flood slope between Eighth Avenue and the RMA-2 boundary, 2.6km downstream of Richmond Road.

The resulting tailwater levels were applied as a static water surface level boundary within the MIKEFLOOD model and are shown in Table 3-1, for each of the six scenarios.

- Table 3-1 Adopted tailwater levels in MIKEFLOOD model

Event	Local Flood Conditions (mAHD)	Regional Flood Conditions (mAHD)
5% AEP	9.1	13.7
1% AEP	9.8	17.3
PMF	13.3	26.4

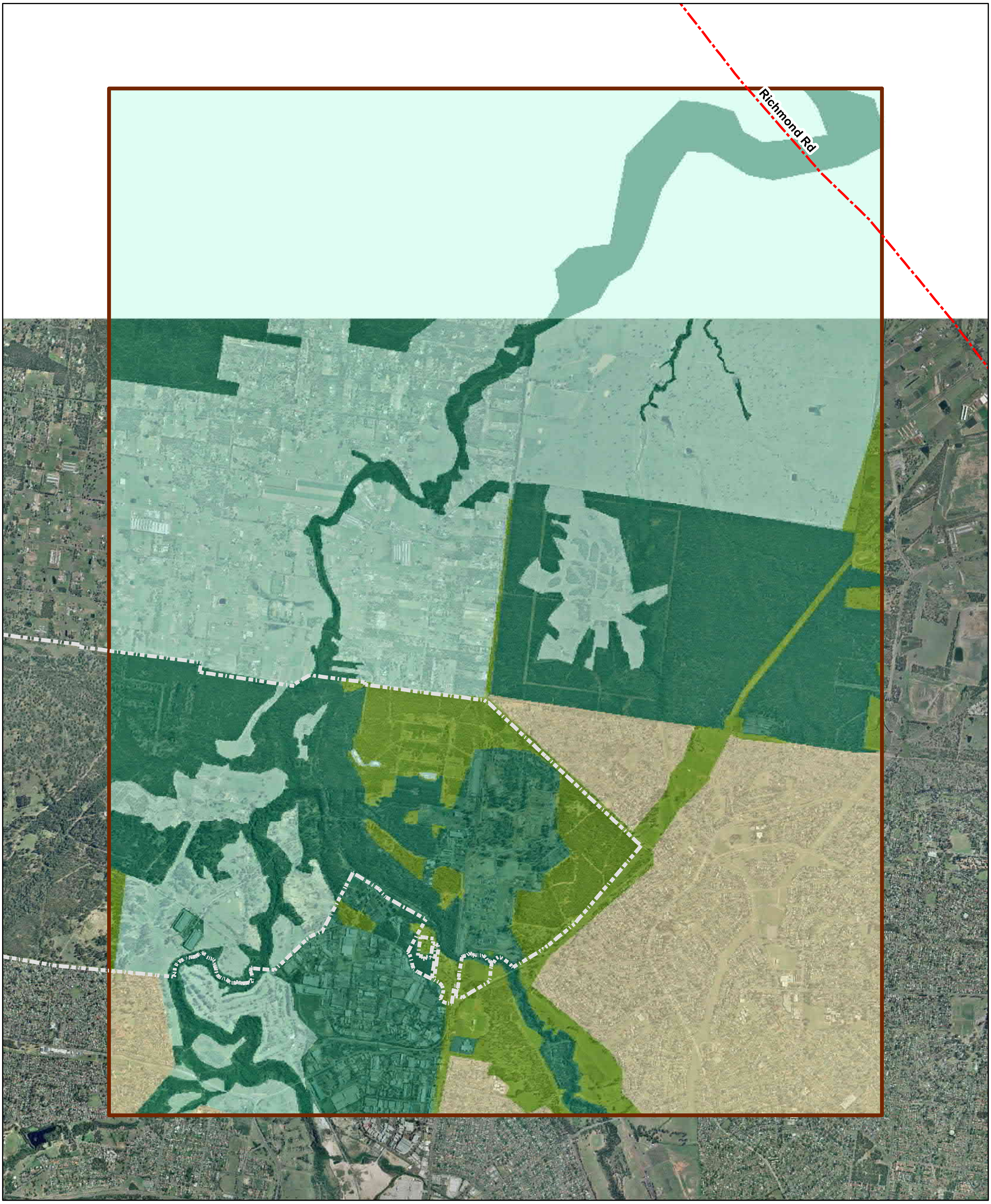
The sensitivity of the model to the adopted downstream boundary conditions was tested in the MIKEFLOOD model and was found to have little impact on the model results upstream on Richmond Road. This is due to the action of the Richmond Road Bridge as a hydraulic control.

3.6 Land Use and Manning’s roughness

Hydraulic roughness values (Manning’s M) were used to describe the different surfaces in the hydraulic model. Manning’s M values were initially estimated based on an analysis of the existing land use and vegetation cover in aerial photography. The final adopted values were defined through the calibration process to produce results consistent with the South Creek Flood Study (Worley Parsons, 2015). The adopted Manning’s roughness values are presented in Table 3-2 and Figure 3-3.

- Table 3-2 Adopted Manning’s roughness in MIKEFLOOD model

Land Use	Manning’s M	Manning’s n
Urban	40	0.025
Floodplain A	12.5	0.08
Floodplain B	11	0.09
Riparian (including channel)	10	0.1



Legend

- Roads
- Site Boundary
- MIKEFlood Extent

Manning's n value

- 0.025
- 0.080
- 0.091
- 0.100

0 300 600 1,200 1,800 2,400 3,000 Meters

Scale in A3

Figure 3-3: Hydraulic Roughness (Manning's n)

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3.7 Eddy viscosity

A uniform eddy viscosity of 0.5 was applied across the full model area, consistent with the 10 m resolution grid adopted, as recommended by the model developers.

3.8 Hydraulic structures

There are numerous drainage structures located within the investigation area with the potential to impact on flooding behaviour. These include major structures such as bridges and culverts.

Site survey was undertaken to collect information on six structures within the model area. Information on the structure type, location, size and invert levels was collected. These details were used to build structure code in MIKE11 which was linked to the MIKE21 model though MIKEFLOOD coupling software.

Where required, the 10 m model terrain was adjusted to equal the invert levels of the detailed structure survey and ensure smooth connection between the 2-D and 1-D models.

The modelled head loss across each structure was expressed as a multiplier of the velocity head through the structure and found to be within reasonable bounds. It can therefore be concluded that the 1D-2D linkages are appropriately representing flow through the structures.

3.9 Model calibration results

The model was required by PCC to provide good agreement with the RMA-2 model results produced by Worley Parsons in 2013. The parameters within the Jacobs MIKEFLOOD hydraulic model were adjusted to create a model that had similar hydraulic behaviours to the RMA-2 model. This process was undertaken with a focus on the 1% AEP event with local tailwater conditions, which was provided by Worley Parsons.

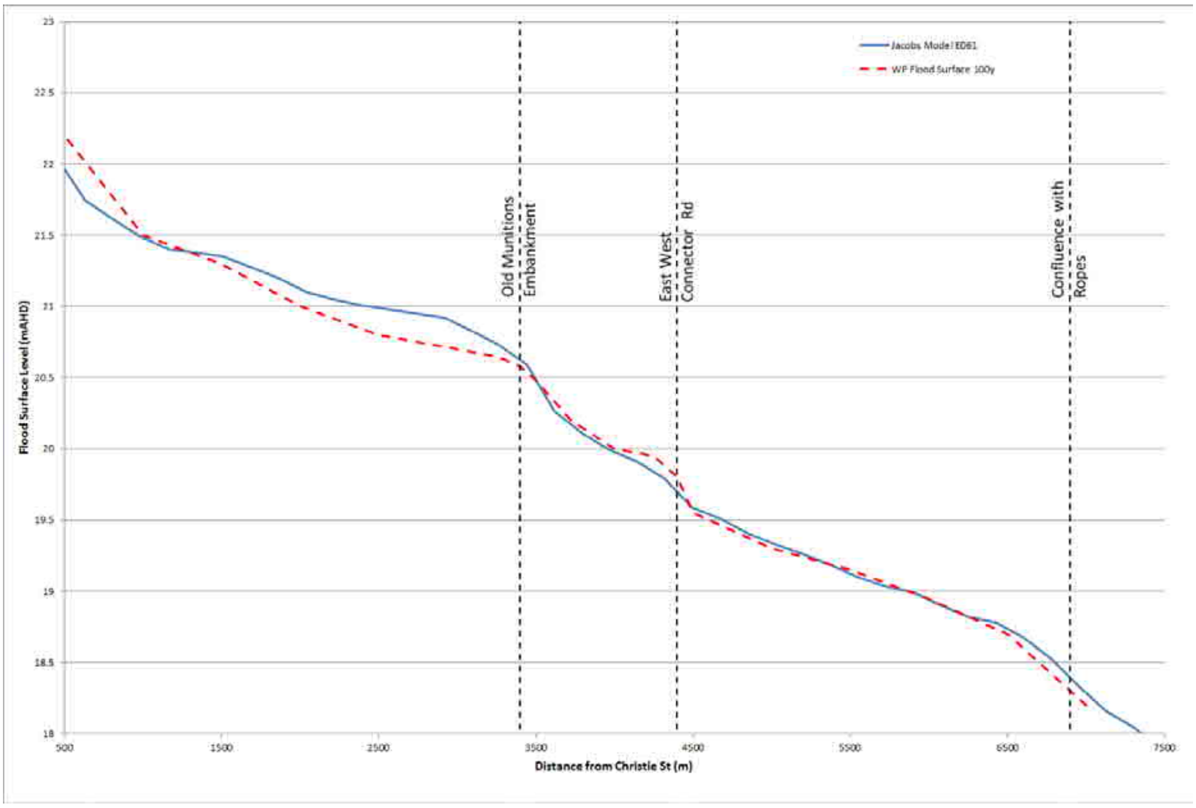
The process of matching hydraulic model results was done by considering both the flooding contours across the floodplain and the flooding profile along the length of the two creeks.

Peak water surface level long sections were extracted from the Jacobs model for the 1% AEP local tailwater event and compared to the Worley Parsons RMA-2 model results for the same event. A comparison of the Jacobs long section with the long section provided by Worley Parsons for South Creek is shown in Figure 3-4. This shows a maximum difference in flood surface level of 200 mm with the majority of the profile being in close agreement.

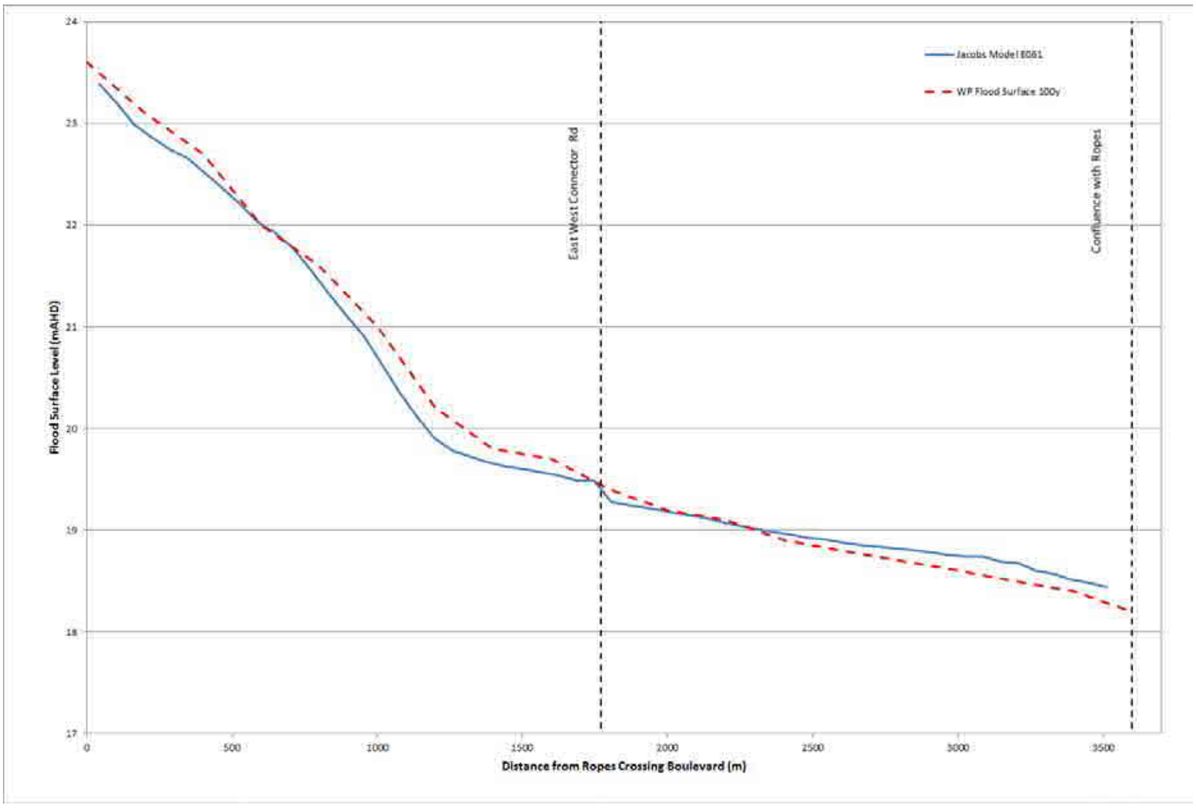
Figure 3-5 shows a comparison of the Jacobs long section with the long section provided by Worley Parsons for Ropes Creek in the 1% AEP local tailwater event. Within Ropes Creek there was a maximum difference of less than 300 mm in the upstream extent of Ropes Creek.

Figure 3-6Figure 3-6 shows a comparison of the peak water surface level contours between the two models for the 1% AEP local tailwater event. This plot shows good agreement between the two models with a similar pattern of flooding. Similar long section comparisons were undertaken for the 5% AEP and PMF events, with the Jacobs levels generally within +/- 400 mm of the Worley Parsons levels. Differences between the two models are likely to be due to differences in the representation of hydraulic structures and other landform features within the model area.

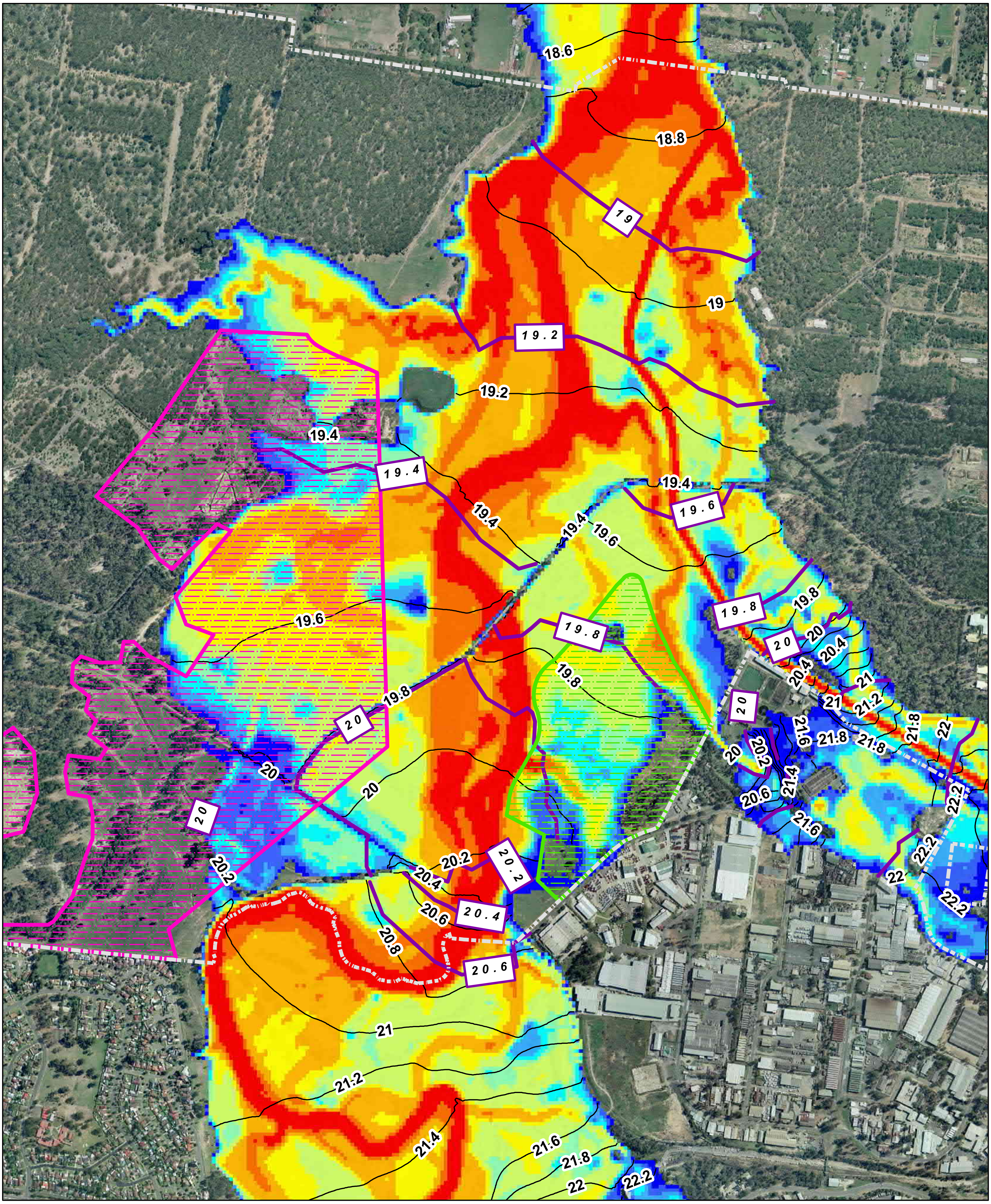
The independent peer review by Worley Parsons (2015) provided a more detailed comparison between the peak 1% AEP flood levels (Regional Tailwater) across the site predicted by the MIKEFLOOD and RMA-2 models, in Figure 1 of their review (reproduced here as Figure 3-7).



- Figure 3-4 South Creek Long Section Comparison for 1% AEP Local Tailwater Event



- Figure 3-5 Ropes Creek Long Section Comparison for 1% AEP Local Tailwater Event



Legend

- Site Boundary
- Dunheved Precinct
- Central Precinct
- WP 2013 Flood Contour (mAHD)
- Jacobs Flood Contour (mAHD)

Peak Flood Depth (m)

- < 0.2
- 0.2 - 0.4
- 0.4 - 0.6
- 0.6 - 0.8
- 0.8 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 3.0
- 3.0 - 4.0
- > 4.0

0 100 200 400 600 800 1,000 Meters

Scale in A3

Figure 3-6: Peak Flood Inundation Comparison of Jacobs and WP Models 100 year ARI Local Tailwater Existing Conditions - E061

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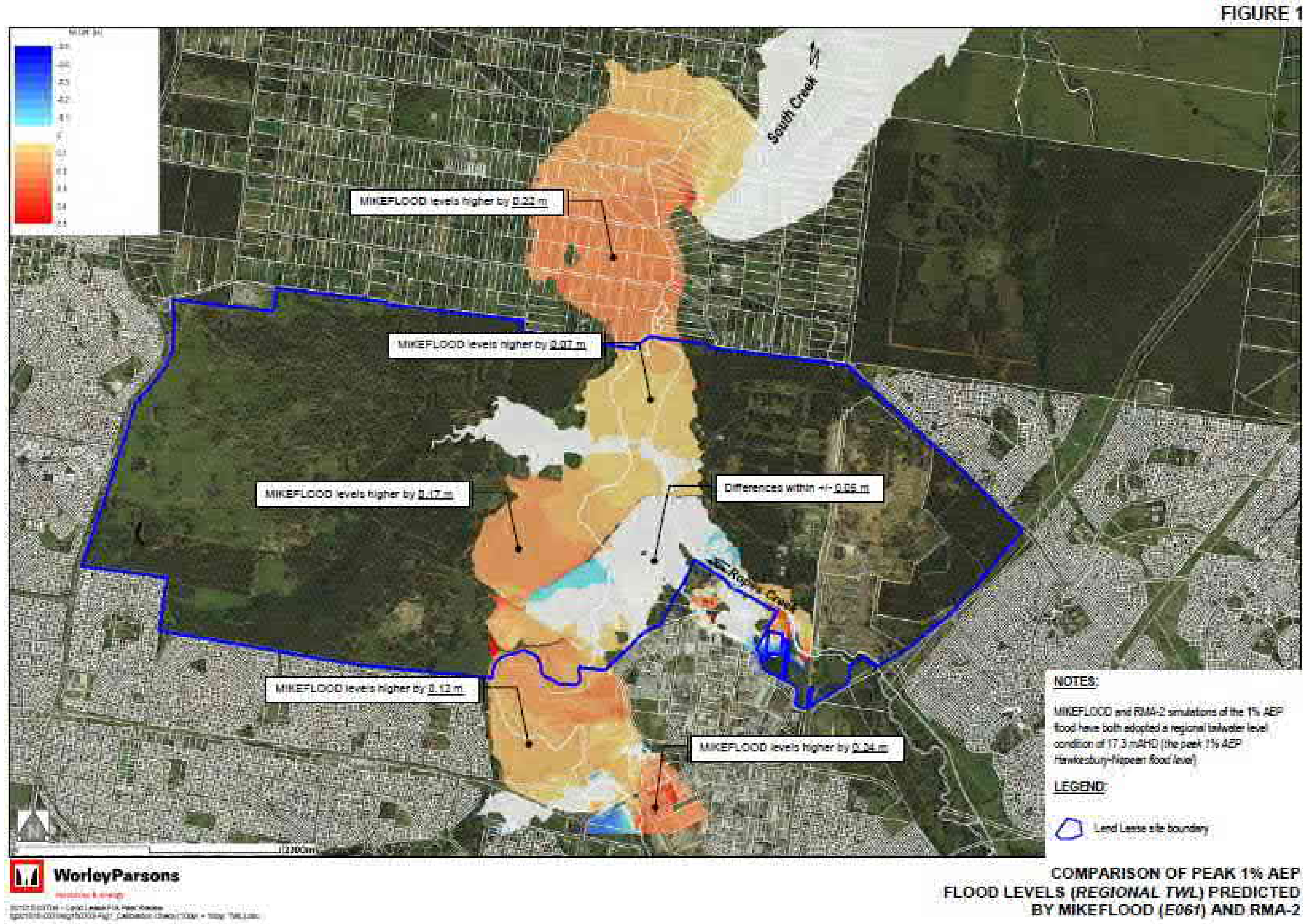

- Figure 3-7 Comparison of flood levels for 1% AEP Regional Tailwater Event (Worley Parsons, 2015)

In the Links Road industrial area, it was identified that the ALS terrain levels did not accurately reflect actual levels around one key building and detailed survey was incorporated into the MIKEFLOOD model in this area.

With detailed survey incorporated, the Jacobs flood model shows that Lot 13 DP31908 is not flooded and not located within the Flood Planning Area (FPA) in either existing or developed scenarios. This lot is identified as flooded by Worley Parsons RMA-2 model and was originally identified as flooded in the Jacobs model when only ALS data was used to represent terrain levels in this area. It is considered that the Jacobs model provides a more accurate representation of the flood extent within this area.

3.10 Conclusions on model calibration

The model developed by Jacobs for the purpose of this study was found to have good agreement with the flood model produced by Worley Parsons. It was considered a consistent tool for the assessment of the potential flood impacts of the Dunheved and Central Precincts, as required by PCC.

Some small differences exist in the representation of hydraulic structures; however, these differences are not expected to influence the ability of the model to assess the impacts of development scenarios.

The independent peer review by Worley Parsons concluded that “*the MIKEFLOOD model predicts peak 100 year ARI flood levels that are generally in good agreement with the RMA-2 modelling results that were generated for the South Creek Flood Study (2015), particularly within the subject site*”.

4. Existing flood behaviour

4.1 General

The project sites spans an area of floodplain across South Creek and Ropes Creek in St Mary's, west of Sydney.

Current land use at the project site is currently a mixture of open space and remnant vegetation. The areas upstream of the project site are made up of open space, residential and commercial areas. The area downstream of the project site consists of less densely populated rural residential area.

The existing flooding conditions have been characterised for the following six events:

- 5% AEP with local tailwater
- 5% AEP with regional tailwater
- 1% AEP with local tailwater
- 1% AEP with regional tailwater
- PMF with local tailwater
- PMF with regional tailwater

4.2 Long section profiles

Long section profiles of both South and Ropes Creek were created for the six existing case flood events. The long section distance reflects the flow path length at the invert of the channel.

4.2.1 South creek

Within South Creek, the long section profile was taken along the centre line of the creek from Christie Street to just downstream of Richmond Road. The long section profiles for the six events are shown in Figure 4-1.

4.2.2 Ropes Creek

Within Ropes Creek, the long section profile was taken along the centre line of the creek from Ropes Crossing Boulevard to the confluence with South Creek. The Long section profiles for the six events are shown in Figure 4-2.

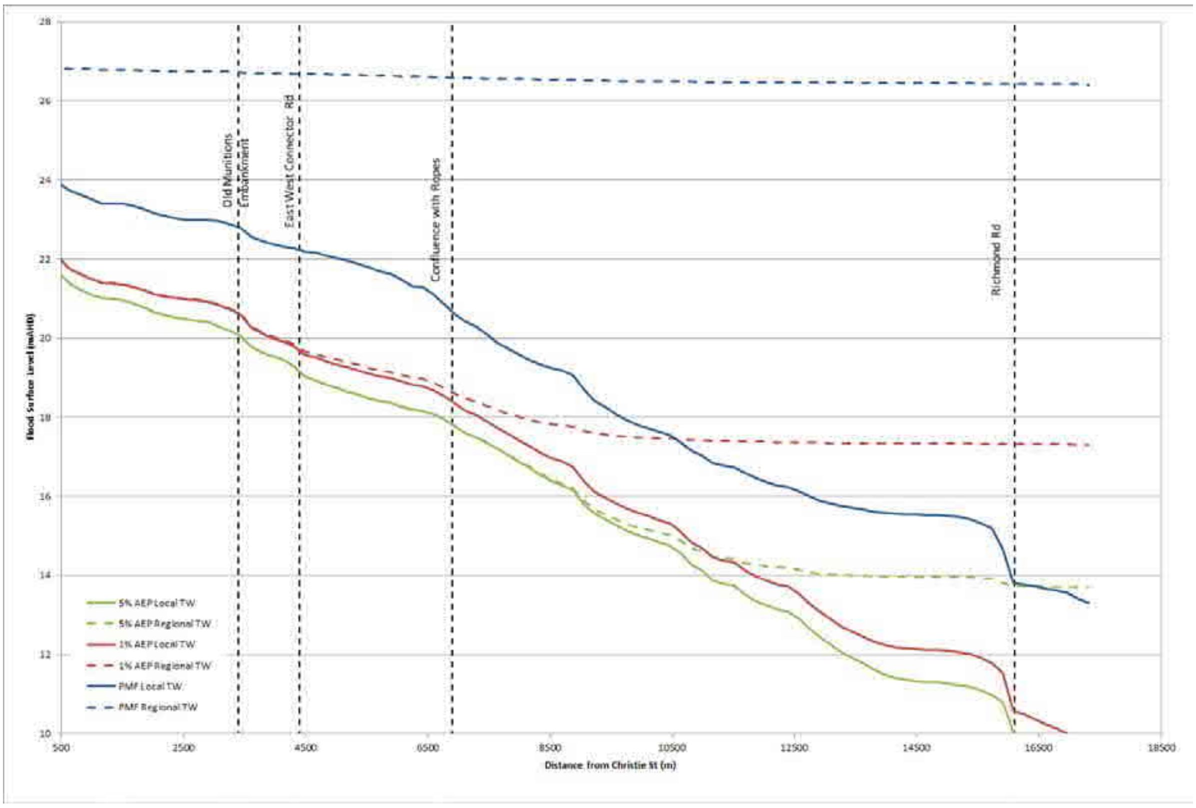

Figure 4-1 Long section profile in South Creek for existing flooding conditions (E061)

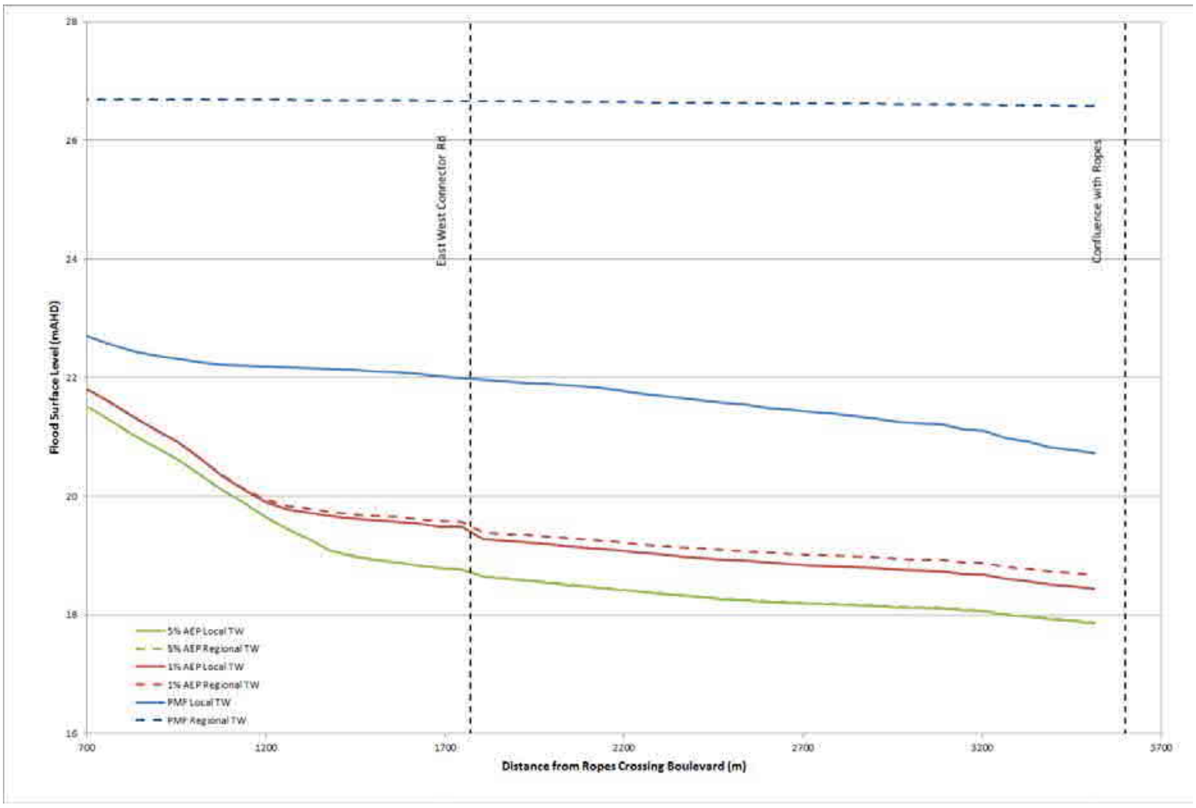

Figure 4-2 Long section profile in Ropes Creek for existing flooding conditions (E061)

4.3 Flood extents and flood level contours

Peak flood inundation depth and water surface level contours were produced for the six existing case flood events. Maps displaying these inundation extent results are presented in **Appendix A** of this report.

4.4 Velocity

Peak velocities were extracted from the MIKEFLOOD results for the six flood events discussed above. Maps displaying the peak velocities for the six events are presented in **Appendix B** of this report.

4.5 Flow intensity (Velocity Depth Product)

The NSW Government's Floodplain Development Manual (2005) (the Manual) identifies three hydraulic categories within the floodplain:

- Floodways, defined as "areas conveying a significant proportion of the flood flow … where partial blocking will adversely affect flood behaviour to a significant and unacceptable extent."
- Flood storage areas, defined as "areas outside floodways which, if completely filled with solid material, would cause peak flood levels to increase anywhere by more than 0.1m and/or would cause the peak discharge anywhere downstream to increase by more than 10%."
- Flood fringe areas, defined as "the remaining area affected by flooding".

The South Creek Flood Study (Worley Parsons, 2015) included hydraulic category mapping for the South Creek floodplain through an iterative process based on Velocity X Depth product mapping and encroachment analysis.

It was not possible to reproduce this iterative process as part of this Flood Impact Assessment. However, an assessment of the peak velocity-depth product was used as a proxy to further verify the flooding behaviour found in the Jacobs model against the flood behaviour in the RMA-2 model. Mapping of the velocity-depth product for the Existing (E061) and Developed (D209) scenarios is shown in **Appendix C** for the 1% AEP regional tailwater event.

5. Flood assessment for development scenario

5.1 Proposed Development

Lend Lease proposes to develop the Dunheved and Central Precinct areas of the St Mary's Project. The proposed development includes filling within the floodplain, modifications to road embankments and hydraulic structures and some channel works within South Creek.

5.2 Developed case modelling

The current proposed development of the Dunheved and Central Precinct areas has been modelled in the MIKEFLOOD model to assess potential flooding impacts.

The model bathymetry and representation of specific hydraulic structures has been modified to represent the proposed developed conditions.

The modelled head loss across each structure was expressed as a multiplier of the velocity head through the structure and found to be within reasonable bounds. It can therefore be concluded that the 1D-2D linkages are appropriately representing flow through the structures in the developed case model.

5.2.1 Proposed Design

The preferred proposed design (D209) has been represented in the following way:

- inclusion of the proposed bulk earthworks DEM for Central Precinct into the model;
- inclusion of the Dunheved fill into the model at levels at or above the developed Flood Planning Level;
- removal of the abutment and embankments of Old Munitions Rd;
- removal of the stockpiles on the north-western South Creek floodplain;
- raising the crest level of the East West Connector Road to the 1% AEP level with no freeboard (crest levels vary from ~19.7 – 20.4 mAHD);
- expansion of the South Creek waterway opening under the East West Connector (120 m top width);
- inclusion of the Dunheved Link Road with a crest level to provide 1% AEP immunity with no freeboard;
- inclusion of an M-Lock bridge under the Dunheved Link (~150 m wide);
- minor re-shaping of the opening at the Dunheved Link to provide a trapezoidal opening with an invert at ~18 mAHD;
- inclusion of a landscaped berm at the southern end of the Dunheved fill area; and
- inclusion of a bank of 9/ 4200 X 2700 mm RCBC under the East West Connector to the west of the South Creek Bridge with an upstream invert of 16.8 mAHD.

The layout of the proposed fill and the location of the modifications described above are shown in Figure 5-1.

A fill levels of 21.1 mAHD has been adopted to represent the finished platform level of the Dunheved fill area

Some terrain modification will be required associated with the proposed M-lock bridge under the Dunheved Link Road. However, this area is not a formed channel and should require limited if any clearing. Flood impacts upstream of the site are sensitive to changes in the waterway opening available under the Dunheved Link.

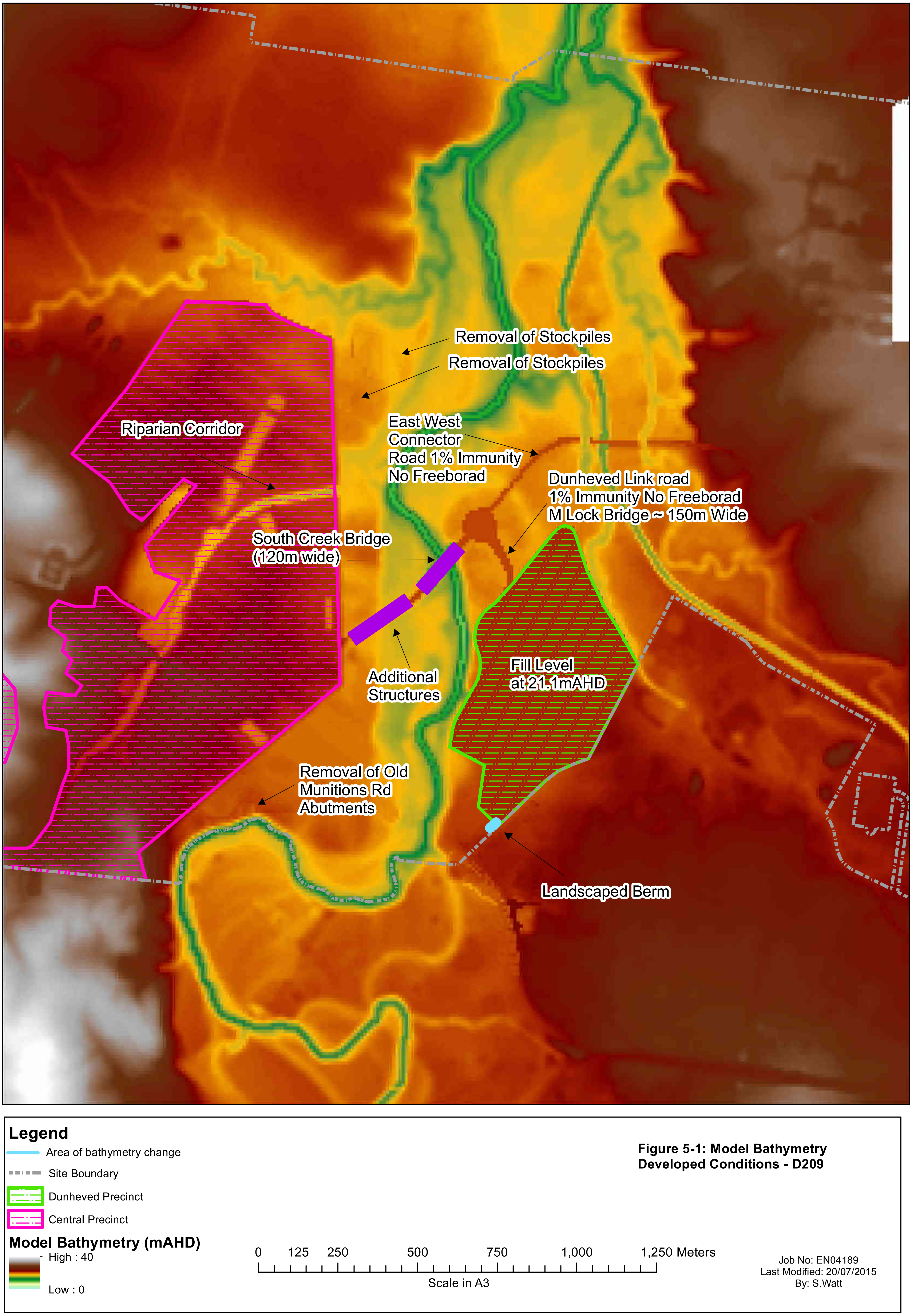

Figure 5-1: Model Bathymetry Developed Conditions - D209

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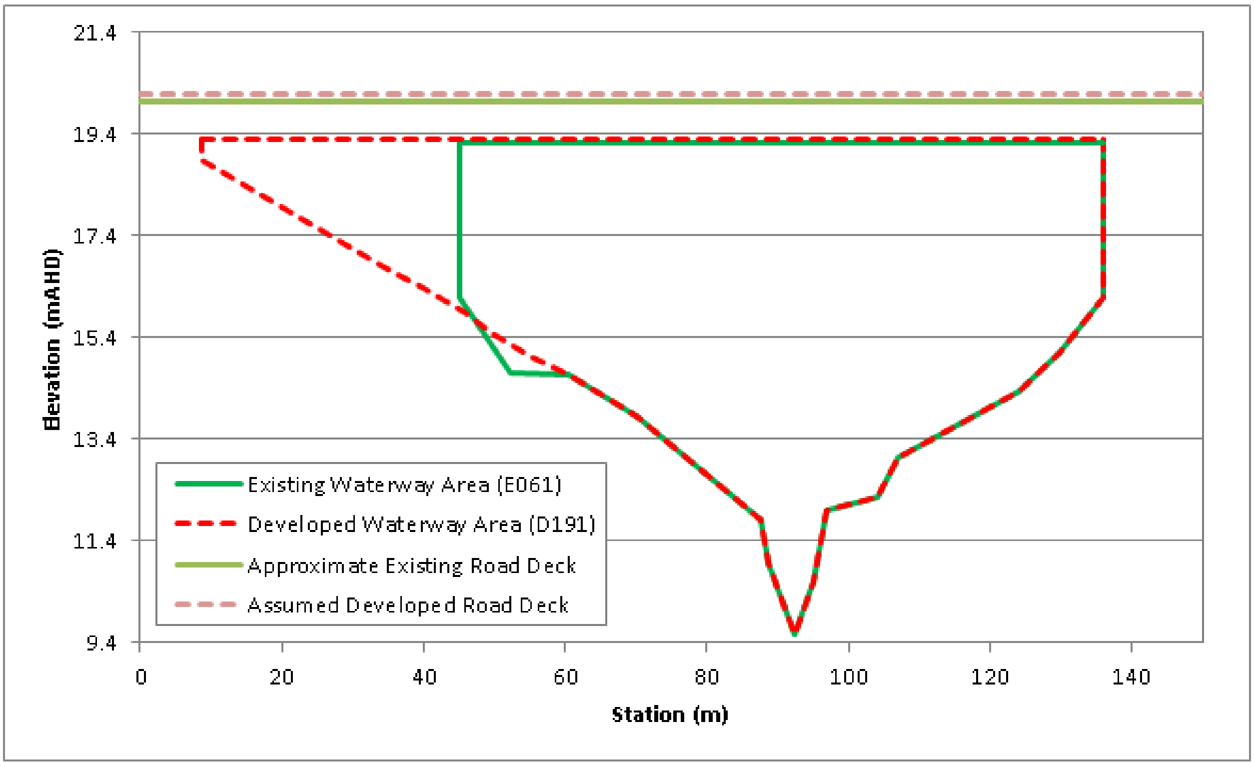
The increases to the South Creek Bridge waterway opening mimic as far as possible the existing waterway opening, increasing the structure opening for high flows on the left bank. Figure 5-2 shows a cross sectional representation of the changes to the waterway opening at South Creek under the East West Connector Road while details the proposed changes to the waterway area.

■ Table 5-1 Proposed changes to waterway openings under the East West Connector Road in the developed case

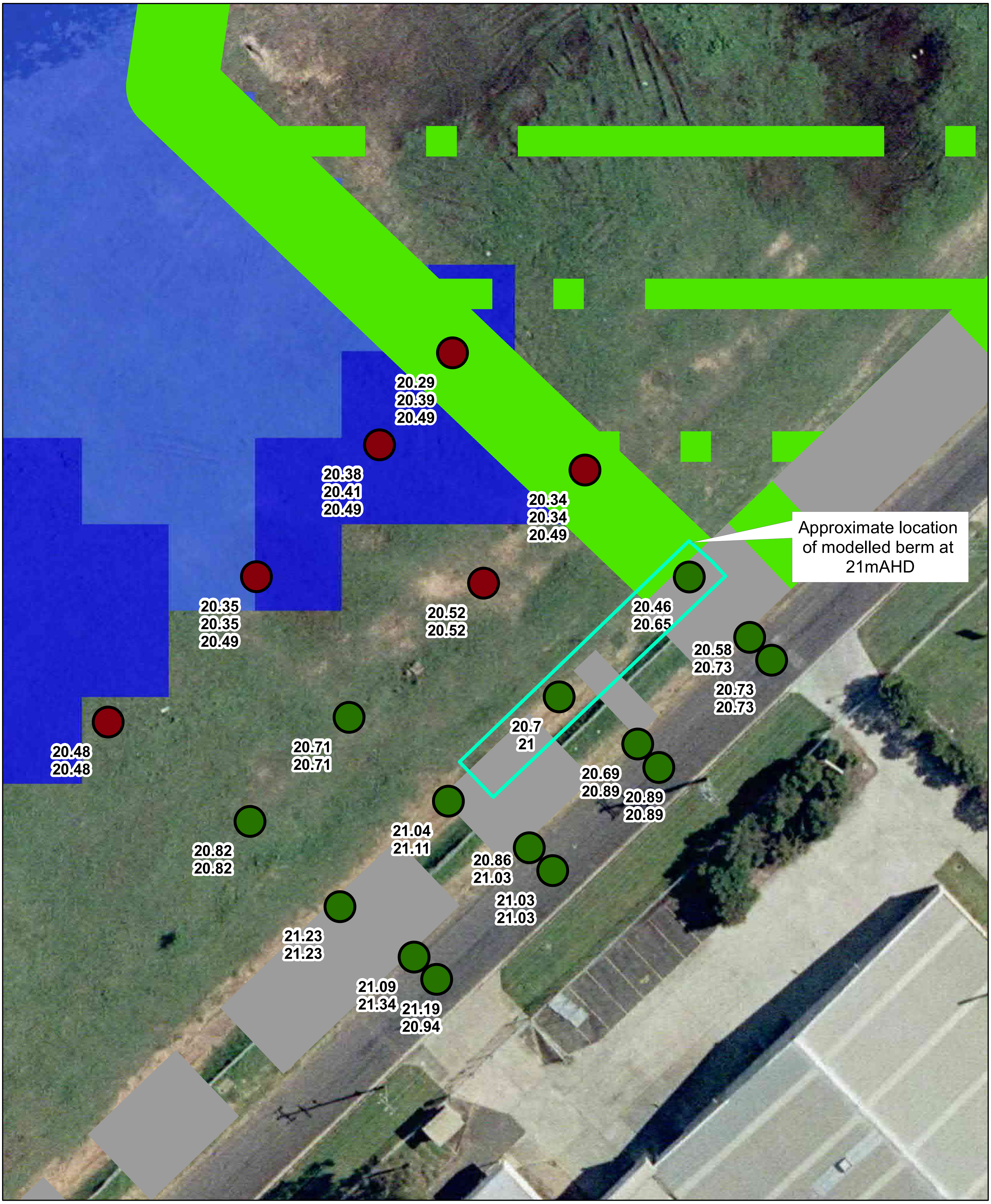
Model Definition	South Creek
Original waterway opening (m^2)	490
Approximate developed case waterway opening (m^2)	559
Developed waterway – maximum top width (m)	120



■ Figure 5-2 Representation of the East West Connector Road crossing of South Creek

A small landscaped berm has been included in the model to stop floodwaters backing up onto Links Road at the southern end of the Dunheved Precinct. The modelled berm is approximately 30 m long, 10 m wide and 300-500 mm high, with a modelled crest level of 21 mAHD. The modelled berm location, local survey and flood levels are presented on Figure 5-3.

Detailed survey around the proposed berm area shows that a localised low spot exists in Links Road and the footpaths on either side which, without mitigation, may allow flood water to flow around the end of the Dunheved fill area and onto Links Road and nearby industrial properties. No local cross-drainage structures under Links Road were identified in the survey that would allow flow to enter these properties if a berm was in place.



Legend

Site Boundary
Dunheved Precinct
Peak Flood Depth (m)
< 0.2
0.2 - 0.4
0.4 - 0.6
0.6 - 0.8
0.8 - 1.0
1.0 - 1.5
1.5 - 2.0
2.0 - 3.0
3.0 - 4.0
> 4.0

Survey Level
Model Terrain — Survey Points

Survey Level
Model Terrain
1% WSL D191 — Inundated Survey Points

0 5 10 20 30 40 50 Meters

Scale in A3

Figure 5-3: Berm Analysis
1% AEP Regional Tailwater
Developed Conditions - D209

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Based on the survey information, localised raising of a 30-40 m section of Links Road to tie into the Dunheved fill levels would likely provide a similar benefit to the modelled berm. Inclusion of detailed survey into the model has identified that this berm may not be required depending on the final configuration of the Dunheved Precinct and any associated road works to Links Road. The exact configuration of the berm or road raising can be determined during detailed design.

The developed flooding conditions for the preferred option (D209) have been modelled for the 5% and1% AEP and the PMF with local and regional tailwater conditions.

5.2.2 Proposed phasing of development (interim scenarios)

The bulk earthworks activities at the Central and Dunheved Precincts and the implementation of flood mitigation measures will be phased. A phasing plan has been prepared by Lend Lease to represent the planned chronology of fill activities and flood mitigation measures. The proposed phasing will result in a series of interim scenarios which have been modelled to estimate the potential impacts at the completion of each phase. The following four phases are planned and have been modelled.

The Phase 1 works (D210) are shown in Figure 5-4 and have been represented in the following way:

- establishment of a temporary haul road between the Links Road and the East West Connector;
- formation of a bund along the south-east boundary of the Dunheved Precinct along Links Road to exclude any temporary flood impacts to industrial properties on Links Road;
- establishment of a temporary stockpile within the Dunheved Precinct;
- filling of Stage 1 – 3 areas of Central Precinct;
- establishment of Central Precinct riparian corridor; and
- removal of the abutment and embankments of Old Munitions Rd.

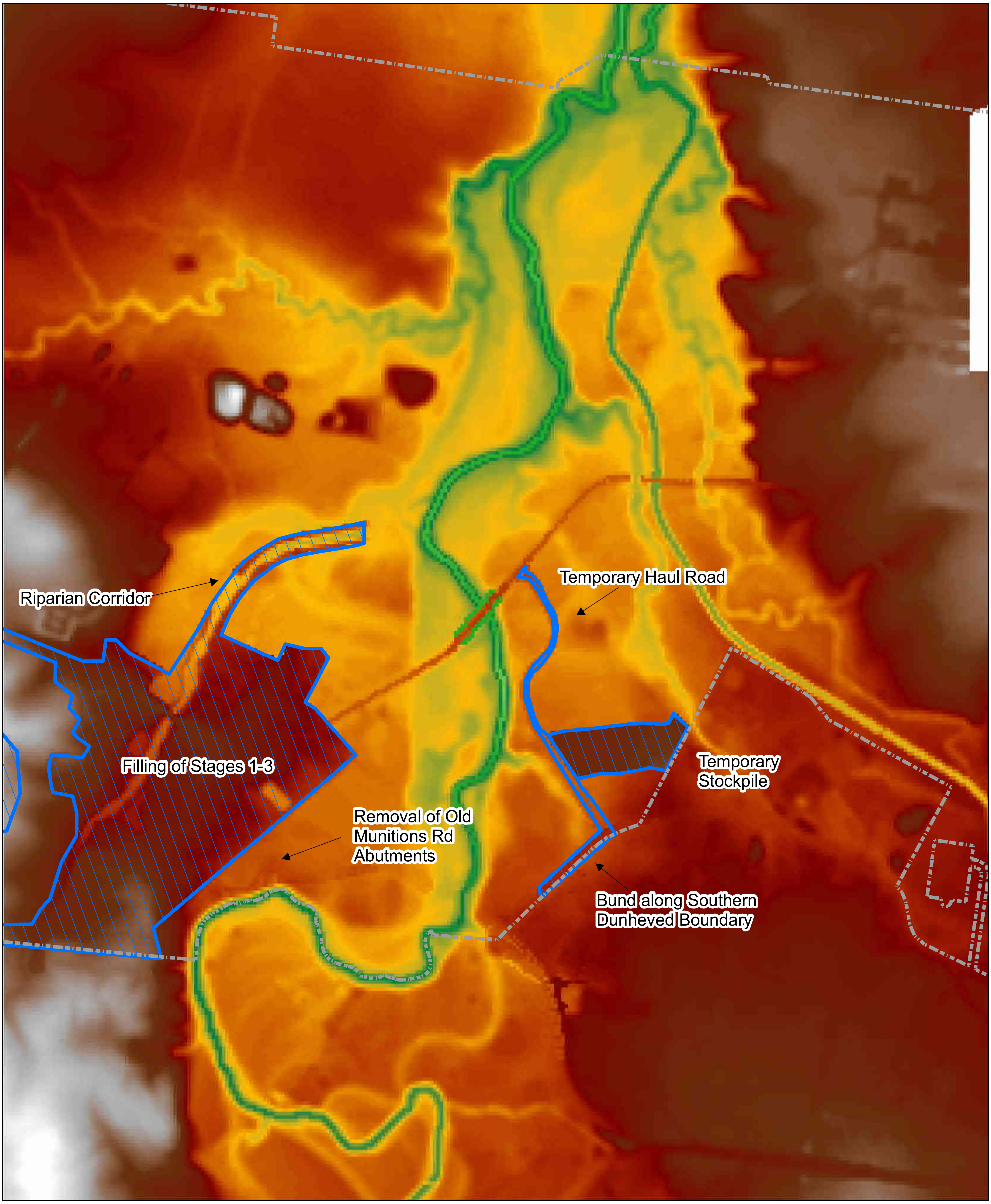
The Phase 2 works (D211) are shown in Figure 5-5 and have been represented in the following way:

- all Phase 1 works;
- filling of Stage 4-5 areas of Central Precinct; and
- removal of the stockpiles on the north-western South Creek floodplain.

The Phase 3 works (D212) are shown in Figure 5-6 and have been represented in the following way:

- all Phase 1 and 2 works;
- raising the crest level of the East West Connector Road to the 1% AEP level with no freeboard (crest levels vary from ~19.7 – 20.4 mAHD);
- expansion of the South Creek waterway opening under the East West Connector (120 m top width);
- inclusion of a bank of 9/ 4200 X 2700 mm RCBC under the East West Connector to the west of the South Creek Bridge with an upstream invert of 16.8 mAHD.
- inclusion of the Dunheved Link Road with a crest level to provide 1% AEP immunity with no freeboard;
- inclusion of an M-Lock bridge under the Dunheved Link (~150 m wide);
- minor re-shaping of the opening at the Dunheved Link to provide a trapezoidal opening with an invert at ~18 mAHD;

The Phase 4 works consist of the filling of the Dunheved Precinct. This scenario is consistent with the preferred developed scenario (D209) presented elsewhere in this report.



Legend

Site Boundary

Phase 1 Works

Model Bathymetry (mAHD)

High : 40

Low : 0

0 125 250 500 750 1,000 1,250 Meters

Scale in A3

Figure 5-4: Development Staging Phase 1 Conditions - D210

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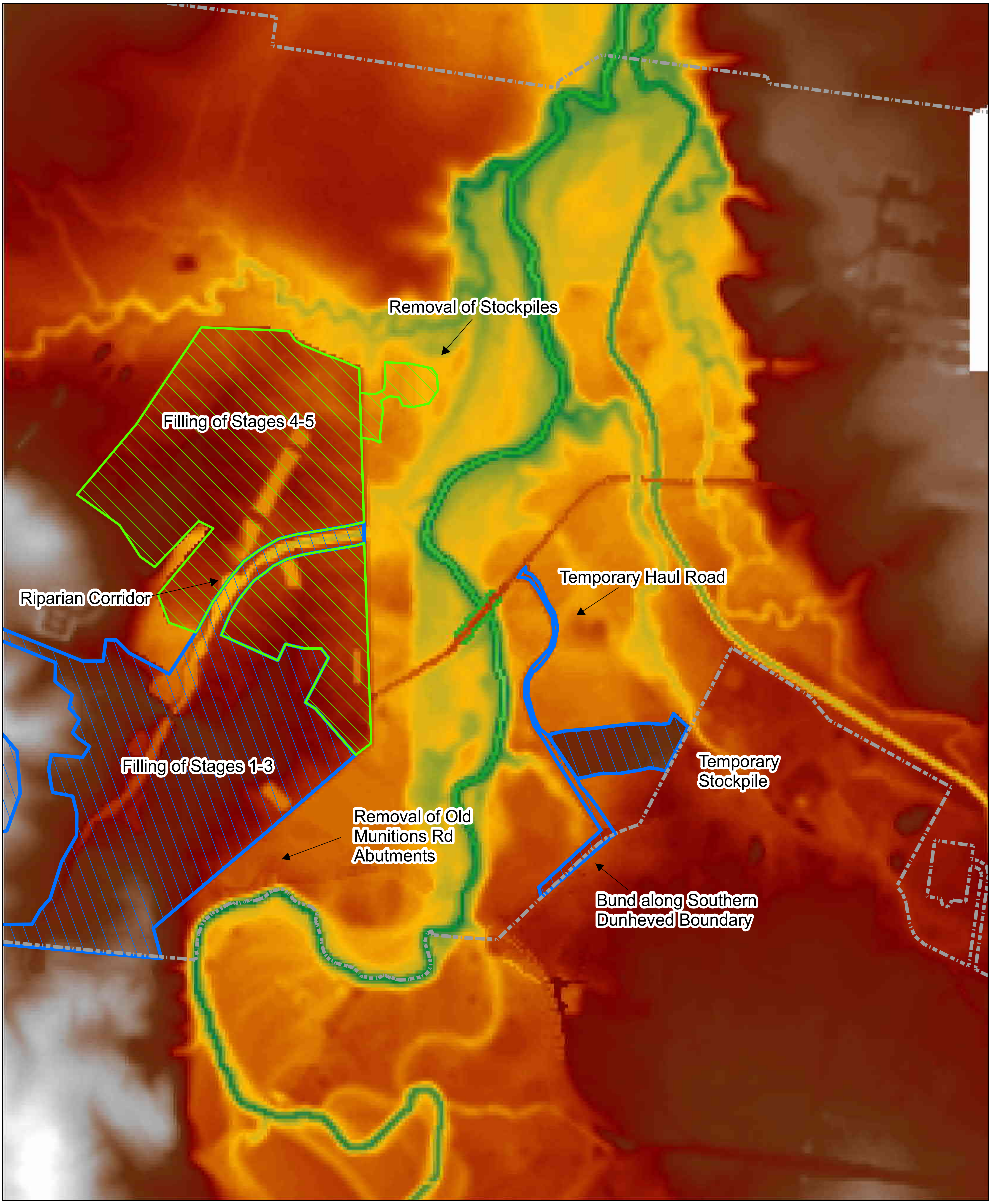

Legend

Site Boundary

Phase 1 Works

Phase 2 Works

Model Bathymetry (mAHD)

Value

High : 40

Low : 0

0 125 250 500 750 1,000 1,250 Meters

Scale in A3

Figure 5-5: Development Staging Phase 2 Conditions - D211

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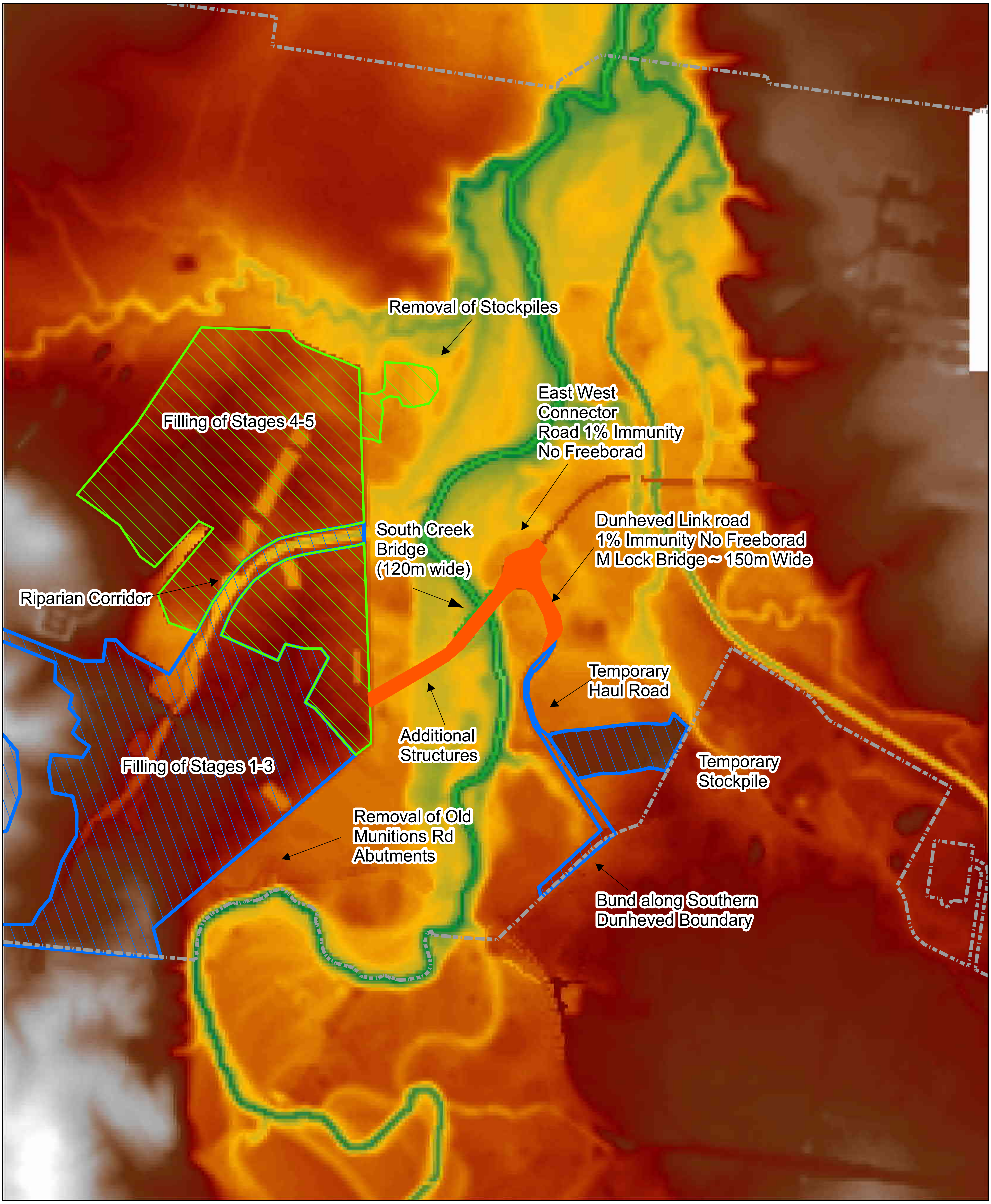

Legend

Site Boundary

Phase 1 Works

Phase 2 Works

Phase 3 Works

Model Bathymetry (mAHD)

Value

High : 40

Low : 0

0 125 250 500 750 1,000 1,250 Meters

Scale in A3

Figure 5-6: Development Staging
Phase 3 Conditions - D212

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By: S.Watt

5.2.3 Alternative options considered

An assessment of a series of alternative development options was undertaken in determining the preferred option. This included consideration of the following:

- level of immunity required for the East West Connector Road (5% and 1% AEP immunity investigated);
- maintenance of the existing East West Connector Road levels and South Creek Bridge;
- potential for wider bridge options for the East West Connector Road Bridge over South Creek;
- additional structures under the East West Connector Road to the west of the bridge over South Creek;
- realignment and widening of South Creek upstream and downstream of the East West Connector Bridge;
- a low immunity option at 19 mAHD for the Dunheved Link Road; and
- alternative structures under the Dunheved Link Road (culvert or bridge).

The preferred developed scenario (D209) was adopted based on its suitability to meet a wide-range of criteria:

- maintenance of connectivity between the Central Precinct and Dunheved areas;
- availability of alternative evacuation routes from both Central Precinct and Dunheved areas;
- minimisation of upstream and downstream flood impacts; and
- constructability and maintenance considerations.

A discussion of the flooding behaviour and associated impacts of alternative options considered can be found in Section 5.4.3.

5.2.4 Comparison with MIKE11 model (SKM, 2007)

A full comparison between the MIKE11 and MIKEFLOOD model files was undertaken to understand key differences between the two models.

The modifications and mitigation options included in the MIKE11 model were all included in the MIKEFLOOD model with the exception of widening the structure at Ropes Creek, which is no longer part of the preferred design. The representation of the modifications and mitigation options was found to be broadly consistent, with some variation due to the nature of 1D/2D models.

The key differences between the 2007 MIKE11 model and the current MIKEFLOOD model are as follows:

- the representation of the existing South Creek structure under East West Connector Road (due to the survey available at the time of model development);
- the hydrology and downstream boundary conditions (as those in the MIKEFLOOD model were adopted from the WP South Creek Flood Study); and
- the adopted assumptions around hydraulic roughness (as this parameter was modified in the MIKEFLOOD model to match the WP South Creek Flood Study results).

5.3 Developed conditions flood behaviour

Mapping of peak flood depths and velocities under developed conditions can be found in Appendix D and Appendix E respectively.

Flood behaviour is modified in the developed case model due to placement of the fill areas. This has resulted in a constriction in the waterway width, particularly in the area around the East West Connector Road. However, this constriction is partially offset by a number of hydraulic improvements within the floodplain proposed as part of the planned development. These include:

- Removal of Old Munitions Road embankment
- Removal of stockpiles on the western floodplain

- Increased waterway area through the South Creek bridge
- Additional culverts under the western section of the East West Connector

The fill in the Central Precinct causes modification to the flow patterns on the western bank of South Creek with existing overland flow paths interrupted by the proposed fill, including the filling of one drainage line that currently acts as an overflow path for South Creek in larger events. This results in localised flood impacts directly to the south of Central Precinct.

It should be noted that while the developed scenario for Central Precinct causes constriction of flows through the site, it does not change the distribution of flows as the floodplain flows as one through this section of the creek.

The East West Connector Road acts a hydraulic control within the South Creek and Ropes Creek floodplains. Under existing conditions, flow is constricted through the existing bridges and culverts under the road, sections of the road are overtopped in a 5% AEP event, and it acts as a large weir in larger events. Under developed conditions, the proposed raising of the road increases the immunity of the road and decreases the weir flow over the road. This is offset by the proposed increase in waterway area through the South Creek Bridge and additional culverts under the western section of the road. A localised increase in velocity is observed downstream of the additional structures under the East West Connector. During detailed design, further analysis will be undertaken to determine appropriate scour protection, if necessary.

During flood events, flow breaks out of South Creek, flowing east along the upstream toe of the East West Connector Road to join Ropes Creek. Under proposed developed conditions, the Dunheved Link Road which will connect the Dunheved Precinct with the East West Connector crosses this overflow path. An M-lock bridge structure has been proposed along this link to limit the constriction of this overflow path.

5.4 Potential flood impacts

The potential flood impacts of the development have been assessed through production of afflux mapping and velocity impact mapping. The regional tailwater condition produces critical flood levels throughout the model area. The 1% AEP event with regional tailwater conditions has been adopted for all detailed analyses of flood impact.

For properties where potential impacts in excess of 20 mm have been identified, more detailed survey has been obtained, and the potential impacts on over-floor flooding have been assessed.

The existing and developed Flood Planning Area (FPA) have been mapped and a comparison undertaken to identify any properties that may be impacted by additional planning constraints under the developed scenario. The FPA is an area that would be inundated in a flood event with a peak level 500 mm higher than the 1% AEP event.

5.4.1 Impacts for preferred developed scenario (D209)

Impact mapping of the six modelled flood events has been provided for the preferred option (D209), with mapping of impacts on flood depths in Appendix F and mapping of velocity impact maps in Appendix G of this report.

These afflux maps show the changes to the flow patterns due to the filling and constriction of the flood plain, and changed configuration of the East West Connector Road. The impacts of the proposed filling on flood levels in the 1% AEP flood event are generally limited to within the Central Precinct boundaries (i.e. within the land area owned by Lend Lease).

There are no newly flooded properties in the developed case for the 1% AEP flood event. That is, the flood impacts do not inundate any properties that are not affected by flooding under existing conditions.

For properties where potential impacts in excess of 20 mm were identified, more detailed survey was obtained, and it was determined that no buildings on these properties will be flooded above surveyed floor levels.

Figure 5.7 presents the potential upstream impacts for the preferred option, identifying the maximum afflux at the upstream site boundary for the 1% AEP regional tailwater conditions as 38 mm.

Increased flooding upstream of the Lend Lease site is limited to 5 lots and the Links Rd road reserve, detailed in **Table 5.2**and **Figure 5.7**. A reduction of flood levels is also experienced on several lots.

Table 5.2 Summary of impacted lots upstream

Lot	Plan	Description	Impact on Lot (mm)*
101	DP1202567	St Marys Wastewater Treatment Plant (Sydney Water)	89
20	DP773781	Dunheved Golf Course	38
1	DP234336	Dunheved Golf Course	38
1	DP600517	Sydney Water Recycled Water Scheme	33
1	DP31908	Links Road footpath	-
-	-	Links Road	18

* Impact in 1% AEP Regional TW event (D209).

An area of impact has been identified within the St Marys Wastewater Treatment Plant (WWTP) to the east of the Dunheved Precinct. This impact occurs within a breakout path from Ropes Creek which joins a formed channel with the St Marys WWTP site. The formed channel flows through a culvert at the site boundary and continues along the north-western boundary of the Dunheved Precinct, joining Ropes Creek near the East West Connector. Within the model, the north-eastern corner of the Dunheved fill area clips the channel around the culvert location, causing a localised increase in upstream water levels.

Further definition of this impact may be possible by incorporating detailed survey and structure details into the model in this area. The 10 m grid resolution within the model is coarse in comparison to the width of the formed channel and culvert, and therefore may over-estimate the impacts on this lot.

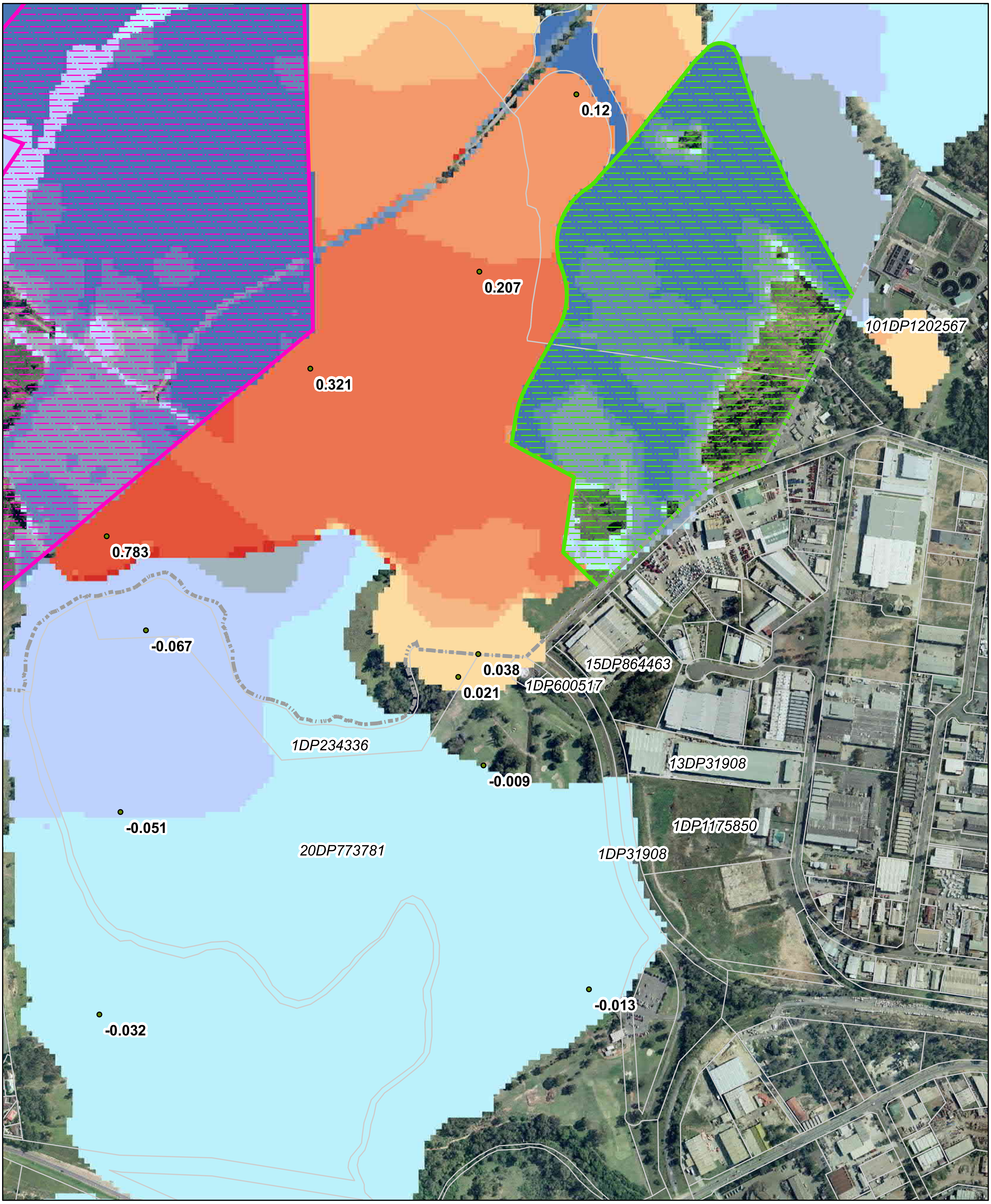
Modelled impacts within this site are generally in the order of 20-40 mm in a localised low area with a very small area of 89 mm where the channel is very constrained. No buildings are impacted. Therefore, these impacts are not considered a material impact on this site. However, if required, these impacts could likely be mitigated by minor refinements to the Dunheved fill area during detailed design.

The remainder of upstream impacts are substantially limited to within the Dunheved Golf Course. Under existing conditions, the majority of the Golf Course adjoining the site is inundated by water over 1m deep in the 1% AEP event, with depths of up to 3 m in some areas. The increase in flood levels across the Golf Course is unlikely to change the effects of flooding on the Golf Course. There are no impacts on the buildings or carpark associated with the Golf Course. The area of increased flooding on the Golf Course (~1ha) is also offset by an area (~54ha) of reduced flooding (10-70mm).

The duration of flooding on the Golf Course is unchanged under developed conditions at approximately 20 hours in the regional 1% AEP flood event. Similarly, there is no change in the duration of flooding or the timing of the arrival of flooding at the downstream site boundary in the 1% AEP flood event.

Tanks and plant, but no buildings were identified on the Sydney Water property (1 DP600517) with what appears to be a pump station elevated above ground level. No site survey was available for the site.

There are no buildings on the remaining impacted lot, Lot DP31908, which currently forms the Links Rd footpath.



Legend

- Spot Impacts (m)
- Site Boundary
- Dunheved Precinct
- Central Precinct
- Cadastre

Afflux (m)

- < -1.00
- -1.00 - -0.50
- -0.50 - -0.20
- -0.20 - -0.10
- -0.10 - -0.05
- -0.05 - -0.01
- -0.01 - 0.01
- 0.01 - 0.05
- 0.05 - 0.10
- 0.10 - 0.20
- 0.20 - 0.50
- 0.50 - 1.00
- > 1.00

Figure 5-7: Upstream Flood Impacts
1% AEP Regional Tailwater
Jacobs Model D209

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Flood impacts at the downstream boundary of the Lend Lease site in the 1% AEP event (**Appendix F**) are 11 mm and 16 mm under regional and local tailwater conditions, respectively. The characteristics of the floodplain result in relatively little attenuation, meaning these small impacts propagate at approximately the same magnitude for 400 m (regional tailwater) and to Richmond Road, which acts as a hydraulic control (local tailwater). Downstream of this, the impacts are less than 10 mm and within the accuracy limit of the model.

These impacts represent less than 0.2% of the total flood depth in the channel and do not cause a significant increase in flood extent. There are no newly flooded properties upstream or downstream in the developed case for the 1% AEP flood event. That is, the flood impacts do not inundate any properties that are not affected by flooding under existing conditions. No material flood impact is therefore expected at downstream properties.

It is noted that while the local tailwater impacts extend further, the peak levels for the regional tailwater event significantly exceed those in the local tailwater event and are therefore critical.

Existing and developed Flood Planning Areas (FPA) were generated based on the modelled peak water surface levels for the 1% AEP Regional tailwater, using WaterRide.

Council provided Jacobs with their current FPA for the Lend Lease site and upstream area as an ESRI shapefile format on 30 April 2015. A comparison was undertaken between the Council's FPA and the FPA estimated based on the existing (E061) Jacobs flood modelling. **Figure 5.8** presents this comparison, as well as the developed FPA. This comparison showed good agreement between the two FPA layers, with the two FPA lines generally within 10-20 m of each other, except for on Lot 13 DP31908 which is identified as not flooded and not within the FPA area in the Jacobs model due to the incorporation of detailed survey.

To identify the potential for property impacts based on changes to the FPA and associated Section 149, the existing and developed FPAs were intersected with the cadastre (2015) and the area affected by the FPA calculated for each property. Properties with a change in the area affected by the FPA were then identified.

Under the developed case, there are no additional properties affected by the FPA compared to existing conditions.

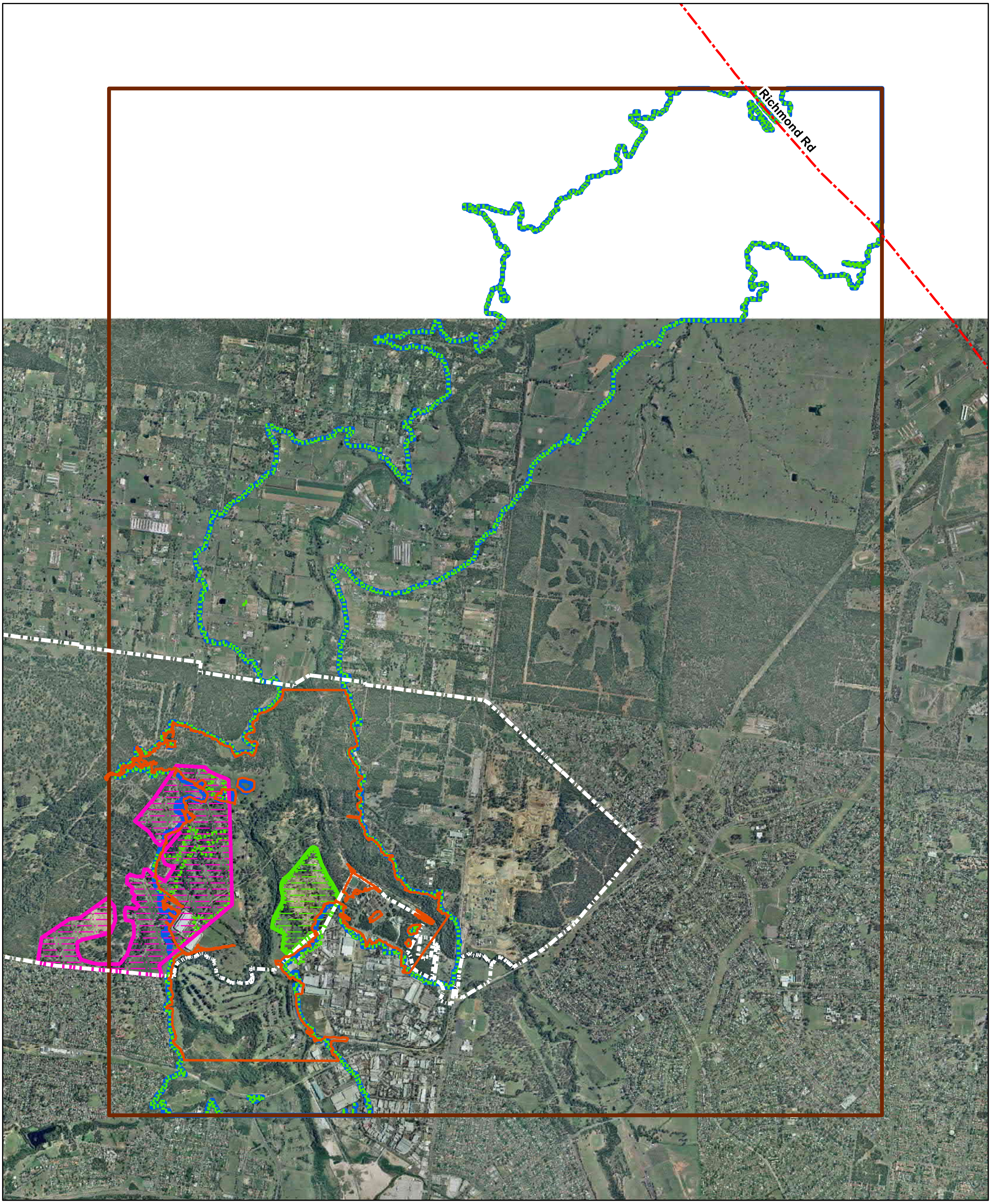
Several properties have been identified with a negligible increase in area affected by the FPA and eight (8) properties with a negligible decrease in area affected by the FPA. A negligible change has been defined as less than 5% of the total area AND less than 3 grid cells affected. Changes of this magnitude are within the tolerance of the analysis techniques.

Five (5) properties have been identified upstream of the Lend Lease site that will potentially be affected by a change in the area affected by the FPA, with four (4) impacted by an increase and one (1) by a decrease. These properties are all industrial properties located upstream of the Lend Lease site on Links Road.

Table 5.3 and **Figure 5.9** present the details of these impacted lots.

Table 5.3 Summary of lots impacted by increase in area affected by the FPA

Lot	Plan	Description	Total Lot Area (Ha)	Area Affected by FPA (Ha)		% Area Affected by FPA	
				Existing	Developed	Existing	Developed
192	DP1135763	Industrial	0.56	0.06	0.48	11%	85%
1	DP1191285	Industrial	0.90	0.24	0.58	26%	64%
44	DP1185482	Industrial	1.15	0.42	0.66	36%	57%
45	DP1185482	Industrial	1.09	0.05	0.24	5%	22%
15	DP864463	Industrial	2.84	0.81	0.76	28%	27%



Legend

Roads
Site Boundary
MIKEFlood Extent
Council Existing FPA
Dunheved Precinct
Central Precinct
Existing Model FPA
Developed Model FPA (D209)

0 300 600 1,200 1,800 2,400 3,000 Meters

Figure 5-8: Flood Planning Area (FPA) Comparison - Overview

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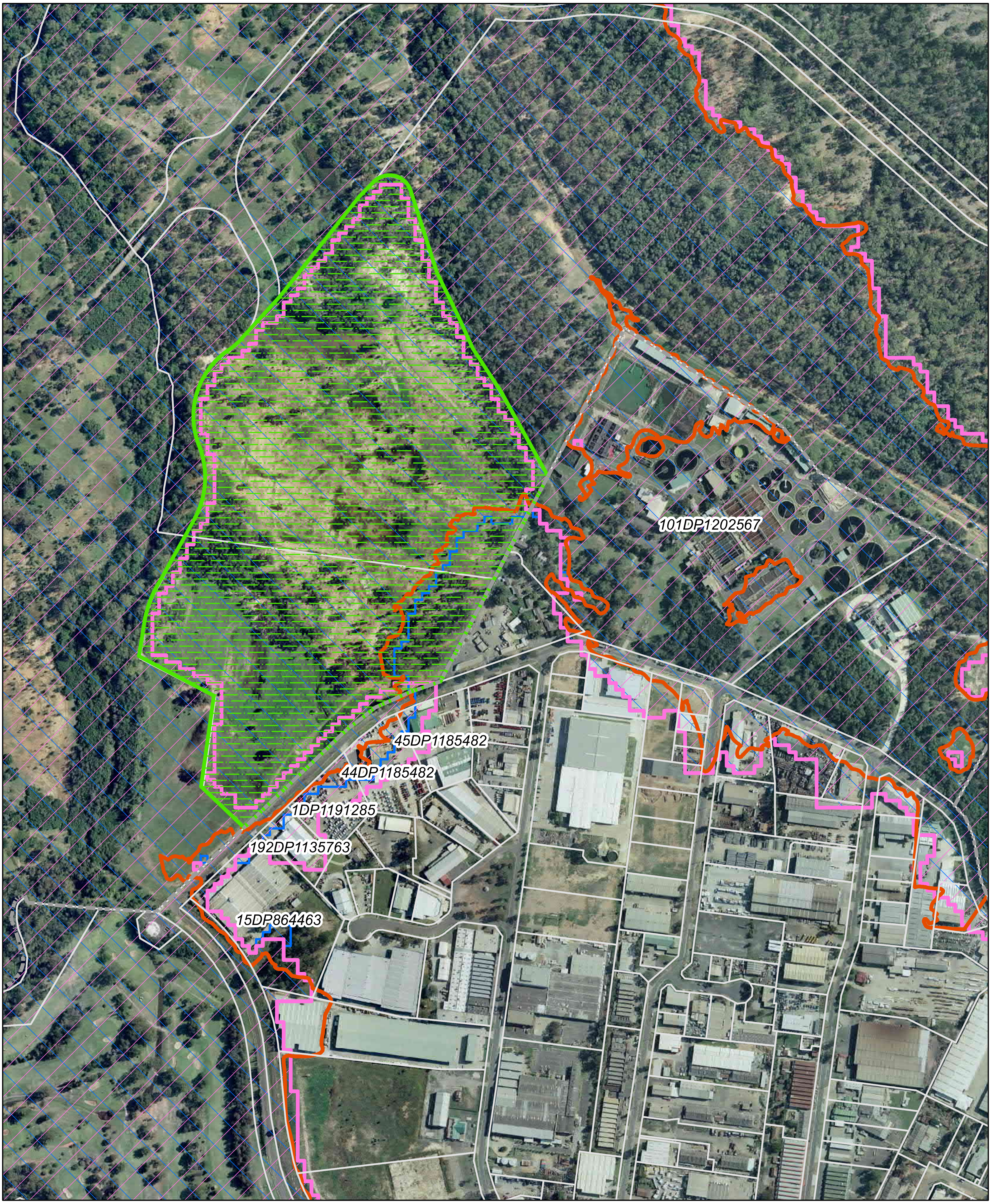

Legend

- Site Boundary
- Dunheved Precinct
- Cadastre
- Council Existing FPA
- Developed Model FPA (D209)
- Existing Model FPA

0 50 100 200 300 400 500 Meters

Scale in A3

Figure 5-9: Flood Planning Area (FPA) Comparison - Upstream

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Maps detailing the changes in velocity are provided in **Appendix G** of this report. Velocity mapping for the existing and developed scenarios can be found in **Appendix B** and **Appendix D** respectively.

Existing peak velocities at the downstream site boundary vary between 0.8-1 m/s within the channel and are generally less than 0.8 m/s on the floodplain. This same velocity profile is maintained under developed conditions with a maximum velocity increase of 0.01 m/s under developed conditions (local and regional tailwater conditions). These velocity changes at the downstream boundary of the site represent less than a 10% increase, generally in the order of 1-5%.

At the upstream site boundary, there is a minor change to the velocity profile due to the increased conveyance along the western bank of South Creek where the Old Munitions Road embankment and abutment will be removed. This embankment is currently acting as a localised levee limiting flow on the western bank. When it is removed, this causes increases on the western bank and decreases on the eastern bank of South Creek of up to 0.02 m/s in the 1% AEP event.

5.4.2 Impacts during development (interim scenarios)

To reflect the programming of work, three interim phases of development were modelled to assess the potential impacts during each phase prior to full development of the preferred developed scenario D209.

The impacts of Phases 1 – 3 of development are presented in Figure 5.10 to Figure 5.12 for the 1% AEP Regional Tailwater event. These afflux maps show the changes to the flow patterns during each Phase of development.

The impacts at the upstream and downstream site boundaries are identified in Table 5.4 for each Phase of development including the final developed scenario.

This demonstrates that the flood impacts during development are consistent with or less than those of the fully developed scenario. Specifically, the modelling demonstrates that the Central Precinct area can be fully developed prior to the upgrade works to the East West Connector without impacts materially exceeding, within the sensitivity of the model, those of the preferred developed option.

Table 5.4 Summary of flood impacts during development (1% AEP Regional Tailwater)

Phase of Development	Model Scenario	Upstream Impact (mm)	Downstream Impact (mm)
Phase 1	D210	-0.052	+0.006
Phase 2	D211	-0.008	+0.012
Phase 3	D212	-0.035	+0.009
Phase 4	D209	+0.038	+0.011

5.4.3 Impacts for alternative options

A range of alternative options were considered and modelled to arrive at the preferred developed option.

As a key hydraulic control within the floodplain, the configuration of the East West Connector Road and associated hydraulic structures was investigated. The preferred option (D209) has the road at a level to provide immunity in the 1% AEP with no freeboard. Under this option, the South Creek Bridge is widened from a top width of approximately 90 m to 120 m and an additional bank of 9/ 4200 X 2700 mm box culverts added under the section of the East West Connector to the west of the bridge. This configuration causes 38 mm of potential impact at the upstream site boundary.

In comparison, with no additional culverts under the East West Connector, the potential impacts at the upstream site boundary are 145 mm, while with 4/ 4200 X 2700 mm box culverts impacts are 85 mm.

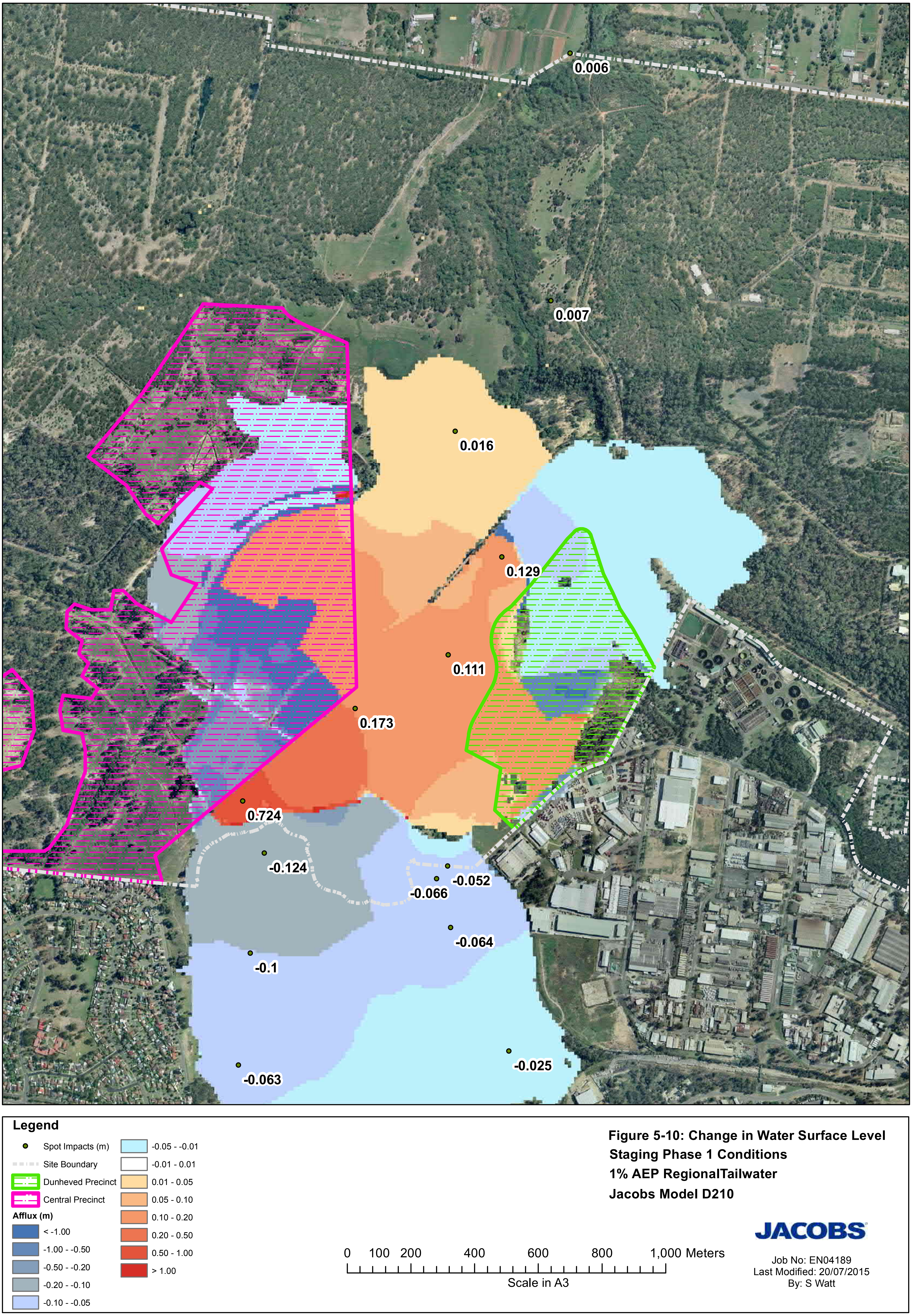
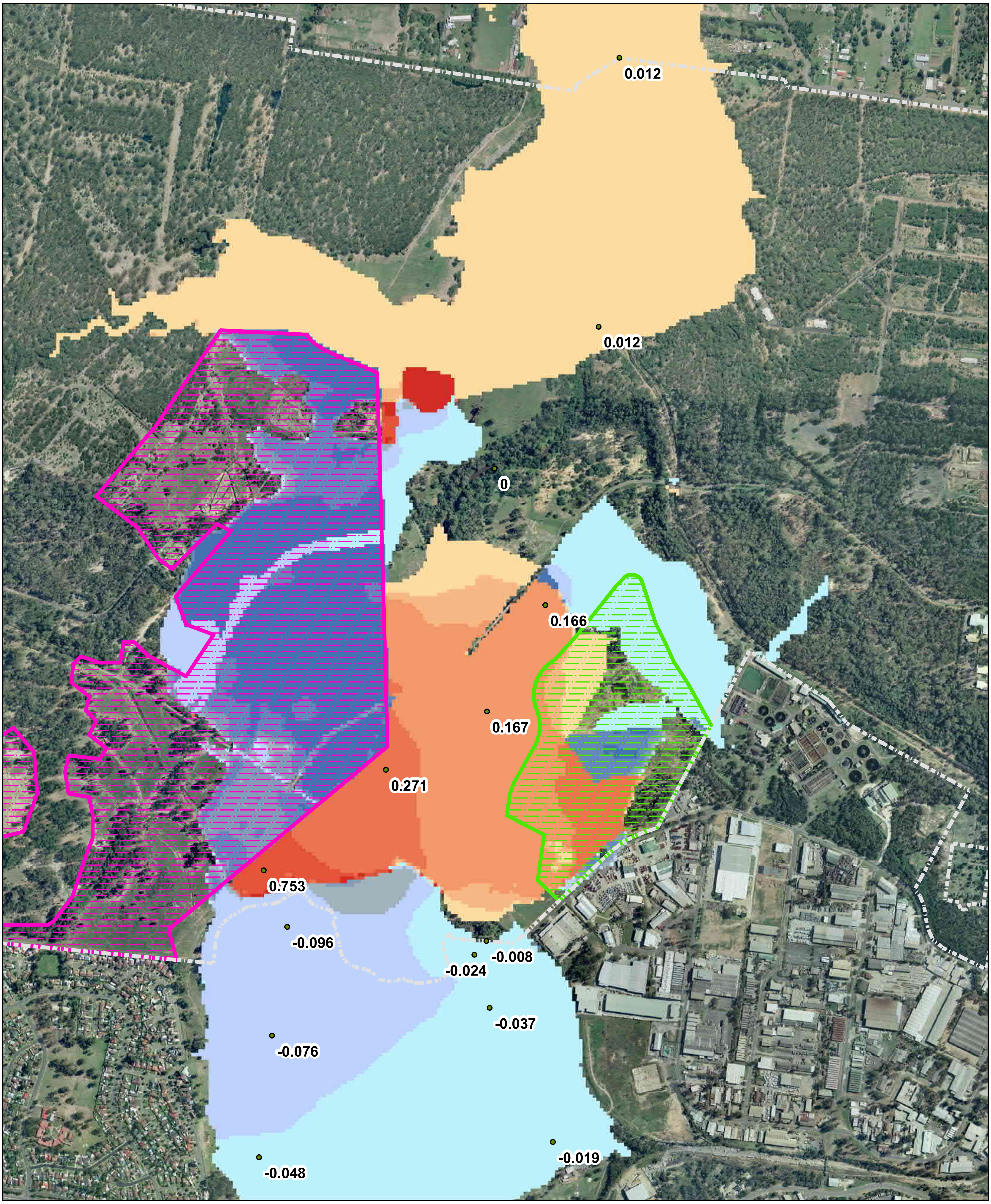


Figure 5-10: Change in Water Surface Level
Staging Phase 1 Conditions
1% AEP RegionalTailwater
Jacobs Model D210



Legend

- Spot Impacts (m)
- Site Boundary
- Dunheved Precinct
- Central Precinct

Afflux (m)

< -1.00	-0.05 - -0.01
-1.00 - -0.50	-0.01 - 0.01
-0.50 - -0.20	0.01 - 0.05
-0.20 - -0.10	0.05 - 0.10
-0.10 - -0.05	0.10 - 0.20
	0.20 - 0.50
	0.50 - 1.00
	> 1.00

0 100 200 400 600 800 1,000 Meters

Scale in A3

Figure 5-11: Change in Water Surface Level
Staging Phase 2 Conditions
1% AEP RegionalTailwater
Jacobs Model D211

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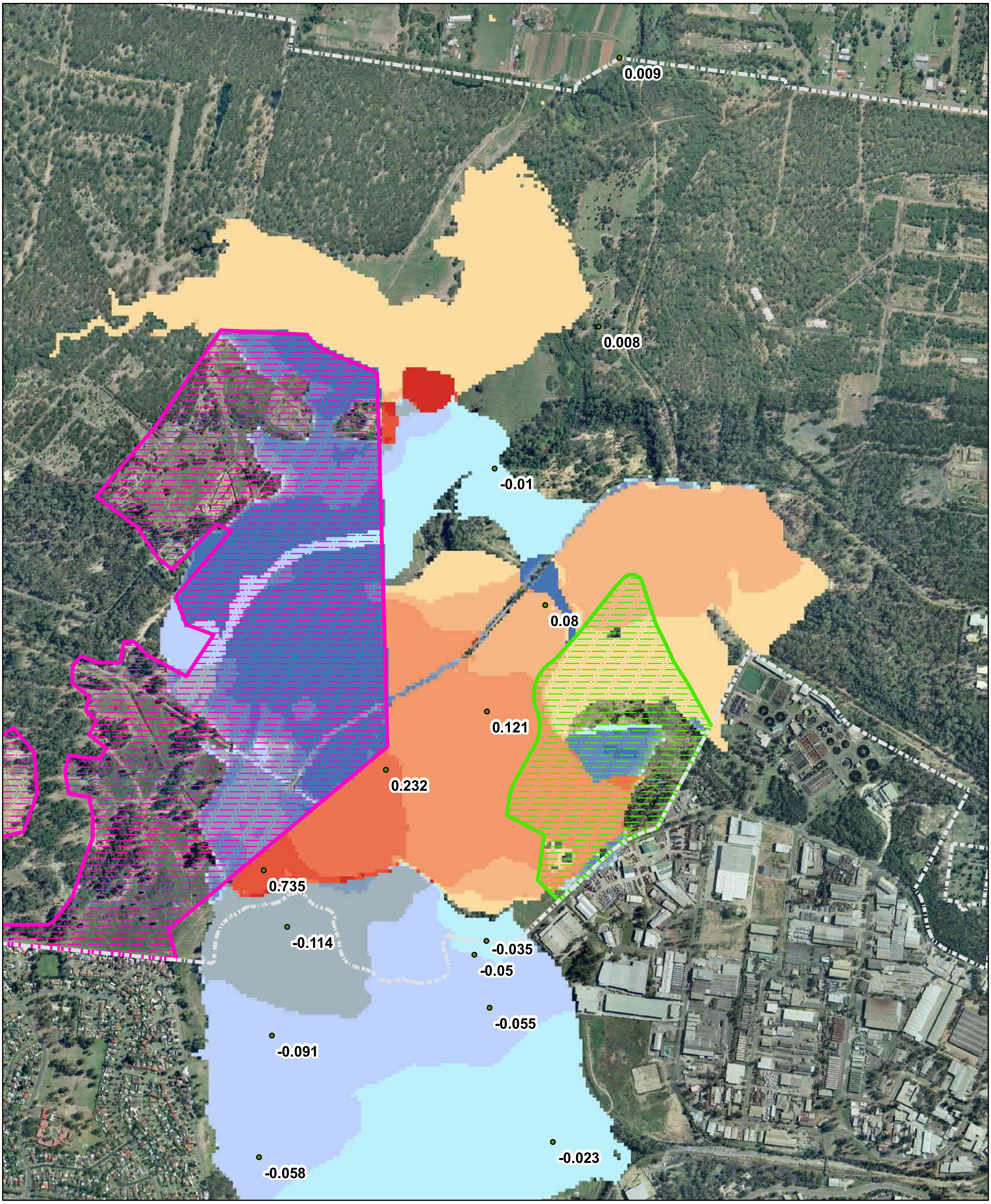
By: S Watt



Legend

Spot Impacts (m)
Site Boundary
Dunheved Precinct
Central Precinct

Afflux (m)

< -1.00
-1.00 - -0.50
-0.50 - -0.20
-0.20 - -0.10
-0.10 - -0.05
-0.05 - -0.01
-0.01 - 0.01
0.01 - 0.05
0.05 - 0.10
0.10 - 0.20
0.20 - 0.50
0.50 - 1.00
> 1.00

0 100 200 400 600 800 1,000 Meters
Scale in A3

Figure 5-12: Change in Water Surface Level
Staging Phase 3 Conditions
1% AEP RegionalTailwater
Jacobs Model D212

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By: S Watt

With the East West Connector at 19.9 mAHD, the road has 5% immunity with 500 mm freeboard and is likely to be trafficable in the 1% AEP. In this scenario, with similar bridge widening but no additional structures under the East West Connector, upstream impacts at the boundary are in the order of 85 mm.

The potential to minimise upstream flood impacts via channel widening through the East West Connector and channel realignment upstream and downstream of the bridge. This was modelled as an additional 10 m channel width for 100 m upstream and downstream of the bridge. Channel widening was tested with a range of other configurations and was found to decrease upstream flood impacts by only 10 mm. This requires significant earthworks within the waterway corridor with potential high environmental impacts.

Several alternative road and structure configurations were assessed for the Dunheved Link road. These included a low immunity option, culvert option and M-lock bridge option. Inclusion of the Dunheved Link road in any form adds at least 20 mm of upstream impact. This impact is sensitive to the size of the waterway area available under the road. Some terrain modification will be required to provide the required waterway area. However, this area is not a formed channel and should require limited if any clearing. The preferred option includes an M-lock bridge structure. The exact road and structure configuration will be determined during detailed design.

The preferred developed option (D209) provides a balance between the flood immunity provided and upstream/downstream flooding impacts. The preferred developed option demonstrates that an appropriate balance of flood immunity, flood impacts, constructability and environmental impact is achievable. Furthermore, the preferred developed option can be implemented without unacceptable impacts occurring during development prior to upgrade of the East West Connector.

The exact structure configurations will be determined and modelled in more detail during detailed design.

5.4.3 Hydraulic Category Mapping

The NSW Government's *Floodplain Development Manual* (2005) (the Manual) identifies three hydraulic categories within the floodplain:

- Floodways, defined as "*areas conveying a significant proportion of the flood flow … where partial blocking will adversely affect flood behaviour to a significant and unacceptable extent.*"
- Flood storage areas, defined as "*areas outside floodways which, if completely filled with solid material, would cause peak flood levels to increase anywhere by more than 0.1m and/or would cause the peak discharge anywhere downstream to increase by more than 10%.*"
- Flood fringe areas, defined as "*the remaining area affected by flooding*".

The definition of these hydraulic categories provides a guide for best practice floodplain risk management. However, the Manual does not prohibit filling within the floodway or flood storage area. It advocates a merit-based approach which takes into consideration social, economic and ecological factors, as well as flooding characteristics.

The *South Creek Flood Study* (Worley Parsons, 2015) included hydraulic category mapping for the South Creek floodplain through an iterative process based on Velocity X Depth product mapping and encroachment analysis. Parts of the Central Precinct and Dunheved fill platforms lie within areas identified by this study as floodway or flood storage.

The NSW Government's *Floodplain Risk Guideline – Floodway Definition* (2007) identifies that approximate limits of the floodway can be identified through an iterative method by modelling altered cross-sections (simulating filling of the floodplain) and examining whether:

- There is a significant affect [sic] on upstream flood levels; and/or
- There is a significant diversion to an existing flowpath; and/or
- A significant new flowpath or floodway develops due to the change.

While this process is aimed at identifying the extents of the floodway, it can equally be applied to identifying whether filling of the floodplain constitutes acceptable encroachment or not.

Without mitigation, the filling of the Central Precinct and Dunheved areas constricts the floodplain and produces impacts in excess of 100mm upstream of the site boundary. However, there are a number of hydraulic improvements to the floodway proposed as part of the planned development to partially counter the loss of conveyance due to filling in the floodway areas. These include:

- removal of Old Munitions Road embankment;
- removal of stockpiles on the western floodplain;
- increased waterway area through the South Creek bridge; and
- additional culverts under the western section of the EWC road.

An analysis of the conveyance of South Creek upstream, downstream and through the site has been undertaken. This assessment demonstrates that the proposed measures included within the development to manage encroachment in the floodplain would maintain the conveyance capacity through the South Creek floodplain.

The unit flow (Velocity X Depth Product) was plotted for a series of cross-sections along South Creek. A comparison of the existing (E061) and developed (D209) unit flow across the floodplain demonstrates that the conveyance upstream and downstream of the site is not impacted by the proposed development. Within the site, the analysis demonstrates that the reduction in conveyance through the fill platforms would be partially offset by the increased conveyance provided through the compensatory measures detailed above.

The other key issue relevant to merit-based consideration of filling in a floodway area is cumulative fill based on assumptions regarding possible future filling scenarios. The Lend Lease site encompasses the width of the South Creek floodplain through this reach. The Central Precinct and Dunheved fill areas form part of a larger regional plan which includes handover of the residual floodplain areas to the NSW Government to manage as National Park. Therefore, future filling and further encroachment within this section of South Creek is highly unlikely. Hence, this assessment can be considered to be a cumulative assessment of the total fill possible in this reach of the floodplain.

This analysis for the proposed development scenario (D209) has shown that:

- There is NOT a significant affect [sic] on upstream flood levels; and
- There is a NOT a significant diversion to an existing flowpath; and
- A significant new flowpath or floodway DOES NOT develop due to the change.

6. Conclusions and recommendations

The following conclusions and recommendations are made from this investigation.

- A new MIKFLOOD hydraulic model was developed for the St Mary's area. The purpose of this model was to meet PCC's requirement to assess the proposed developments at Central Precinct and the Dunheved development in a manner consistent with Worley Parsons RMA-2 model.
- The MIKEFLOOD model produced design event peak flood levels consistent with those produced by the Worley Parsons RMA-2 model for existing flooding conditions. The MIKEFLOOD model was also deemed to be adequately consistent with the previous MIKE11 model (SKM 2007).
- The model was used to assess the proposed developed scenario including proposed fill layout, upgrades to the East West Connector Road, widening of the South Creek Bridge and addition of high flow culverts on the west bank, addition of Dunheved Link, removal of the abutments and embankment at Old Munitions Road and removal of existing stockpiles on the western bank of South Creek.
- Several alternative options have been modelled to identify the preferred option. The preferred developed option demonstrates that an appropriate balance of flood immunity, flood impacts, constructability and environmental impact is achievable. Exact structure configurations will be determined and modelled in more detail during detailed design.
- Three interim scenarios have been modelled based on detailed planning of the proposed phasing. This modelling demonstrates that development of the preferred option can be phased such that impacts during development do not materially exceed the impacts of the final development.
- This modelling takes into account the proposed filling of the Central Precinct together with the approved filling of the North and South Dunheved Precincts.
- The proposed development in its current form produces water level impacts limited to 38 mm at the upstream site boundary and 11 mm at the downstream site boundary in the 1% AEP regional tailwater event.
- The impacts of the proposed development meet PCC's Development Control Plan (DCP) requirements of afflux not exceeding 100 mm at the upstream boundary (noting that the DCP is not explicitly applicable).
- The impact at the upstream site boundary is 38 mm, consistent with the level of impact adopted by PCC in the 2009 Central Precinct Plan and Development Control Strategy (DCS).
- Upstream impacts are limited to the Sydney Water Recycled Water Project site, a low-lying area and formed channel within the St Marys WWTP, the Dunheved Golf Course which is already inundated under existing conditions, and the Links Rd road reserve and footpath.
- The preferred developed scenario does not inundate any additional buildings in the 1% AEP event.
- No additional properties are affected by the Flood Planning Area in the developed scenario. Four (4) upstream industrial properties on Links Road will potentially be affected by an increase in the area affected by the FPA. One property will potentially be affected by a decrease in the area affected by the FPA.
- Impacts at the downstream boundary are 11 mm and 16 mm in the regional and local tailwater scenarios. The characteristics of the floodplain result in relatively little attenuation, meaning these small impacts propagate for a significant distance downstream. However, these impacts do not cause a significant increase in flood extent, or result in inundation of additional properties. No material flood impact is therefore expected at downstream properties.
- An independent peer review by Worley Parsons using the *South Creek Flood Study* RMA-2 model predicted "*impacts that are equal to or lesser than those documented in the FIA Report".* Furthermore, the review considered the impacts *"minor for all areas outside of the Lend Lease site."*

7. References

DHI (2011) *MIKE11 Reference Manual*

DHI (2011) *MIKE21 Reference Manual*

NSW Government (2005) *Floodplain Development Manual: the management of flood liable land*

NSW Government (2007) *Floodplain Risk Management Guideline – Floodway Definition*

SKM (May 2009) St Marys Project – Central Precinct Plan, Water, Soils, and Infrastructure Report

SKM (30 Mar 2007) *Dunheved Precinct Development Application, Flood Impact Assessment*

SKM (18 Dec 2007) *Dunheved Precinct Development Application Flood Impact Assessment Addendum*

Worley Parsons (2013) *South Creek Flood Study: Provision of DRAFT Results at ADI Site, St Marys*

Worley Parsons (2015) *St Marys Central Precinct Flood Impact Assessment: Peer Review* 3 July 2015

Appendix A. Existing flood mapping