St Marys Project
Central Precinct Plan

WATER, SOILS AND INFRASTRUCTURE REPORT

- Final
- May 2009

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Sinclair Knight Merz
ABN 37 001 024 095
100 Christie Street
PO Box 164
St Leonards NSW
Australia 1590
Tel: +61 2 9928 2100
Fax: +61 2 9928 2500
Web: www.skmconsulting.com

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Executive Summary

Sinclair Knight Merz (SKM) has prepared this report for Maryland Development Company (MDC) to provide background information, describe existing and proposed conditions and provide Water, Soil and 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 the water cycle and water;
- Management strategies for stormwater trunk drainage system;
- Management strategies for groundwater and salinity;
- Essential services infrastructure (water, sewer, and electricity);
- Filling of Land; and
- Flood Evacuation

The proposed stormwater quality management strategy for the Western Precinct is based on the principles of Ecologically Sustainable Development (ESD) and Water Sensitive Urban Design (WSUD). This strategy includes the use of water quality controls such as gross pollutant traps, constructed wetlands and biofiltration basins.

The proposed development involves changes to the local catchments, including an increase in the amount of impervious area. Stormwater quantity would be managed via the use of detention basins. Runoff would be conveyed to the detention basins via an underground pipe network and above-ground overland flow paths. The lots would remain flood-free in events up to and including the 100 year ARI event. Detention of stormwater runoff would ensure that peak flows do not increase in storm events up to and including the 100 year ARI event.

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 limiting infiltration will further reduce this possibility.

Sydney Water and Integral Energy have indicated that they are able to service the Central Precinct with extensions to their existing networks. In brief, water would be from existing reservoir at Cranebrook immediately adjacent the site. Sewer would be transferred to existing St Marys Sewage Treatment Plant via pumping stations, rising mains and carriers. Electricity would be from existing zone substation at Cambridge Gardens to the south of the site.

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 for the adjoining Dunheved Precinct has recently been approved by Penrith City Council 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 further described in the report.

A portion of the Central Precinct would be subject to the Probable Maximum Flood (PMF) event (i.e. greater than the 100 year ARI event) and evacuation would be necessary. The flood evacuation strategy for residents and workers is to evacuate by car, which can be achieved and is described in the report. The approach taken is consistent with the NSW Floodplain Development Manual.

These measures proposed would achieve SREP30 and EPS requirements and objectives the details are further described in the report.

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 development "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.

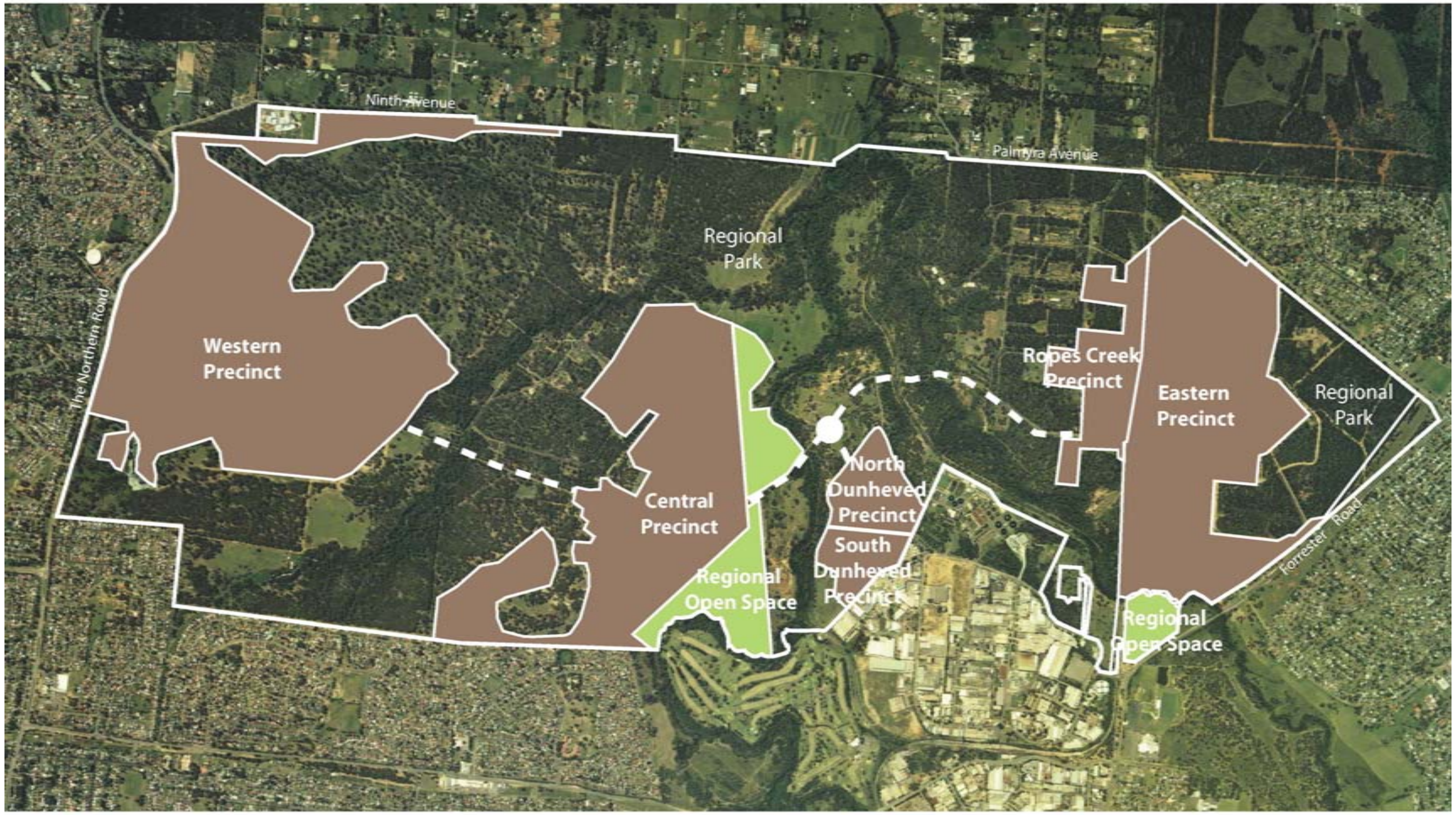
SREP 30 is accompanied by the St Marys Employment Planning Strategy (EPS) which identifies the aims for the future use and management of the site and sets out specific performance objectives and strategies to address key planning issues, including: conservation, cultural heritage, water and soils, transport, 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 Penrith City Council (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 currently zoned part Urban (129.7 ha) and part Employment (3.4 ha). Under a draft amendment to SREP30 currently being prepared, the land zoned Employment in the Precinct will increase to 38.4 ha, with a corresponding reduction in the land zoned Urban to 94.7 ha. Land zoned Urban is intended to accommodate primarily residential uses, with limited non-residential uses such as local retail and commercial uses. The Employment zone is intended to accommodate primarily employment generating land uses which are compatible with surrounding development and which will complement established employment areas and retail and commercial centres in the Blacktown and Penrith Local Government Areas.

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

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

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; and,
- Future integration with the balance of the site and the existing surrounding neighbourhoods.

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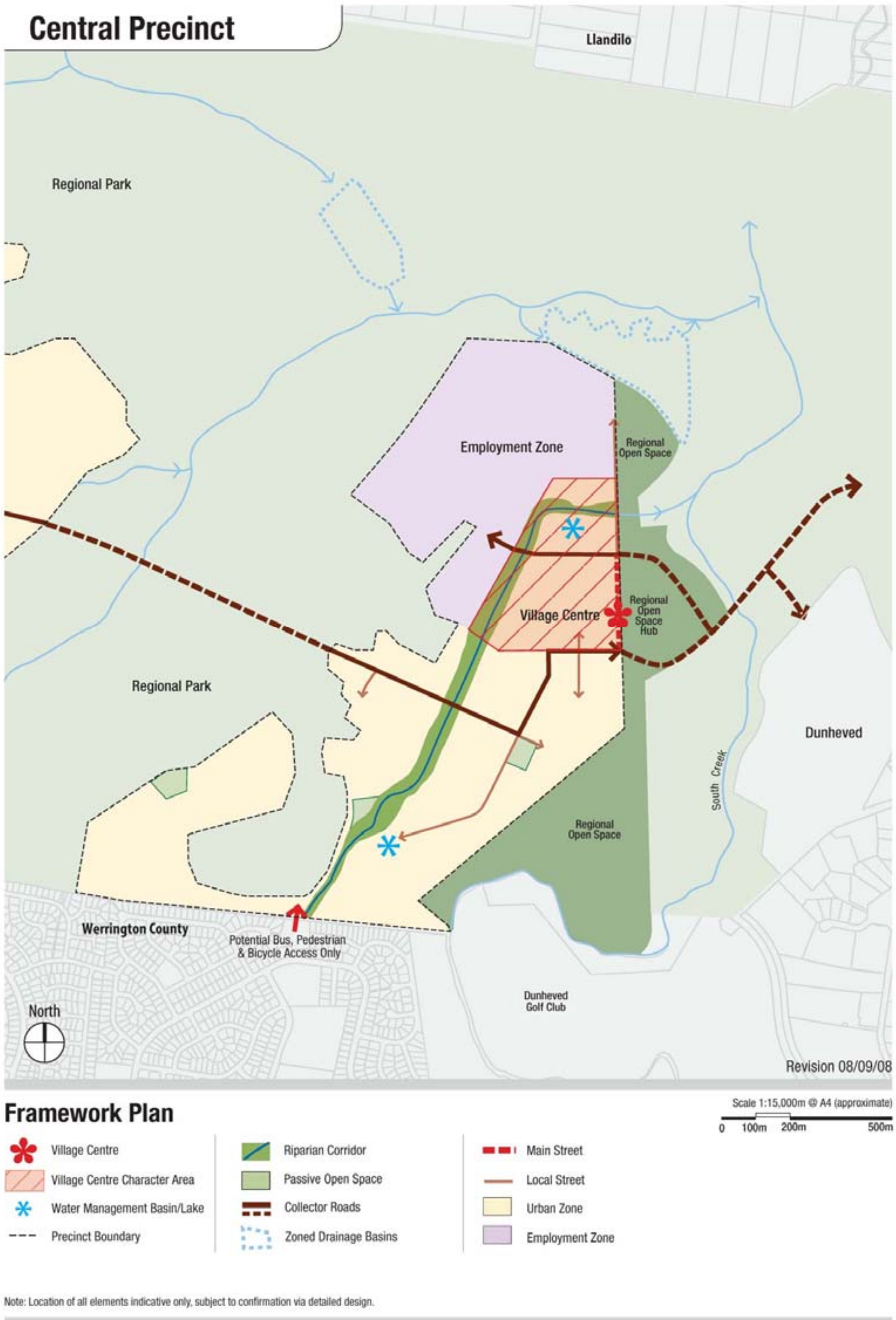
Figure 1-2 Framework Plan



2. Existing Environment

2.1 Topography

The Central Precinct occupies approximately 170 hectares of the St Marys development site. The 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 (SLs). 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.

2.4 Hydrology Runoff 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.

A RAFTS model was set up to predict existing peak flows from the site for a range of storm events. Details and results of the RAFTS model are included in **Appendix A**. Runoff quantities were determined at key locations points where runoff leaves the Central Precinct.

2.5 Hydrology Runoff 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.

2.7 Services

The existing infrastructure in and around the Central Precinct have been identified. The trunk components such as water reservoirs, sewage treatment plants and zone substations exist in close proximity to site. Other services such as communications and gas also exist in the area.

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3. Performance Objectives

The performance objectives for water, soils and infrastructure components are detailed in the SREP30 and the EPS. The objectives are summarised in this section along with an overview of the proposed management strategies are outlined in **Table 3-1**. Sections of the report are referenced to identify where more information can be found.

- **Table 3-1 Performance Objectives**

SREP 30 Clause Number / EPS Clause No	Requirement	Where Addressed
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

SREP 30 Clause Number / EPS Clause No	Requirement	Where Addressed
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
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

SREP 30 Clause Number / EPS Clause No	Requirement	Where Addressed
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
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

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SREP 30 Clause Number / EPS Clause No	**Requirement**	**Where Addressed**
EPS - Water & Soils		
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 b. incorporate measure in the infrastructure design, which ensure that changes in local flow regimes which result from the proposed development are minimised c. identify arrangements for the ongoing maintenance and monitoring of the watercycle management system d. ensure constructed trunk drainage channels are designed to convey the 100 year average recurrence interval (ARI) e. 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

<table>
<tr><th>SREP 30 Clause Number / EPS Clause No</th><th>Requirement</th><th>Where Addressed</th></tr>
<tr><td colspan="2">EPS - Water & Soils</td><td></td></tr>
<tr><td>6.4.10</td><td>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.</td><td>Soils, Groundwater & Salinity</td></tr>
<tr><td>6.4.11</td><td>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.</td><td>Filling of Land</td></tr>
</table>

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SREP 30 Clause Number / EPS Clause No	Requirement	Where Addressed
EPS - Water & Soils		
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
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

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SREP 30 Clause Number / EPS Clause No	**Requirement**	**Where Addressed**
EPS - Water & Soils		
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 Strategy
6.4.18	All trunk drainage infrastructures will be designed to reduce constraints on the flow of floodwaters, especially in relation to events above 1 percent AEP.	Catchment Management Strategy
6.4.19	Measures will be incorporated into infrastructure design to minimise demand for potable water. These will include: a) specifying low water demand fixtures in all dwellings and other buildings where appropriate b) limiting maximum pressure by managing system zonings (pressure zoning) having regard to critical water supply needs such as pressure for fire fighting c) including above ground rainwater tanks for dwellings on lots greater than 400m^2 d) using stormwater for irrigating open space areas e) incorporating other best practice measures at the time of development.	Catchment Management Strategy

4. Catchment Management Strategy

The objectives of the total catchment management strategy are to:

- Ensure peak flow rates do not increase for all storms up to the 100 year ARI event;
- Maximise source controls for runoff quantity and quality;
- Achieve a no net increase in the annual pollutant load exported from the site;
- To achieve efficient use of water and minimise demand for potable water;

The relevant measures listed below could be adopted for the Central Precinct. The performance of the proposed water quantity and quality controls was assessed and the results demonstrate that the proposed total catchment management plan meets the required objectives.

The 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;
- Recycled water (treated effluent) for toilet flushing, irrigation in public and private spaces use and other suitable activities such as washing cars;
- Water saving fixtures within the buildings;
- Bioretention vegetated areas in open space areas;
- Gross pollutant traps;
- Constructed stormwater wetlands or dry infiltration bioretention basins; and
- Detention storage intergrated into the wetlands or dry infiltration basin areas.

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4.1 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**.

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

Stormwater Management Pond ID	1998 Study (Basis of SREP 30) Wetlands Land Take (ha) [1]	SREP 30 Amendment (2005) Drainage Zones within Regional Park Land Take (ha)	Current Precinct Plan [2] Minimum Land Take (ha)
A1	2.2		2.5
A2	3.7		2.8
B	6	8	8
C1	3.4		2
C2	2.8	4.5	4.5
C3	1.4		0
D	0.6		2
E	1.4		1
F	0.6		0
G	0.7		0
H	1.6		0
I	4	7.4	7.4
EX1	2.6		0
Total	**31**	**19.9**	**30.2**

1- These 1998 Study landtake estimates are for water quality and detention requirements. These areas do not include benching or pathway areas.

2- For this Precinct Plan assessment, it has been assumed that the actual stormwater management wetland surface area is approximately 75% of the land take.

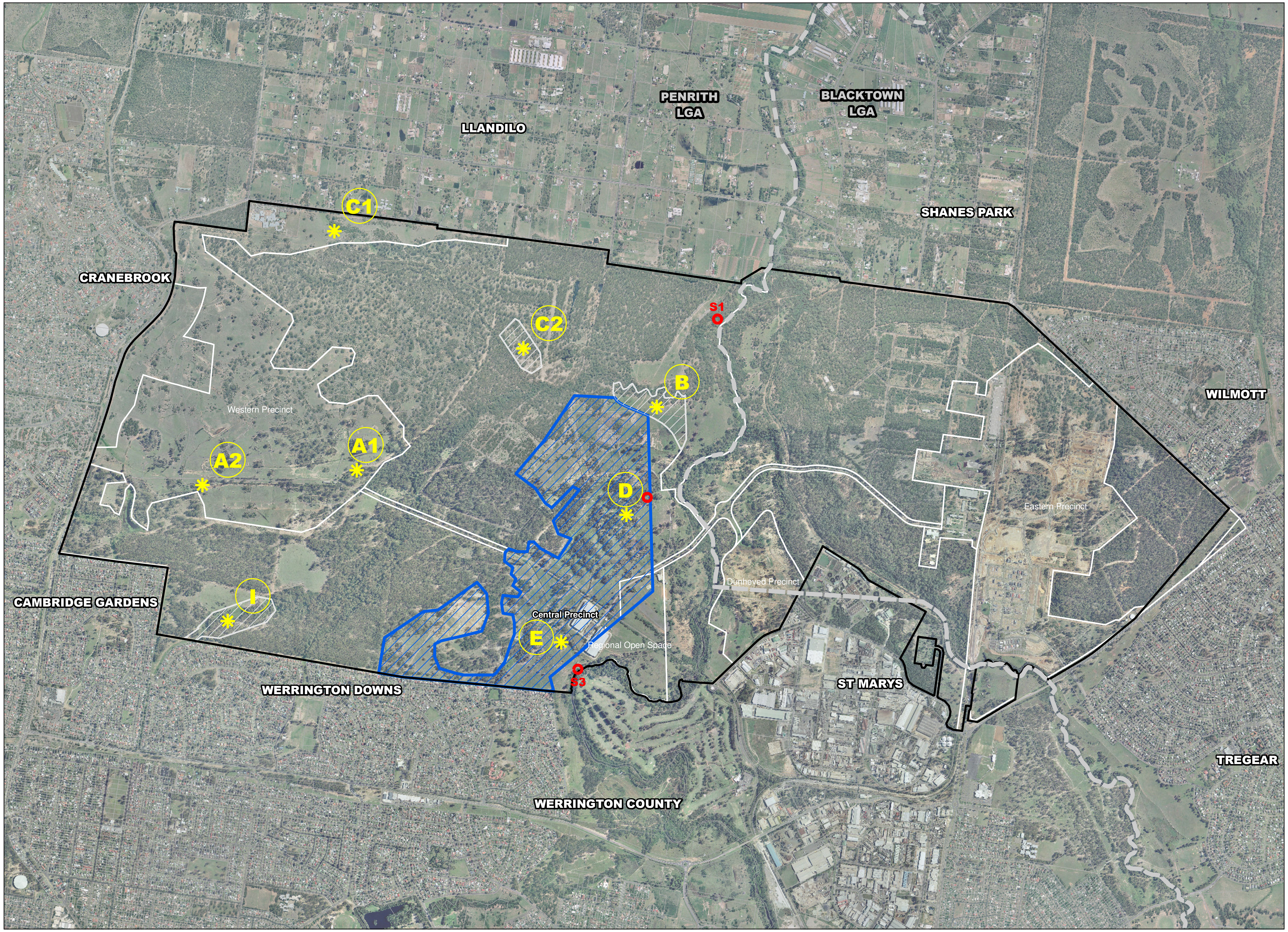
Many similarities can be drawn between the previous (1998) work and the assessment detailed in this Precinct Plan. The primary function of the wetland/detention basins remains as peak flow mitigation and water quality control. The basins within the Regional Park may need to be online basins as they are fixed zoned areas. The approximate locations of the proposed basins are shown in **Figure 4-1.**

Following recent consultation with Penrith City Council it was agreed that a similar approach to the this watercycle management would be taken whereby;

- Water quality is assessed for Central and Western Precincts together at a discharge point situated at South Creek;
- Water quantity is assessed for the Western and Central Precincts separately.

Volumes and areas required for detention and water quality purposes are based upon currently available information for input to the respective models. The basin volumes will be refined during detailed design as models are further developed to include the internal piping system, more sub catchment areas and parameters and maybe reduce as a result. During the detailed design stage, the use of onsite detention (OSD) and open space areas for detention may also be explored. Open space areas (for example grassed recreational areas) located in close proximity to creek lines can be utilised to detain floodwater temporarily, thus further reducing the detention volumes required to meet the objectives.

The location of the proposed basins is provided in **Figure 4-1**. The locations of the basins within the Precinct are indicative only thus allowing basin distribution and arrangements to remain flexible at this stage and more or less basins maybe required which would be determined at the detailed design stage.



Legend

SREP 30 - St Marys Amendment No.1 (11.04.2006)

- SREP 30 boundaries
- Site boundary
- Drainage Zone

(Sydney Regional Environmental Plan No 30 – St Marys Structure Plan Amendment No 1. Environmental Planning and Assessment Act, 1979. 11/04/2006. NSW Department of Planning.)

Stormwater Management Strategy

- Proposed detention / wetlands June 2008
- S1 Water Quality Control Points

(The St Marys Project, Delfin Lend Lease / Lend Lease Developments. Review and Assessment of Stormwater Requirements for the Western, Central and Ropes Creek Precincts. Final Draft 8 August 2005.)

General

- Property boundaries (LPI 2007)
- LGA boundaries (LPI 2007)

Note: Location of Detention/Wetlands within the precincts are indicative only and subject to change as part of detailed design

2007 Aerial Photography by AUSIMAGE

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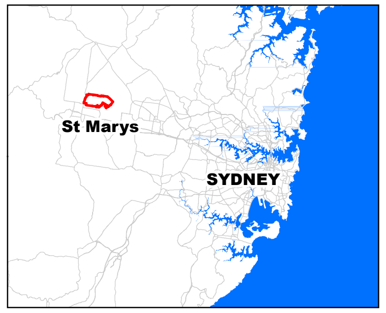

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Figure 4.1 Proposed Basin Locations Western & Central Precincts

St Marys Development Project - Central Precinct

GDA 94 MGA Zone 56

June 18, 2008
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4.2 Stormwater Quantity Management

To achieve the management objectives specified by SREP30 and EPS, detention basins have been proposed for the St Marys site to convey stormwater runoff from the proposed development to downstream discharge points on South Creek. Detention basins within the Precinct will be constructed off line with a low flow bypass to ensure that the peak flow following development does not exceed the peak flow under existing conditions.

A hydrological model (XP-RAFTS) was set up to assess the required detention volume of each basin for 2 yr to 100 year ARI events with details provided in **Appendix A**. The required volume of detention for each basin is shown in **Table 4-2**.

Overview

The objectives of the stormwater trunk drainage system are to:

- Safely convey runoff through the proposed development;
- Integrate with the road and lot layout; and
- Integrate with the water cycle management system such that runoff quality and quantity are controlled efficiently.

Water Quantity Management Objectives

Watercycle management objectives are outlined in two documents SREP30 and EPS, both prepared by the then Department of Urban Affairs and Planning. The following objectives refer to the management of stormwater quantity.

Changes in local flow regimes due to the development are to be minimised for rainfall events up to the 50% AEP rainfall event; i.e. from 2 yr to 100yr Average Recurrence Interval (ARI events).

Proposed Drainage System

The following components would make up the drainage system:

- Pit and pipe system able to carry flows up to the 10 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; and
- Combined detention/wetland basins able to provide the necessary quality and quantity controls, while also coping safely with the 100 year ARI flow.

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Proposed Detention Basin Volumes

Two detention basins are proposed for the Central Precinct for peak flow mitigation for 2 year to 100 year ARI storm events. The two basins (D and E) are located within the Central Precinct as shown on **Figure 4-1**. Required detention volumes to mitigate peak flows have been derived using a hydrology model and are reported in **Table 4-2.**

- **Table 4-2 Proposed Water Quantity Detention Basin Volumes Central Precinct**

Detention Basin	Detention Depth (m)	*Water Surface Area (ha)	Detention Volume Required (ML)
D	1.4	1.6	22
E	2.0	1	18

*Surface area of water in detention basin at maximum detention depth

The volumes for the Central Precinct would be refined at the design stage by further modelling and detailing of the outlet controls for the basins.

Hydraulics

Channel top widths will be defined for the trunk drainage system during further consultation with the Department of Water and Energy (DWE) 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.

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 Department of Water and Energy (DWE), the "rivers" for the Central Precinct as shown on **Figure 4-2** were identified. It was agreed with DWE that the "rivers" will be refined during further consultation with DWE.

Maintenance of Water Quantity Controls

Proposed detention basins/wetlands will be maintained by MDC for an initial three year period following construction. After this time, Penrith City Council will be responsible for the ongoing maintenance of the basins.

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Figure 4.2 Identified "Rivers" within the Central Precinct

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4.3 Stormwater Quality Management

Overview

The water cycle management strategy for the Central Precinct development will be based on design principle to meet the stormwater management objectives described in the following documents:

- *SREP No 30, 2001; and*
- *St Marys Environmental Planning Strategy, 2000*

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

- *Penrith City Council ,Water Conservation and Water Action Plan – Water Way -– Sustainable Penrith series*
- *Penrith City Council, Sustainability Blue Print for Urban Release Areas, June 2005 – Sustainable Penrith series.*
- *Penrith City Council, Erosion and Sediment Control DCP, December 2006- section 2.4*
- *South Creek Stormwater Management Plan, 1999-2000, Stormwater Trust*
- *Department of Environment & Climate Change (DECC), Managing Urban Stormwater, Environmental Targets, Draft October 2007.*
- *Penrith City Council, Stormwater Quality Control Draft Policy*
- *Landcom, Soils and Construction, 2004*
- ANZECC Guidelines for Fresh and Marine Water Quality, 2000

Water Quality Management 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. There will be no increase in the annual pollutant loads in the developed case compared to the existing case. This objective will be applied to all runoff into South Creek entering the creek along the St Marys site from the west. This includes runoff from the Western Precinct, the Central Precinct and any existing urbanised areas located further upstream of this catchment.

To meet this objective, a water quality assessment has been undertaken for the Western and Central Precincts. These models were combined into one assessment to represent runoff from all catchments entering South Creek from the west. A series of stormwater management wetlands have been identified across the Western Precinct, Central Precinct and areas in the Regional Park.

The MUSIC water quality model (eWater CRC, Version 3.01) has been used in the water quality assessment. The water quality modelling details are given in **Appendix B**.

The proposed water quality measures on site are not limited to wetlands. The additional controls are described in the following section. For water quality modelling purpose, only wetlands were included in the assessment. This would result in relatively conservative sizing for the proposed wetlands.

Proposed Water Management System

A number of stormwater management controls would be integrated into the overall drainage concept to manage stormwater quality and quantity where appropriate and to achieve the required objectives. 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 roofwater 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 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 or a wetland. The GPTs would remove coarse sediment, litter and debris that are generated on the roads.
- Dry infiltration basins or wetlands 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, or if a wetland is proposed instead of a dry infiltration basin, then it would offer a similar treatment of polishing the runoff.

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.

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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; and
- 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.

Size of Proposed Water Quality Controls

The land take requirements of the proposed stormwater wetlands in the Western and Central Precincts (Central Precinct basin is highlighted in bold) that would meet the water quality objectives for South Creek are shown in **Table 4-3.**

Table 4-3 Proposed Water Quality Stormwater Management Wetland Sizes for the Western and Central Precincts

Stormwater management wetland ID	Minimum* land take (ha) for water quality purposes only
A1	1
A2	1.8
B	8
C1	1
C2	4.5
D	**2**
I	7.4

***** Refer to Table 4.1 for the landtake requirements that include the additional areas required for detention purposes

Wetlands "I" and "B" are required to meet to achieve the project water quality objectives and would be progressively constructed during the development. Wetlands have been proposed in this Precinct Plan but it should be noted that other WSUD water control measures such as biofiltration basins may also be considered as an alternative during the detailed design stage.

Maintenance of Water Quality Controls

The pollutant retention capability of any 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.
Constructed Wetlands	The wetlands area should be inspected annually for trapped sediments. Excessive sediment should be removed and disposed of properly to maintain the design volume of the wetland. Excessive dead plant debris should be removed to reduce the organic material and nutrient loads in the wetland area.

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 wetlands;
- the maintenance regime for heavy and light machinery for cleaning of sediments and organic material deposited within all parts of the wetland;
- litter and sediments trapped in gross pollutant traps;
- monitoring of vegetation type and growth;
- maintenance of conditions to ensure mosquito control; and
- appropriate safety measures.

4.4 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.

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; and
- 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 : *Penrith City Council, Erosion and Sediment Control DCP, December 2006- section 2.4,* and the Landcom "*Soils and Construction* " *Manual, 2004,* 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.

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; and
- Stockpiles should be seeded.

Erosion and Sediment Control Measures

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

- Stabilised entry/exit point;
- Sediment filter fences;
- Weed-free straw bales;
- Barrier fences;
- Diversion drain banks/channels;
- Check dams;
- Temporary sedimentation basins; and
- Top soil stockpiles.

These control structures are described in the following sections.

Stabilised Entry/Exit Point

A 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 ($0.5L/m^2$). 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. 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.

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.

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; and
- Obtain additional information needed from the *"Soils and Construction", Landcom 2004* manual.

4.5 Conclusion

The MUSIC model results, as provided indicate that the proposed stormwater management wetlands would meet the SREP30 water quality objectives of ensuring that there is no net increase in the annual pollutant load in the developed case compared to the existing case.

This assessment identifies fewer stormwater management ponds across the St Marys Project site compared with the 1998 Study. This result is an expected one, as the proposed area to be developed by MDC has been reduced since the 1998 SKM report was produced. In summary, the modelling results indicate that the proposed stormwater management wetlands would meet the water quality and quantity objectives.

4.6 References

1) ANZECC Guidelines for Fresh and Marine Water Quality, 2000
2) Environmental Investigation Services, *Soil and Groundwater Investigation*, December 2006
3) eWater, *MUSIC User Guide*, Version 3.1
4) Healthy Rivers Commission of New South Wales, *Independent Inquiry into the Hawkesbury Nepean River System, Final Report*, August 1998.
5) Landcom, *Soils and Construction*, 2004
6) Penrith City Council, *Erosion and Sediment Control DCP, December 2006- section 2.4*
7) Penrith City Council, *Stormwater Quality Control Draft Policy*
8) Sinclair Knight Merz, *Flood Assessment in South Creek*
9) Stormwater Trust, *South Creek Stormwater Management Plan*, 1999-2000,
10) SREP No 30
11) *St Marys Environmental Planning Strategy*, 2000

5. Soils, Groundwater & Salinity Management Strategy

5.1 Background to Soils, Groundwater and Salinity

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; and,
- 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-evapo-transpiration 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.

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; and,
- A Groundwater Management Strategy should be prepared for the site.

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

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

5.3 Precinct Description

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.

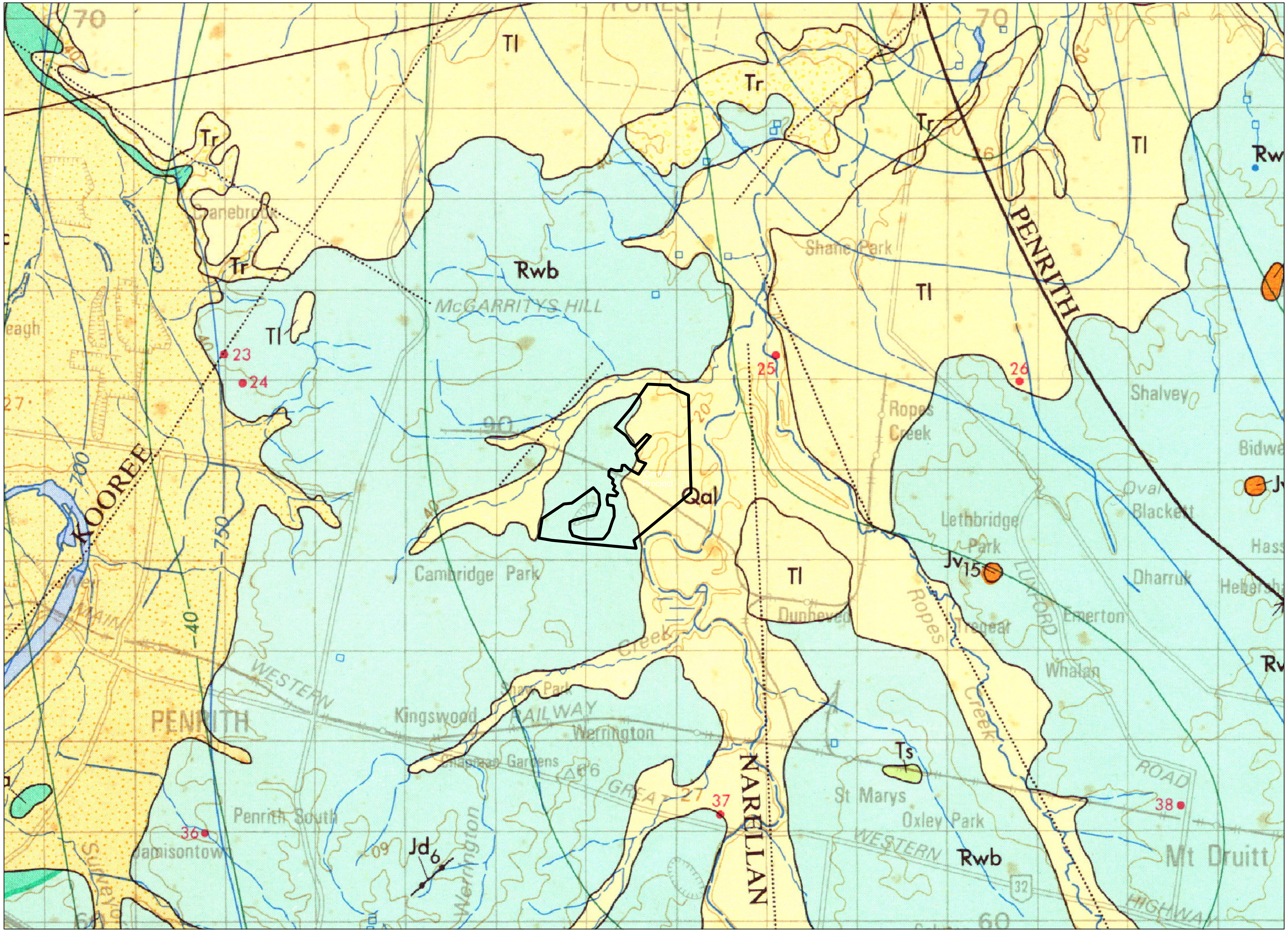
The surface of the alluvial terrace is nearly level to undulating, with a number of very shallow wet depressions (relief 0.2 to 0.4 m), resembling gilgais. They differ from gilgais in that the soil is not noticeably expansive, shrinkage cracks are relatively uncommon and generally less than 10 mm wide, 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.

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.



Legend

Tl	Clay, patches of ferruginized, consolidated sand
Tr	Conglomerate, matrix supported
Rwb	Shale, carbonaceous claystone, claystone, laminite, fine to medium-grained lithic sandstone, rare coal and tuff
Qal	Fine-grained sand, silt and clay
Jd	Basalt, dolerite
Ts	Laterized sand and clay with ferricrete bands; Includes silcrete, sandstone and shale boulders
Jv	Volcanic breccia, varying amounts of sedimentary breccia and basalt

0 1
Kilometres

Figure 5.1 Regional Geology Map (Extract From Penrith 1:100,000)

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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.

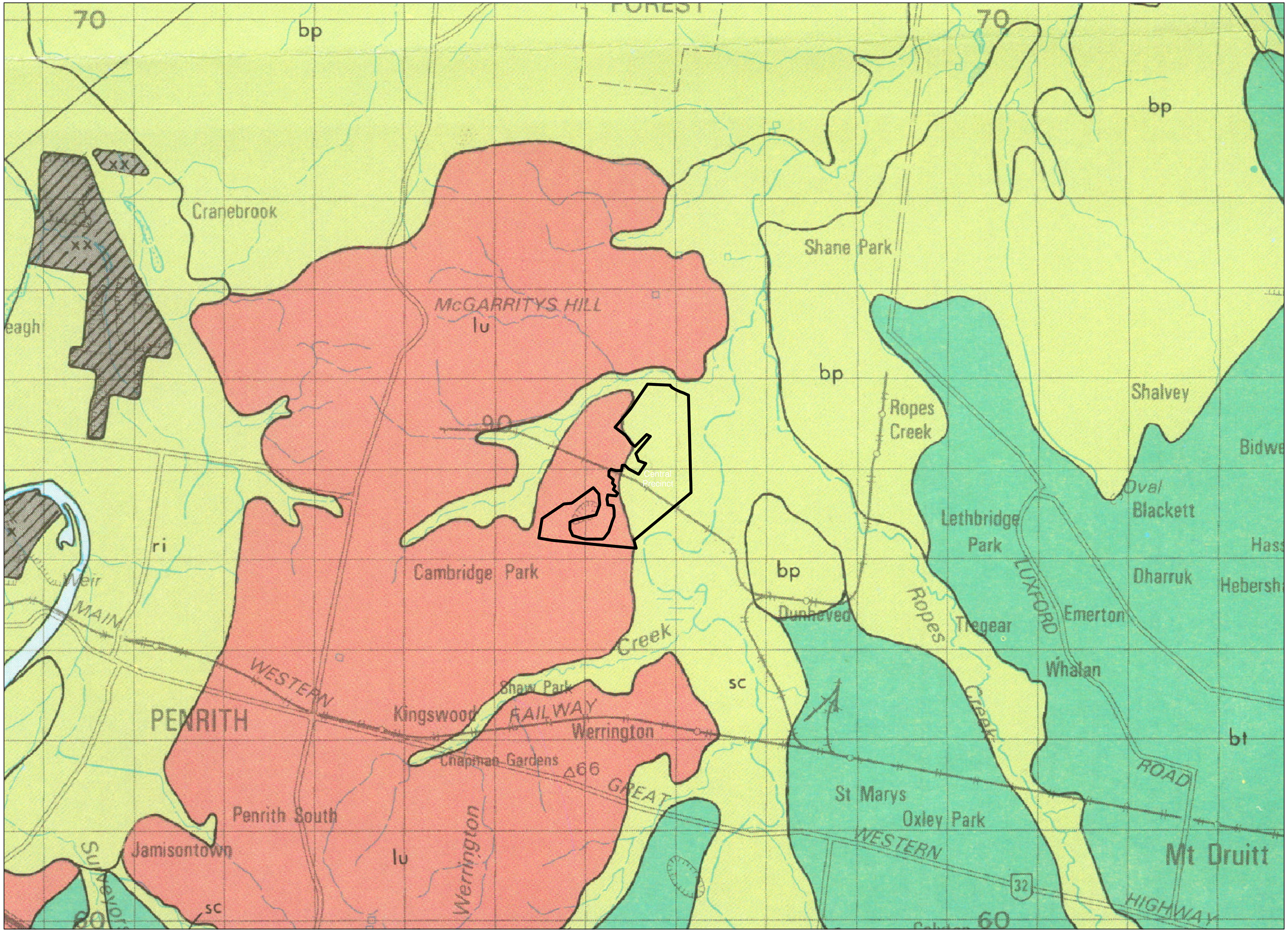
The lower slopes of the hills are generally mantled by 1 to 4 m of clay colluvium, which is being moved slowly downslope by soil creep and is merging with the floodplain alluvium that it closely resembles.

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.



Legend

bp Berkshire Park (150 km2)

lu Luddenham (285 km2)

sc South Creek (150 km2)

bt Blacktown (670 km2)

0 1 Kilometres

St Marys

SYDNEY

Figure 5.2 Soil Landscape Map (Extract From Penrith 1:100,000)

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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 40 cm	Very hard setting surface
			Low available water capacity
	lu3	>50 cm	Low wet strength
			Low permeability
			Low fertility
			High shrink-swell
			Low available water capacity
	lu4	<90 cm	Low wet strength
			Low permeability
			Low available water capacity
			High shrink-swell
South Creek (sc)	sc2	15 cm	High erodibility
			Hard setting surface
			Strongly Acid
			Low fertility
	sc3	60-85 cm	Shrink-swell potential
			Very high erodibility
			Low fertility

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.

Regional 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. 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.

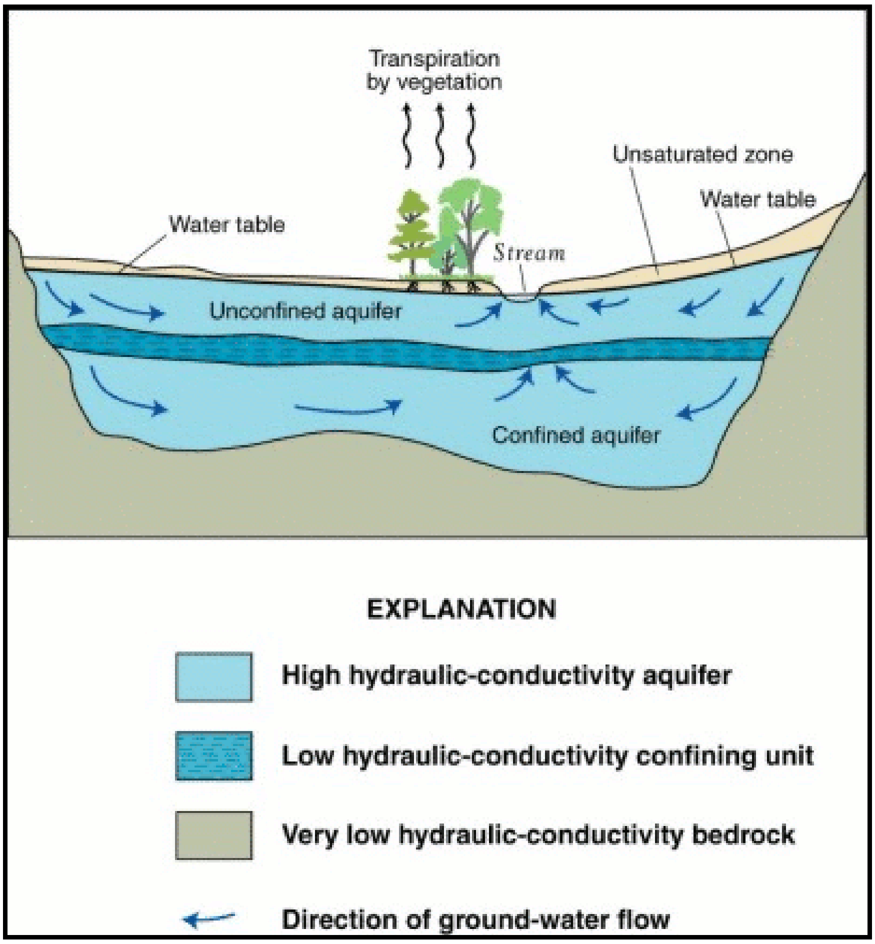
The main difference between these two 'aquifer systems' is that the shallow ones are more-or-less fresh, relatively permeable, but only ephemerally saturated; while the deeper aquifers are tighter, permanently saturated and much more saline (with salt content approaching that of sea water in places). The use of the plural recognises 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.

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 **Figure 5-3**. 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**



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 footslopes 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 permeabilty.

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.

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 (m^2 per m^3), 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.

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.

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

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**.

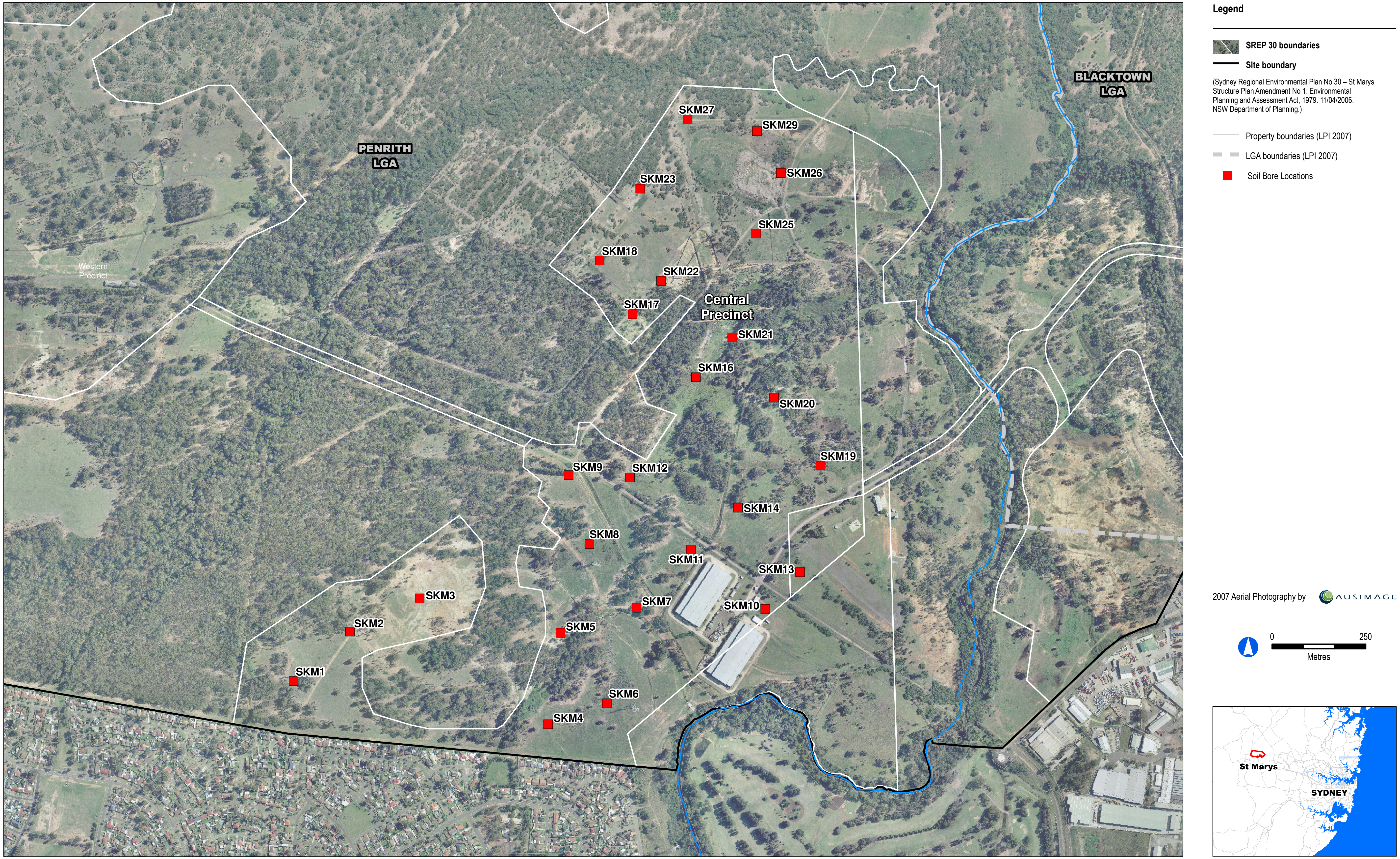


Fig 5.4 : Soil Bore Locations

St Marys Development Project - Central Precinct

GDA 94 MGA Zone 56

June 24, 2008
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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.

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 5-2** 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.

SKM

▪ Table 5-2 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			

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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 5-3**.

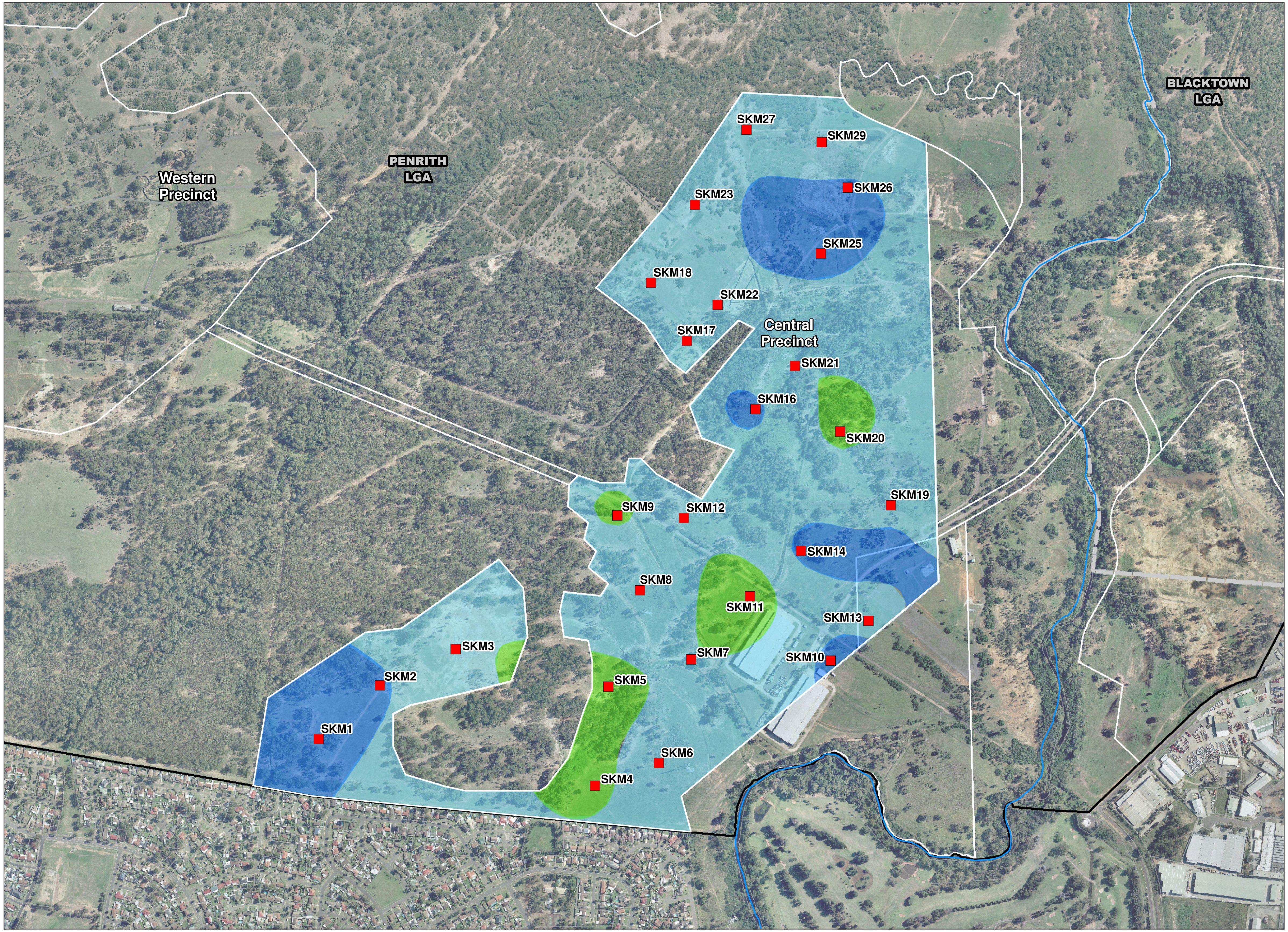
- **Table 5-3 EC_e Values of Soil Salinity Classes (DLWC 2002)**

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	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

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; and,
- 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.



Legend

SREP 30 boundaries

Site boundary

(Sydney Regional Environmental Plan No 30 – St Marys Structure Plan Amendment No 1. Environmental Planning and Assessment Act, 1979. 11/04/2006. NSW Department of Planning.)

Property boundaries (LPI 2007)

LGA boundaries (LPI 2007)

Soil Bore Locations

Soil Salinity

Class	EC_e (dS/m)
Non-Saline	<2
Slightly Saline	2-4
Moderately Saline	4-8
Very Saline	8-16
Highly Saline	>16

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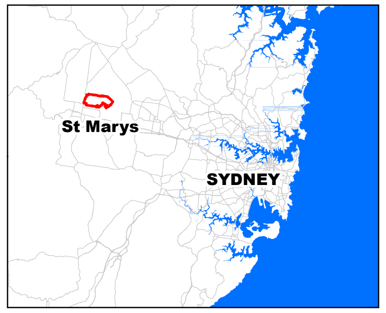

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Fig 5.5 : Soil Salinity at a Depth of 0.25m (A-Horizon)

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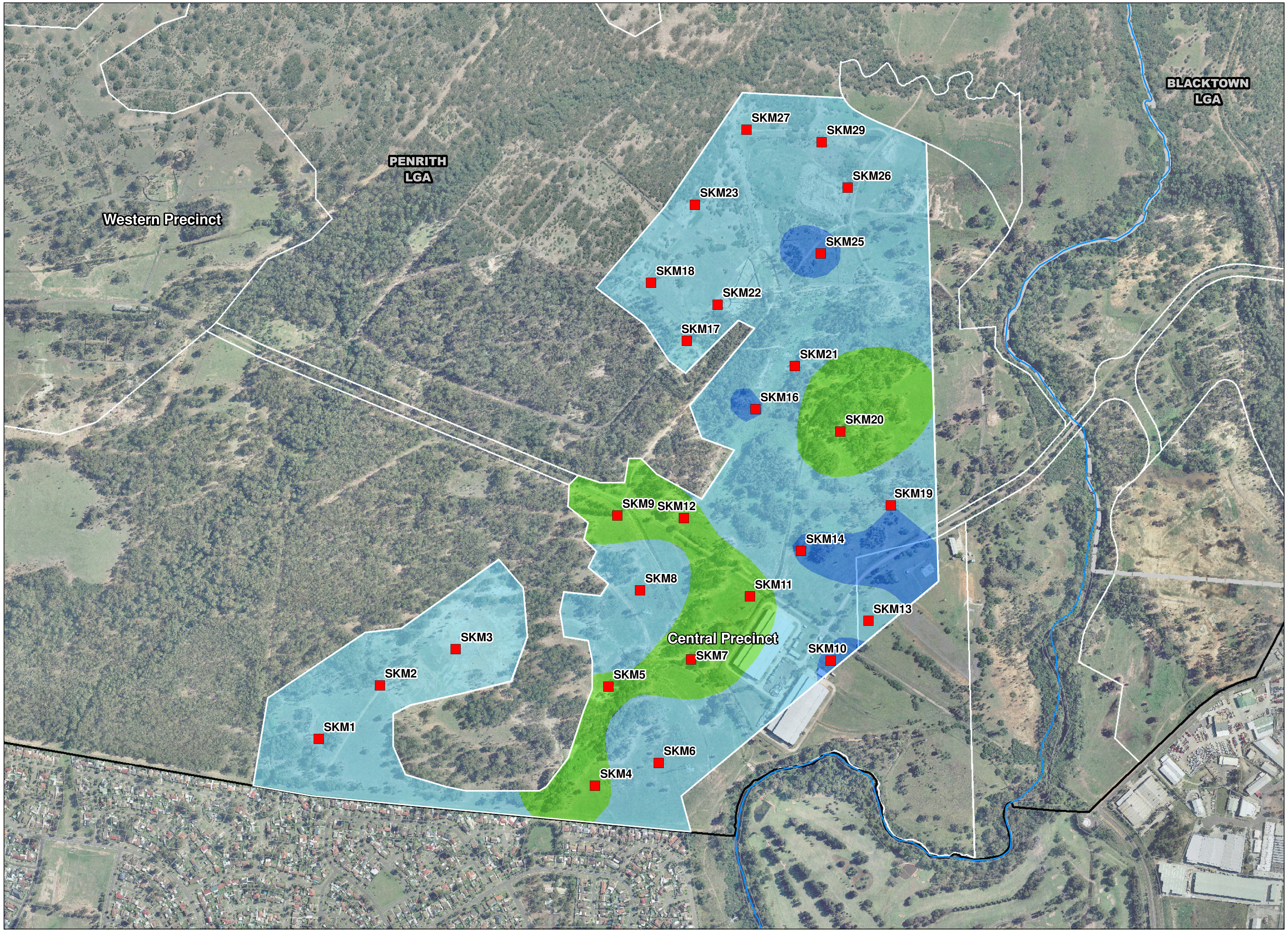

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SREP 30 boundaries

Site boundary

(Sydney Regional Environmental Plan No 30 – St Marys Structure Plan Amendment No 1. Environmental Planning and Assessment Act, 1979. 11/04/2006. NSW Department of Planning.)

Property boundaries (LPI 2007)

LGA boundaries (LPI 2007)

Soil Bore Locations

Soil Salinity

Class	EC_e (dS/m)
Non-Saline	<2
Slightly Saline	2-4
Moderately Saline	4-8
Very Saline	8-16
Highly Saline	>16

2007 Aerial Photography by AUSIMAGE

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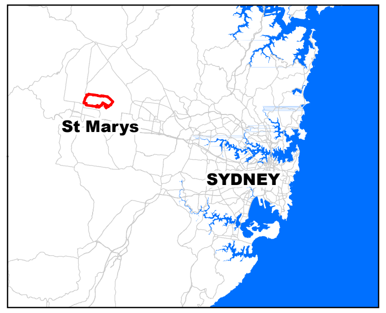

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Fig 5.6 : Soil Salinity at a Depth of 0.5m (B-Horizon)

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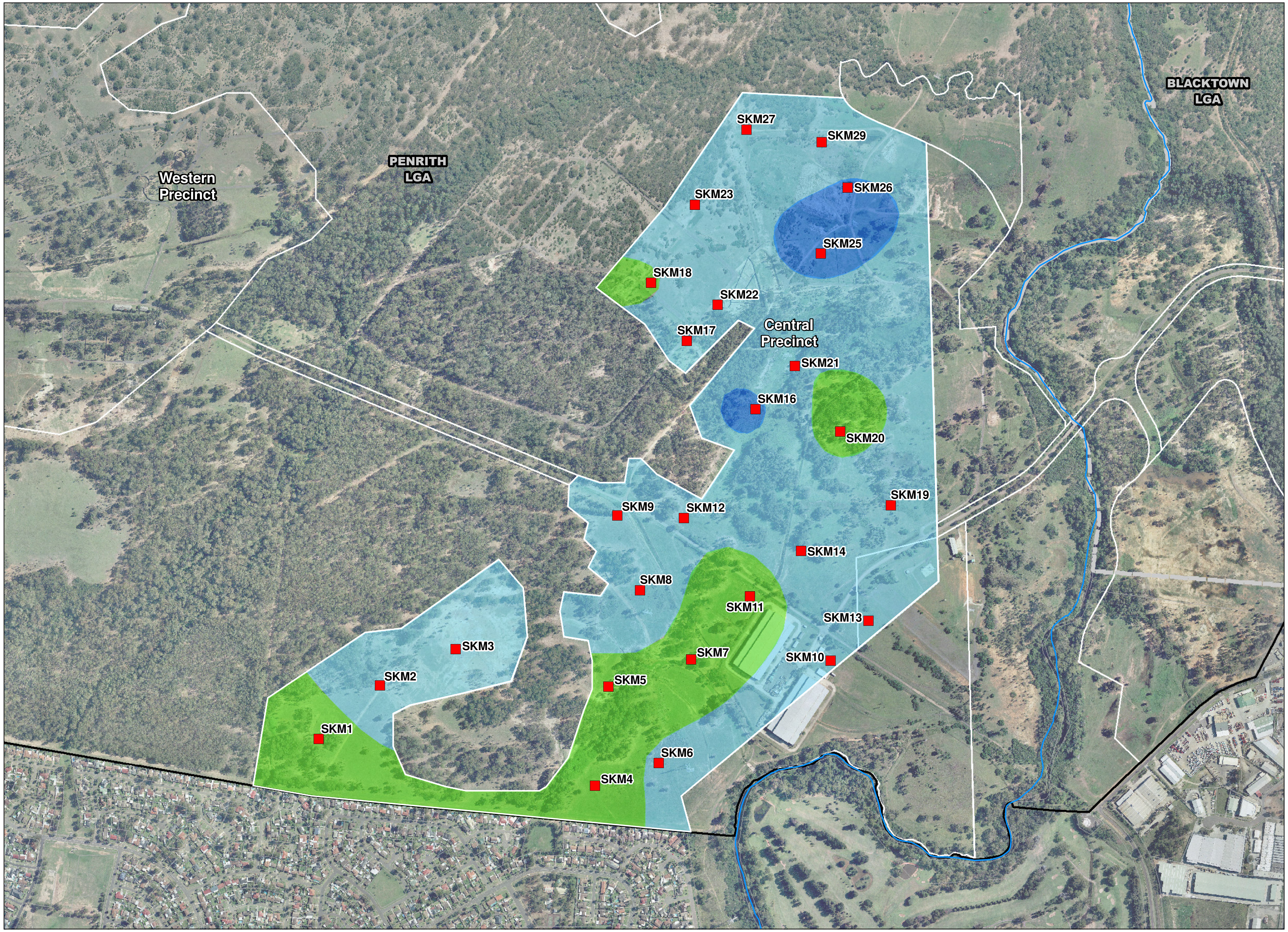

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SREP 30 boundaries

Site boundary

(Sydney Regional Environmental Plan No 30 – St Marys Structure Plan Amendment No 1. Environmental Planning and Assessment Act, 1979. 11/04/2006. NSW Department of Planning.)

Property boundaries (LPI 2007)

LGA boundaries (LPI 2007)

Soil Bore Locations

Soil Salinity

Class	EC_e (dS/m)
Non-Saline	<2
Slightly Saline	2-4
Moderately Saline	4-8
Very Saline	8-16
Highly Saline	>16

2007 Aerial Photography by AUSIMAGE

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Metres

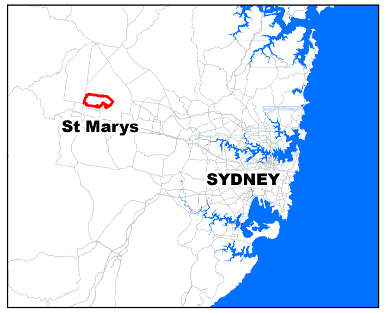

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Fig 5.7 : Soil Salinity at a Depth of 1m (Lower B-Horizon)

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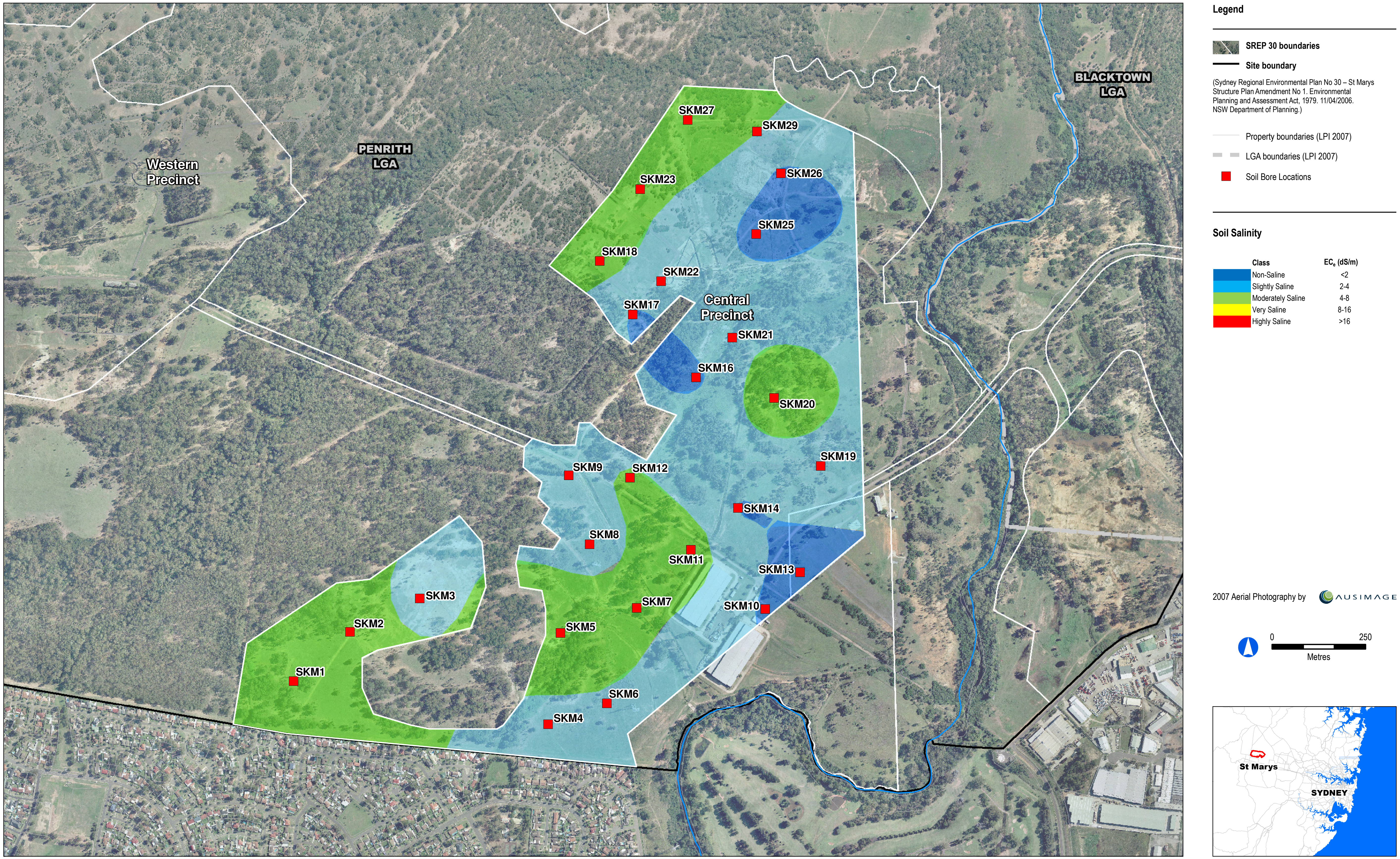

SINCLAIR KNIGHT MERZ SKM

Fig 5.8 : Soil Salinity at a Depth of 3m (Weathered Shale)

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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; and
- 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

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 RL 19 to 20 m), followed by throughflow across the A/B soil profile interface and temporary storage in shallow perched aquifers at depth raging 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, EC_e 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.

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 5-2**);
- 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**; and,
- Electro-magnetic induction (EMI) surveys across the Precinct area to measure ground conductivity, carried out by Douglas Partners in 2008 and reported separately.

Fig 5.9 : Piezometers (Mackie Martin, 1991)

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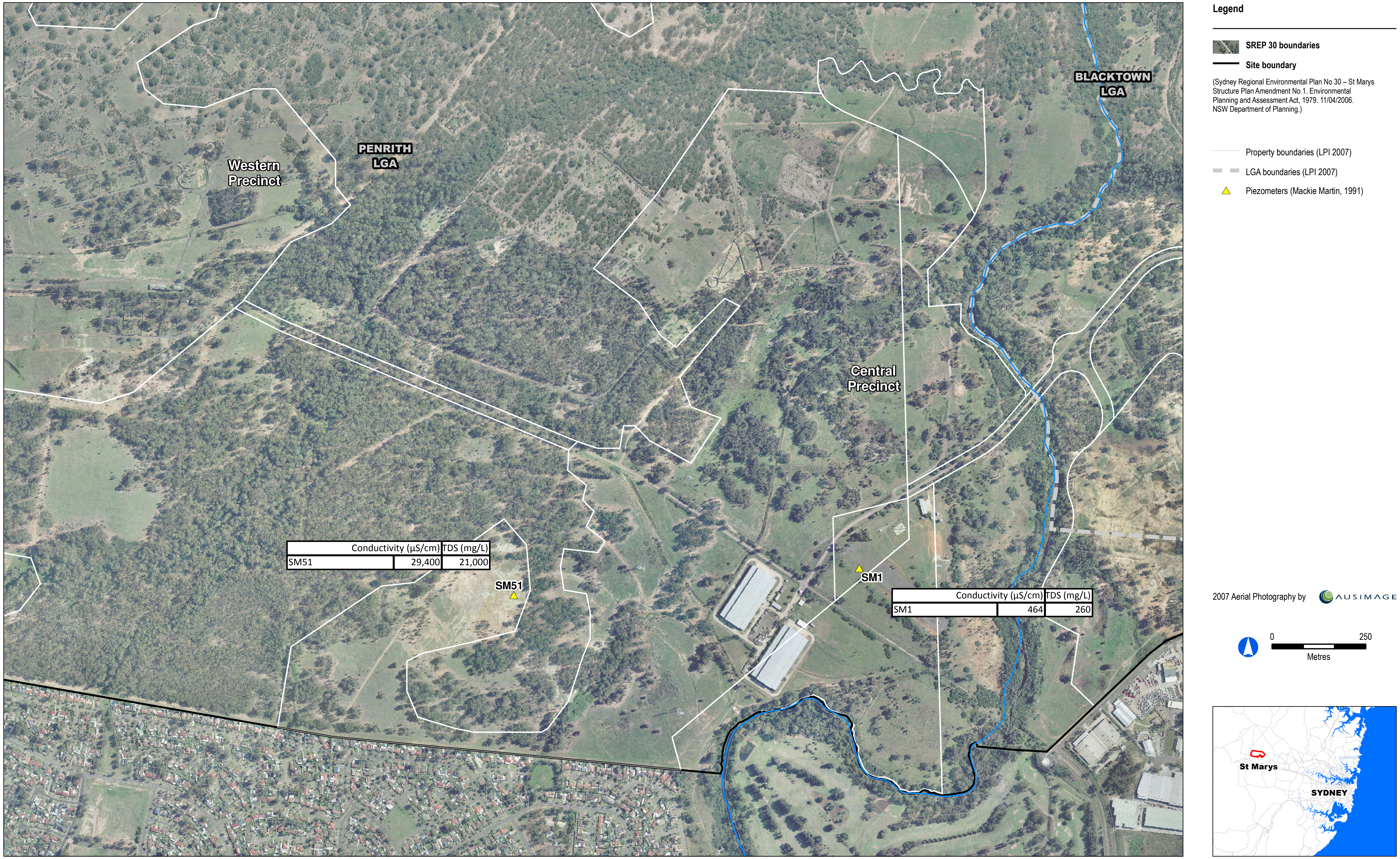

	Conductivity (µS/cm)	TDS (mg/L)
SM51	29,400	21,000

	Conductivity (µS/cm)	TDS (mg/L)
SM1	464	260

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Fig 5.10: Groundwater Salinity (Mackie Martin, 1991)

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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); and,
- 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.

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.

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 by all practical means. The proposed filled landform within the eastern portion of the Central Precinct and the management measures indicated below present opportunities for achieving these goals.

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); and
- Increased groundwater recharge from wetlands, stormwater detention basins, unlined drainage lines and ponded runoff generally.

5.9 Management Measures

The specific measures 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 so far as 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 sealed drains to approved discharge points along natural drainage lines.
- All basins and swales may need to be lined with an impermeable liner to prevent infiltration into groundwater.
- 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.

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.

Stormwater Conduits

All paved areas such as roads and carparks should 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; and
- 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.

Wetlands

The key groundwater management issue with respect to wetlands is to provide a liner to prevent any interaction between groundwater and the water in the wetland.

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.

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 SKM during this investigation (refer **Figure 5-14)** and any other existing piezometers across the site.

Legend

SREP 30 boundaries

Site boundary

(Sydney Regional Environmental Plan No 30 – St Marys Structure Plan Amendment No 1. Environmental Planning and Assessment Act, 1979. 11/04/2006. NSW Department of Planning.)

Property boundaries (LPI 2007)

LGA boundaries (LPI 2007)

Piezometers (SKM, 2008)

2007 Aerial Photography by AUSIMAGE

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Metres

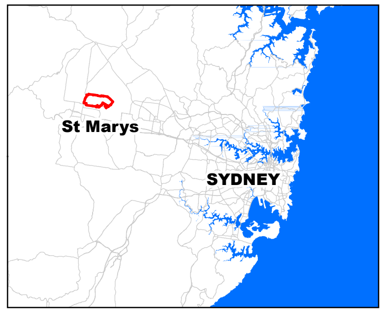

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Fig 5.11 : Piezometers (SKM, 2008)

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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.

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.

Table 5-4 Salinity, Erosion and Sediment Management Strategy Overview

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 ONTO 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 GARDEN/LAWN AREAS • ESTABLISH LOW WATER REQUIREMENT PLANTS • USE MULCH COVER • USE LOW FLOW WATERING FACILITIES		
		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		
		PREVENT LEAKAGE FROM WETLAND AND DRAINAGE FACILITIES	• LINE ALL PERMANENT STORMWATER RETENTION STRUCTURES AND WETLANDS		
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; and,
- 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 comparitively 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 and limiting infiltration – will further reduce this possibility.

5.12 References

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6. SERVICES INFRASTRUCTURE

6.1 Proposed Infrastructure

Sewer

The recent Developer Servicing Plan for the St Marys Wastewater System 2006 identified sewage from the St Marys Project (which includes the Central Precinct) would be treated at St Marys Sewerage Treatment Plant (STP). The St Marys (STP) has sufficient capacity to accommodate the additional flows from the Central Precinct.

Discussions with Sydney Water have revealed that sewerage from the Central Precinct could be delivered to the STP by either tapping into the carrier that runs through the St Marys Project "Werrington Downs Carrier", direct connection to the treatment plant or connection to existing pumping station SPS366. Further investigations would be required to ascertain the appropriate method of transferring sewage and connection to Sydney Water system.

Drinking Water

The Precinct will be link with the Western Precinct and will be serviced from the Orchard Hills drinking water supply system It is likely that upgrades to the existing system will be required,including potentially an additional reservoir at Cranebrook and trunk watermains.

Sydney Water is undertaking investigations which will confirm the required major infrastructure necessary to service the Central Precinct. Easements over public or private lands will be created where absolutely necessary as a last resort.

Electricity

Discussions with Integral Energy have revealed that they are able to service the Central Precinct subject to some augmentations to their existing network. Integral Energy advised that ultimately the Central Precinct would be serviced from Cambridge Gardens Zone Substation situated south of the site once the Claremont Meadows Zone substation is established in 2010. Establishment of the Claremont Meadows zone would free up capacity at Cambridge Gardens zone. Feeders (11kV) from Cambridge Gardens zone would be required and the entire Central Precinct would be supplied from this zone.

Development within the Central Precinct will require the extension of the electricity reticulation network throughout the project. Internal electricity reticulation within the Central Precinct will be provided under Integral Energy's usual developer arrangements for the supply of underground electricity. Easements over public or private lands will be created where absolutely necessary as a last resort.

Communications

Underground telecommunications cables (optical fibre and/or copper cables) will be extended throughout the Central Precinct under the usual developer arrangements. Telstra will be updated when more accurate data on the number and type of users are known. Easements over public or private lands will be created where absolutely necessary as a last resort.

Gas

Agility Management Pty Ltd provides network management expertise for AGL, the organisation responsible for the extension and reticulation of the gas supply network. Agility will be updated when more accurate data on the number and type of users are known. Easements over public or private lands will be created where absolutely necessary as a last resort.

6.2 Design and Ecological Sustainable Development Initiatives

An opportunity exists to incorporate Ecologically Sustainable Development (ESD) principles in the services infrastructure for the Central Precinct.

Sewer

The following initiatives could be used in the design and construction of sewerage infrastructure:

- The gravity reticulation system for the site could be a ‘Low Infiltration System’ or 'Low pressure System' to reduce ground-water infiltration.
- Vitreous clay pipes should not be utilised in the construction of sewerage reticulation systems. uPVC or similar pipes should be used for all sewerage construction with compatible access chambers and house connections.

Drinking Water

The following initiatives could be used in the design, construction and use of potable water infrastructure:

- Specifying the use of low water demand fixtures (showerheads, toilets and other AAA rated devices etc) and appliances in buildings where appropriate.
- Rainwater collection tanks on lots for irrigation.

Recycled Water

The following initiatives could be used in the design and construction of infrastructure:

- The potential future use of treated effluent, if available from Sydney Water for toilet flushing, irrigation (when rainwater is unavailable) and industrial purposes will reduce potable water demand and reduce the pollution load on South Creek.

Electricity

The following initiatives could be used in the supply and reticulation of electricity:

- Passive design and built form controls that reduce the demand for electricity should be promoted as an integral requirement for the Precinct.
- Specifying the use, where appropriate, of "energy efficient" electrical appliances in buildings.
- Examining the use of solar powered and water heating systems lighting where appropriate.

Communications

The following initiatives could be used in the design and construction of telecommunications infrastructure:

- Provide adequate 'spare' conduit capacity in all street reticulation networks to facilitate future expansion and technology.
- Provide an optical fibre network throughout the site.

Gas

Gas reticulation is recommended for the development due to:

- Provision of gas services reduces the expected load on Electricity Infrastructure and therefore reduces the emission of greenhouse gases.
- Gas reticulation provides commercial customers within the development with options and pricing power, particularly for contestable works.

Common Trenching

Best practice development allows for "Common Trenching Agreements" between the developer, Telstra, AGL and Integral Energy. Benefits of Common Trenching Agreements include:

- Reduced costs due to a shared trench between the three service providers.
- Lower land take within the road reserves throughout the site.
- Increased efficiency and shorter time frame for provision of services.

6.3 Conclusion

Essential services, (water, sewer and electricity) would be made available for the development. Sydney Water and Integral Energy have indicated that they are able to service the Central Precinct.

SKM

7. Filling of Land

7.1 Existing Flood Risk

The site is located on the floodplain of South Creek (a tributary of the Hawkesbury-Nepean River). South Creek runs along the eastern boundary of the site. Ropes Creek joins South Creek downstream of the site. South Creek flows northwards from this point to join the Hawkesbury River near Windsor. Flooding may be caused by rainfall in the catchments of Ropes and/or South Creeks themselves, and also by backwater flooding from major events in the Hawkesbury-Nepean River.

7.2 Flood Modelling Background

Dunheved Precinct Plan Model

An existing hydraulic computer model of South Creek including the lower section of Ropes Creek was used to define flood behaviour in the vicinity of the site for the 100 year Average Recurrence Interval (ARI) and Probable Maximum Flood (PMF) design flood events. The development of the existing model is described in the *North and South Dunheved Precinct Plan Water, Soils and Infrastructure Report* (*SKM, May 2006*). The May 2006 Precinct Plan report includes detailed information and results for flood modelling results of the existing situation (or Base Case). The Preferred Development Option for the combined Dunheved and Central Precincts is outlined including the following mitigation measures:

- Removal of the approach embankment for the Old Munitions Bridge; and
- Raising the bridge deck of both the South Creek and Ropes Creek road crossings.

The key flood impacts of the Dunheved and Central Precincts for the Preferred Development Option in the 100 year ARI event was generally a small increase in flood levels outside of the site. The maximum increase in 100 year ARI flood level at South Creek cross section CH 31.778 km, upstream of the boundary of the site, was 37 mm. The maximum increase at South Creek cross section CH 34.778 km downstream of the boundary of the site was 11 mm. In a PMF event in the Hawkesbury-Nepean River, the proposed development is likely to cause negligible changes in flood levels on the site. These flood impacts were reviewed by both Blacktown and Penrith City Councils as part of the Dunheved Precinct Plan and the small increase in peak flood levels has been approved.

Dunheved Precinct Development Application Model

A Flood Impact Assessment Report dated 30th March 2007 was prepared and submitted to Penrith City Council for the Dunheved Precinct Development Application. Following issues raised by Council An addendum (*Dunheved Precinct Development Application – Flood Impact Assessment*

Addendum Additional Information dated 18 December 2007) was prepared that covered the scenario which is outlined below:

- Assumptions and inputs as described in *Dunheved Precinct Development Application – Flood Impact Assessment Report dated 30th March 2007*; and
- Assumptions relating to the proposed filling for Central Precinct and two other mitigations which were raising of both South and Ropes Creek bridges as per Dunheved Precinct Plan, (*North & South Dunheved Precinct Plan Water, Soils and Infrastructure Report May 2006*).
- The Addendum concluded that the proposed filling of the Dunheved Precincts (which was the subject of the development application) in combination with the proposed filling in the Central Precinct (not subject of the development application) resulted in similar flood levels reported in the "*Water, Soils and Infrastructure Report, SKM, May 2006*" report with a small increase in flood level upstream of the site and negligible downstream of the site. The upstream impact was limited to within the Dunheved Golf Course.

7.3 Proposed Fill Area

The existing topography of the Central Precinct is dominated by the major natural drainage lines nearby. There is an area of higher ground on the western and southern side of the Precinct, from where the site slopes downward towards the drainage lines and creeks. A portion of the Precinct is located below the 100 year ARI flood level, and filling of the floodplain is required to place the proposed development above this level. Similar to the Dunheved Precinct, protective fencing will be provided around the Central Precinct.

The fill area has now been refined through more detailed Precinct planning. The initial structure plan for the Central Precinct identified an education and village centre further to the north. The park area has therefore been moved from its previous location (in the southern portion of the Regional Open Space) to the northern portion of the Regional Open Space adjoining the education and village centre.

For the purposes of the flood impact assessment, a conservative approach was taken, assuming that filling would be maximised in the Central Precinct. Previous approved areas of filling and the proposed new area of filling are shown in the following **Figure 7-1**and **Figure 7-2**.

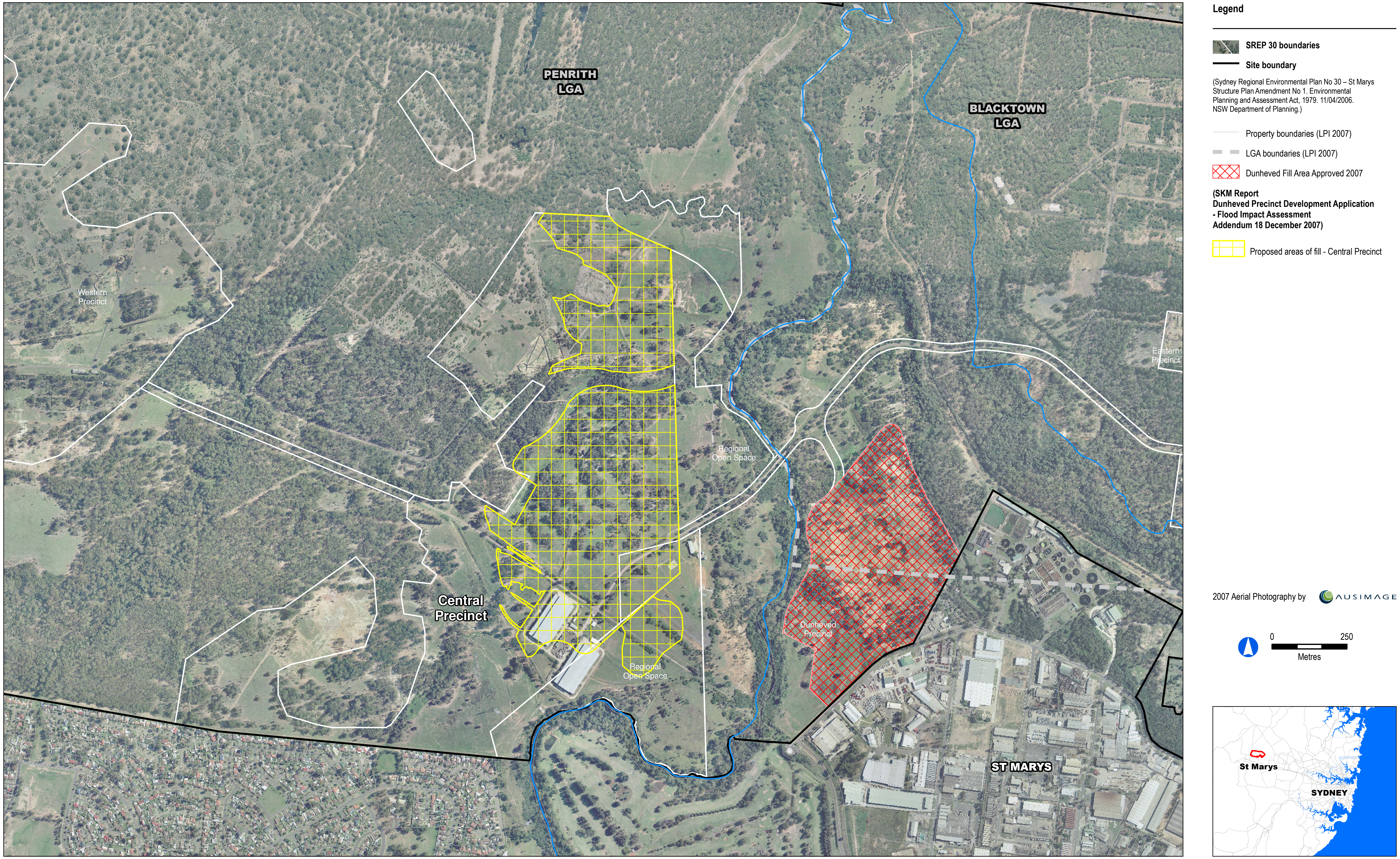

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Fig 7.1 : Approved Dunheved DA Fill Area

St Marys Development Project - Central Precinct

GDA 94 MGA Zone 56

June 18, 2008
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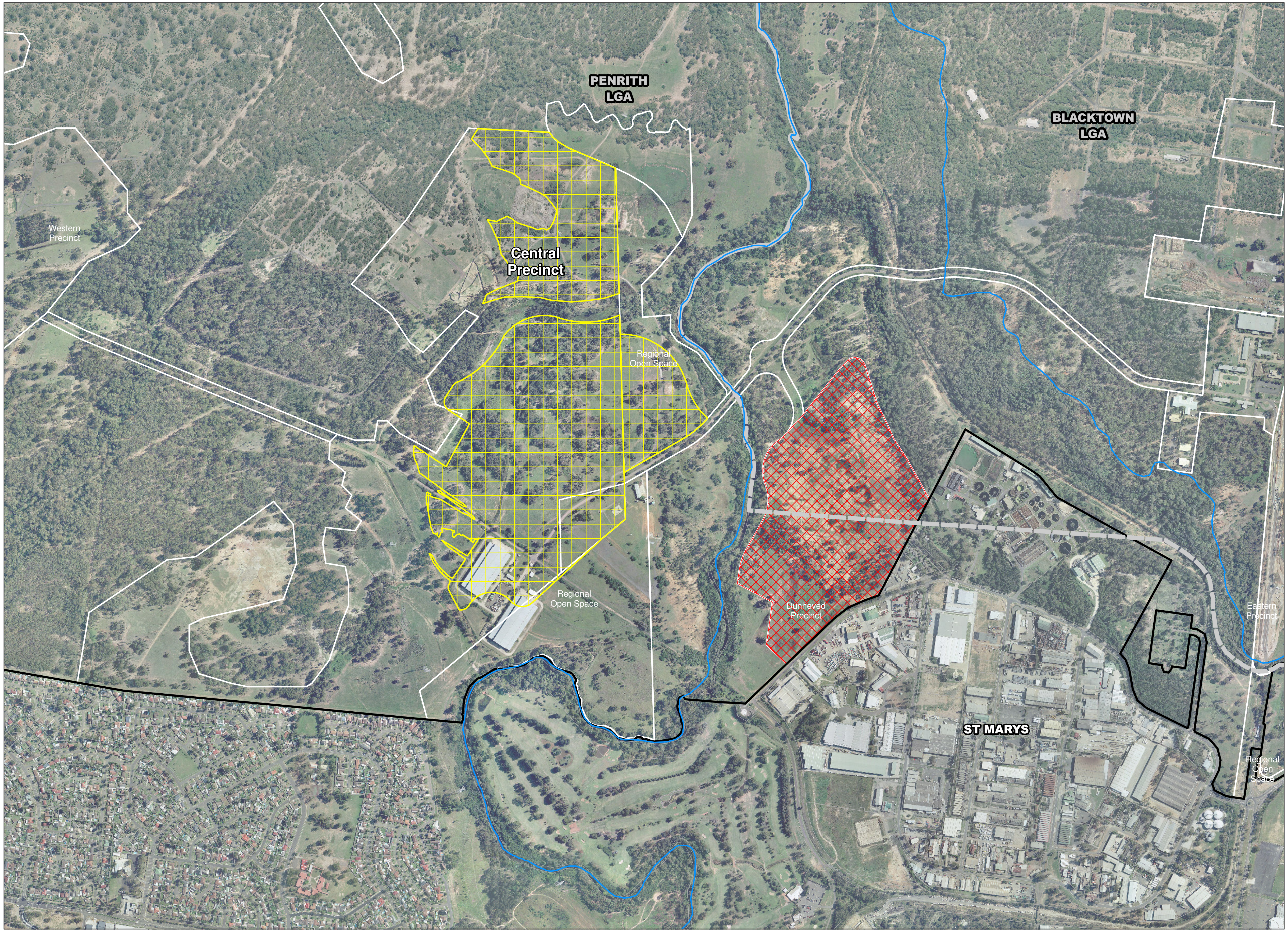

Legend

- **SREP 30 boundaries**
- **Site boundary**

(Sydney Regional Environmental Plan No 30 – St Marys Structure Plan Amendment No 1. Environmental Planning and Assessment Act, 1979. 11/04/2006. NSW Department of Planning.)

- Property boundaries (LPI 2007)
- LGA boundaries (LPI 2007)
- Dunheved Fill Area Approved 2007

**(SKM Report
Dunheved Precinct Development Application
- Flood Impact Assessment
Addendum 18 December 2007)**

- Proposed areas of fill - Central Precinct Plan 2008

2007 Aerial Photography by AUSIMAGE

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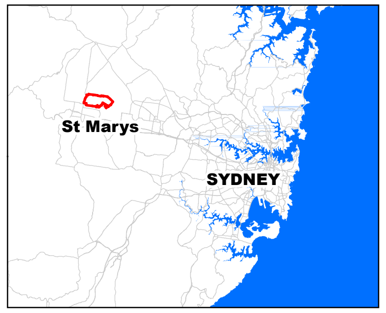
Metres



SINCLAIR KNIGHT MERZ

Fig 7.2 : Proposed Central and Approved Dunheved DA Fill Area

St Marys Development Project - Central Precinct

GDA 94 MGA Zone 56

June 18, 2008
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7.4 Hydraulic Modelling

The MIKE-11 model used in the (*Dunheved Precinct Development Application – Flood Impact Assessment Addendum Additional Information dated 18 December 2007*) was updated to incorporate the proposed filling on the Central Precinct as discussed above. Cross-sections used in the MIKE-11 model for South Creek to represent the combined development of Dunheved and Central Precincts are shown in **Appendix D**.

The updated MIKE-11 model was used to investigate flood impacts resulting from the combined development of the Dunheved and Central Precincts for the following events:

- 100 year ARI flood in South Creek and a concurrent 20 year ARI flood in the Hawkesbury-Nepean River; and
- PMF in South Creek and a concurrent 100 year ARI flood in the Hawkesbury Nepean River.

All the assumptions as part of the previous modelling were adopted. It is to be noted that mitigation options used in the Dunheved DA were included in the model. The mitigation measures represented in the model include the following:

- Removal of the approach embankment for the Old Munitions Bridge; and
- Raising the bridge deck of both the South Creek and Ropes Creek road crossings to provide waterway areas of approximately 980m^2 and 100 m^2 respectively in the 100 year ARI event.

As part of this recent modelling an additional assumption was made that the Transmission Easement would be blocked off and hence would not act as a floodway in the event of a 100 year ARI flood event in South Creek catchment. Details on MIKE-11 model runs for the above scenarios are given in **Appendix D**.

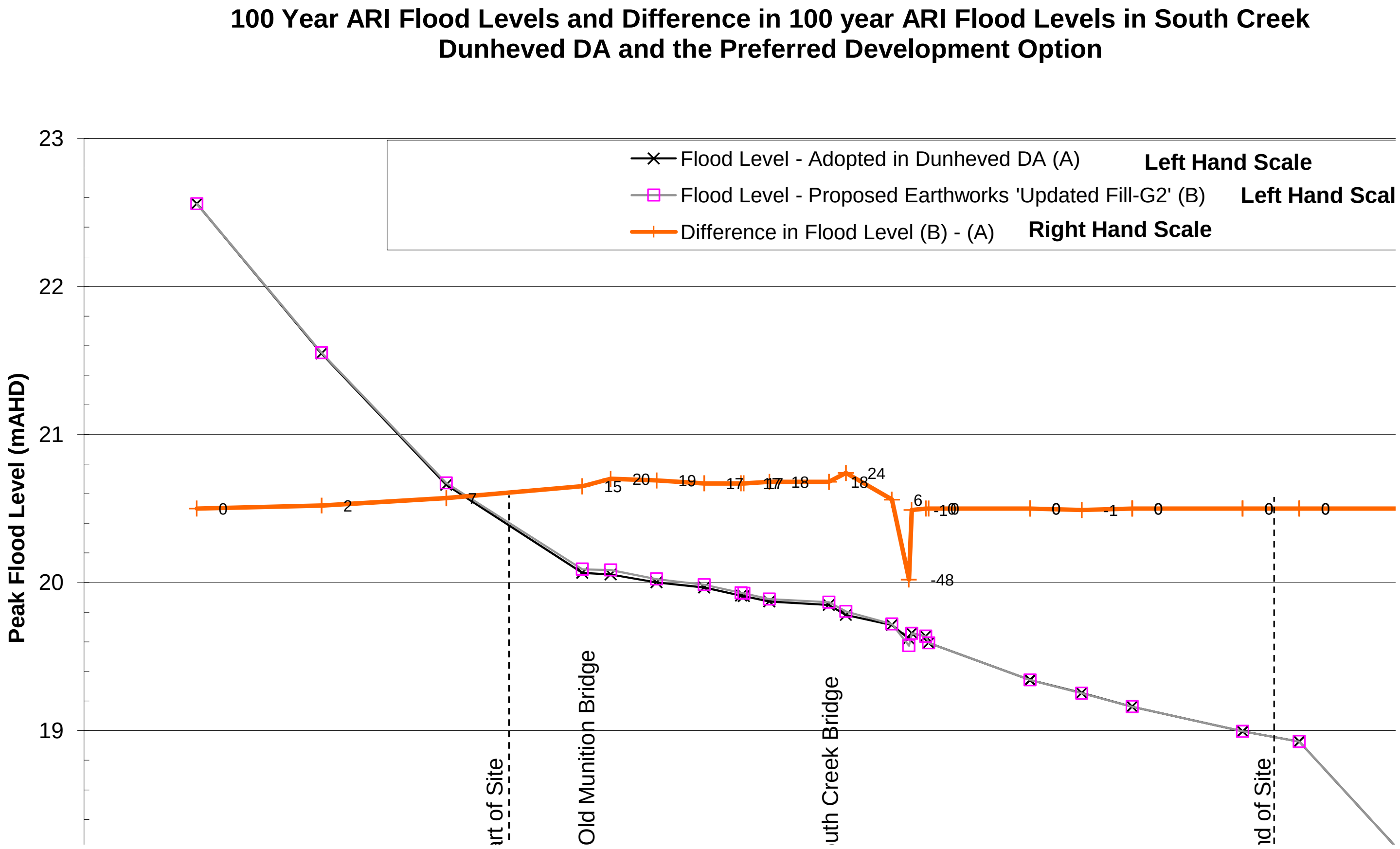
7.5 Impacts on Flood Levels

Peak flood levels at selected MIKE-11 cross sections in the vicinity of the St Marys Project site are given in **Appendix D**. Long section plots of the flood levels in the vicinity of the site are shown **Figure 7-3** (100 year ARI) and **Figure 7-4** (PMF). **Figure 7-5** shows approximate 100 year ARI flood inundation for the preferred development.

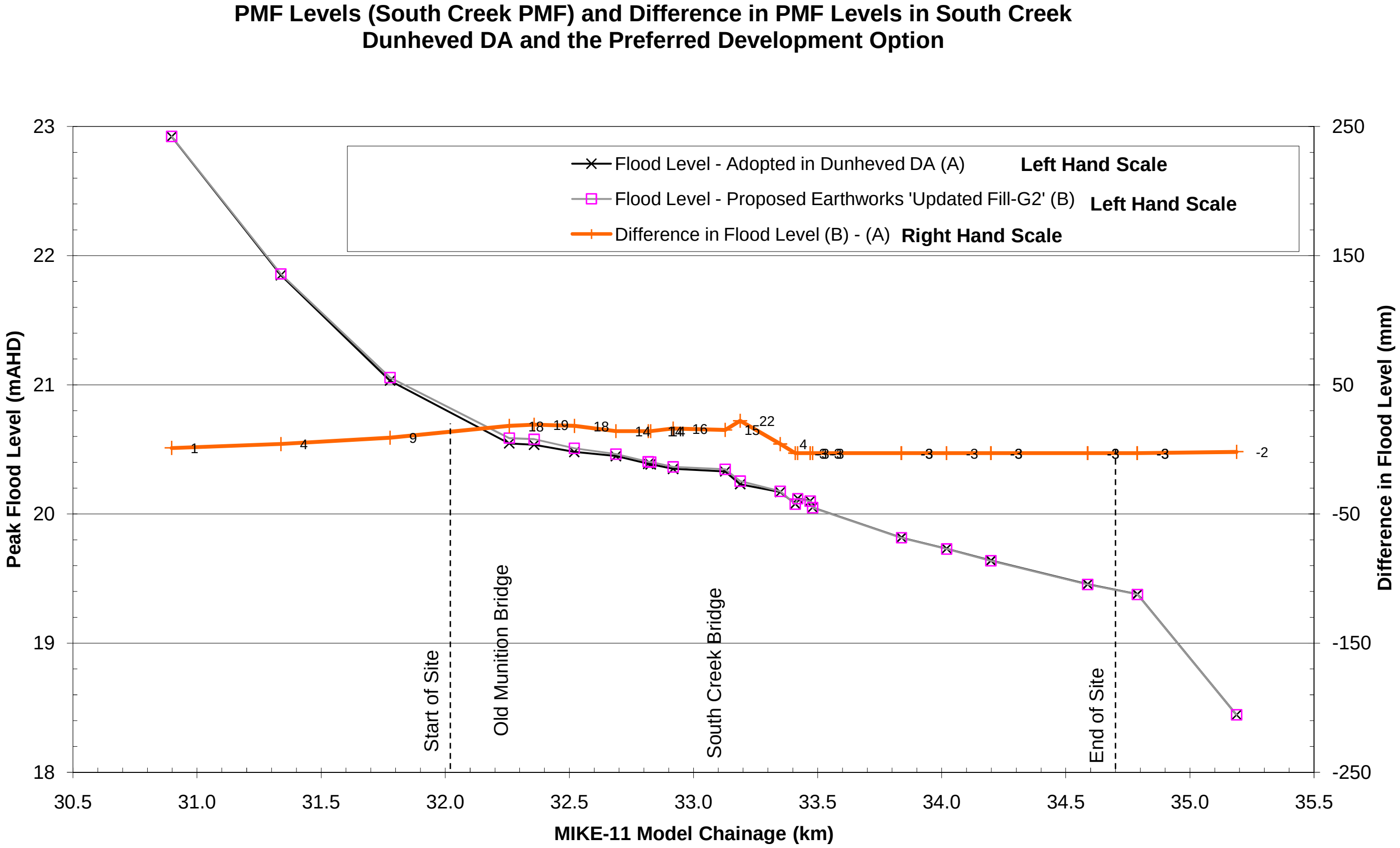
The flood modelling results indicate that the impacts of the proposed development would be:

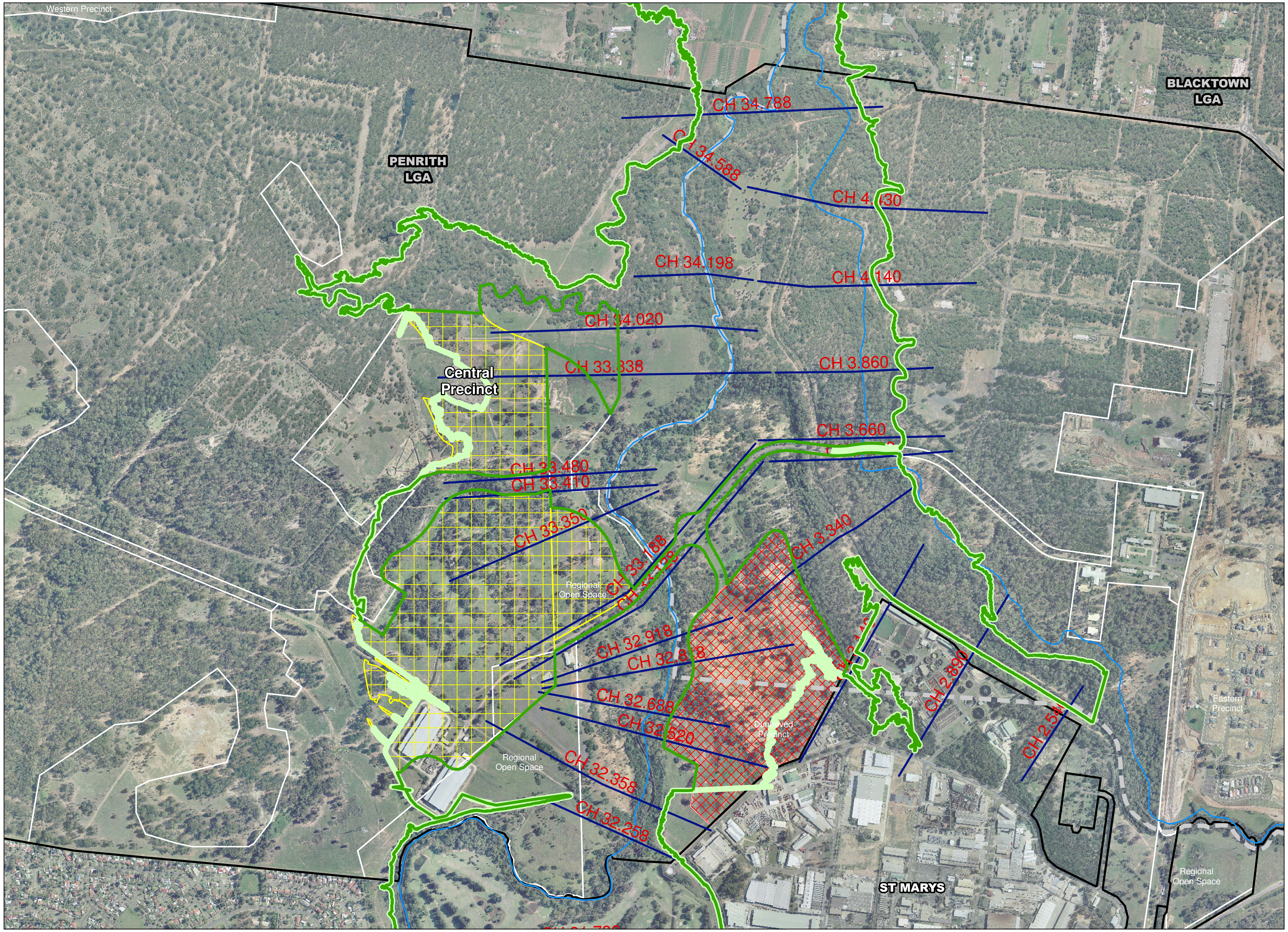
- A minor increase in flood levels upstream of the St Marys Project site in the 100 year ARI event. The maximum increment in flood level would be 7mm upstream (south) of the site at CH 31.778. The upstream impact is limited to within the Dunheved Golf Course.
- There would be no increase in flood levels downstream of the St Marys Project site in the 100 year ARI event (north) of the site at CH 34.778.
- In the South Creek PMF event, there would be a minor increase in flood levels upstream of the St Marys Project site. The maximum increment in flood level at CH 31.778 would be 9mm upstream (south) of the site. The upstream impact is substantially limited to within the Dunheved Golf Course. The largest increase in flood level would be 22mm immediately upstream of the South Creek Bridge. There would be a slight reduction downstream of the site for the PMF event.

- **Figure 7-3 100 Year ARI Flood Levels and Difference in 100 Year ARI Flood Levels in South Creek and the Preferred Development Option**



- **Figure 7-4 PMF Levels (South Creek PMF) and Difference in PMF Levels in South Creek and the Preferred Development Option**





Legend

SREP 30 boundaries

Site boundary

(Sydney Regional Environmental Plan No 30 – St Marys Structure Plan Amendment No 1. Environmental Planning and Assessment Act, 1979. 11/04/2006. NSW Department of Planning.)

Property boundaries (LPI 2007)

LGA boundaries (LPI 2007)

Dunheved Fill Area Approved 2007

(SKM Report Dunheved Precinct Development Application - Flood Impact Assessment Addendum 18 December 2007)

Proposed areas of fill - Central Precinct

Base case 1:100 year ARI flood extent

(North & South Dunheved Draft Precinct Plan – Water, Soils & Infrastructure Report 17/05/2006 Figure 6.4)

Preferred development without old munition bridge option 1:100 year ARI flood extent 2008

2007 Aerial Photography by AUSIMAGE

0 250
Metres

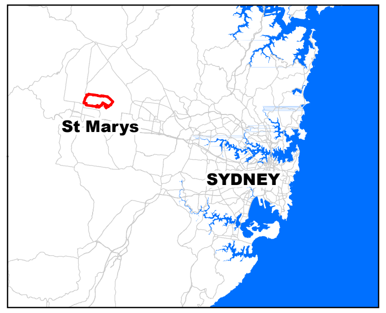


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Fig 7.5 : 100 year ARI Flood Inundation Map for the Preferred Development Option

St Marys Development Project - Central Precinct

GDA 94 MGA Zone 56

June 18, 2008
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7.6 Conclusion

The proposed filling of the Central Precinct in combination with the already approved Dunheved Precinct results in similar flood levels reported in *"Dunheved Precinct Development Application – Flood Impact Assessment Addendum Additional Information dated 18 December 2007"* report, which has been approved by Council. The upstream impact would be limited to the site and to within the Dunheved Golf Course.

8. Flood Evacuation Strategy

The overall flood evacuation objectives in the development are:

- To provide safe conveyance of local runoff;
- To bring ground levels on the developed lots on site are to least 500mm above the 100 year ARI flood level; and
- To conform to the requirements of the NSW Government Floodplain Management Manual.

8.1 Overall Approach

The site is in the Sydney Western Division of the State Emergency Service (SES) and within Penrith Local Government Area. The existing regional flood plan and local flood plans relevant to the site would be:

- Regional: Sydney Western Division Flood Plan; and
- Penrith Local Flood Plan.

The flood evacuation plan for the proposed development would be consistent with these regional and local plans.

Local Runoff

The site drainage system would be designed to convey runoff from storm events up to the 10 year ARI within the pipe system and up to the 100 year ARI within the overland system.

Development lot and floor levels would be at least 500mm above the 100 year ARI flood levels throughout the Precinct.

Evacuation is necessary in events larger than the 100 year ARI event. In a PMF event, a portion of the Central Precinct would become inundated by regional flooding, preventing local runoff from flowing away from the site.

8.2 Regional Flooding

Regional flooding is affected by two main types of events:

1) Type 1 - Floods in the Hawkesbury-Nepean River system. Ropes Creek and South Creek, which pass to the east of Central Precinct, are part of this system and are affected by backwater flooding from the Hawkesbury River at Windsor.
2) Type 2 - Floods due to storm events in the local catchments of the South Creek and/or Ropes Creek system/s.

Type 1 flooding is governed by the levels in the Hawkesbury River at Windsor. The Hawkesbury River has a large catchment and there are a number of flood gauges in the catchment, including one at Windsor. The Bureau of Meteorology issue flood warnings for the Hawkesbury River, including predictions of the likely flood level at the Windsor Bridge gauge at Windsor. The Bureau may provide 9-12 hours of warning for Windsor with greater warning time for the site.

A major flood in the Hawkesbury-Nepean system would affect many areas, including Windsor, Richmond and possibly parts of Penrith, lower South Creek and Eastern Creek around Riverstone. Flood warning information would be available over the radio and television and the SES would be conducting extensive evacuations of the likely affected suburbs. Flood levels may remain high for several days.

Type 2 flooding would occur quickly due to the relatively small catchment sizes of South Creek and Ropes Creek at the Central Precinct, and there would not be any specific flood warning available. There may be a Bureau of Meteorology Severe Weather Warning for the area, indicating the likelihood of severe storms and flooding; this may be issued up to 12-24 hours before such an event.

Floods on the South Creek and/or Ropes Creek system would rise and fall quickly, in a matter of hours. There would be little or no warning time.

Evacuation may be necessary during either type of flooding. The most logical evacuation route for the proposed development site would be to the west via the proposed roads.

An alternative evacuation route would be over both South Creek and Ropes Creek and hence towards high ground in the Eastern Precinct. This would be possible as the creek crossings would be passable and there is sufficient warning time of a major flood event.

Information on flooding and evacuation presented to businesses and residents in the Central Precinct would be consistent with Penrith Council flood education programmes.

8.3 Evacuation Strategy

The preferred strategy for residents and workers is to evacuate by car which is achievable and is described below. The approach taken is described in the NSW Floodplain Development Manual.

SES has developed an evacuation model for preparing Flood Evacuation Plans. The general process is as follows:

1) Decision to evacuate;
   - The Bureau of Meteorology provides forward warning advice to the SES to enable them to make decisions regarding the evacuation of the site.
2) Mobilisation of SES personnel;
   - The SES would organise staff to evacuate the site.
3) Communicating the need to evacuate the site by door-knocking each employment unit within the site;
   - There are multiple means of warning dissemination, including mass broadcasting of warning messages. The evacuation plan requires that sufficient time be allowed for every building to be door-knocked. Communication is usually done with volunteers working in pairs. It is estimated that on average it will take 1 minute for each employee to be warned at business premises and 5 minutes to warn each household by door-knocking.
   - One vehicle per employee and 1.8 vehicles per dwelling was assumed for the analysis.
4) Overseeing the flood evacuation traffic as evacuees leave the site in their vehicles. The total time to evacuate includes an allowance of 2 hours for evacuees to accept the fact that they need to evacuate (WAF), plus 1 hour allowance to provide for evacuees to organise themselves (WLF), their possessions and their property before leaving and a 1 hour travel safety factor (TSF) that allows for interruptions in the evacuation process due to temporary blockages of the route.

The analysis below is based on two evacuation routes one via the proposed Precinct connector road to the west, and one to the east via the zoned road corridor over both South Creek and Ropes Creek. There is also a third route available via the proposed “bus only” access at Leichhardt Avenue to the south. However this route was not included in the evacuation strategy. A typical lane capacity of 600 vehicles per hour for mid-block and intersection would be adopted for the Central Precinct evacuation (in comparison a normal lane capacity is around 1800 vehicles per hour for mid block). Allowing one lane in-bound for SES vehicles and assuming the evacuation routes mentioned above we have identified the required time to evacuate the entire Precinct even though only a portion of the Precinct is actually affected. A total evacuation capacity of 1200 vehicles per hour was assumed for the analysis.

Total evacuation time is defined to be the greater of the following:

Door-knocking + Warning Acceptance Factor (WAF) + Warning Lag Factor (WLF)

Or

Warning Acceptance Factor + Warning Lag Factor + Travel Safety Factor (TSF) + Travel Time

Assuming ten 2-person teams could be mobilised, Central Precinct's 760 employees and 967 dwellings could be warned of the need to evacuate in approximately 9.2 hours. The 760 employees figure is based on advice provided by the developer. The number of affected dwellings is based on the Central Precinct Structure Plan prepared by the developer. Whilst approximately 60% of residents would be required to evacuate, conservative estimates were prepared and this analysis assumed all residents would be evacuated.

The Precinct would generate approximately 2437 vehicles that would require 2 hours to evacuate (assuming 2 lanes of traffic was available for evacuation). Traffic would however be released at the door-knocking rate. Adding WAF (2 hours), WLF (1 hour) and TSF (1 hour) for evacuation and considering the above-mentioned formula is estimated that the site could be evacuated in approximately 12.2 hours.

As the structure plan for Central Precinct is developed a more refined road layout pattern including road levels would be developed. Considering all roads will be above the 100 year ARI levels for the site generally the lowest point in the Central Precinct would be the northern most point near flood cross section CH 34.020. Adopting this flood level plus 500mm freeboard generally makes the lowest point in the Central Precinct approximately RL 19.8m to RL 20.0m.

If we were to assume this level as the critical level at which access is cut (this would be conservative considering that the entire Precinct is still flood free including both South and Ropes Creek bridges) preventing evacuation by car we would require at a minimum approximately 12.8 hours warning time to evacuate the Precinct.

It is understood from the SES that the Bureau of Meteorology can forecast 100 year Average Recurrence Interval (ARI) peak flood level in the Nepean River at Victoria Bridge in Penrith 7 hours in advance. A peak flood level at Windsor for the 100 year ARI would occur approximately 12 hours after the 100 year ARI peak flood level is reached in the Nepean River at the Penrith gauge (Source: Water Board (1994) Warragamba Flood Mitigation Dam EIS Flood Study, Part E, Flood Mitigation Dam). It would take another 6 hours for the PMF hydrograph to approximately reach RL 20m at Windsor from the 100 year ARI peak flood level (Source: Water Board, 1994).

Ignoring the travel time between Windsor to Central Precinct, the warning time would be approximately 25 hours. The available 25 hours is significantly greater than the 12.8 hours of warning time required to evacuate a portion of the Precinct. On this basis it can be concluded that there is sufficient time for vehicular evacuation of the site. Moreover the flood evacuation could be accomplished using one road route and thus is responsive to infrastructure staging needs should only access to the Central Precinct be initially from the west only.

8.4 Conclusion

On the above basis, it can be concluded that there is sufficient time to vehicular evacuate the site for the Probable Maximum Flood, using prescribed flood evacuation methodology.

Appendix A Assessment of Drainage Controls

A.1 Hydrological Model

A XP-RAFTS model was developed for the Central Precinct to represent the hydrological network. 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 flows at specified locations in the drainage system for the following conditions;

- Existing catchment conditions
- Proposed developed catchment conditions (without flow mitigation)
- Proposed developed catchment conditions with flow mitigation

A.2 Model Input Data

Catchment Data

Catchment delineation was undertaken for the previous St Marys study in 1998. These catchment boundaries were reviewed using 2m contours from Airborne Laser Survey (ALS) data. Some adjustments were made to ensure contributing areas to proposed wetland/detention basins were correct. Each catchment was subdivided to represent the rural and urban portion in the existing and developed case. The percentage impervious adopted in the model is as follows;

Existing Case

Urban Area outside the site – 50% impervious

Rural (within and outside the site) – 5% impervious

Developed Case

Urban (within the site) – 70% impervious

Urban (south catchment overlapping site boundary) – 60% impervious

Rural – 5% impervious (unchanged from existing case)

These values are based on the following assumptions:

- No development will occur in the regional park therefore % impervious does not change;
- Areas allocated for urban development (including education and road areas) will have varying impervious percentages between 50-70%. For the purpose of the Precinct Plan the more conservative 70% has been adopted for all areas; and
- Existing urban areas external to the site will be unchanged from existing, i.e. 50% impervious.

Rainfall Intensities and Loss Parameters

Penrith City Council IFD data was used in the RAFTS model. A suite of storm durations were input for each ARI rainfall event. IFD data is shown in **Table A 1** below.

Table A 1 Penrith City Council IFD Rainfall Data

Duration (min)	2yr ARI	5yr ARI	10yr ARI	20yr ARI	50yr ARI	100yr ARI
20	52.82	69.66	79.08	91.89	108.85	121.9
30	42.83	56.47	64.09	74.46	88.19	98.75
60	29.05	38.28	43.43	50.44	59.72	66.86
90	23.04	30.31	34.36	39.89	47.19	52.81
120	19.48	25.6	29	33.65	39.79	44.51
180	15.33	20.12	22.78	26.41	31.21	34.89
360	10.16	13.3	15.04	17.42	20.56	22.97
720	6.75	8.81	9.95	11.51	13.57	15.15

Loss parameters used in the model are as follows;

- Impervious Losses; Initial 1.0mm Continuing 0.5mm
- Pervious Losses; Initial 10.0mm Continuing 2.5mm
- Bx factor 1.0

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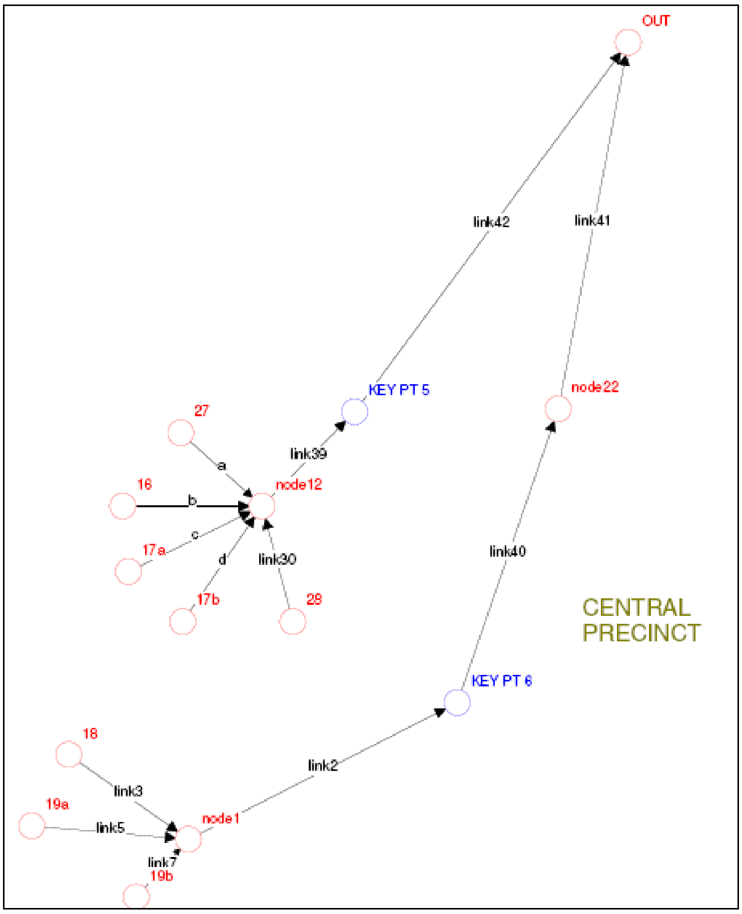
A.3 Existing Model

The layout of sub catchments of the existing RAFTS model is shown in **Figure A 1**. Sub catchment parameters are listed in **Table A 2.**

- **Figure A 1 RAFTS Model Schematic Layout – Existing**



- **Table A 2 Sub-catchment Parameters – Existing**

Catchment	Area (ha)	% Impervious
16	9.1	0
17a	4.8	0
17b	4.2	0
27	58.5	5
18	19	0
19a	43.6	50
19b	43.87	50
28	16	5

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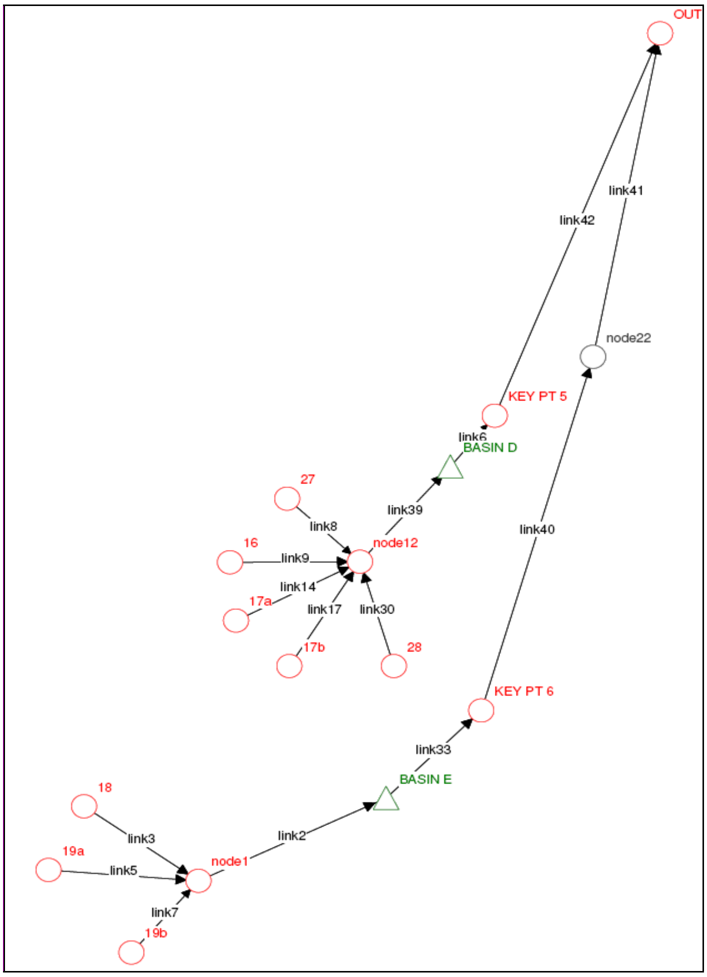
A.4 Proposed Model

The layout of sub catchments of the existing RAFTS model is shown in **Figure A 2**. Sub catchment parameters are listed in **Table A 3.**

- **Figure A 2 RAFTS Model Schematic Layout – Proposed**



- **Table A 3 Sub-catchment Parameters – Proposed**

Catchment	Area (ha)	% Impervious
16	9.1	8
17a	4.8	61
17b	4.2	14
27	58.5	81
18	19	71
19a	43.6	78
19b	43.87	75
28	16	69

A.5 Existing Peak Flows

In order to meet the water quantity objective, post development peak flows must not exceed existing peak flows for a range of events from 2 year to 100 year ARI. The existing RAFTS model was run for a range of storm durations and events. The existing peak flows at a two key points in the catchment for the 100 year and 2 year storms are presented in **Table A 4** and **Table A 5** respectively.

A.6 Developed Site Peak Flows

Hydrological analysis of the developed site conditions was undertaken using the RAFTS model (initially with no onsite detention included). Peak flows were extracted at the fore-mentioned key locations and compared to the existing case. A comparison of developed (without detention) and existing flows for the 100 year and 2 year events are provided in **Table A 4** and **Table A 5.**

- **Table A 4 100 Year ARI Existing and Developed (with no detention) Peak flows**

Event	Peak flows (m^3/s)	
	Existing	Proposed (no detention)
Key Point 5	10	32
Key Point 6	26	44

- **Table A 5 2 Year ARI Existing and Developed (with no detention) Peak flows**

Event	Peak flows (m^3/s)	
	Existing	Proposed
Key Point 5	3	14
Key Point 6	10	20

The results in indicate that without detention, the proposed development would increase peak flows within the site for a range of storm events. This is due to the increase in impervious catchment area attributed to the proposed Precinct development. Detention facilities are required to reduce the peak flows from the development to ensure they do not exceed existing flows.

A.7 Detention Basins

Two detention basins are proposed for the Central Precinct for peak flow mitigation for 2 year to 100 year ARI storm events. The two basins (D and E) are located within the Central Precinct as shown on **Figure 4-1**. The detention basins have been designed for events up to and including the 100 year ARI storm; peak flows were checked in the 2, 10 and 100 ARI events, to ensure that peak developed flows would not exceed peak existing flows.

Each of these basins would have both a low-level outlet and a spillway. In most storm events, the low-level outlets would control the flow and the basins would not fill to the level of the spillway. However in the case that the low-level outlets are fully or partially blocked submerging the low-level outlets, storm flows could still safely exit the site via the spillways. The detained water will be discharged within a day and be temporarily stored above the permanent pools in the basin (which are present for water quality treatment).

Results

Peak flows for the developed case in comparison to the existing case are presented in **Table A 7** and **Table A 6** for the 2yr and 100 yr ARI events.

- **Table A 6 Predicted Developed Peak Flows – 100 year ARI**

Event	Peak flows (m^3/s)	
	Existing	Proposed
Key Point 5	10	9
Key Point 6	26	26

- **Table A 7 Predicted Developed Peak Flows – 2 year ARI**

Event	Peak flows (m^3/s)	
	Existing	Proposed
Key Point 5	3	2
Key Point 6	10	10

The results indicate that the proposed detention system attenuates all flows up to and including the 100 year ARI events. Detention storage will occur above a permanent wetland area, the size of which has been determined from the water quality assessment.

Appendix B Assessment of Water Quality Controls

B.1 MUSIC Modelling

A water quality assessment was undertaken using the MUSIC water quality model (eWater CRC, Version 3.01). The main purpose of the modelling was to determine the land take required for the stormwater management wetlands to ensure that the water quality objective of no net increase in annual pollutant load into the receiving waterways is met.

Data

The following data were used in the model:

- **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 1996 to November 2003. Since the model was run at a small (6 minute) timestep, one year of rainfall data was used with 1997 chosen as the average rainfall year.
- **Catchment areas:** The study area was split into smaller catchment areas as used in the 1998 SKM report. The catchment characteristics were then updated according to information from the latest land use plan. **Table B 1** provides all the subcatchment areas used in the Music model; these are shown in **Figure B 1**.
- **Event Mean Concentrations**: Long term water quality monitoring data for the site is currently not available. In order to estimate the existing pollutant runoff loads and determine the effectiveness of the proposed stormwater management ponds, the Event Mean Concentrations (EMCs) for Total Suspended Solids (TSS), Total Phosphorus (TP) and Total Nitrogen (TN) have been based on data from the 1998 SKM report with some modifications made. The EMCs used in the model for the existing and developed cases are provided in **Table B 2**. Data from *Stormwater Flow and Quality and the Effectiveness of Non-Proprietary Stormwater Treatment Measures* (Monash University and CRC for Catchment Hydrology, 2004) was reviewed. The CRC data on EMCs was similar to the concentrations given in **Table B 2**. These EMCs are also similar to the measured stromwater concentrations for typical urban catchments in Sydney in the early 1990s by Sydney Water. For consistency purposes, the previously adopted EMC in the 1998 report were used.

- **Table B 1 Music Model Catchment Areas**

Catchment Name	Area (ha)
1	61.7
2	176.3
3	13.6
4	21.4
5	8.7
6,7,8,25	137.2
9a 10a	83.6
9a 10b	49.5
9b,11,12a	102.4
1.2,12-15,20-22	308.7
C3	55
23-24	74.9
17ab,16	18.1
27	58.1
18,19ab	42.5
19a	22.3
28	21.2
26	47.1
20	22.2

- **Table B 2 Event Mean Concentrations**

Site conditions	TSS (mg/L) Storm Flow (Wet)	TSS (mg/L) Base Flow	TP (mg/L) Storm Flow (Wet)	TP (mg/L) Base Flow	TN (mg/L) Storm Flow (Wet)	TN (mg/L) Base Flow
Existing	50	7.9	0.075	0.075	1	0.75
Developed	110	12.6	0.2	0.1	1.5	1.0

B.2 Methodology

The following methodology was adopted in the MUSIC model:

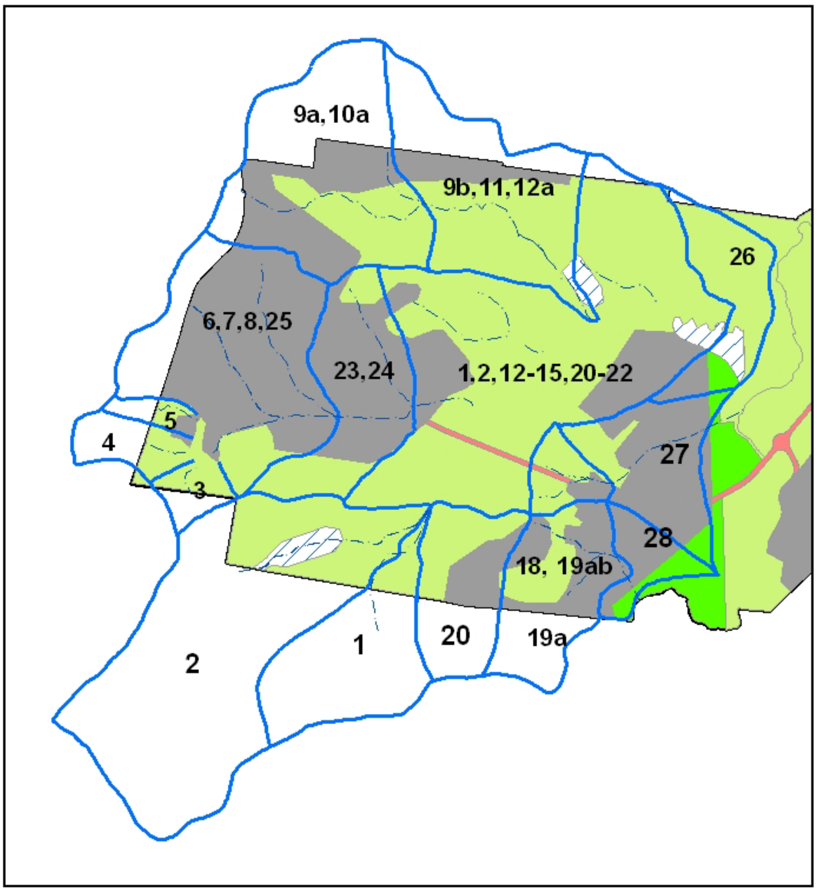
- The Western and Central Precincts have been considered together for water quality purposes. There are three discharge areas for these two Precincts: at S1, S2 and S3 as shown **Figure 4-1.** The combined annual pollutant load at the discharge points for the existing case was compared to the combined annual pollutant load in the developed case. This is similar to the approach that was adopted in the 1998 SKM Watercycle Management Report. The objective for the Western and Central Precincts is that the combined annual pollutant export from the developed site does not exceed the existing.

- It has been estimated that the actual stormwater management wetland surface area is approximately 75% of the land take required. The remaining approximated area would be required for detention, pathways and benching purposes. The modelling assumes a concept design whereby twenty percent of the total wetland area would be an inlet zone. The remaining 80% represents the open water and macrophytes zone areas. 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.

- Other WSUD water quality controls such as those listed in this report have not been included in the Music model. These details will be considered during the subsequent stages (ie: development application) when other water quality controls such as the additional WSUD controls and GPTs on site would also be assessed. This represents a conservative modelling approach for the Precinct Plan assessment.

- **Figure B 1 Music Model Sub-catchment Areas**



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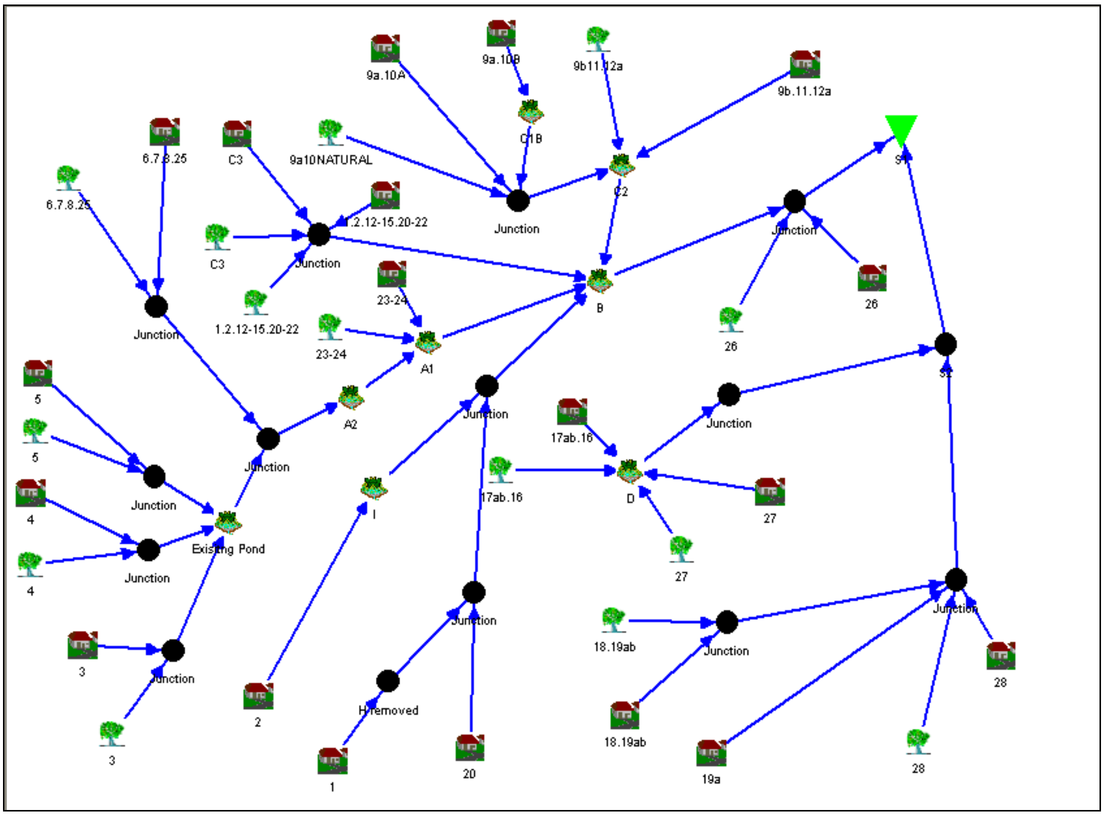
- **Figure B 2 Water Quality MUSIC Model Layout for the Western and Central Precinct**



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B.3 MUSIC Model Results

Western and Central Precincts

The indicative locations of the proposed stormwater management wetlands that would meet the water quality objective for the Western and Central Precinct are shown in **Figure 4-1.** The exclusion of the other WSUD controls from the water quality modelling provides a conservative approach and hence the results in this Precinct Plan report would be conservative. The estimated land take for the proposed wetlands ponds for water quality purposes only are provided in **Table B** 3**.**

- **Table B 3 Proposed Stormwater Management Pond Sizes for the Western and Central Precincts (Water Quality Only)**

Stormwater management pond ID	1998 Study (Basis of SREP 30) Wetlands Land Take (ha) [1]	SREP 30 Draft Amendment (2005) Drainage Zones Land Take (ha)	Precinct Plan [2] Minimum [3] land take (ha) for water quality purposes only
A1	2.2		1
A2	3.7		1.8
B	6	8	8
C1	3.4		1
C2	2.8	4.5	4.5
C3	1.4		0
D	0.6		2
E	1.4		0
F	0.6		0
G	0.7		0
H	1.6		0
I	4	7.4	7.4
EX1	2.6		0
Total	**31**	**19.9**	**25.7**

1- These 1998 Study landtake estimates are for water quality and detention requirements. These areas do not include benching or pathway areas.

2- For this Precinct Plan assessment, it has been assumed that the actual stormwater management wetland surface area is approximately 75% of the land take required shown in the above table.

The MUSIC model can provide the annual pollutant load exported for Total Suspended Solids (TSS), Total Phosphorus (TP) and Total Nitrogen (TN). The results for the existing case, the developed case with no water quality controls and the developed case with controls are provided in **Table B 4.** The values in brackets are the results compared to the existing case.

SKM

- **Table B 4 MUSIC Results for the Western and Central Precincts**

	TSS (kg/year)	TP (kg/year)	TN (kg/year)
Existing	240,000	426	3,900
Developed, no controls	357,000 (+50%)	620 (+46%)	4,920 (+26%)
Developed, with controls	113,000 (-53%)	290 (-32%)	3,620 (-7%)

Note: The % values in brackets are the results compared to the existing case. The target reduction is -5% for the worst pollutant which provides a safety margin. The actual margin is in the range of approximately 5% for TN and upto 50% for TSS.

Appendix C Groundwater and Soils

C.1 Douglas Partners Report & 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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Douglas Partners Pty Ltd
ABN 75 053 980 117

96 Hermitage Road
West Ryde NSW 2114
Australia

PO Box 472
West Ryde NSW 1685

Phone (02) 9809 0666
Fax (02) 9809 4095

sydney@douglaspartners.com.au

The Association of Consulting Engineers Australia

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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.

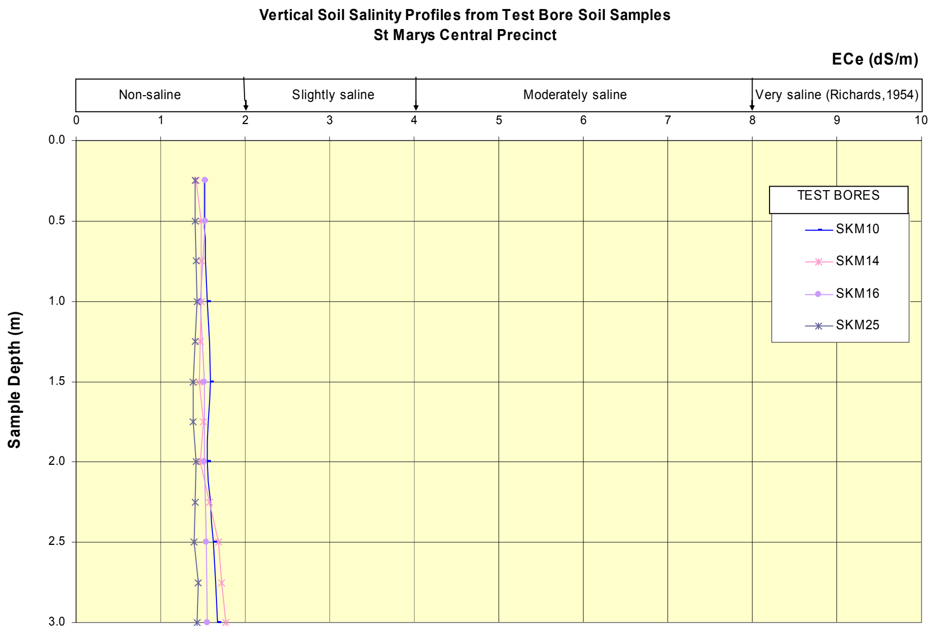

Figure 2a – "Uniform" Vertical Soil Salinity Profiles

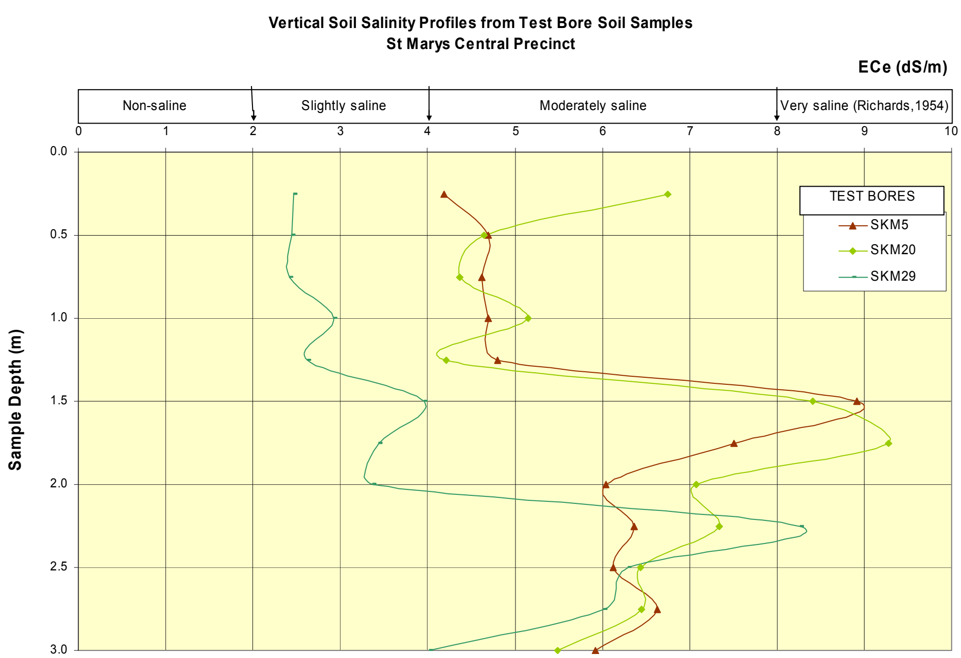

Figure 2b – "Intermittent" type Vertical Soil Salinity Profiles

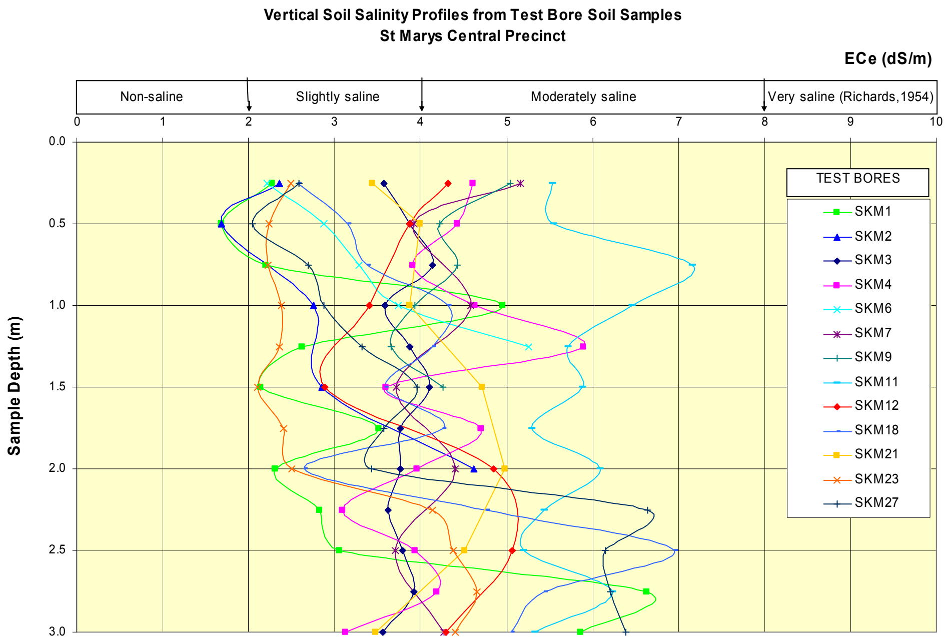

Figure 2c – "Mixed" Vertical Soil Salinity Profiles

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 <u>at the test bore locations</u> 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.

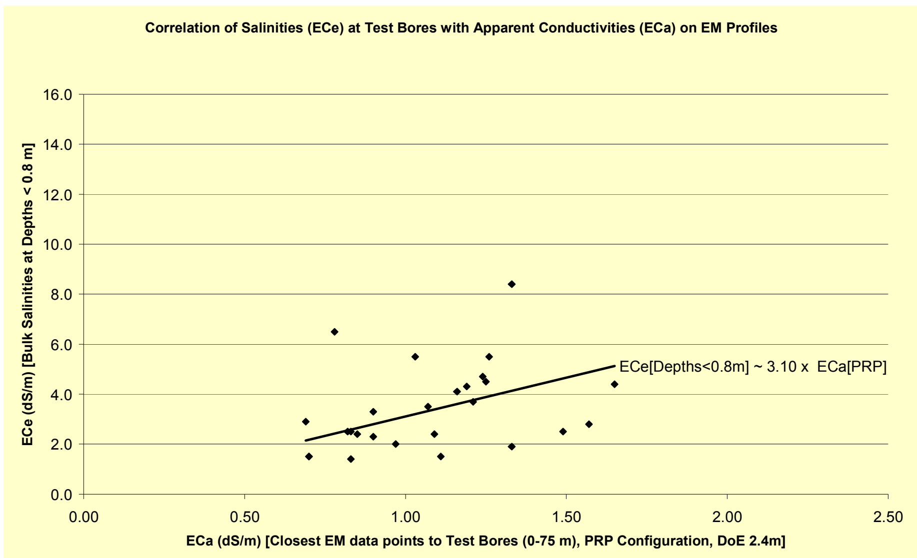

Figure 3 – Correlation of Bulk ECe (above 0.8m) and ECa (PRP) data

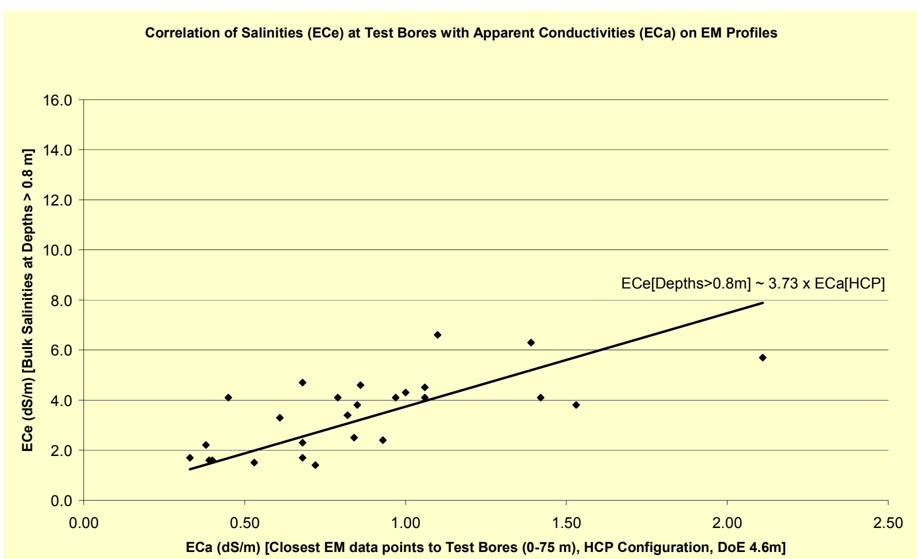

Figure 4 – Correlation of Bulk ECe (below 0.8m) and ECa (HCP) data

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:

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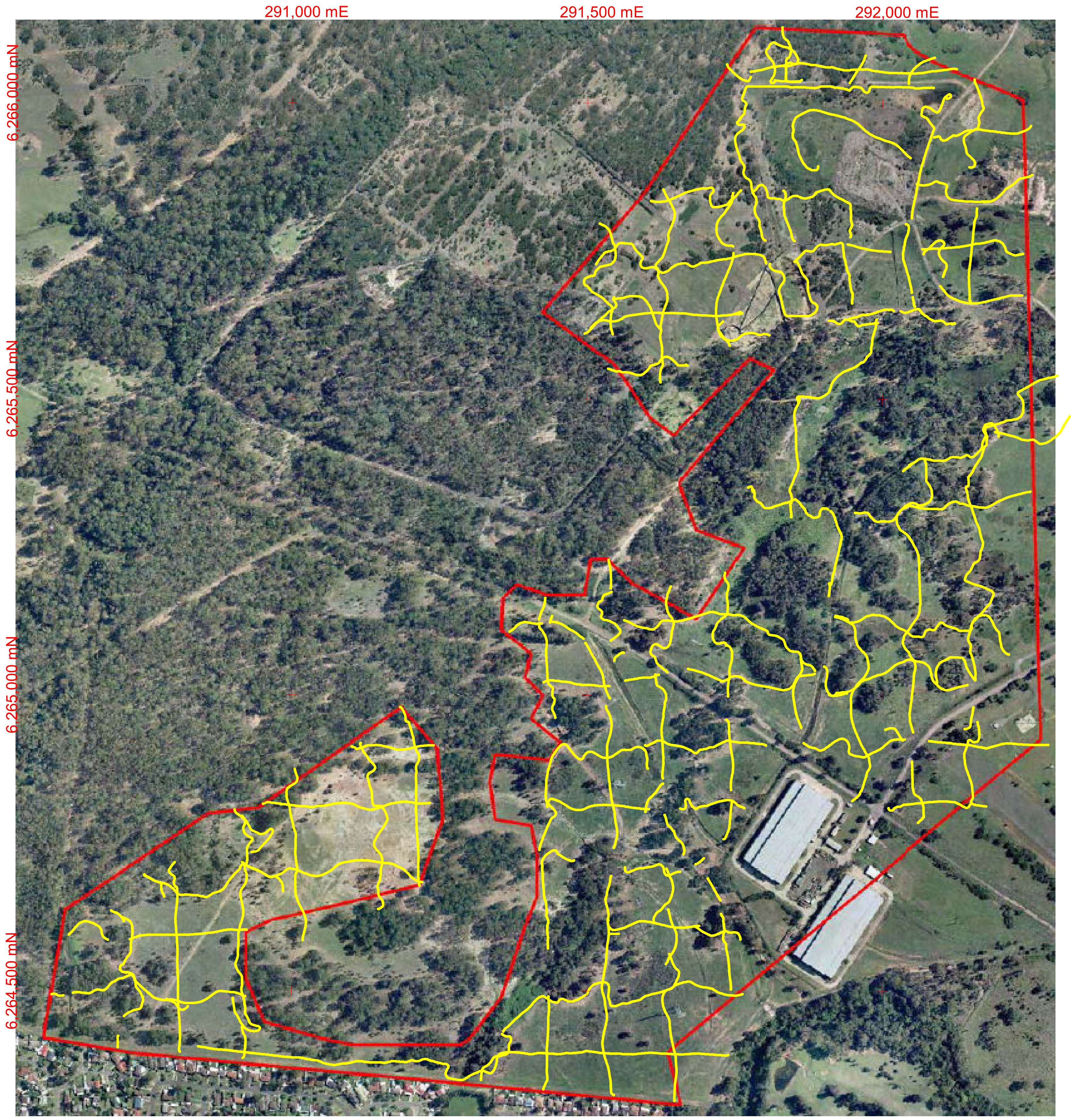
APPENDIX A
Drawings 1 to 7



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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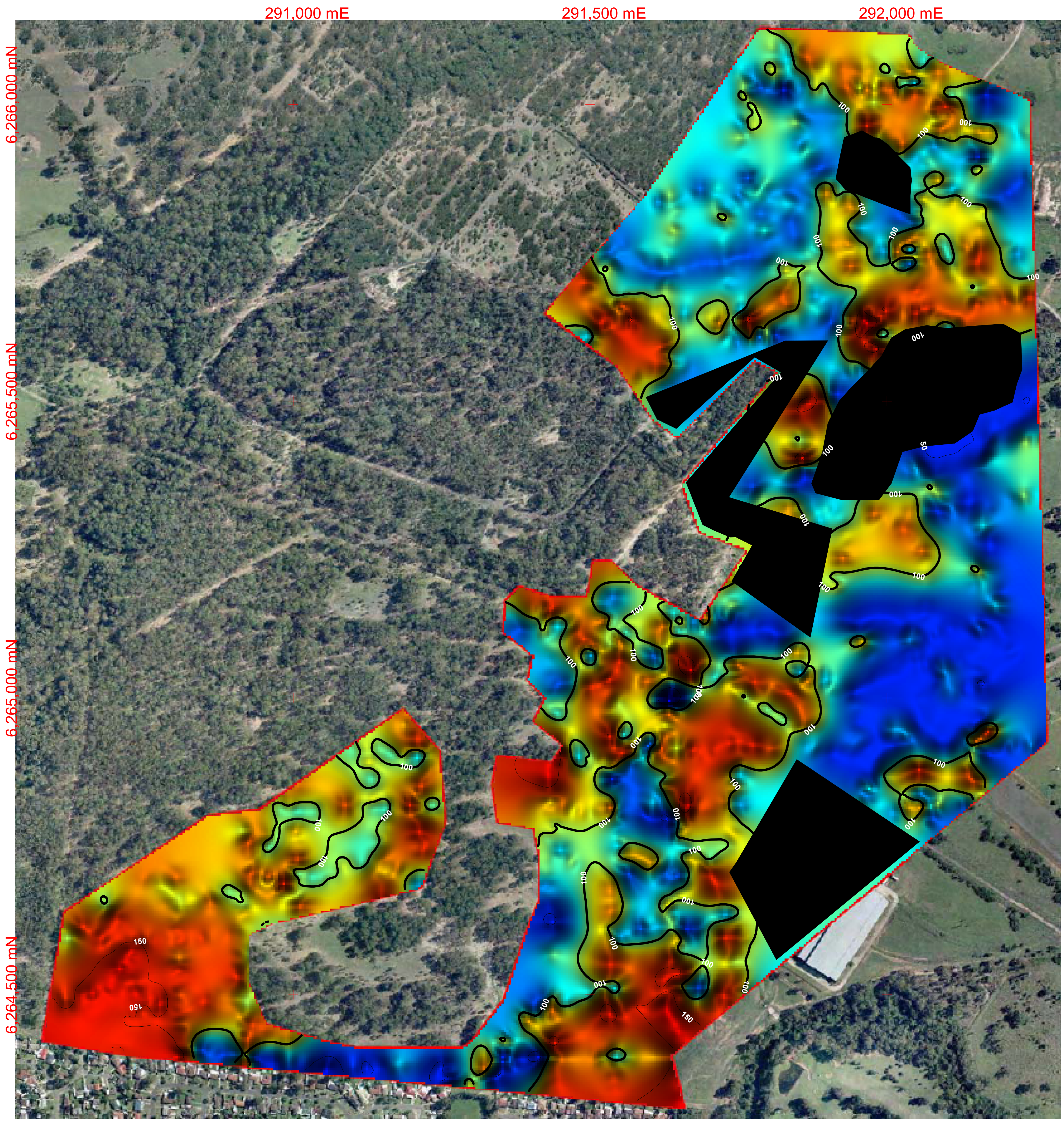
TITLE:

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.

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

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

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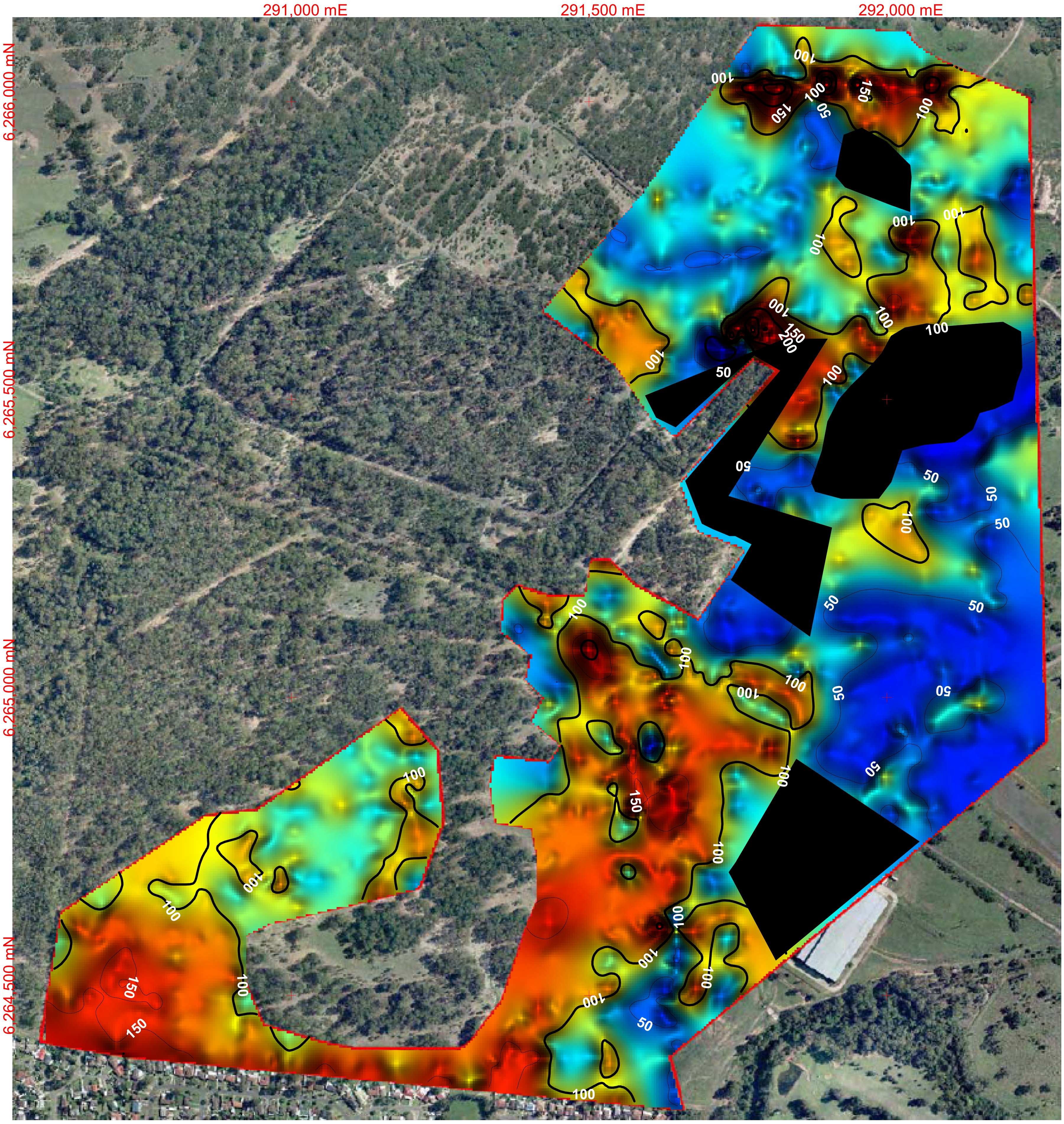
TITLE:

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.

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

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

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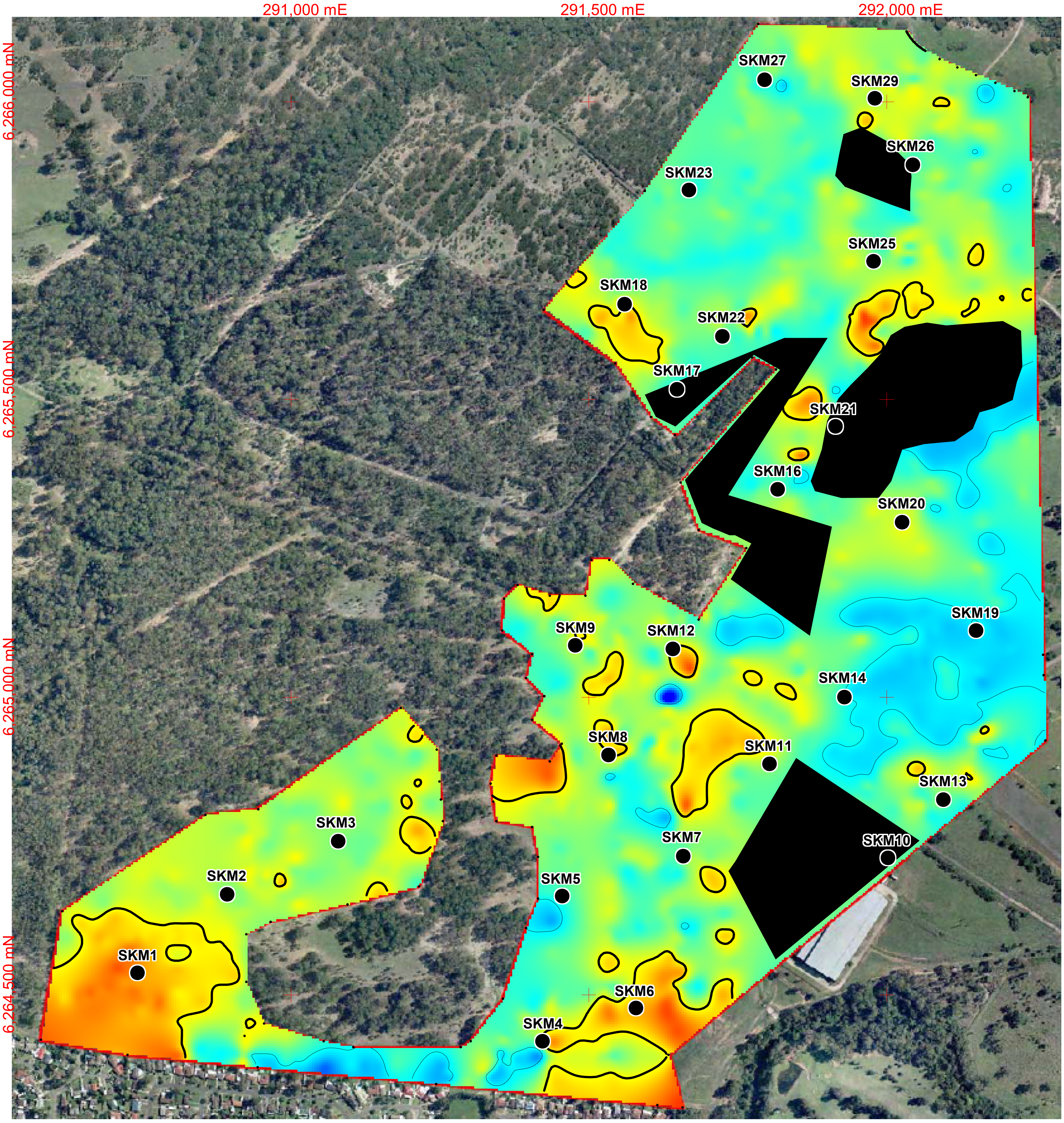
TITLE:

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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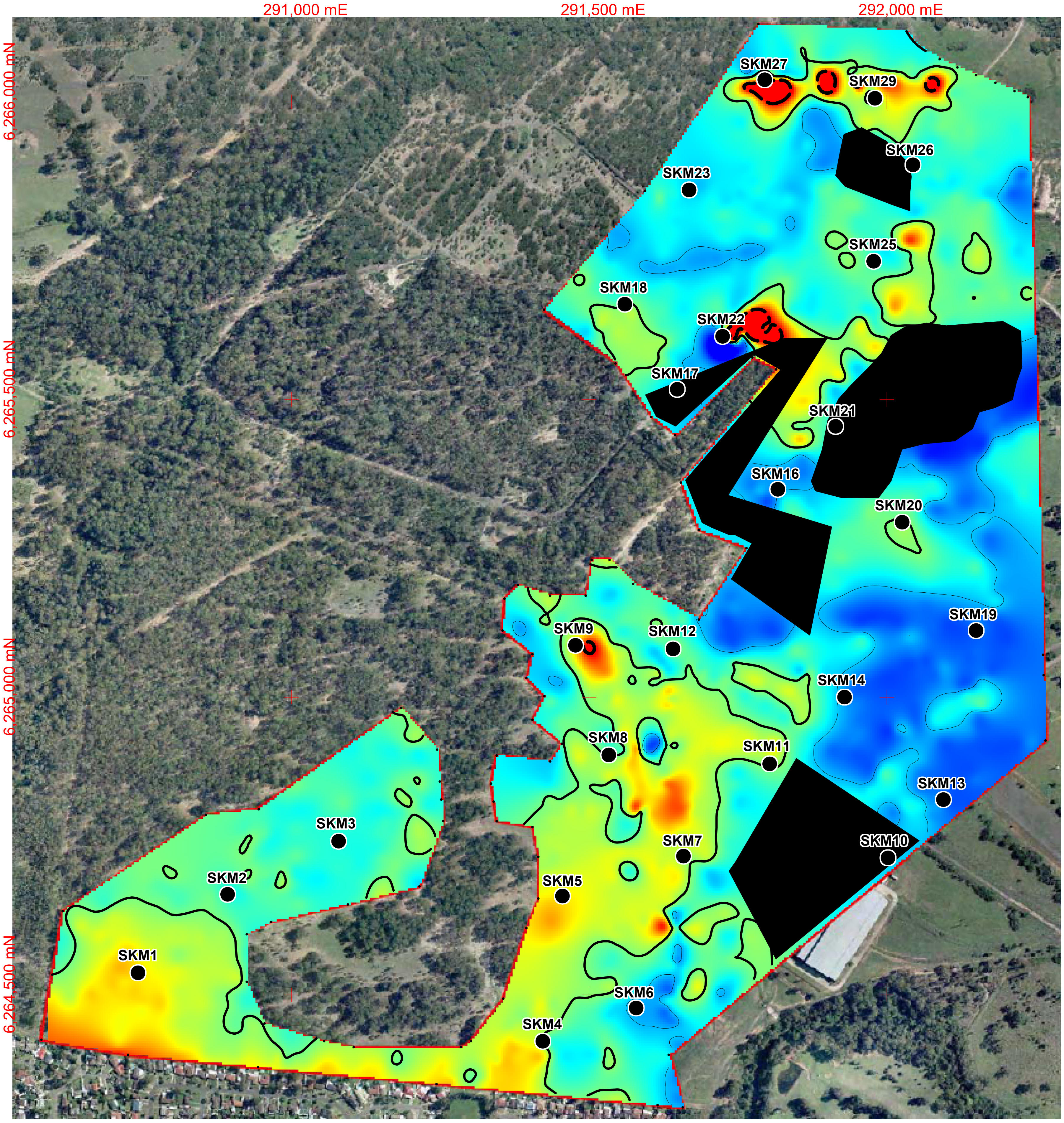
TITLE:
APPARENT SALINITIES AT DEPTHS < 0.8 m
SALINITY INVESTIGATION
CENTRAL PRECINCT
ST MARYS, NSW

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.

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

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

8	Very saline
6	Moderately saline
4	
	Slightly saline
2	
0	Non-saline

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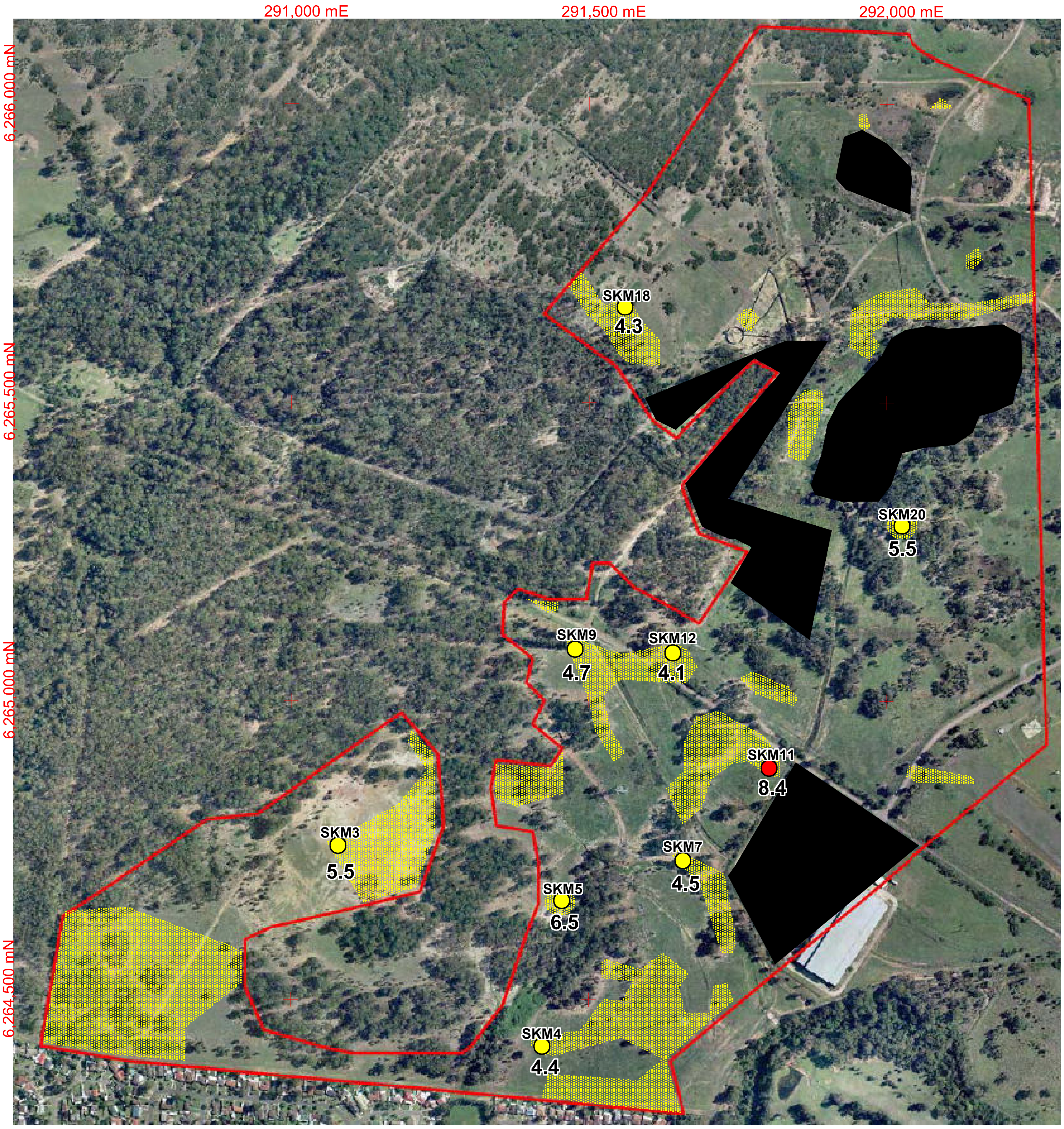
TITLE:

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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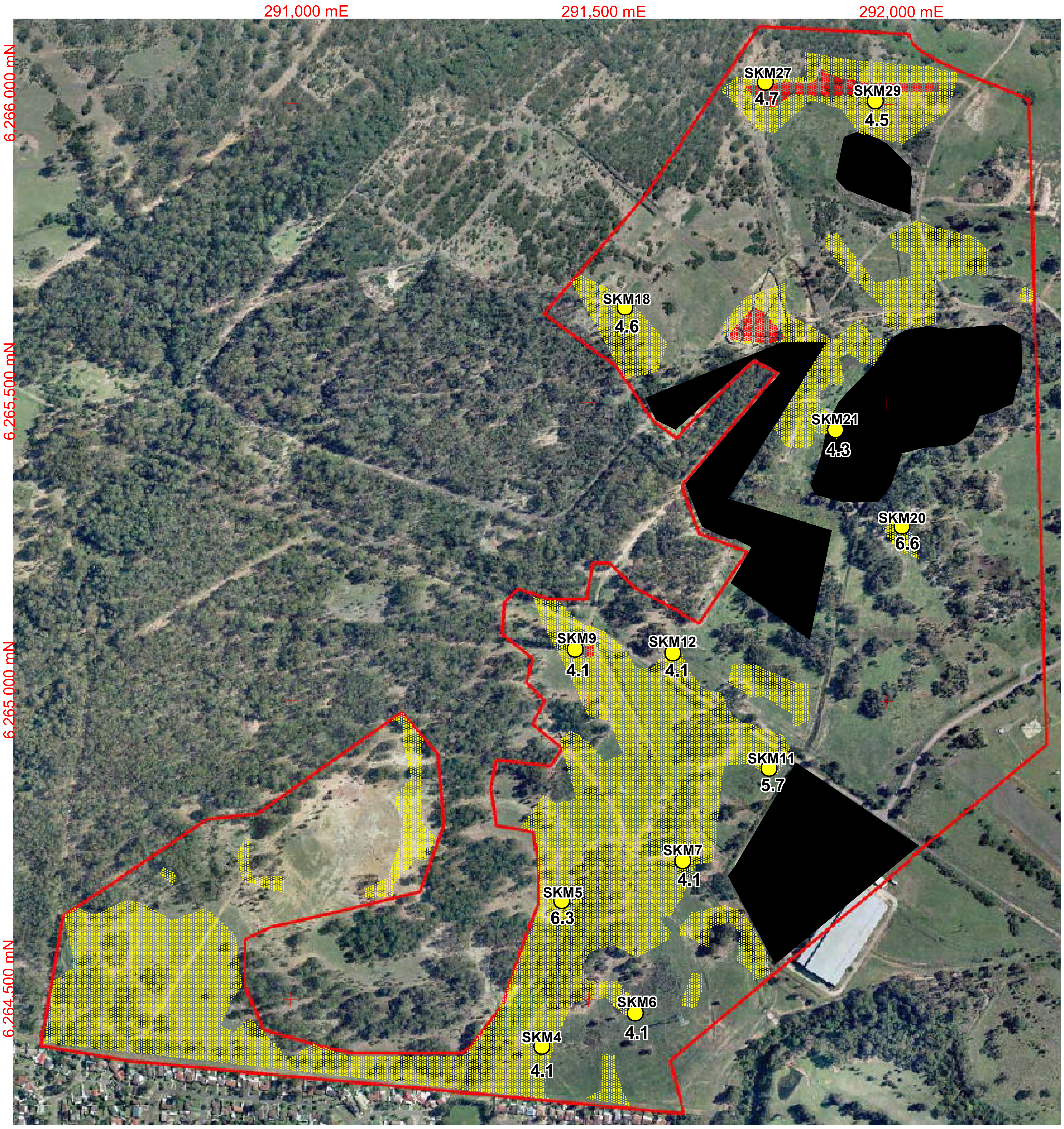
TITLE:

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		
				0.50	6.8	B	Non-Aggressive	Non-Aggressive	Clay loam	9	187	1.7	Non Saline	2.8	Slightly 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		
				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	3.8	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		
				1.50	4.8	B	Mild	Non-Aggressive	Clay loam	9	317	2.9	Slightly Saline	3.4	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		
				0.50	8.9	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	277	3.9	Slightly Saline	5.5	Moderately 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		
				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	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		
				0.50	7.5	B	Non-Aggressive	Non-Aggressive	Clay loam	9	492	4.4	Moderately Saline	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		
				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	4.1	Moderately 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		
				0.50	7.2	B	Non-Aggressive	Non-Aggressive	Clay loam	9	522	4.7	Moderately Saline	6.5	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		
				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	6.3	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		
				1.00	9.4	B	Non-Aggressive	Non-Aggressive	Loam	10	374	3.7	Slightly Saline	4.1	Moderately 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		
				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	4.1	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	East (m MGA94)	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 [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		
				1.00	8.4	A	Non-Aggressive	Non-Aggressive	Sandy loam	14	281	3.9	Slightly Saline	4.1	Moderately 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)	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]
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 Flood Modelling Details

D.1 Model Cross Sections

Cross sections used in the hydraulic model representing the existing South Creek and Ropes Creek under the existing conditions and with the proposed development involving the following earthworks:

- Filling in Central Precinct according to **Figure 7-2**
- Filling of Dunheved Precinct according to "Dunheved Precinct Development Application – Flood Impact Assessment"SKM, 30 March 2007
- Removal of part of the existing Old Munitions Road Embankment according to "Dunheved Precinct Development Application – Flood Impact Assessment" Report of 30 March 2007

Cross section plots are shown in the following pages.

D.2 Details on MIKE-11 Model Runs

Details of South Creek MIKE-11 (Version 2003 SP1) Model Runs

a) Proposed Development

.sim11	Updated_Fill_g2_no_MRB_SthCreekBR_1P5P	Updated_Fill_g2_no_MRB_SthCreekBR_S_PMF_1PH
.nwk11	Fill-G2-no-munition-br-SthCreekBr	Fill-G2-no-munition-br-SthCreekBr_PMF
.xns11	Updated_Fill-g2	Updated_Fill-g2-PMF
.bnd11	BASE1%5%	S_PMF1%H
.hd11	Base_r	Base_r
Timestep (Sec)	6	6
Start Time	1/01/2000 4:10	1/01/2000 4:10
End Time	2/01/2000 19:00	2/01/2000 19:00
Saving of Results (No. of Time Steps)	300	150
.res11	Updated_Fill_g2_no_MRB_SthCreekBR_1P5P	Updated_Fill_g2_no_MRB_SthCreekBR_S_PMF_1PH
Initial Conditions	Hotstart	Hotstart
Hotstart File	HOT_UPDATED_FILL_G2_NO_MBR.res11	HOT_UPDATED_FILL_G2_NO_MBR.res11
Hotstart Time	2/01/2000 18:10	2/01/2000 18:10
Design Flood Event:		
Catchment	1% AEP	PMF
Downstream	5% AEP	1% AEP

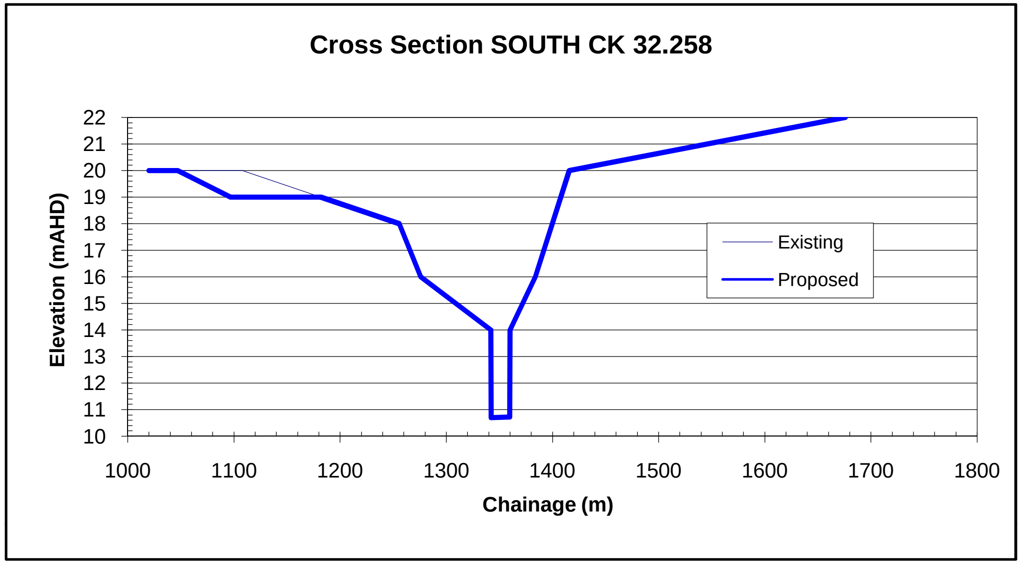
D.3 Peak Flood Levels

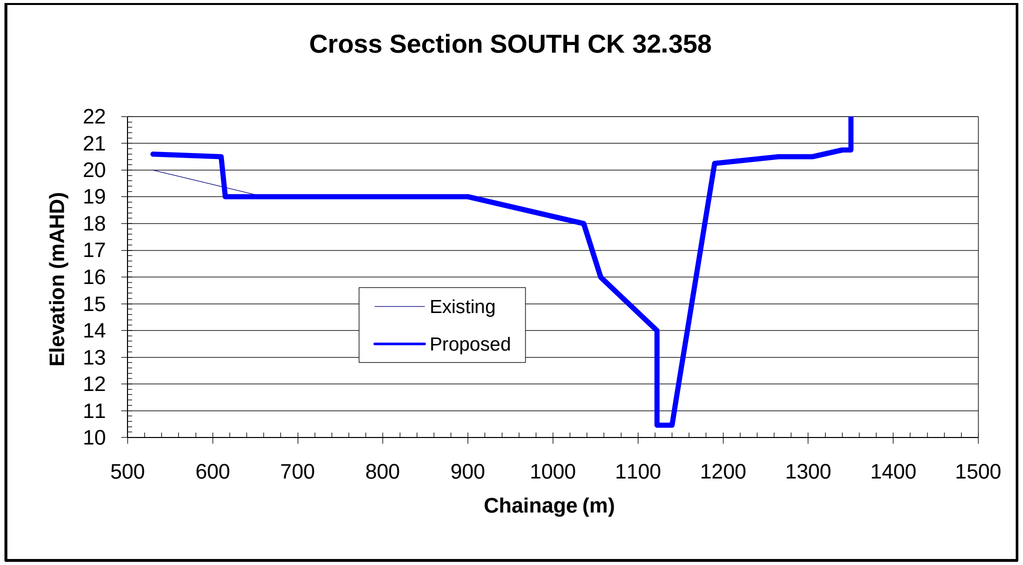
SKM

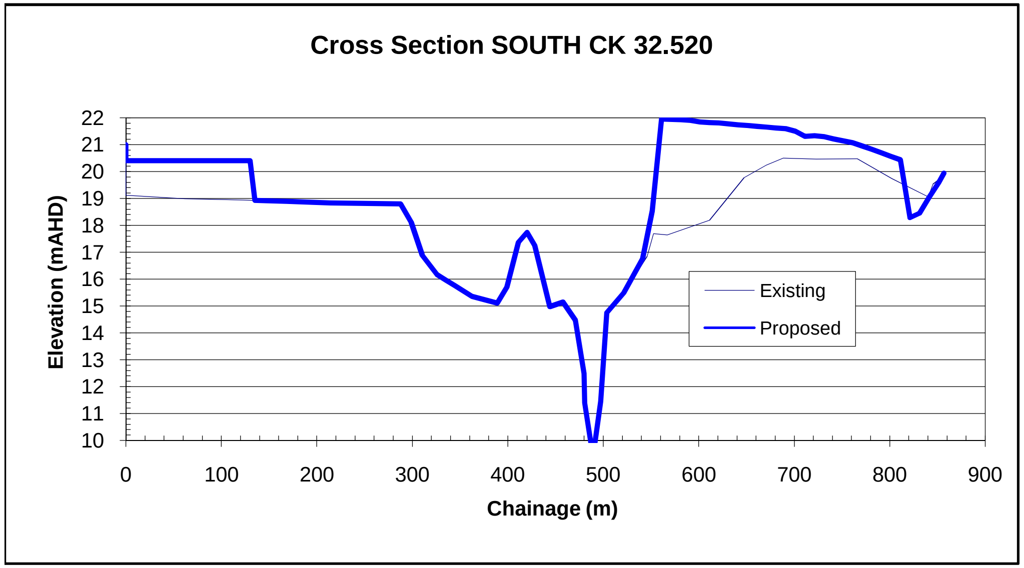
Table D3-1 : Modelled Peak Flood Levels - Adopted Dunheved DA and the Preferred Development Option : South Creek						
Cross-sections (Chinage in m)	Adopted Dunheved DA (peak water levels in m AHD)		Preferred Development Option Results (peak water levels in m AHD)		Impact of the Preferred Development Option on Peak Flood Level (mm)	
	100 year ARI	PMF in South Creek	100 year ARI	PMF in South Creek	100 year ARI	PMF in South Creek
SOUTH CK 30898.00	22.559	22.921	22.559	22.922	0	1
SOUTH CK 31338.00	21.55	21.853	21.552	21.857	2	4
SOUTH CK 31778.00	20.666	21.046	20.673	21.055	7	9
SOUTH CK 32258.00	20.075	20.568	20.09	20.586	15	18
SOUTH CK 32358.00	20.065	20.558	20.085	20.577	20	19
SOUTH CK 32520.00	20.005	20.489	20.024	20.507	19	18
SOUTH CK 32688.00	19.968	20.448	19.985	20.462	17	14
SOUTH CK 32818.00	19.913	20.389	19.93	20.403	17	14
SOUTH CK 32828.00	19.91	20.386	19.927	20.4	17	14
SOUTH CK 32918.00	19.87	20.347	19.888	20.363	18	16
SOUTH CK 33128.00	19.849	20.329	19.867	20.344	18	15
SOUTH CK 33188.00	19.78	20.23	19.804	20.252	24	22
SOUTH CK 33350.00	19.713	20.17	19.719	20.174	6	4
SOUTH CK 33410.00	19.621	20.078	19.573	20.075	-48	-3
SOUTH CK 33420.00	19.658	20.12	19.657	20.117	-1	-3
SOUTH CK 33470.00	19.637	20.1	19.637	20.097	0	-3
SOUTH CK 33480.00	19.592	20.05	19.592	20.047	0	-3
SOUTH CK 33838.00	19.342	19.817	19.342	19.814	0	-3
SOUTH CK 34020.00	19.254	19.731	19.253	19.728	-1	-3
SOUTH CK 34198.00	19.162	19.639	19.162	19.636	0	-3
SOUTH CK 34588.00	18.994	19.456	18.994	19.453	0	-3
SOUTH CK 34788.00	18.926	19.378	18.926	19.375	0	-3
SOUTH CK 35188.00	18.09	18.445	18.09	18.443	0	-2
SOUTH CK 35458.00	17.568	17.951	17.568	17.949	0	-2
SOUTH CK 35798.00	17.236	17.658	17.236	17.656	0	-2
SOUTH CK 36143.00	16.922	17.483	16.922	17.483	0	0
SOUTH CK 36488.00	16.764	17.483	16.764	17.483	0	0
SOUTH CK 36978.00	16.582	17.483	16.581	17.483	-1	0
SOUTH CK 37388.00	16.25	17.482	16.25	17.482	0	0
SOUTH CK 37598.00	15.997	17.481	15.997	17.481	0	0
SOUTH CK 38098.00	15.583	17.481	15.582	17.481	-1	0
SOUTH CK 38588.00	15.135	17.481	15.135	17.481	0	0
SOUTH CK 39078.00	14.464	17.48	14.464	17.48	0	0
SOUTH CK 39398.00	14.152	17.48	14.152	17.48	0	0
SOUTH CK 39753.00	14.007	17.48	14.007	17.48	0	0
SOUTH CK 40108.00	13.929	17.48	13.929	17.48	0	0
SOUTH CK 40443.00	13.885	17.48	13.885	17.48	0	0
SOUTH CK 40778.00	13.855	17.48	13.855	17.48	0	0
SOUTH CK 41143.00	13.826	17.479	13.826	17.479	0	0
SOUTH CK 41508.00	13.809	17.479	13.809	17.479	0	0
SOUTH CK 41878.00	13.802	17.479	13.802	17.479	0	0
SOUTH CK 42248.00	13.797	17.479	13.797	17.479	0	0

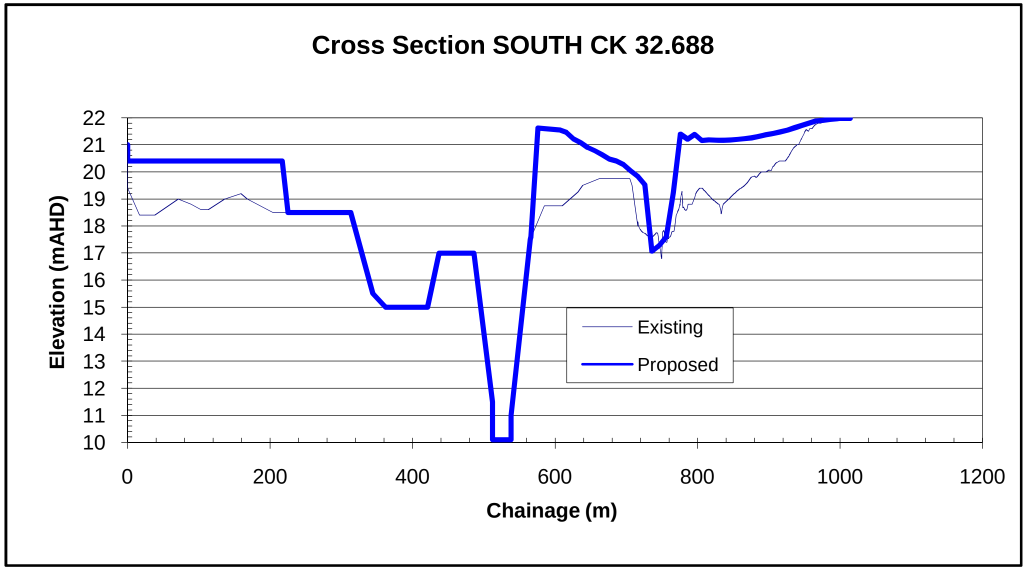
Table D3-2 : Modelled Peak Flood Levels - Adopted Dunheved DA and the Preferred Development Option : Ropes Creek						
Cross-sections (Chinage in m)	Adopted Dunheved DA (peak water levels in m AHD)		Preferred Development Option Results (peak water levels in m AHD)		Impact of the Preferred Development Option on Peak Flood Level (mm)	
	100 year ARI	PMF in South Creek	100 year ARI	PMF in South Creek	100 year ARI	PMF in South Creek
ROPES CK 0.00	28.601	29.352	28.601	29.352	0	0
ROPES CK 170.00	27.947	28.819	27.947	28.819	0	0
ROPES CK 390.00	27.317	28.451	27.317	28.451	0	0
ROPES CK 760.00	26.195	27.708	26.195	27.708	0	0
ROPES CK 900.00	25.779	27.07	25.779	27.07	0	0
ROPES CK 1250.00	25.251	26.314	25.251	26.314	0	0
ROPES CK 1520.00	24.674	25.77	24.674	25.77	0	0
ROPES CK 1560.00	24.574	25.714	24.574	25.714	0	0
ROPES CK 1840.00	23.959	25.304	23.959	25.304	0	0
ROPES CK 1950.00	22.689	23.994	22.689	23.994	0	0
ROPES CK 2010.00	22.578	23.819	22.578	23.819	0	0
ROPES CK 2230.00	22.146	23.377	22.146	23.377	0	0
ROPES CK 2540.00	21.194	22.534	21.194	22.534	0	0
ROPES CK 2890.00	20.087	21.788	20.087	21.788	0	0
ROPES CK 3146.00	19.786	21.53	19.786	21.53	0	0
ROPES CK 3156.00	19.763	21.475	19.763	21.475	0	0
ROPES CK 3340.00	19.583	21.344	19.583	21.344	0	0
ROPES CK 3590.00	19.479	21.219	19.479	21.219	0	0
ROPES CK 3660.00	19.315	19.863	19.315	19.861	0	-2
ROPES CK 3860.00	19.283	19.774	19.283	19.77	0	-4
ROPES CK 4140.00	19.15	19.63	19.15	19.626	0	-4
ROPES CK 4430.00	18.988	19.45	18.988	19.447	0	-3
ROPES CK 4760.00	18.926	19.378	18.926	19.375	0	-3

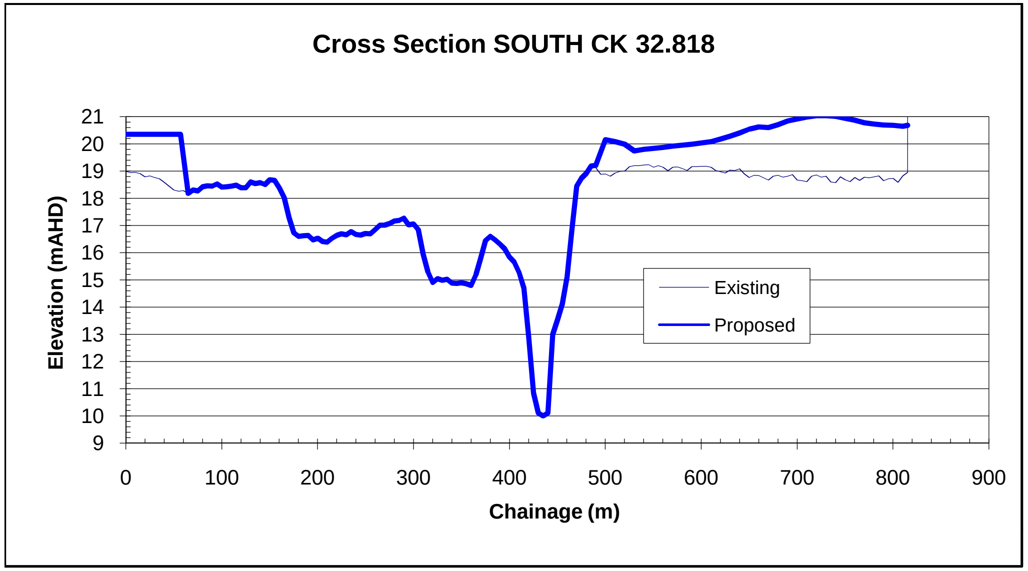
D.4 Cross Section Plots

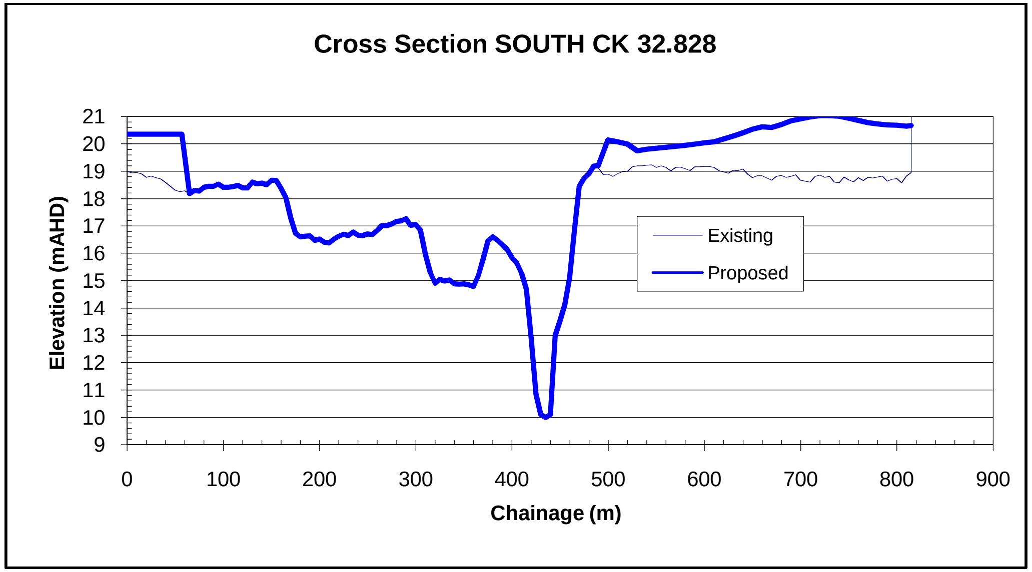

Cross Section SOUTH CK 32.258
Elevation (mAHD)
Chainage (m)
Existing
Proposed
22
21
20
19
18
17
16
15
14
13
12
11
10
1000
1100
1200
1300
1400
1500
1600
1700
1800

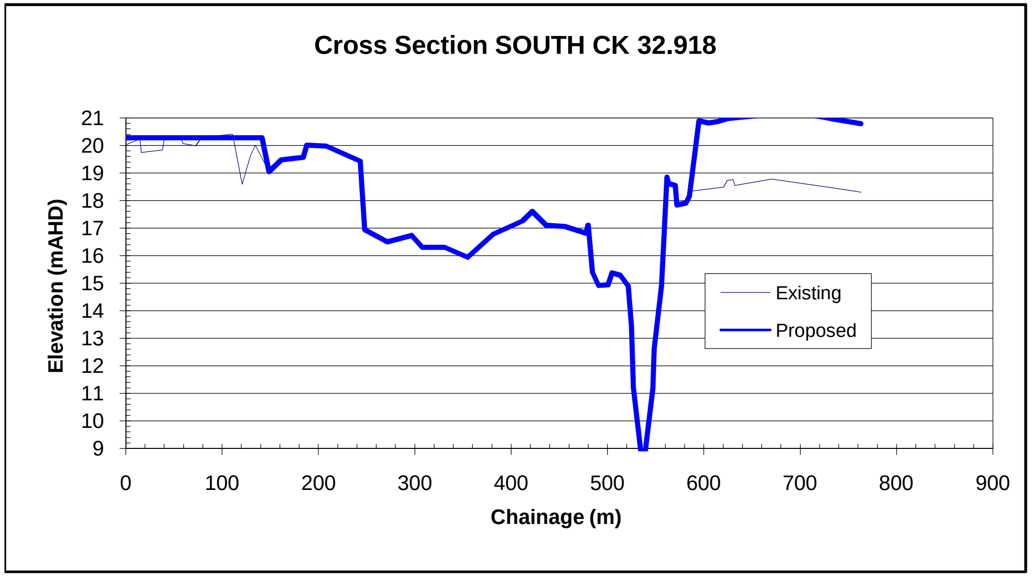

Cross Section SOUTH CK 32.358
Elevation (mAHD)
Chainage (m)
Existing
Proposed
22
21
20
19
18
17
16
15
14
13
12
11
10
500
600
700
800
900
1000
1100
1200
1300
1400
1500

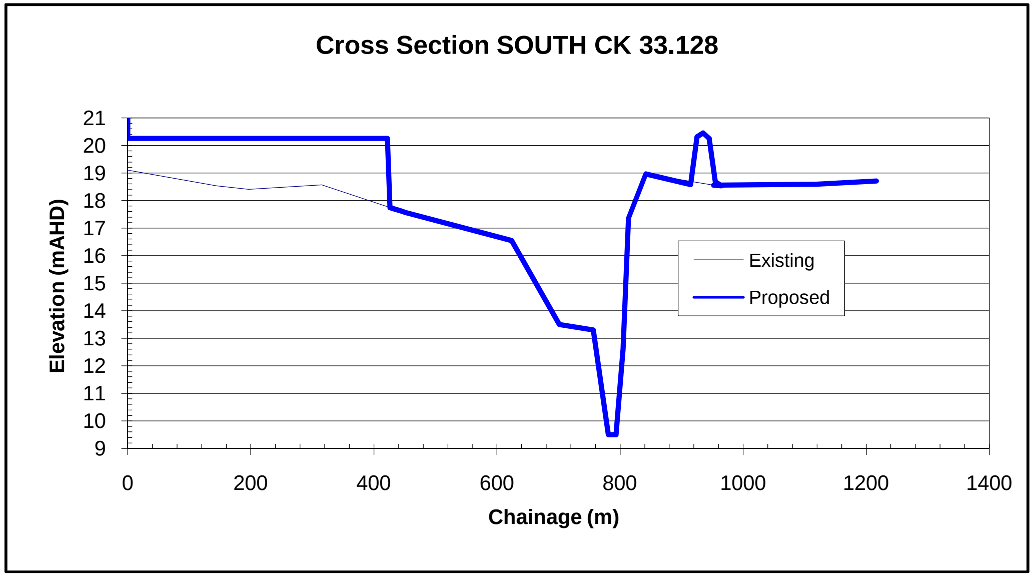

Cross Section SOUTH CK 32.520
Elevation (mAHD)
Chainage (m)
Existing
Proposed
22
21
20
19
18
17
16
15
14
13
12
11
10
0
100
200
300
400
500
600
700
800
900

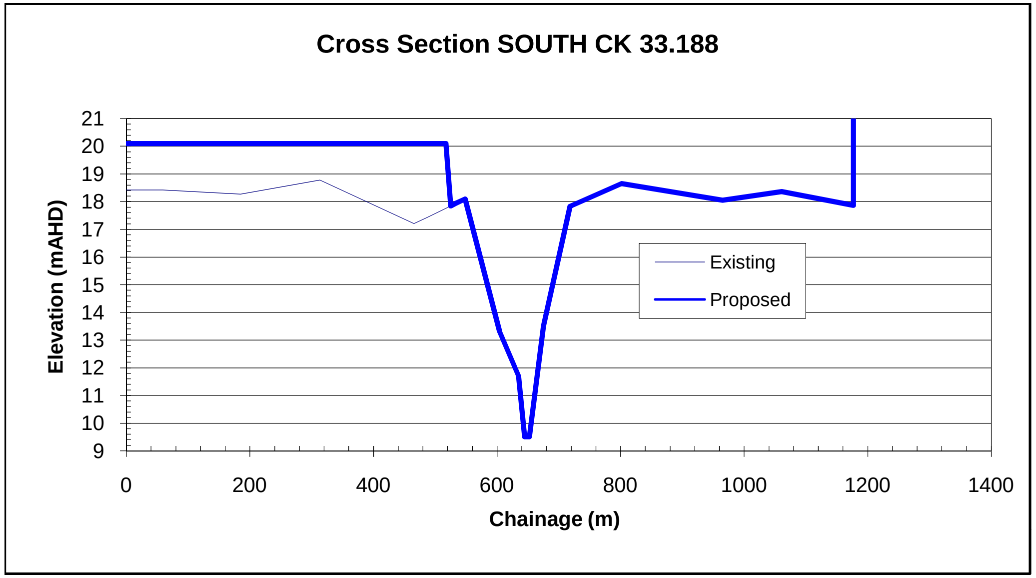

Cross Section SOUTH CK 32.688
Elevation (mAHD)
Chainage (m)
Existing
Proposed
22
21
20
19
18
17
16
15
14
13
12
11
10
0
200
400
600
800
1000
1200


Cross Section SOUTH CK 32.818
Elevation (mAHD)
21
20
19
18
17
16
15
14
13
12
11
10
9
0
100
200
300
400
500
600
700
800
900
Chainage (m)
Existing
Proposed

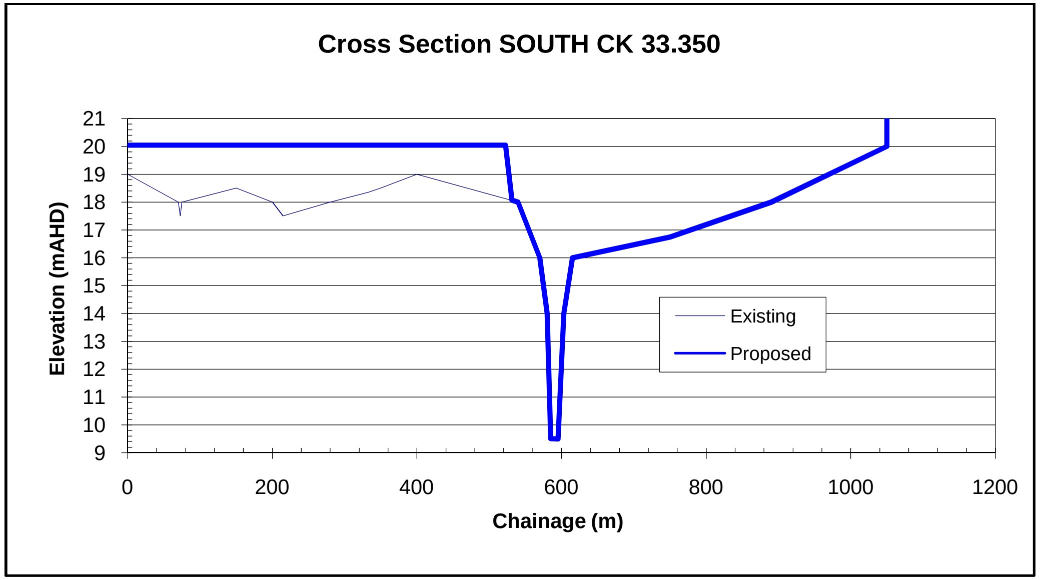

Cross Section SOUTH CK 32.828
Elevation (mAHD)
21
20
19
18
17
16
15
14
13
12
11
10
9
0
100
200
300
400
500
600
700
800
900
Chainage (m)
Existing
Proposed


Cross Section SOUTH CK 32.918
Elevation (mAHD)
21
20
19
18
17
16
15
14
13
12
11
10
9
0
100
200
300
400
500
600
700
800
900
Chainage (m)
Existing
Proposed

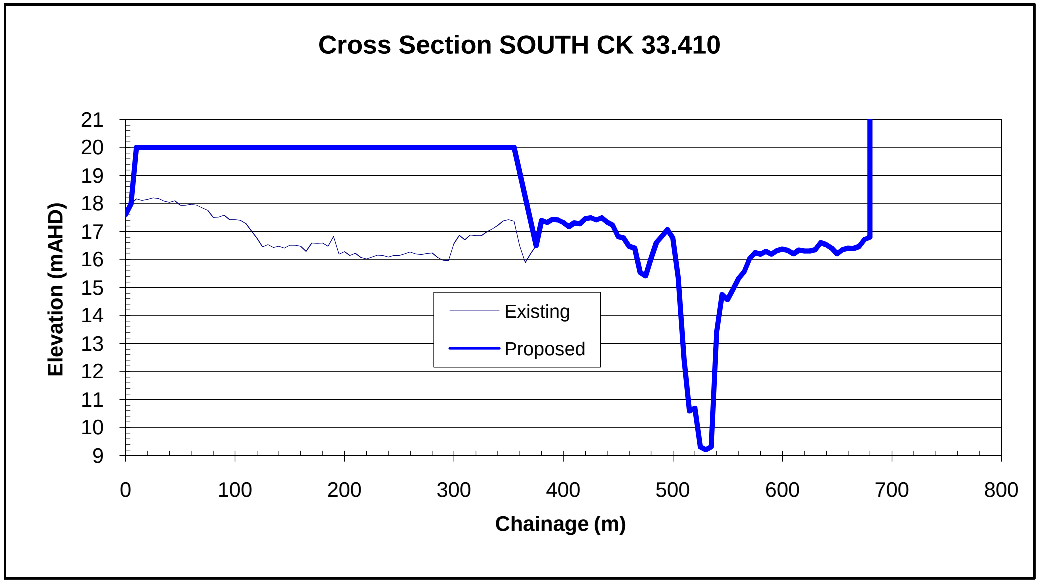

Cross Section SOUTH CK 33.128
Elevation (mAHD)
21
20
19
18
17
16
15
14
13
12
11
10
9
0
200
400
600
800
1000
1200
1400
Chainage (m)
Existing
Proposed



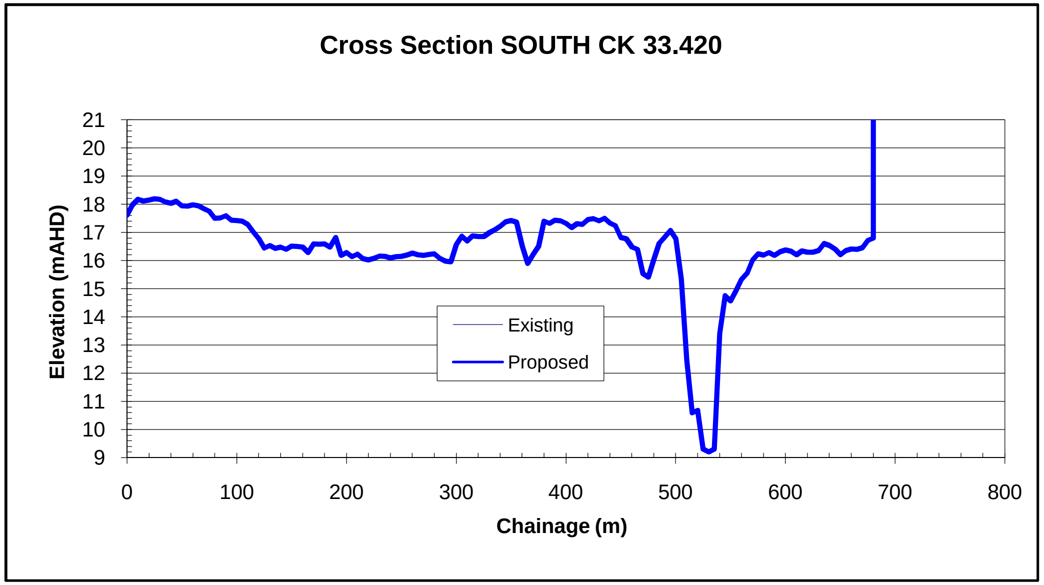
Cross Section SOUTH CK 33.188
Elevation (mAHD)
Chainage (m)
Existing
Proposed
21
20
19
18
17
16
15
14
13
12
11
10
9
0
200
400
600
800
1000
1200
1400



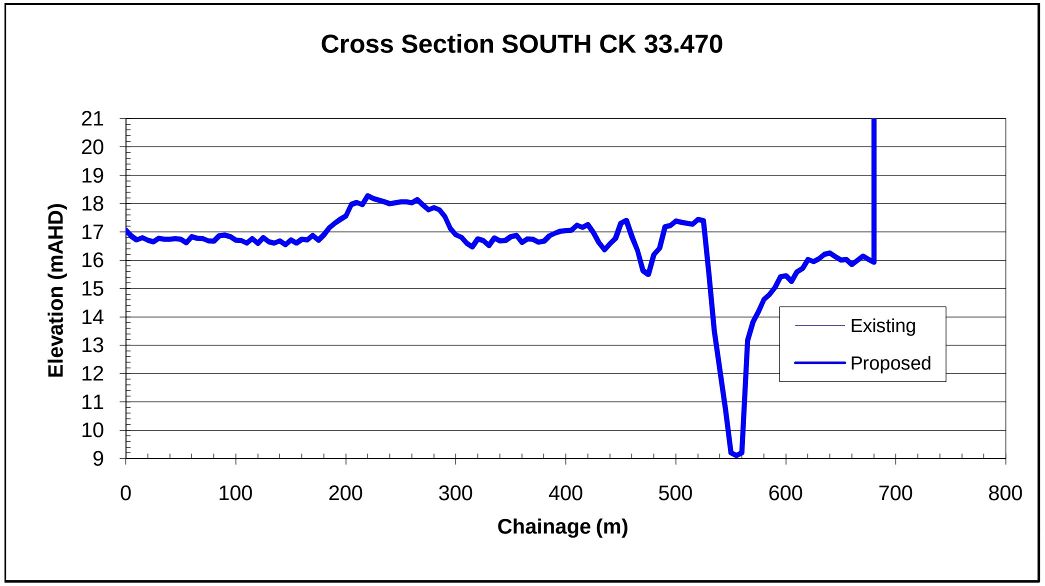
Cross Section SOUTH CK 33.350
Elevation (mAHD)
Chainage (m)
Existing
Proposed
21
20
19
18
17
16
15
14
13
12
11
10
9
0
200
400
600
800
1000
1200

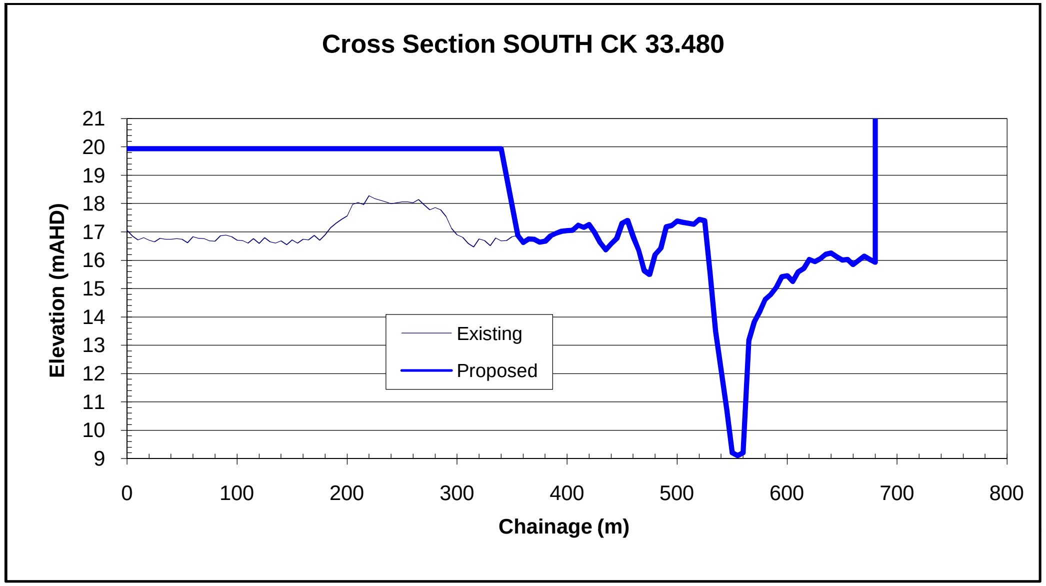


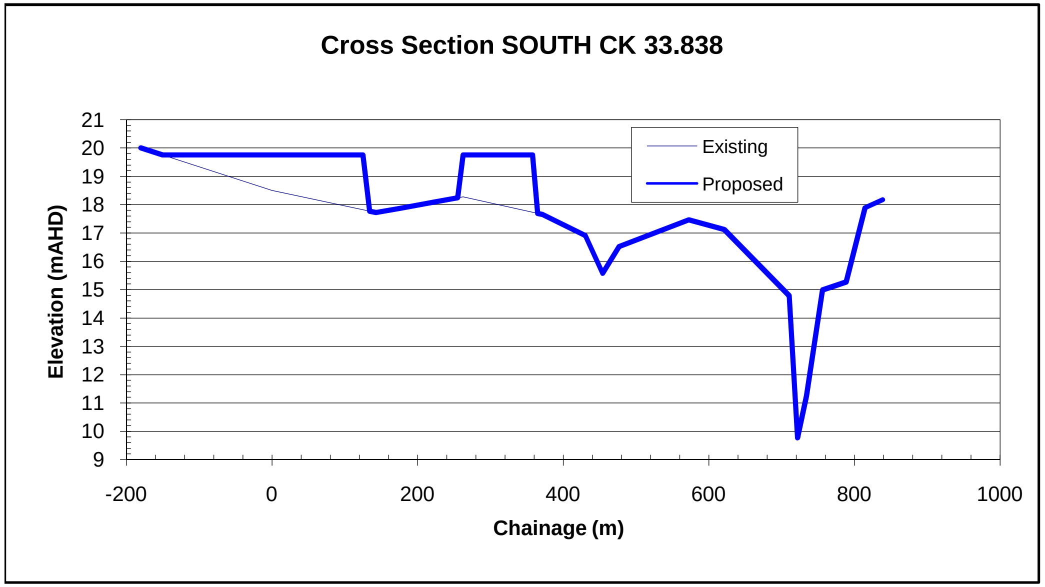
Cross Section SOUTH CK 33.410
Elevation (mAHD)
Chainage (m)
Existing
Proposed
21
20
19
18
17
16
15
14
13
12
11
10
9
0
100
200
300
400
500
600
700
800

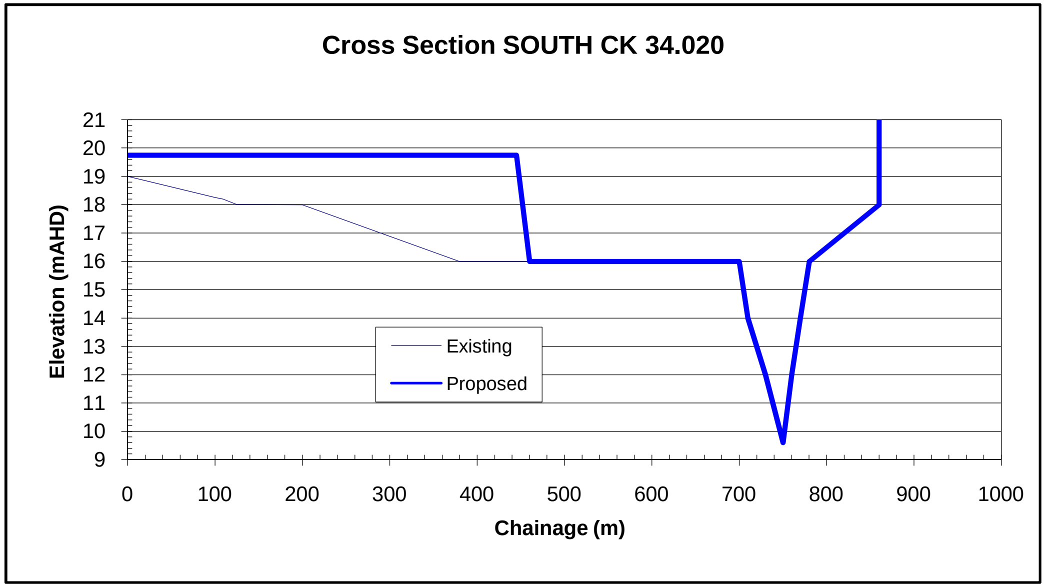


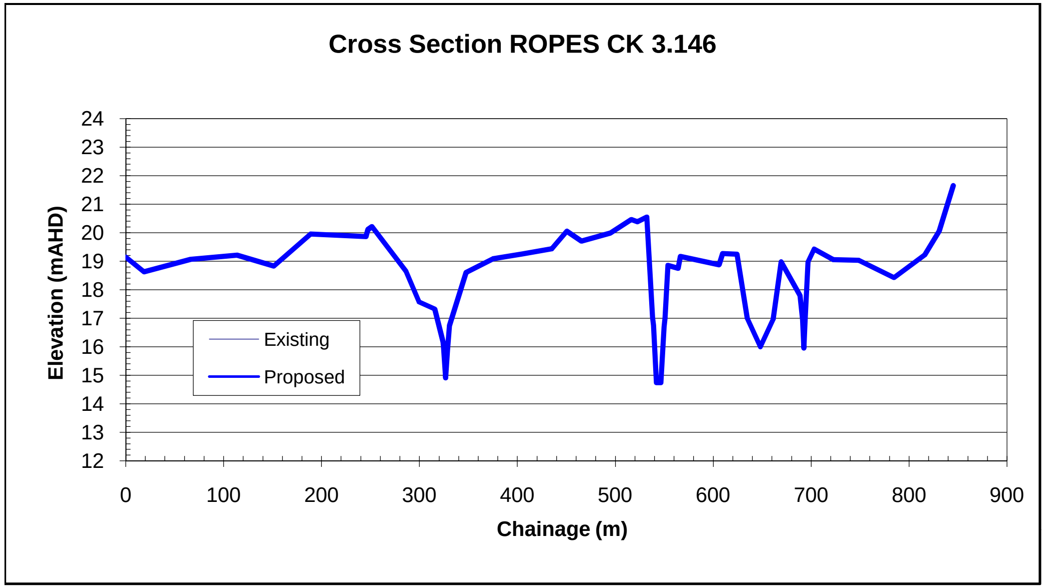
Cross Section SOUTH CK 33.420
Elevation (mAHD)
Chainage (m)
Existing
Proposed
21
20
19
18
17
16
15
14
13
12
11
10
9
0
100
200
300
400
500
600
700
800


Cross Section SOUTH CK 33.470
Elevation (mAHD)
Chainage (m)
Existing
Proposed

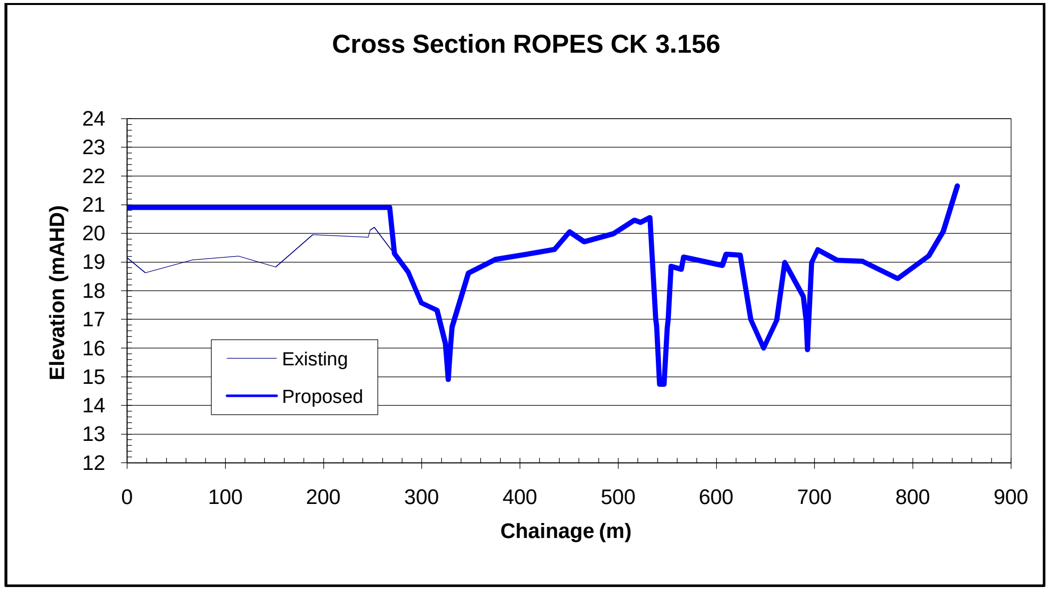

Cross Section SOUTH CK 33.480
Elevation (mAHD)
Chainage (m)
Existing
Proposed


Cross Section SOUTH CK 33.838
Elevation (mAHD)
Chainage (m)
Existing
Proposed

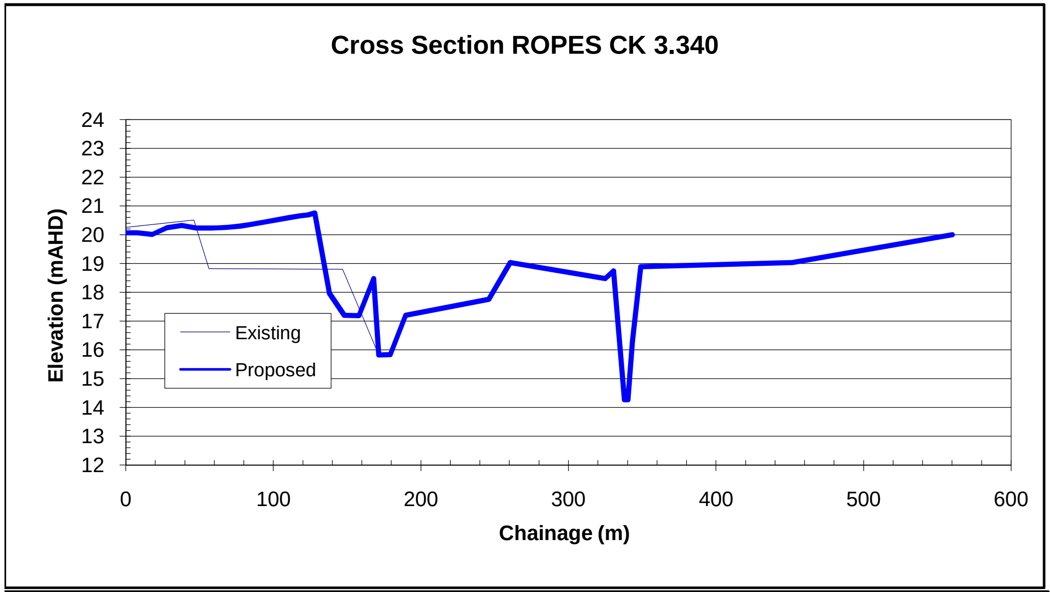

Cross Section SOUTH CK 34.020
Elevation (mAHD)
Chainage (m)
Existing
Proposed



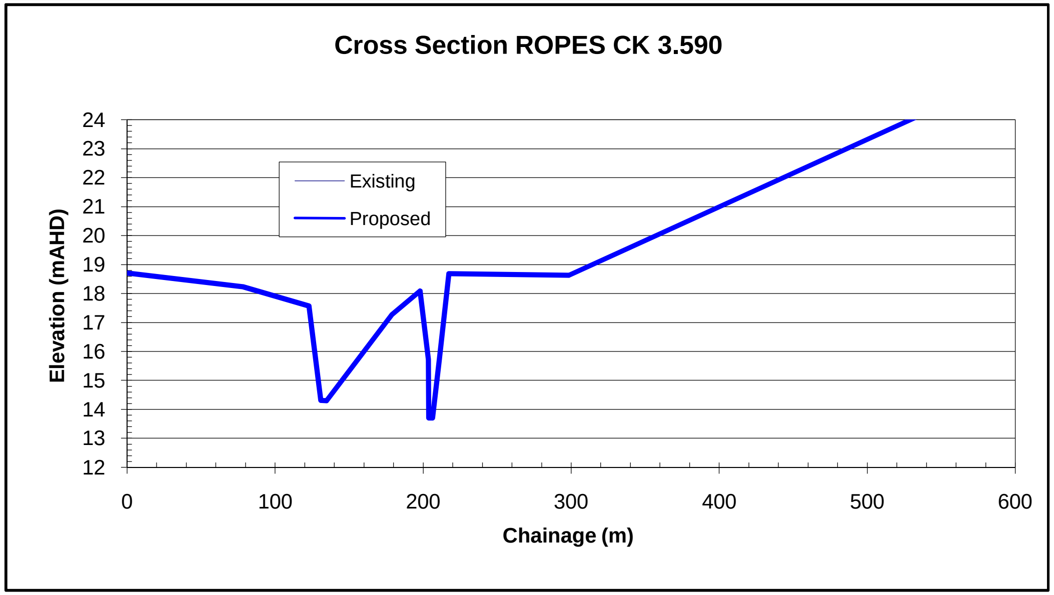
Cross Section ROPES CK 3.146
Elevation (mAHD)
Chainage (m)
Existing
Proposed



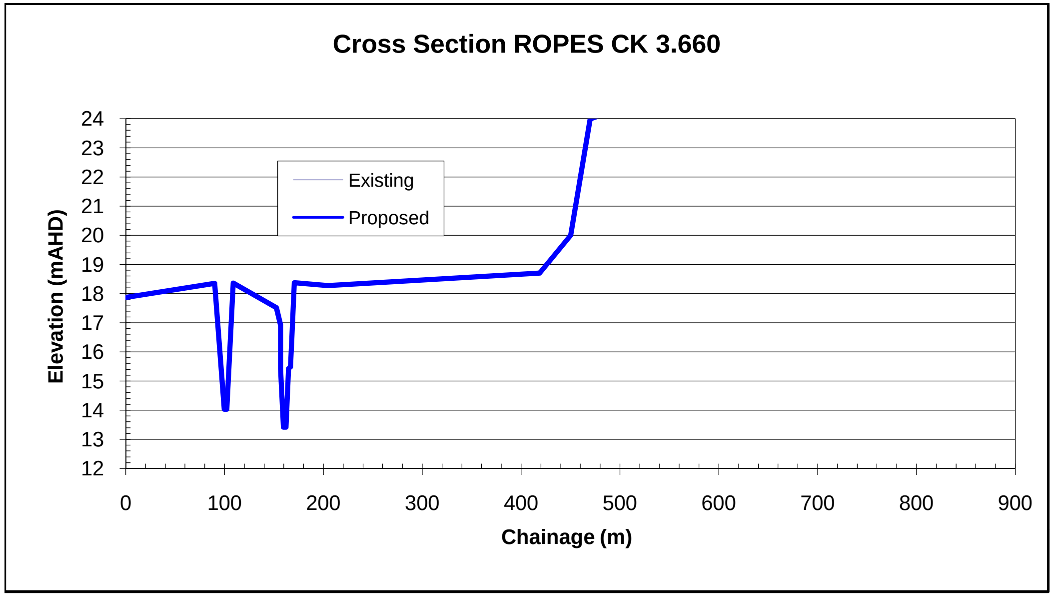
Cross Section ROPES CK 3.156
Elevation (mAHD)
Chainage (m)
Existing
Proposed



Cross Section ROPES CK 3.340
Elevation (mAHD)
Chainage (m)
Existing
Proposed



Cross Section ROPES CK 3.590
Elevation (mAHD)
Chainage (m)
Existing
Proposed


Cross Section ROPES CK 3.660
Existing
Proposed
Elevation (mAHD)
24
23
22
21
20
19
18
17
16
15
14
13
12
0
100
200
300
400
500
600
700
800
900
Chainage (m)