Stormy Water Solutions

Hydrological and Environmental Engineering

Macarthur Memorial Park

Water Sensitive Urban Design Strategy

and

Storm Water Management Plan

7 November 2018

Report by: **Stormy Water Solutions**

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Contents

1 Introduction

This Macarthur Memorial Park Storm Water Management Plan (SWMP) has been prepared to complement the drainage design prepared by Warren Smith and Partners (WS&P). Stormy Water Solutions (SWS) have worked closely with WS&P, Florence Jaquet Landscape Architect (FJLA), GRC Hydro (GRC) and Travers Bushfire & Ecology (TB&E) to ensure an integrated approach to this unique site.

This SWMP report and associated plans specifically applies to the flood storage and Water Sensitive Urban Design (WSUD) requirements of the site once it is developed as a cemetery. The specific piped drainage network pertaining to individual catchments and roads are as developed by WS&P. Crucial to the development of the SWMP is integration with the site drainage, catchment, landscape, ecological and riparian zone considerations. This has been achieved through an iterative process to ensure all objectives and constraints have been captured.

This report considers the major WSUD elements, retarding basin flood storage and water quality management issues within the subject site. The aim of the SWMP is to clearly define the potential land footprint requirements of major drainage assets so that the site can be developed as proposed without adverse downstream or upstream drainage impacts.

WS&P will be preparing documentation and plans relating the road and development piped network. Similarly, it is understood that GRC will be conducting a flood analysis of the waterways and dams affecting the site to confirm the flood levels and spillway requirements detailed in this SWMP.

All assets detailed in this report are at the strategy functional design stage. As such, all proposals are subject to change as the planning and design process continues. Notwithstanding this, the following items have been considered used to ensure all SWMP assets are realistic in regard to sizing and placement within the site:

- Site survey information,
- Ecological constraints,
- Flooding constraints,
- The current details of the building architectural designs (specifically the floor levels detailed in the DA),
- Civil drainage and road designs, and
- Current Landscape proposals.

1.1 Stormy Water Solutions Description

The primary author of this report is Valerie Mag, principal of Stormy Water Solutions. Valerie is a hydrologist with the following educational qualifications:

- Bachelor of Civil Engineering, Monash University (1989)
- Master of Water Resources and Environmental Engineering, Monash University (1993)

Valerie has twenty-nine years' experience and expertise in hydrologic and hydraulic engineering, particularly in the areas of:

- Preparing complex urban and rural flood plain strategies,
- Preparing Water Sensitive Urban Design Strategies,
- Major catchment analysis, including flood flow and flood level estimation,
- Planning and assessment of development within flood plain and overland flow path systems,
- Reviewing drainage strategies prepared by other consultants for Melbourne Water and various councils, and
- Regularly preparing and conducting training in drainage and WSUD for the Municipal Association of Victoria, Vic Roads, Melbourne Water, the Department of Tourism Arts and the Environment (Tasmania), ARRB Group (run twice in Sydney), Austroads and others.

Projects the Stormy Water Solutions team have completed include, but are not limited to, those listed below.

- Audits of drainage and WSUD elements, with a particular emphasis on clearly identifying ongoing maintenance issues and recommending cost effective remedial works,
- Development of WSUD maintenance schedules for bioretention systems, wetlands, sediment ponds and swales,
- Safety Audits of pond and wetland systems,
- Hydraulic assessment and/or functional design of rock chutes, weirs, culverts, bridges, spillways and other hydraulic structures,
- Specialist advice on all aspects of Water Sensitive Urban Design,
- Pollutant modelling using the MUSIC model,
- Functional and functional design of best practice stormwater system elements such as retarding basins, wetlands, bioretention systems, swales, gross pollutant traps and rainwater storage tanks.

The Stormy Water Solutions team have used the above experience, together with the extensive knowledge within the consultant team for this project (WS&P, TB&E and FJLA and in particular), to ensure drainage functional designs are to best practice and to Council and state requirements.

2 Background

Figure 1 below details the subject site and the main drainage and waterway features located in and around the area of interest. This report specifically relates to Stage 1 of the cemetery development proposed for the site.

Previously Alluvium had prepared a report entitled “Macarthur Memorial Park – Catholic Cemeteries and Crematoria, Water Cycle Management Plan for D.A., October 2017”. This is referred to as the 2017 Alluvium DA Report in this document. The current SWMP incorporates all the original objectives of the 2017 Alluvium DA report.

However, the SWMP as was proposed in the **2017 Alluvium DA Report** has been modified to:

- Ensure all contributing catchments are modelled,
- Reduce the complexity in design, construction and maintenance of 26 bioretention systems, three swales, and two wetlands (Dam 5 and Dam 3) as proposed by Alluvium,
- Include the provision of flood retardation on site (the 2017 Alluvium SWMP proposed no retarding basins due to a stated “no change in site fraction imperviousness”, however this is not the case once the site is developed),
- To include the “real” consideration of the fact that the dams are, and will always be part of the drainage system, and therefore need to be suitably considered in the SWMP, and
- Specifically incorporate the water requirements and duel uses which can be attributed to the many dams on site.

The subject land is located within 166-176 St. Andrews Road, Varroville. The land is undulating, with some steep areas. The site has a total area of 113 ha. Stage 1 has a total area of approximately 85 ha.

Primarily the site defines the headwaters of many small drainage lines which outfall Stage 1 at St Andrews Road (the south western corner of the site, Point X in Figure 1). A small external catchment also enters the site at St Andrews Road in the north western portion of the site. Bunbury Curran Creek, the receiving water for most flows from the proposed development, runs on the southern side of the Hume Highway. Bunbury Curran Creek continues north-east (concrete-lined in places), to the suburb of Glenfield where it meets the Georges River.

There are currently 10 dams on site. These dams have historical and heritage significance, possibly going back to what remains of the handmade dams first established by Dr Robert Townson (1812).

As such, a key constraint in the design of the SWMP is to retain (as far as possible) and restore the dams and surrounding landscape in what is an important part of NSW’s pastoral history. The aim is to minimise impact on these important heritage works, while ensuring they are retrofitted to current safety standard regarding flood protection and structural integrity. In addition, minimising ongoing risks in regard to the water quality in the dam systems has been examined in detail. SWS have worked closely with FJLA, the ecology team and the heritage team to ensure the land and dams will be remediated to show what 19th century pastoral lands looked like in their prime.

The site is currently used as a working cattle farm. There are significant stands of vegetation in the site, especially in the many drainage lines and associated riparian zones. However, largely the site consists of cleared paddocks. This offers the opportunity to increase the environmental and ecological diversity of the site going forward.

Key to the design is to use the existing drainage infrastructure as an opportunity to assist to the landscape and ecological diversity of the site, while meeting drainage and WSUD objectives.

In producing this WSUD strategy and SWMP SWS has used:

- Australian Rainfall and Runoff: A Guide to Flood Estimation, Commonwealth of Australia Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors), 2016,
- The “The Campbelltown (Sustainable City) Development Control Plan 2015” document, referred to as the CDCP 2015 in this SWMP report,
- Campbelltown (Sustainable City) Development Control Plan Volume 2 Engineering Design for Development 2009 (contained as part of the CDCP 2015),
- Macarthur Memorial Park – Catholic Cemeteries and Crematoria, Water Cycle Management Plan for D.A., Alluvium October 2017 (2017 Alluvium DA Report),
- Draft NSW MUSIC Modelling Guidelines, Sydney Metropolitan Catchment Management authority, August 2010,
- Austroads Publication “Guide to Road Design Part 5A: Drainage – Road Surface, Networks, Basins and Subsurface” 2013,
- NSW Office of Water’s “Guidelines for riparian corridors on waterfront land (July 2012)”,
- Site survey information and various working plans developed by WS&P, TB&E and FJLA in 2017 and 2018,
- The documented photos and notes from a site inspection conducted by SWS in January 2018,
- A RORB model (an industry-standard Runoff Routing Model originally developed by Monash University (Laurenson EM and Mein RG)) developed for this study by SWS to estimate flood flows and provide flood storage capacity requirements,
- Various hydraulic formula (including Manning’s formula) to estimate required swale dimensions, and
- A MUSIC Model V6 (Model for Urban Stormwater Improvement Conceptualisation) software developed for this study by SWS to simulate runoff and pollutant load regimes and to design the Water Sensitive Urban Design (WSUD) elements on site.

All elements proposed as part of this drainage strategy have been fully considered in regard to their applicability. As much as possible final design invert levels, normal water levels, batter requirements etc. have been set at this stage to ensure all elements can be constructed and will not be constrained by outfall invert levels, reserve widths, buffers, ecological constraints etc. Notwithstanding the above, all designs are at the functional design stage only and are subject to change during the design process.

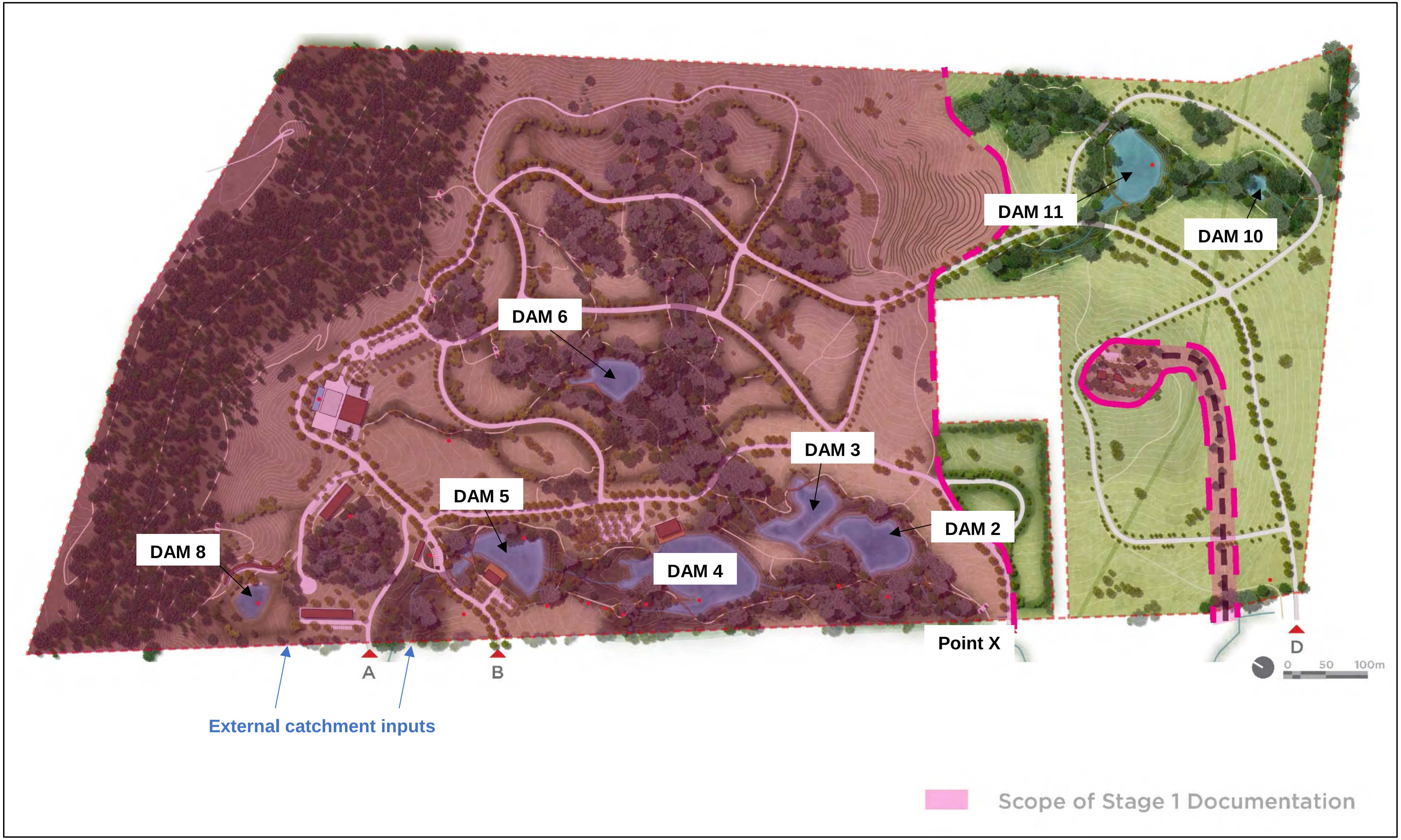


Figure 1 **Subject Site – Staging and Dam Definitions**
Refer to Figure B.4 for catchment Delineation

3 WSUD Strategy SWMP Objectives

All WSUD and SWMP elements referred to in this section are described in detailed Section 4 below.

The drainage requirements applicable to this SWMP are as defined in the “Campbelltown (Sustainable City) Development Control Plan 2015”. This document is referred to at CDCP 2015 in this report.

This is generally a summary of the requirements as detailed by 2017 Alluvium in the DA WSUD report. However, the summary has been modified to concentrate of the major and relevant requirements and objectives.

3.1 Stormwater Harvesting and Site Irrigation

Where possible rainwater tanks will supply water for toilet flushing in all proposed new buildings. All rainwater tanks will be designed in accordance with Section 2.4 (Sustainable Building Design) of the CDCP 2015.

As detailed in Appendix D, the catchment contributing to the major dams on site is extremely small (compared to the size of the dams). As such, in this SWMP the dam system is considered the first user of the stormwater on site. This is required to minimise dam water quality issues in the long term.

Given this constraint, it is understood that the consultant team is investigating alternative sources of water for site irrigation. At this stage treated bore water is probably to most feasible source of water to allow:

- Dam top up (if required), and
- Site irrigation.

3.2 Erosion and sediment control

In line with CDCP 2015 Section 2.7, a sediment and erosion control plan will be developed prior to construction to prevent loss of soil from site during construction. Proposed stormwater treatment systems (e.g. swales and wetlands) will capture sediment from site runoff after construction.

3.3 Water Cycle Management

In line with CDCP 2015 Section 2.10, the proposed revised drainage strategy ensures that water cycle management incorporates enhanced existing site conditions (water courses as swales, dams as wetlands etc.) to naturally drain and treat runoff. Impervious surfaces will almost all not be directly connected to the onsite outfalls (i.e. they will be separated by swales, wetland or buffers in line with current best practice).

The revised drainage strategy also ensures that Water Sensitive Urban Design (WSUD) principles are incorporated into development.

In addition, the strategy ensures that the development is protected from mainstream, local catchment and overland flow aspects of flooding by requiring all designs will be modelled with TUFLOW to set final floor levels for all buildings and to ensure all roads are flood free in the 1% AEP event. This SWMP has defined 1% AEP flood levels in all dam systems using the RORB model and associated design outlet configurations in each dam. It is understood that GRC Hydro will be confirming these flood levels (and the extreme spillway requirements) with a separate TUFLOW Model.

3.4 Watercourse and Riparian Corridor Protection

Council require natural creeks to be retained and comply with requirements of the Office of Water.

The watercourse assessment undertaken by TB&E (September 2013) identified drainage lines, first order streams, second order streams and third order streams within the site. Riparian protection zones were identified, and acceptable works noted in accordance with the NSW Office Of Water Controlled activities on waterfront land – Guidelines for Riparian Corridors on Waterfront Land (July 2012). Figure 2 shows the outcome of the assessment, with validated watercourses highlighted and associated riparian buffers.

Design Guide Section 4.15 requires retention of vegetation on site. Vegetation and significant trees will be retained wherever possible. The riparian corridors of the existing water courses include zones of protection. Plants within the vegetated stormwater treatment systems will be selected to include Use of indigenous (local) vegetation.

In relation to activities within the vegetated riparian zone, such as cycle ways and paths, detention basins, stormwater management devices and essential services, compliance is required with the 'riparian corridor matrix' in the NSW Office of Water's "Guidelines for riparian corridors on waterfront land (July 2012)".

As defined in these 2012 guidelines, the riparian corridor matrix enables applicants to identify certain works and activities that can occur on waterfront land and in riparian corridors.

The Vegetated riparian zone (VRZ) is the required width of the VRZ measured from the top of the high bank on each side of the watercourse.

The riparian corridor matrix states:

1. Stormwater outlet structures and essential services can be proposed within the riparian zone waterways of stream orders 1 – 4,
2. Detention basins can be proposed only within 50% outer Vegetated Riparian Zone for waterways of stream orders 1 – 4,
3. Detention basins can be online to 1st and 2nd order streams,
4. Online basins must:
    - Be dry and vegetated
    - Be for temporary flood detention only with no permanent water holding
    - Have an equivalent VRZ for the corresponding watercourse order and
    - Not be used for water quality treatment purposes.

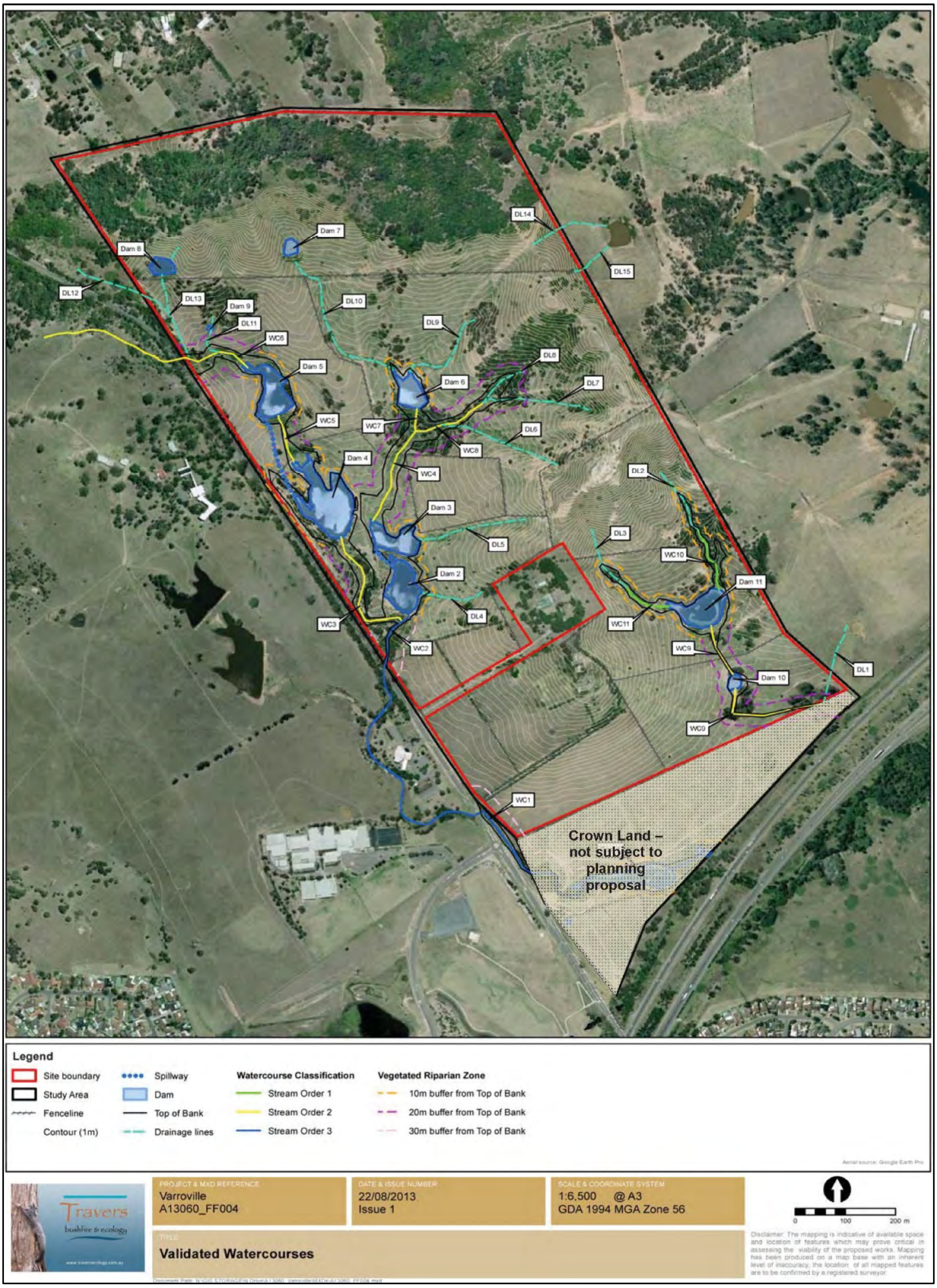


Figure 2 **Site Watercourse Definitions and Riparian Buffers (TB&E 2013)**

There are three WSUD elements (wetlands) and two detention (retarding) basins proposed within the riparian zone of the second order streams located in the site. These are:

- Dam 3 – reconfigured to ensure structural integrity of embankment, minimise dam water quality issues going forward and to provide wetland treatment for stormwater,
- Dam 5 – reconfigured to ensure structural integrity of embankment, minimise dam water quality issues going forward and to provide wetland treatment for stormwater,
- Dam 6 – reconfigured to ensure structural integrity of embankment, minimise dam water quality issues going forward and to provide wetland treatment for stormwater,
- Dam 2 – reconfigured to ensure structural integrity of embankment, minimise dam water quality issues going forward and to provide a flood storage role for site runoff, and
- Dam 4 – reconfigured to ensure structural integrity of embankment, minimise dam water quality issues going forward and to provide a flood storage role for site runoff.

These WSUD/Drainage elements will be utilising existing water features to perform the functions of the WSUD strategy and SWMP. However, works will be limited to “stormwater outlet structures and essential services” as defined in the first dot point above. The only works proposed on all systems are:

- Reconstruct the downstream embankments to current structural requirements to ensure the safety of downstream landowners,
- Ensure the outlet from both systems are designed and constructed to meet the WSUD and flood storage requirements as detailed in Appendices B and C, and
- Incorporate remodelled dam edges and bases to:
    - Minimise dam volume and thus maximise water turnover to avoid ongoing stagnation issues in the dams,
    - Maximise vegetation within the dam systems (i.e. wetland planting to avoid ongoing water quality issues in the dams),
    - revegetate the pond edges to ensure they meet current edge safety requirements in relation to “not inviting people to danger”, while
    - ensuring changes to the existing shapes and water levels of all dams are minimised to retain the heritage attributes of all systems.

It should be noted that, if the dams are not remodelled and vegetated, it is expected that current safety requirements will not be met and ongoing water quality issues in all dams will occur (due to the small contributing catchment limiting turnover of stormwater within the systems).

TB&E has advised that the above modifications to the existing water features on the second order streams will ensure they retain their function as a second order streams. TB&E has advised that the WSUD strategy and SWMP can retain all elements in the “intent” of their current form while constructing outlet and revegetation works to ensure the functions required under the SWMP, without compromising the riparian corridor.

Given that there are existing water features in the base of each system it is considered, in this case, that a "wet" retarding basin bases are reasonable. It should be noted that similar designs are advocated in "Australian Rainfall and Runoff: A Guide to Flood Estimation 2016, Book 9, Chapter 4, Table 9.4.4" and in the Austroads Publication "Guide to Road Design Part 5A: Drainage – Road Surface, Networks, Basins and Subsurface (2013)". Basins of this type are seen as best practice examples of incorporating flood storage and WSUD objectives in one site, and should be supported as such. This is an opportunity for council to support such a design in line with current best practice.

All other WSUD elements (vegetated swales) are proposed to be located away from, or within the 50% outer Vegetated Riparian Zone for affected waterways.

3.5 Water Quality Requirements

Council require inclusion of stormwater quality measures for commercial developments greater than 2500 m^2. General water quality objectives from the Design Guide Section 4.15 have been applied as per the points below.

- A treatment train approach has been taken, with treatment systems distributed throughout the site,
- Systems have been designed to take into consideration local and site conditions via ensuring the design responds to the existing site upstream sub catchment types and sizes and the existing drainage system topography and form,
- The drainage and WSUD elements have been designed to be functional and aesthetically pleasing,
- Maintenance requirements have been considered in terms of both plant equipment required and occupational health and safety issues for staff (see section 4.3.6),
- The drainage and WSUD elements have been designed to protect and enhance natural water systems via retention of existing streams within the site, enhanced riparian corridors through weed removal, revegetation and stabilisation works where required,
- The stream environment will be improved as the quality of water discharging to the downstream environment will be improved,
- The drainage and WSUD elements integrate stormwater treatment into the landscape by incorporating multiple-use corridors, that maximise the visual and recreational amenity of the development, and
- The design specifically aims to ensure systems are aesthetically pleasing.

Specifically Design Guide Section 4.15 requires protection of water quality draining from development areas. The quality of water discharging to the downstream environment will be improved by the proposed treatment systems, in accordance with the stormwater quality objectives described below.

Council has previously indicated that suitable pollutant removal targets for the site would be the targets adopted by Landcom. These targets are:

- 85% TSS reduction

- 65% TP reduction
- 45% TN reduction
- 95% Gross pollutant reduction

Modelling for the determination of the mean annual loads of land uses has been undertaken in MUSIC and in accordance with the associated WSUD Technical Guidelines.

In addition, the CDCP 2015 requires impervious areas directly connected to the stormwater system to be minimised. This has been achieved via FJLA specifying (as far as possible) burial areas which direct runoff from headstones etc. onto grass and other landscaped areas designed to accept such flows. Further it is expected almost all of the SW&P piped drainage layout will be connected into a swale before discharge into either, the riparian corridor or a dam system.

CDCP 2015 Section 2.10.2 requires a treatment train approach to water quality should be incorporated into the design and construction of major systems. A treatment train approach has been adopted with decentralised WSUD elements. WSUD have been arranged to first provide primary treatment (buffers), then secondary treatment (vegetated swales), and finally tertiary treatment (wetlands).

The CDCP 2015 Section 2.10.2 requires that water quality control structures shall be located generally off line to creek paths or other watercourses. Major detention storages shall not be located on areas of native vegetation or within riparian areas. The revised drainage strategy does allow for treatment and flood storage in online systems. This is discussed in Section 3.4 above.

3.6 Flood Storage Requirements

Design Guide Section 4.15 requires a reduction in runoff and peak flows from developments by employing local detention measures, minimising impervious areas and maximising re-use (for example through rain water tanks).

At this stage it is assumed that On-Site Stormwater Detention (OSD) or retarding basins (as OSD's are referred to in this report) must be designed and constructed to ensure that

- For all rainwater events up to and including the 1 in 100 Annual Exceedance Probability (AEP) event, do not increase stormwater peak flows from the site, and
- Must be designed using a catchment wide approach (that is, consideration of the total catchment, and external site catchments must be undertaken)

Appendix B details the RORB modelling completed for the SWMP detailing that the above conditions have been met.

The CDCP 2015 Section 2.10.2 requires that major detention storages shall not be located on areas of native vegetation or within riparian areas. The revised drainage strategy does allow for treatment and flood storage in online systems. This is discussed in Section 3.4 above.

Campbelltown (Sustainable City) Development Control Plan Volume 2 Engineering Design for Development 2009 is contained as part of the CDCP 2015. This document highlights specific engineering requirements. These requirements together with the SWMP responses are detailed in Table 1 below.

As detailed in the this SWMP and the functional design drawing set, the existing dams are required to be retained (as far as possible in their current firm) for heritage reasons. However, if the dams are left in their current state (even if no development of the site occurs) the risk of dam embankment failure and ongoing water quality issues is very real. As such, the SWMP aims to retain (as far as possible) the existing dams shape and water level, while upgrading the embankments to current structural standards. In doing this work, incorporating spillways to control the post development flow rates to predevelopment rates has occurred. This is an example of current best practice where drainage assets are used to achieve many objectives (not only drainage objectives).

Table 1 SWMP Responses to Campbelltown (Sustainable City) Development Control Plan Volume 2 Engineering Design for Development 2009 Requirements

Campbelltown (Sustainable City) Development Control Plan Volume 2 Engineering Design for Development 2009 Requirement	SWMP Incorporation and Comments
The basin is to be designed to perform in the full range of flood events up to 100-year ARI. New detention basins and other water quantity control structures should be located generally off line to creek lines.	The revised drainage strategy does allow for treatment and flood storage in online systems. This is discussed in Section 3.4 above.
All structures designed for the detention of stormwater flows are to be designed utilising: • Hydrographs produced by an acceptable method of unit graph theory or mathematical modelling; • Flood routing through the basin/basins; • Designs are to be checked for a range of hydrographs, for floods up to and including the design return periods, and for floods in excess of the design flood; • The design flood is to be passed through a controlled system – no uncontrolled outflow should occur; • Defined spillways should be provided for flows in excess of the design flood; • Under no circumstances should the basin create a situation which would increase flood peaks downstream; and • A multi stage outlet design which reduces all ARI storm flows to, at or below, undeveloped levels is to be provided.	See RORB modelling in Appendix B and Functional Design Drawings 1808/SWS/1-10. All designs and hydrological analysis in accordance with Australian Rainfall and Runoff 2016 procedures.
The high level outlet to any retarding basin must have capacity to contain a minimum of the 100-year ARI flood event. Additional spillway capacity may be required due to the hazard category of the structure. The hazard category should be determined by reference to ANCOLD (1986).	As detailed in Appendix B, all spillways have been designed to set the 1% AEP flood level in the upstream dam and restrict downstream flows to predevelopment flow rates. GRC has informed SWS, that a preliminary assessment of the small population at risk downstream will probably result in spillways being required to be designed for the 1 in 2000 AEP event. The designs in this SWMP have incorporated 1 in 2000 AEP dam spillways. This assessment is required to be confirmed by GRC.
The spillway design must incorporate sufficient capacity to safely convey a minimum of the 0.5 PMF flows without failure of the embankment.	Detailed design of the dam embankments will ensure adequate structural integrity in the 0.5 PMF event.
Special consideration is to be given to erosion protection on the spillways and the techniques proposed require the approval of Council's Development Engineer.	Detailed design of the dam embankments will ensure adequate structural integrity and erosion protection. At this stage, 3 metre structural crest widths and incorporating 1V:4H batters (to minimise downslope velocities) have been incorporated.
Culvert outlets from detention basins are to be rubber ring jointed with no lifting holes. Cut-off walls and seepage collars are to be provided as necessary. Pipe and culvert bedding are to be specified to minimise its permeability.	These considerations are detailed in the functional design drawing set 1808/SWS 1 - 10.
The outlet structure must take into account the upstream catchment land uses in consideration of potential blockage. A minimum blockage factor of 50% is to be assumed. The design is to be such that if no blockage occurs the outflows comply with Council's requirements set out above. Outlets must have debris and scour control along with safety rail where applicable.	As the outlets from this system are spillways, minimal blockage is expected to occur. Notwithstanding the above, 500 mm freeboard to any upstream buildings have been applied.
Grassed external and internal batters are not to be steeper than 1 in 6.	All functional design batters have been set at 1V:4H. This is a compromise given that the existing steep batters (often greater than 1V:3H) of the existing dams are required to be retained for heritage purposes. Embankments can be vegetated with species not requiring mowing to address maintenance issues, and restriction of the public to embankment areas will ensure no public interaction with the batters.

3.7 Flood Protection Requirements

The CDCP 2015 requires that all stormwater systems shall be sized to accommodate the 1 in 100 AEP events (refer to Section 4 of Council's Engineering Design Guide for Development).

A 'major-minor' approach has been taken to the design of the new stormwater systems. The new piped drainage systems servicing the roads and associated upstream sub catchments have been designed for the minor (10% AEP) event and flows from the major (1% AEP) event will be conveyed within natural overland flow paths within the site.

Proposed culverts will accommodate the 1% AEP (100-year ARI) event.

The CDCP 2015 Section 2.10.2 requires that all overland flow be directed to designated overland flow paths such as roads. The development will retain existing overland flow paths as defined by the existing natural water courses.

There are two buildings proposed to be located adjacent to Dam systems. In regard to proposed building floor levels the CDCP 2015 floor level requirement shave been adhered to as per Table 2 below.

Table 2 Required Building Floor Levels
Reproduced from Table 2.8.1, CDCP 2015

Development Criteria	Where the depth of flow is:	Minimum Freeboard above the predicted 100yr ARI Flood level
Floor Level for any dwelling room* including all commercial or industrial areas	< 300mm	300mm
	> 300mm	500mm
Floor Level in relation to any creek or major stormwater line including detention basins for any dwelling room# including all commercial or industrial areas	Any depth	500mm
Garage or shed Floor Level**	<300mm	100mm
	>300mm	300mm
Underside of solid fencing where overland flow is to be accommodated	Any depth	100mm (min)

CDCP 2015 also states that, for development on land not affected by an overland flow path the minimum height of the slab above finished ground level shall be 150 mm.

Given the above, the proposed floor levels of both building:

- Incorporate floor levels 150 mm above the maximum finished surface level at each site, and
- Incorporate 500 mm freeboard to the 1% AEP flood level as detailed in Appendix B. The relevant levels are:
    - o Building located on the southern bank for Dam 5 – Floor Level = 65.30 m AHD, 1% AEP flood level = 64.80 m AHD, and

- o Building located on the northern bank for Dam 4 – Floor Level = 59.90 m AHD, 1% AEP flood level = 59.40 m AHD

It should be noted that all cemetery development will be located outside the 1% AEP flood extent of the local waterways which will be defined by GRC hydro. Examination of the WMA Water August 2017 TUFLOW model results indicates that, given the relatively steep nature of the valley forms etc, this flood extent will be is contained within the defined riparian zones on site.

3.8 Extreme Flows

The CDCP 2015 Section 2.10.2 requires that safe passage of the Probable Maximum Flood (PMF) shall be demonstrated for major systems.

Detailed design of the dam embankments will ensure adequate structural integrity in the PMF event. It is assumed that the functional designs detailed in this report will be used by GRC Hydro to assess embankment and spillway velocities in extreme events to assess if additional armouring etc is required to be specified at the detailed design stage of the project.

3.9 Safety Issues

The CDCP 2015 Section 2.10.2 requires all proposed drainage structures incorporated within new development shall always be designed to maintain public safety.

The key aspects at this site relate to “not inviting people to danger” around permanent water bodies (dams and waterways) via the use of:

- Safe batters (1V:8H below the water line, 1V:6H above the water line except on the embankments, which are 1V:4H)
- vegetated buffers and water body edges (to current best practice), and
- fencing off areas where batters are steeper than 1 in 6 (where appropriate).

3.10 Engineering Design Guide

Council’s Engineering Design Guide for Development (2012, Section 2.5 Stormwater Management Drawings) requires the following aspects of design to be provided as a part of a development application drawings. Table 3 shows how these requirements have been encompassed in this revised SWMP.

Table 3 SWMP Responses to Council's Engineering Design Guide for Development (2012, Section 2.5 Stormwater Management Drawings) Requirements

Council's Engineering Design Guide for Development (2012, Section 2.5 Stormwater Management Drawings) Requirements	SWMP Incorporation and Comments
Catchment Plan showing contours, area of site affected, and area of site not treated.	See Appendix B
Drainage design summary	See Section 4 of this report.
Calculations to confirm volumes, pipe sizes, size of overland flow paths and overflow weirs.	See Appendix C
Detail Plan and sections,	See SWS Functional Design Drawing Set 1808/SWS/1-10
Design Levels for top water/overflow; inverts of all drainage pits, pipelines and storage areas; overflow weir; surface of all drainage pits; and surfaces designed to control, and direct stormwater	See Appendix C and SWS functional design drawing set 1808/SWS/1-10
Details of Water Sensitive Urban Design elements	See SWS Functional Design Drawing Set 1808/SWS/1-10

3.11 System Maintenance

Design Guide Section 4.15 requires that the drainage system adds value while minimising drainage infrastructure development and maintenance costs.

Informal, natural drainage lines have been incorporated in the design where appropriate. Existing drainage lines and water courses have been retained to reduce the need for built drainage infrastructure and to avoid concentrating flows at a small number of large outlets.

All major drainage and WSUD infrastructure have been specifically proposed and designed to:

- Meet the current heritage constraints of the site,
- Provide naturalistic and self-sustaining drainage elements, to
- Minimise ongoing maintenance requirements over time, while
- Maximising site ecological diversity.

This is in line with current best practice approaches to WSUD application in Australia.

Production of detailed inspection and maintenance schedules for all swales, wetlands and dam systems will be prepared at the detailed design stage of the project.

3.12 Landscape, Community Ownership and Education

Design Guide Section 4.15 requires community involvement, understanding and appreciation of the environment.

The proposed development will provide significant opportunities for community appreciation of the environment. Public access will be provided by a significant path network including for observation of the water cycle management elements within the site.

The importance of considering the management objectives, landscape values, heritage requirements and community aspirations is a fundamental part of developing an integrated design solution. To this end, TB&E, WS&P, Florence Jaquet Landscape Architects and Stormy Water Solutions have worked closely to ensure that drainage elements, such as wetland systems and swales, offer the opportunity to complement the landscape amenity and ecological diversity (especially of the riparian zones of the final landscape form). This is in line with best practice application of WSUD in drainage strategies.

Provision of existing and future habitat corridors along existing watercourses and future swales has been seen as a major objective, particularly in terms of providing future habitat for local fauna.

4 Storm Water Management Plan Description

The primary drainage elements proposed within the SWMP are detailed within SWS drainage set 1808/SWS/1-10. These drawings are reproduced in this Appendix A and discussed further below.

To achieve the requirements detailed in Section 3 above, the developed SWMP must achieve multiple objectives. This can be achieved by providing drainage elements:

- Which act together to achieve specific objectives, and
- Incorporate dual functions within each site (if possible).

For example, a Dam can incorporate:

- Retention of dam heritage attributes,
- A retarding basin (flood storage function),
- Contain a wetland (WSUD function), while also ensuring
- landscape enhancement, and
- Increase site ecological diversity.

This is WSUD at its best. The industry is well beyond the time when drainage elements only perform engineering functions alone.

The developed SWMP is aimed at achieving all of the above objectives detailed in Section 3. Specific requirements and the SWMP proposed in the SWMP to address these issues are detailed below.

4.1 Treatment of Development and Burial Areas – All Catchments

In regard to drainage impact, this development largely consists of two types of development being:

- Development resulting in 100% imperviousness areas (roads, car parks, roofs etc), and
- Burial areas.

100% impervious areas (such as roads, car parks and roofs) have been designed by WS&P to drain directly to pipes drainage systems which outfall into the swales, wetlands and/or retarding basin/wetland systems aligned to outfall locations (as described above) for treatment of stormwater.

The remainder of the site will largely be “Burial” areas. Based on other similar cemetery sites, it can be assumed that about:

- 20% of total “Burial” areas could be full monumental (100% impervious), and
- The remainder will be lawn with a concrete beam (0.40 m wide on average concrete beams, running parallel every 5.1 m) This results in a fraction imperviousness in these areas of 8%.

Of course, the total site will not be defined burial areas. As such, a reasonable fraction imperviousness for areas which have burial areas (lawn and monumental) located within them is assumed to be 25% within the WSUD Strategy and SWMP.

Burial areas (catchments), have one additional treatment source in addition to those defined above. All burial areas are assumed to shed stormwater from their impervious areas directly into the surrounding grass, where it eventually makes its way to a local pipe or an outfall treatment element. This shedding of water into surrounding grass can be defined a “buffer” treatment in the MUSIC model and this has been accounted for in the MUSIC model detailed in Appendix D of this report.

Essentially, this “buffer” treatment is the primary treatment element in the WSUD strategy treatment train definition for this site. Secondary and tertiary treatment occurs in the downstream swales and wetland systems as applicable (see below).

In catchments where there are some roads, roofs etc., up to 20% of the catchment may be directly connected to a pipe. In this situation it is assumed that 80% of the catchment is buffered and the buffer area is assumed to be 50% of the upstream impervious area. In catchments only exhibiting burials, none of the catchment will be directly connected to a pipe system, and as such 100 % of the catchment is assumed to be buffered and the buffer area is assumed to be 50% of the upstream impervious area. Both of these assumptions are considered conservative in this application in regard to accounting for burial areas being disconnected from the drainage system.

4.2 Vegetated Swales

The secondary treatment proposed in the treatment train is swale treatment.

Two types of treatment swales are proposed. These are:

1. 5% AEP vegetated swales around the boundary road designed by WS&P and shown as “pink” swales in 1808/SWS/1, and
2. Retaining the current valley floor and vegetating at least 1 metre over the existing bases width in existing drainage lines (shown as “green” swales in 1808/SWS/1.

Crucial to the strategy is to ensure the base of all swales are planted out with dense sedges and rushes. This is crucial to ensuring the flow velocities do not cause erosion of the drainage lines in this relatively steep site.

The detailed design phase of the project may also consider strategic placement of pools and riffles to minimise swale slope in some locations. However, provided planting occurs as described above, this rockwork is not specifically required in the design.

The vegetated swales in existing drainage lines incorporate the existing form of the gully topography along the existing defined drainage lines in line with the riparian zone requirements. In drainage lines defined as “green” in 1808/SWS/1, it is proposed to remodel the watercourses as a swales. In these “green” swales, the watercourse is assumed to be converted to drainage swale definition.

It should be noted that swales defined as “orange” in 1808/SWS/1 may also require the above treatment to control erosion etc. However, no stormwater treatment has been attributed to these swales to negate riparian zone offsets required elsewhere on site.

The primary requirement regarding swale design is that the base of each swale is required to be planted with dense sedges and rushed over at least one metre. This dense planting forms the flood attenuation and pollutant reduction function as detailed in this Appendices B and D.

Based on typical cross sections determined from the Lidar data, these assets will typically incorporate the following parameters:

- Base width = 1 metres (fully vegetated with sedges and rushes),
- Side Batters = 1(vertical) to 6(horizontal), typical based on Lidar information,
- Depth = 0.5 m,
- Top Width = 7 m, and
- Longitudinal slope = 1% to 10% (generally following natural surface slope along the swale).

The typical envisaged form of a vegetated swale configured in an existing drainage line is shown in Figure 3 below.

Figure 3 Typical form of a vegetated swale

4.3 Dam Considerations

Dams 2 and 4 are proposed to be retained as open waterbodies. Dams 3, 5 and 6 are proposed to be remodelled as stormwater treatment wetlands. Dams 7 and 9 are proposed to be removed. Dam 8 is proposed to be filled and planted with ephemeral planting.

4.3.1 Dam Heritage Considerations

Overriding all dam considerations is the requirement, to as far as possible retain the existing normal water level and dam shape for heritage reasons.

4.3.2 Dam Embankment Structural Integrity

The functional design drawing set 1808/SWS/1-10 details the embankment and spillway upgrade required to ensure all dams are bought up to current structural requirements. This includes meeting all requirements as reiterated in sections 3.7, 3.9 and 3.10.

All functional design drawings are based on the hydrological calculations detailed in Appendices B and C, and clearly define:

- 3 metre crest widths on all embankments,
- 1V:4H batters on all embankments,
- Vegetated batters to minimise maintenance (mowing) activities on embankments,
- Restrictive public access to all embankment areas (given embankments are greater than 1V:6H slope),
- Spillways defined to safely convey the 1 in 2000 AEP event within each site.

Structural requirements of the embankment will be finalised once GRC Hydro undertakes assessment of the PMF flood to assess extreme flow velocities on the crest and downslope vegetated batters.

4.3.3 Dam Water Quality

The catchment contributing to the dam systems is relatively small. Investigations undertaken as per Appendix D indicate that the residence time of stormwater within the systems can be very long. This can lead to an increased risk of algal blooms in these systems over time. The primary way this risk is usually mitigated is to minimise the size of the water bodies, and thus minimise water residence times. If water “turns over” in the dams regularly, there is less chance of an algal bloom (or water quality issue) occurring). Minimising dam areas (at NWL) cannot occur in this case due to the heritage constraints of the project.

As such, the dam designs have been formulated to minimise the risk of algal blooms by:

- Minimising systems depths by incorporating shallow wetland systems and raising the base of pond systems within the dams,
- Maximising vegetation within the dams to increase stormwater nutrient uptake via incorporating wetlands within the systems and vegetating all batters with appropriate sedges and rushes,
- Incorporating the provision to circulate water within the systems, possibly by fountains in the sculpture elements of the systems,
- Possibly allowing treated bore water top-up the water over in the dam systems up in very dry times, and
- Incorporating a recirculation pump system to allow recirculation of dam water though the upstream wetland systems. Recirculating water though wetland systems is a proven technology regarding minimising nutrient uptake by algae (as the wetland plants are providing competition for this food source). This minimises the risk of algal blooms in dry times.

The residence time analysis (Appendix D.5) indicates that some of the systems may have relatively long water residence times in dry times. However, given the design considerations listed above (primarily the high degree of vegetation proposed within the systems), it is anticipated that higher than usual residence times could be accommodated without undue risk of algal blooms or water quality issues. This is because these systems will be more in line with “wetland” designs (incorporating shallow water bodies and significant vegetation) rather than lake systems (which are deep with minimal vegetation).

It is recommended that a dam management plan be formulated at the detailed design stage of the project. Mitigation activities (such as initiating recirculation pumps etc) should be initiated when, and if, the risk of water quality issues is identified via ongoing water quality testing. The duration and timing of water quality testing which will be formulated as per the dam management plan.

Notwithstanding the above, some top up with treated bore water may be required in dry years to ensure adequate water quality in these very important landscape features.

4.3.4 Wetland Treatment

Dams 3, 5 and 6 are proposed to incorporate a wetland function. Figure 4 below shows, conceptually, how the outlets from these dams are configured to ensure this function can occur. This is through the provision of extended detention (i.e. storing stormwater for treatment) above the normal water level of the system.

Structural pipe and pit outlet systems will be required from these systems (in addition to a spillway for extreme flows) to ensure this extended detention function can operate effectively.

SWS drawings 1808/SWS/1-10 detail the functional designs of Dams 3, 5 and 6.

It should be noted that planting of Dams 2 and 4 with <u>submerged</u> wetland planting can also be considered going forward to:

- Further treat site stormwater, and
- Mitigate against water quality issues in these assets.

Planting with submerged plants will achieve these objectives without compromising the existing “open water” look of these assets.

4.3.5 Retarding Basin Provisions

Dams 2 and 4 are proposed to incorporate a flood storage (i.e. flood retardation). Figure 4 below shows, conceptually, how the outlets from these dams are configured to ensure this function can occur. This is through the provision of a spillway (set at normal water level) which incorporated a specified width to control the 1% AEP flow to predevelopment levels from the site (i.e. storing stormwater above the NWL for flood storage). The spillway widths then define the 1 in 2000 AEP flood level in the dam, which then results in the required embankment level definition.

SWS drawings 1808/SWS/1-10 detail the functional designs of Dams 2 and 4.

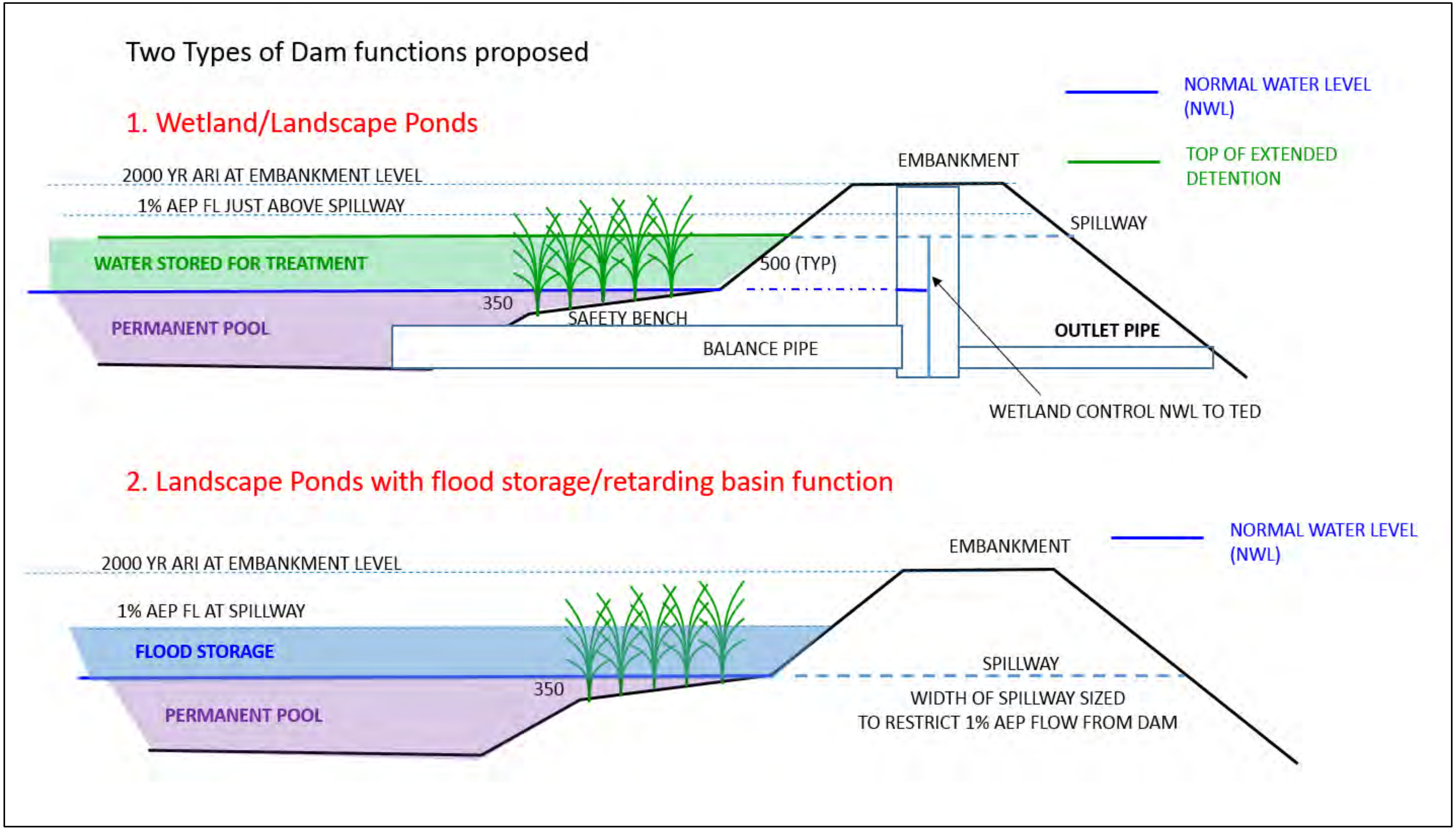


Figure 4 **Dam Outlet Configurations Proposed**

4.3.6 Dam Monitoring and Maintenance Considerations

The SWMP WSUD pre-treatment mechanisms should ensure adequate control of nutrient concentrations within the dam systems. That is, the dam pre-treatment mechanisms, in-dam wetlands and vegetated safety benches (ie water body edge treatments) have been designed within this risk factor in mind.

Notwithstanding the above, good dam water body management will supplement the upstream and “in dam” protection mechanisms. The potential for algal blooms is the major risk the dam system faces. Table 4 details potential risk factors for algal growth. Also detailed are:

- The inbuilt design protection initiatives to address each risk factor, and
- The potential dam management procedures which could be formulated in a future dam management plan to address each risk factor.

A future dam management plan will define and ensure adequate water monitoring requirements. Any possible dam water quality problems will be able to be identified very early and addressed though changes to the recirculation system or, in extreme cases, possibly treated bore water top up.

As detailed in Table 4, additional management techniques can also be applied. The plan will assign the responsibility for its implementation to the Cemetery manager. The Cemetery maintenance staff will be required to be trained to have an intimate knowledge of the dam system and the factors affecting water quality. As such prompt and appropriate action regarding any potential problems is expected to occur.

The combination of upstream pre-treatment, good in dam water body design mechanisms and the existence of the robust dam management plan will minimise the risk of algal blooms and water quality problems within the dam systems.

Table 4 Water Quality and Algal Bloom - Dam Management Risks and Management Options

Water Quality and Algal Bloom Risk Factor	Dam Design Aspects which Minimise Risk	Dam Management Options which Minimise Risk (future Dam Management Plan requirements)
Release of Phosphorus due to thermal stratification and development of anaerobic bottom waters	Design of large, shallow open water bodies maximising exposure to wind action and aeration. Possible fountains in landscape features	Monitoring program Use of the proposed recirculating system to circulate pond waters. Injecting air into the lake to increase dissolved oxygen content.
Imbalance between Nutrient levels and grazing invertebrates	Designing dam planting for fauna habitat to approximately 50% areal cover of total system (including safety benches etc)	Ongoing monitoring of lake fauna concentrations Introduction of fish species Maintaining lake plantings to approximately 50% total dam coverage Infill planting of Dams 2 and 4 with submerged wetland species (if required)
High water residence times and oxygen depletion	Designing lake planting to approximately 50% areal cover to develop viable in dam ecological systems Provision of bypass pipes and valves which can be used to minimise lake volumes in dry times. (that is, artificially decrease lake residence times) Provision of a lake top up system to ensure pond circulation (if required).	Monitoring Program Managing top up water inputs to assist mixing and turnover. Signage if a bloom occurs Regular maintenance activities to remove algal scum, excessive vegetation and litter
Excessive aquatic plant growth	Limiting lake planting to approximately 50% areal cover	Monitoring program Controlling lake planting to approximately 50% areal cover Regular maintenance activities to remove algal scum, excessive vegetation and litter. Infill planting of Dams 2 and 4 with submerged wetland species (if required Introducing fish species

5 Conclusions

The stormwater drainage system proposed for the Macarthur Memorial Park Estate represents a strategy development covering all requirements of best practice floodplain and catchment management.

The WSUD strategy and SWMP has been formulated with full integration with the landscape proposals (developed by FJLA), ecological constraints (defined by TB&E) and internal development drainage proposals (developed by WS&P). As such, the plans clearly show there is enough space allocated on site to ensure all drainage requirements can be met going forward.

The combination of upstream pre-treatment, good in dam water body design mechanisms and the existence of the robust dam management plan will minimise the risk of algal blooms and water quality problems within the dam systems.

It should be noted that the assumptions regarding WSUD strategy and SWMP elements may change over time. However, it is considered at this stage, that the work presented has defined realistic and adequate land footprints required by the major drainage assets required for the Macarthur Memorial Park cemetery.

5.1 Further Work Required

Going forward, the following tasks are required to ensure full implementation of this SWMP:

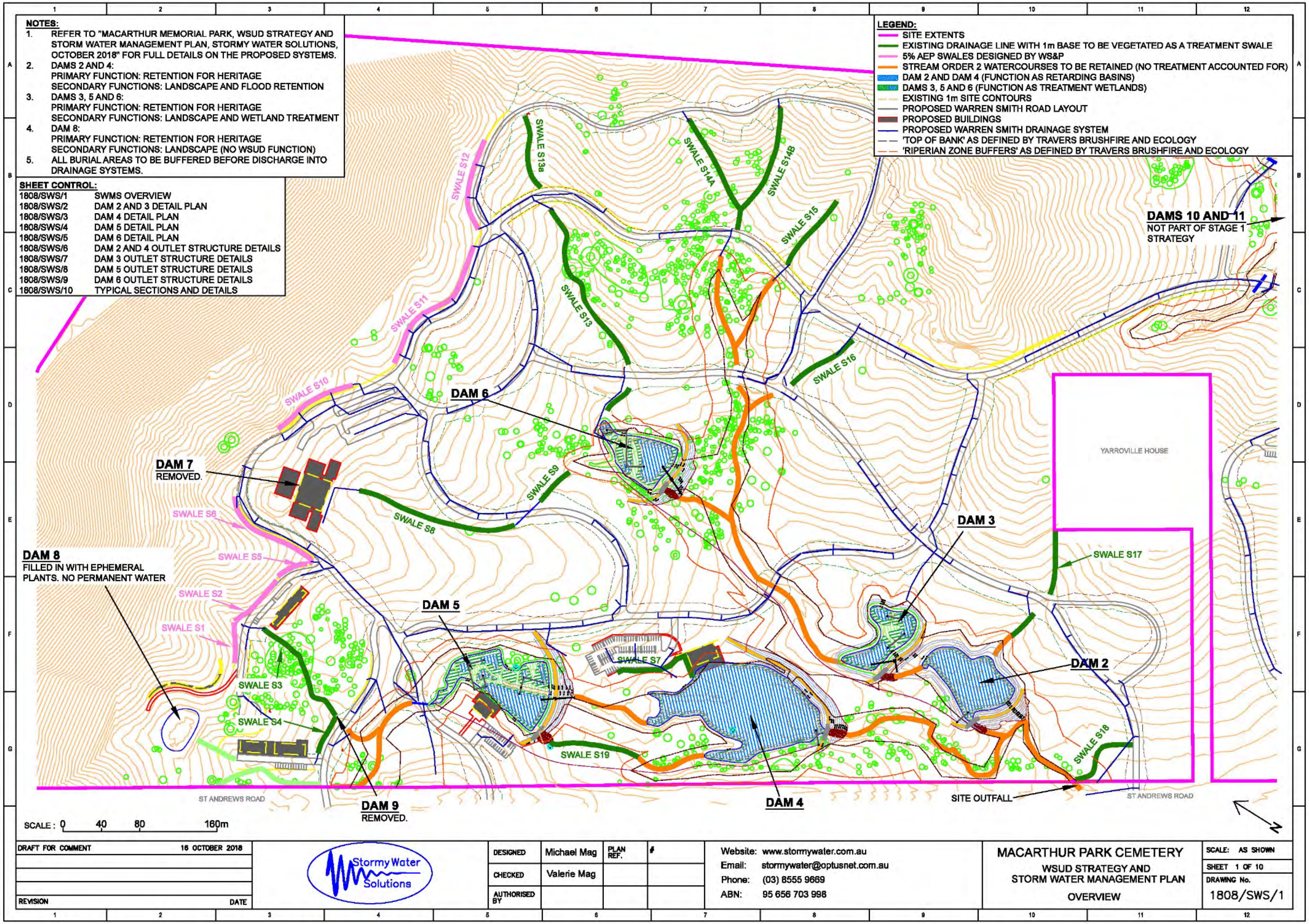
- WS&P will ensure the documentation and plans relating the road and development piped network are aligned with the drainage proposals in this report,
- GRC Hydro is required to conducting an updated flood analysis of the waterways and dams affecting the site to confirm the flood levels and spillway requirements detailed in this SWMP,
- GRC Hydro is required to confirm the small population at risk downstream results in dam spillways being required to be designed for the 1 in 2000 AEP event,
- Geotechnical advice is required to define the final structural form of the embankment design (i.e. clay core requirements etc),
- A dam management plan is required to be formulated in line with the discussion presented in Section 4.3.6 at the detailed design stage of the project,
- A WSUD inspection and maintenance schedules for all WSUD assets (swales, wetlands and ponds) is required to be formulated at the detailed design stage of the project, and
- The project team should provide this report to Council and the relevant authorities for approval.

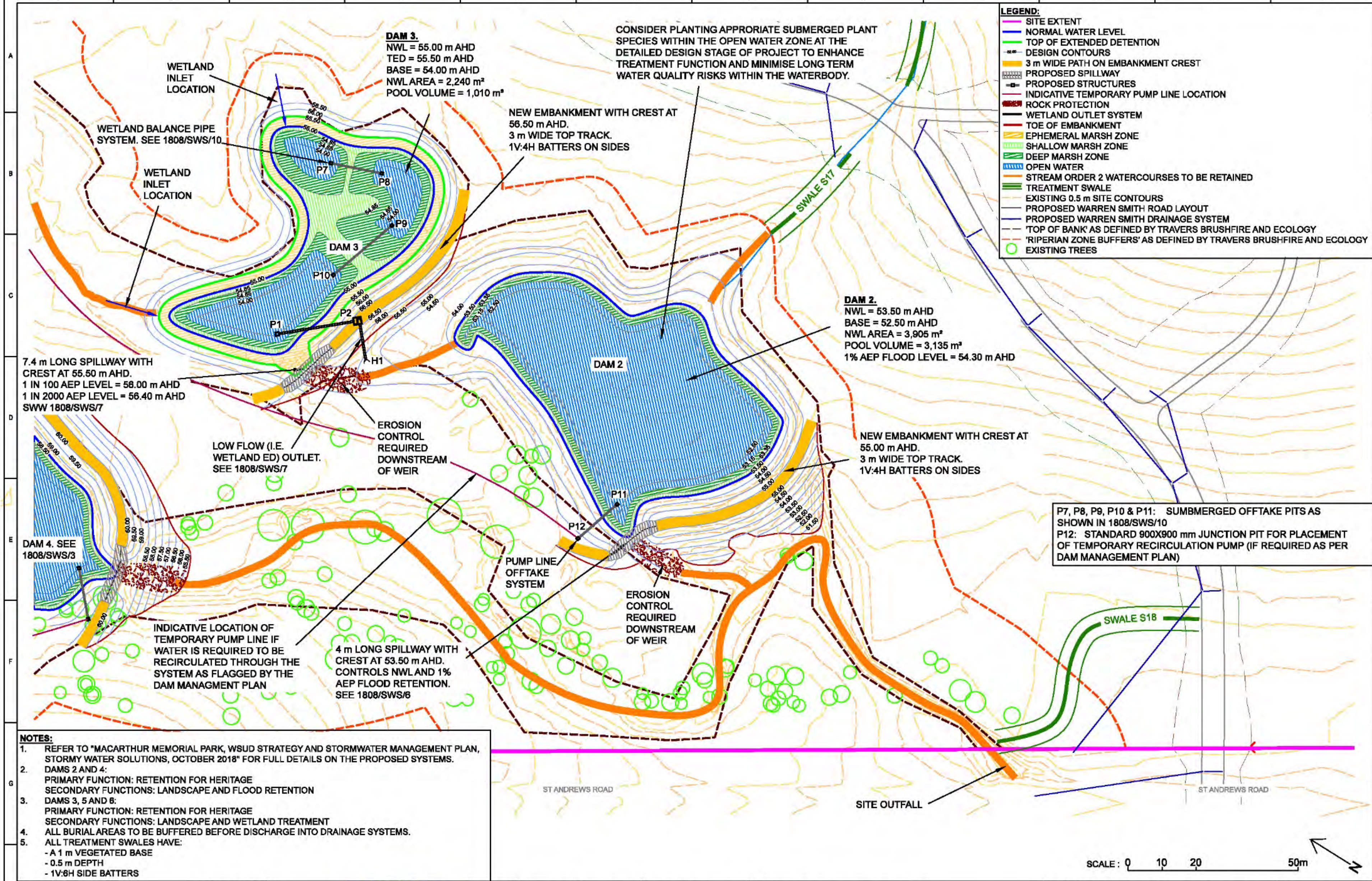
6 Abbreviations and Definitions

The following table lists some common abbreviations and drainage system descriptions and their definitions which are referred to in this report.

Abbreviation Descriptions	Definition
AEP – Annual Exceedance Probability	The probability of an event being equalled or exceeded within a year.
AHD - Australian Height Datum	Common base for all survey levels in Australia. Height in metres above mean sea level.
ARI - Average Recurrence Interval.	The average length of time in years between two floods of a given size or larger
AR&R 2016	Australian Rainfall and Runoff: A Guide to Flood Estimation, Commonwealth of Australia Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors), 2016
BoM	Bureau of Meteorology
Evapotranspiration	The loss of water to the atmosphere by means of evaporation from free water surfaces (e.g. wetlands) or by transpiration by plants
FJLA	Florence Jaquet Landscape Architect
Groundwater	All water stored or flowing below the ground surface level
Hectare (ha)	10,000 square metres
Hec Ras	A one-dimensional, steady state hydraulic model which uses the Standard Step Method to calculate flood levels and flood extents
Kilometre (km)	1000 metres
m^3/s -cubic metre/second	Unit of discharge usually referring to a design flood flow along a stormwater conveyance system
Megalitre (ML) (1000 cubic metres)	1,000,000 litres = 1000 cubic metres Often a unit of water body (e.g. pond) size
MUSIC	Hydrologic computer program used to calculate stormwater pollutant generation in a catchment and the amount of treatment which can be attributed to the WSUD elements placed in that catchment. Can also be used to calculate water body turnover period and wetland drawdowns etc.
NWL	Normal Water Level – invert level of lowest outflow control from a wetland or pond.
PET	Potential Evapotranspiration – potential loss of water to the atmosphere by means of evaporation or transpiration from wetland or pond systems.
Retarding Basin	Drainage element used to retard flood flows to limit flood impacts downstream of a development. Can include complementary WSUD and ecological site benefits if wetland incorporated within the site.
RORB	Hydrologic computer program used to calculate flood flows (m^3/s) and size retarding basins
Surface water	All water stored or flowing above the ground surface level
SWMP	Storm Water Management Plan
TED	Top of Extended Detention – Level to which stormwater is temporarily stored for treatment in a wetland or pond (above NWL).
TB&E	Travers Bushfire & Ecology
TSS	Total Suspended Solids – a term for a particular stormwater pollutant parameter
TP	Total Phosphorus – a term for a particular stormwater pollutant parameter
TN	Total Nitrogen – a term for a particular stormwater pollutant parameter
WS&P	Warren Smith and Partners
Wetland	WSUD elements which is used to collect TSS, TP and TN. Either permanently or periodically inundated with shallow water and either permanently or periodically supports the growth of aquatic macrophyte

Appendix A Storm Water Management Plan Drawings





NOTES:

1. REFER TO "MACARTHUR MEMORIAL PARK, WSUD STRATEGY AND STORMWATER MANAGEMENT PLAN, STORMY WATER SOLUTIONS, OCTOBER 2018" FOR FULL DETAILS ON THE PROPOSED SYSTEMS.
2. DAMS 2 AND 4:
PRIMARY FUNCTION: RETENTION FOR HERITAGE
SECONDARY FUNCTIONS: LANDSCAPE AND FLOOD RETENTION
3. DAMS 3, 5 AND 6:
PRIMARY FUNCTION: RETENTION FOR HERITAGE
SECONDARY FUNCTIONS: LANDSCAPE AND WETLAND TREATMENT
4. ALL BURIAL AREAS TO BE BUFFERED BEFORE DISCHARGE INTO DRAINAGE SYSTEMS.
5. ALL TREATMENT SWALES HAVE:
- A 1 m VEGETATED BASE
- 0.5 m DEPTH
- 1V:6H SIDE BATTERS

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		CHECKED	Valerie Mag		
REVISION	DATE	AUTHORISED BY			

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MACARTHUR PARK CEMETERY
WSUD STRATEGY AND STORM WATER MANAGEMENT PLAN
DAM 2 AND 3 DETAIL PLAN

SCALE: AS SHOWN
SHEET 2 OF 10
DRAWING No. 1808/SWS/2

NOTES:

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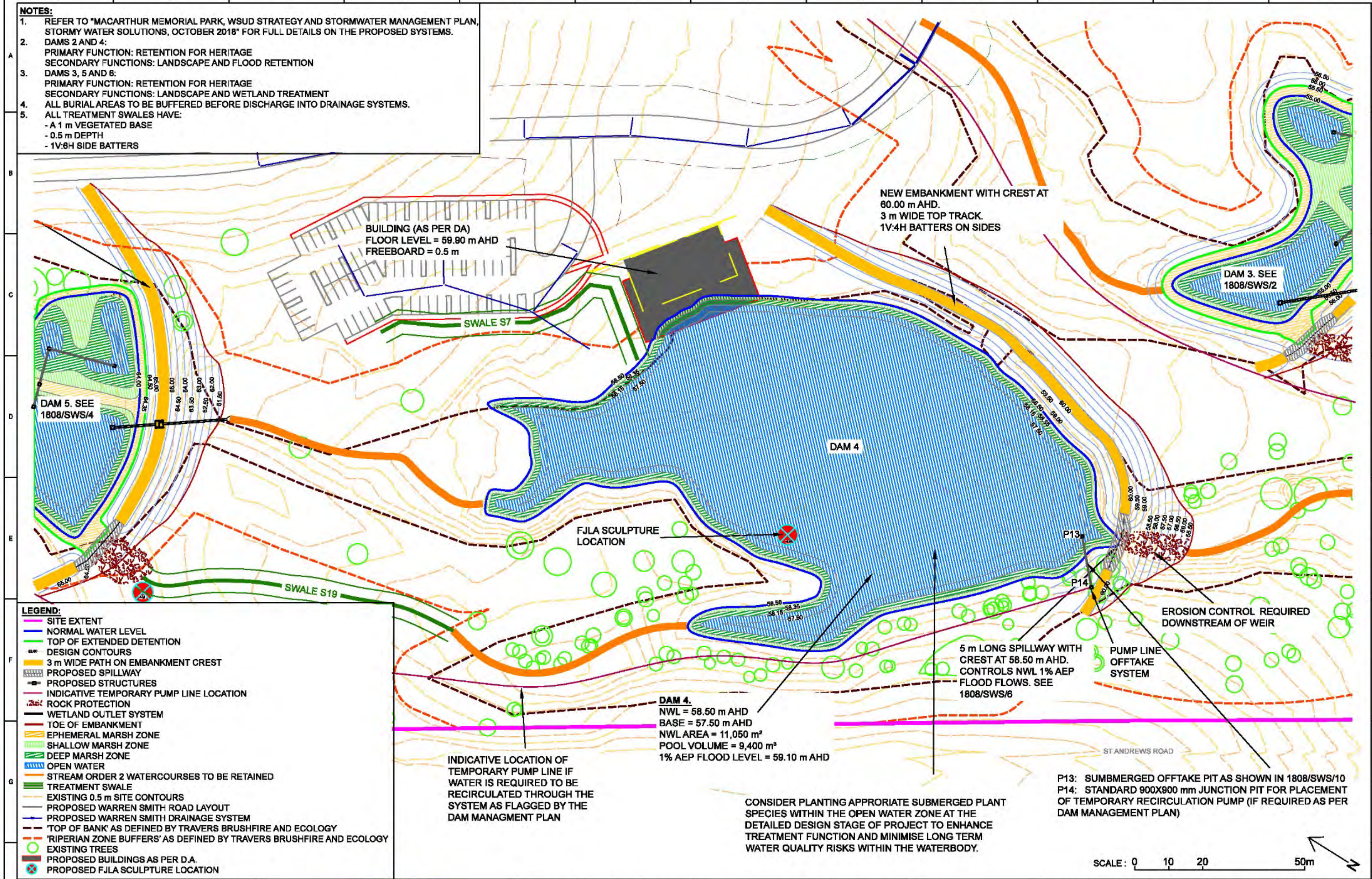

LEGEND:

- SITE EXTENT
- NORMAL WATER LEVEL
- TOP OF EXTENDED DETENTION
- DESIGN CONTOURS
- 3 m WIDE PATH ON EMBANKMENT CREST
- PROPOSED SPILLWAY
- PROPOSED STRUCTURES
- INDICATIVE TEMPORARY PUMP LINE LOCATION
- ROCK PROTECTION
- WETLAND OUTLET SYSTEM
- TOE OF EMBANKMENT
- EPHEMERAL MARSH ZONE
- SHALLOW MARSH ZONE
- DEEP MARSH ZONE
- OPEN WATER
- STREAM ORDER 2 WATERCOURSES TO BE RETAINED
- TREATMENT SWALE
- EXISTING 0.5 m SITE CONTOURS
- PROPOSED WARREN SMITH ROAD LAYOUT
- PROPOSED WARREN SMITH DRAINAGE SYSTEM
- 'TOP OF BANK' AS DEFINED BY TRAVERS BRUSHFIRE AND ECOLOGY
- 'RIPERIAN ZONE BUFFERS' AS DEFINED BY TRAVERS BRUSHFIRE AND ECOLOGY
- EXISTING TREES
- PROPOSED BUILDINGS AS PER D.A.
- PROPOSED FJLA SCULPTURE LOCATION

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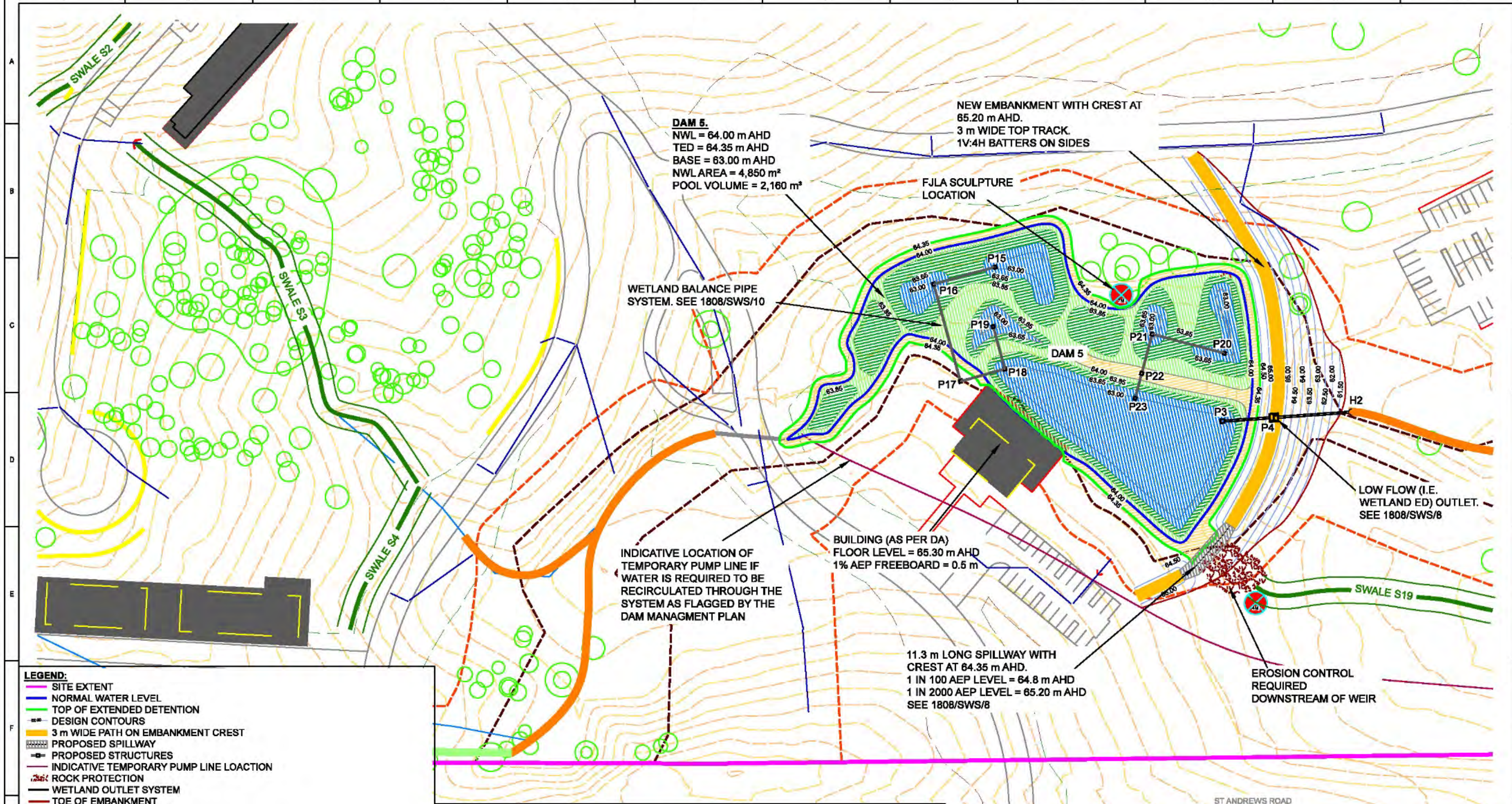
		CHECKED: Valerie Mag		Email: stormywater@optusnet.com.au	WSUD STRATEGY AND STORM WATER MANAGEMENT PLAN
		AUTHORISED BY		Phone: (03) 8555 9669	DAM 4 DETAIL PLAN
REVISION / DATE				ABN: 95 656 703 998	SCALE: AS SHOWN; SHEET 3 OF 10; DRAWING No. 1808/SWS/3



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P15, P16, P18, P19, P20, P21 & P23: SUMBMERGED OFFTAKE PITS AS SHOWN IN 1808/SWS/10.
P17 & P22: STANDARD 900X900 mm JUNCTION PITS WITH GRILLED LID FOR INSPECTION.

SCALE : 0 10 20 50m

DRAFT FOR COMMENT 16 OCTOBER 2018	Stormy Water Solutions	DESIGNED: Michael Mag	PLAN REF.: #	Website: www.stormywater.com.au	MACARTHUR PARK CEMETERY	SCALE: AS SHOWN
		CHECKED: Valerie Mag		Email: stormywater@optusnet.com.au	WSUD STRATEGY AND STORM WATER MANAGEMENT PLAN	SHEET 4 OF 10
		AUTHORISED BY		Phone: (03) 8555 9669		DRAWING No.
REVISION / DATE				ABN: 95 656 703 998	DAM 5 DETAIL PLAN	1808/SWS/4

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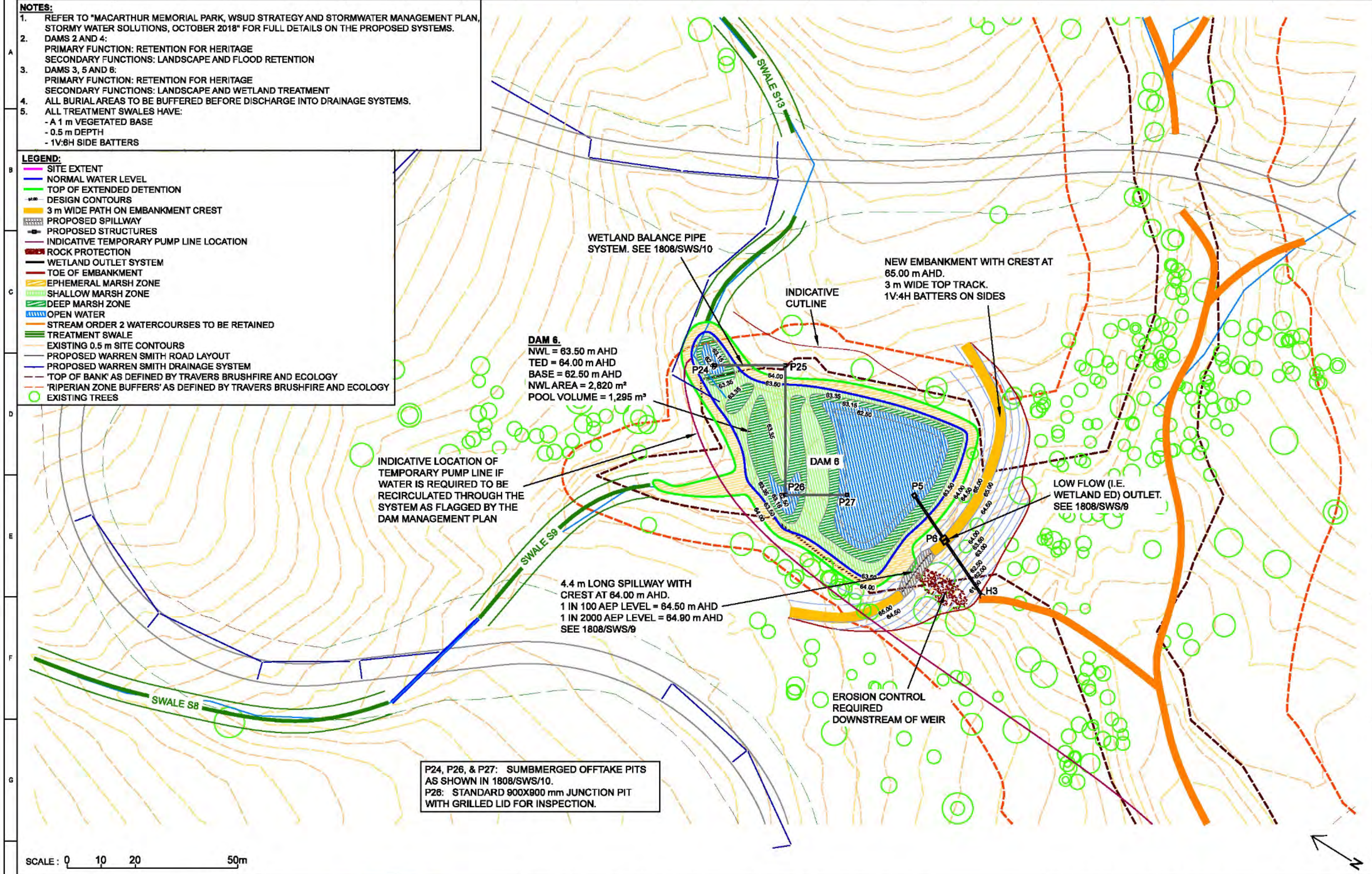

P24, P26, & P27: SUMBMERGED OFFTAKE PITS AS SHOWN IN 1808/SWS/10.
P26: STANDARD 900X900 mm JUNCTION PIT WITH GRILLED LID FOR INSPECTION.

SCALE: 0 10 20 50m

DRAFT FOR COMMENT	16 OCTOBER 2018
REVISION	DATE

Stormy Water Solutions

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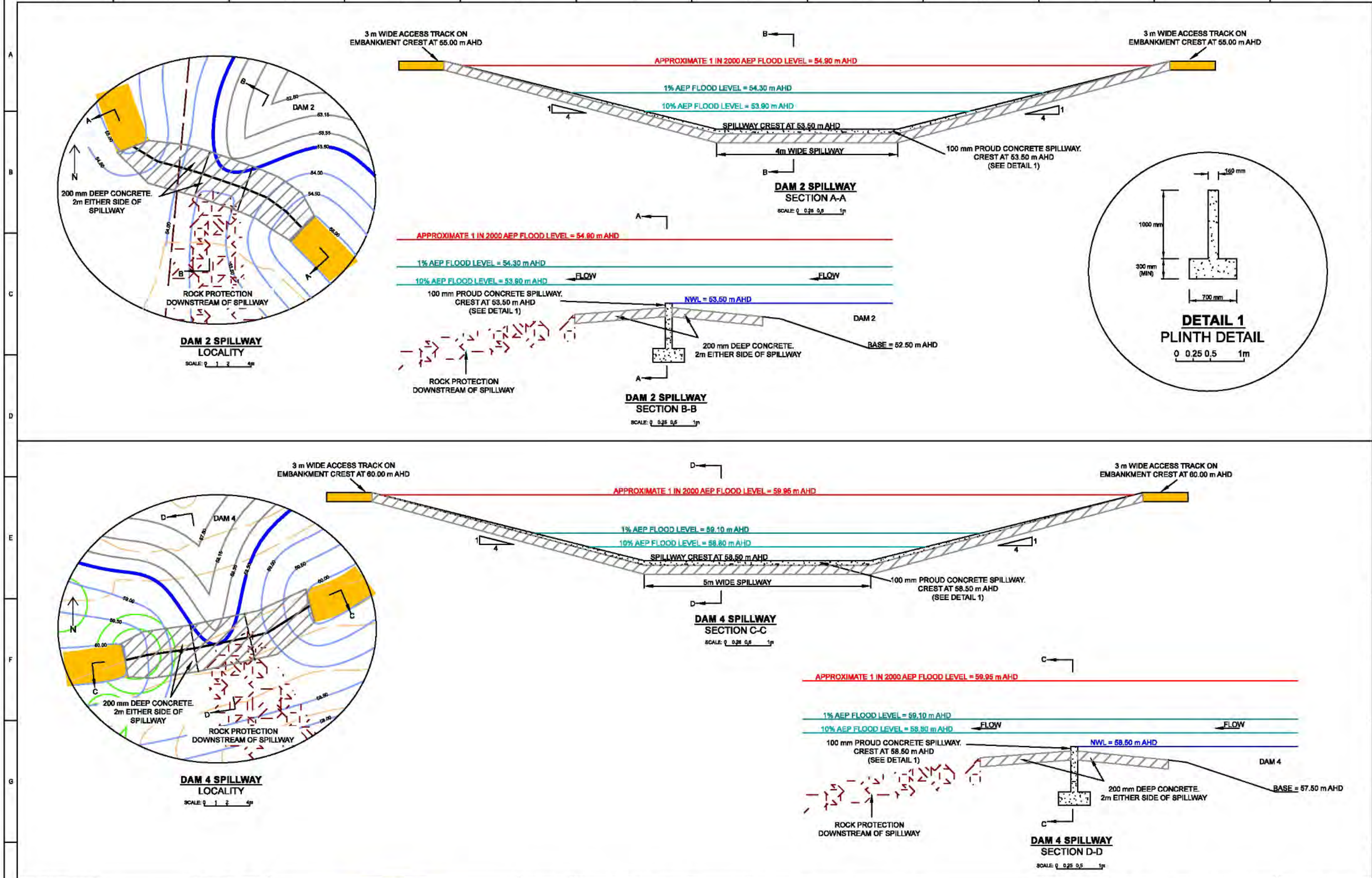
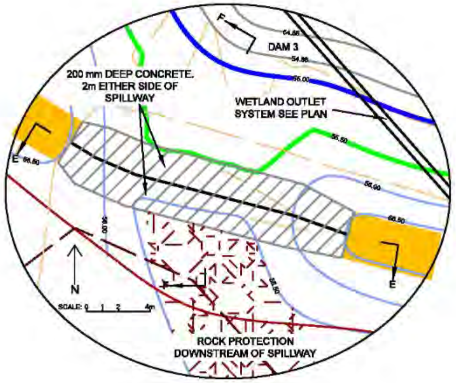
MACARTHUR PARK CEMETERY
WSUD STRATEGY AND STORM WATER MANAGEMENT PLAN
DAM 6 DETAIL PLAN

SCALE: AS SHOWN
SHEET 5 OF 10
DRAWING No. 1808/SWS/5



DRAFT FOR COMMENT 16 OCTOBER 2018	Stormy Water Solutions	DESIGNED: Michael Mag	PLAN REF.: #	Website: www.stormywater.com.au	MACARTHUR PARK CEMETERY	SCALE: AS SHOWN
		CHECKED: Valerie Mag		Email: stormywater@optusnet.com.au	WSUD STRATEGY AND STORM WATER MANAGMENT PLAN	SHEET 6 OF 10
REVISION / DATE		AUTHORISED BY		Phone: (03) 8555 9669 ABN: 95 656 703 998	DAM 2 AND 4 OUTLET STRUCTURE DETAILS	DRAWING No. 1808/SWS/6



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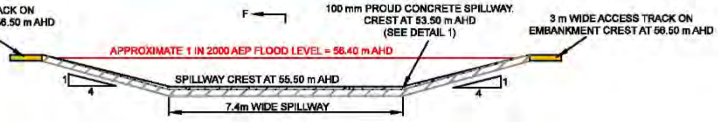
LOCALITY



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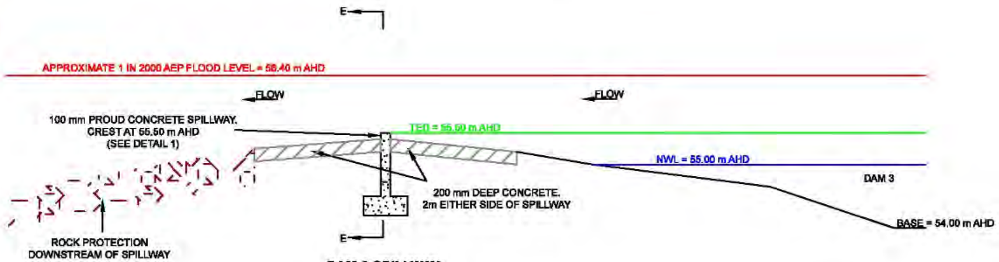
SECTION E-E



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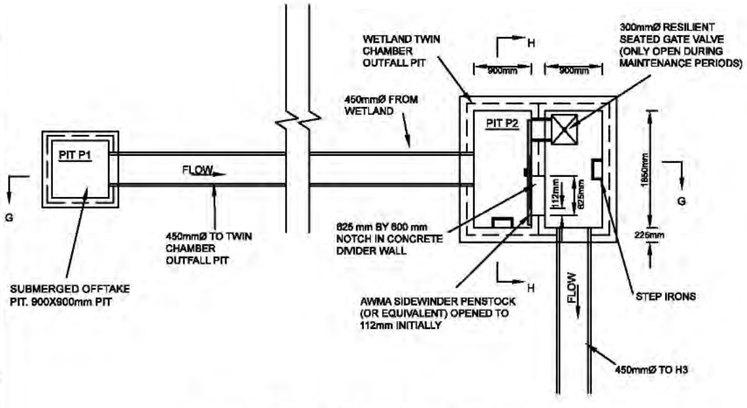
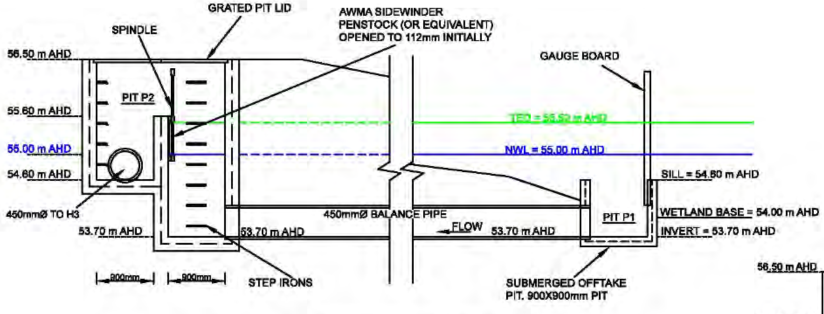
SECTION F-F



DAM 3 WETLAND TWIN CHAMBER OUTFALL PIT
PLAN VIEW



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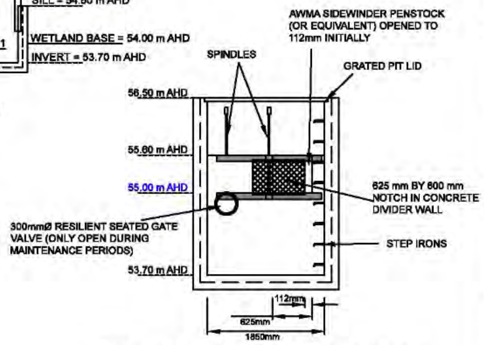
SECTION G-G



DAM 3 WETLAND TWIN CHAMBER OUTFALL PIT
SECTION H-H

NOTE:
OUTLET PIPE SYSTEM TO BE:
- RUBBER RING JOINTED
- INCORPORATE NO LIFTING HOLES
- INCORPORATE CUT OFF WALLS AS SEEPAGE COLLARS AS REQUIRED.

DRAFT FOR COMMENT	16 OCTOBER 2018
REVISION	DATE

Stormy Water Solutions

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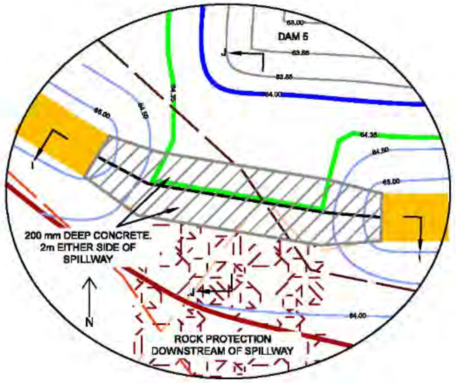
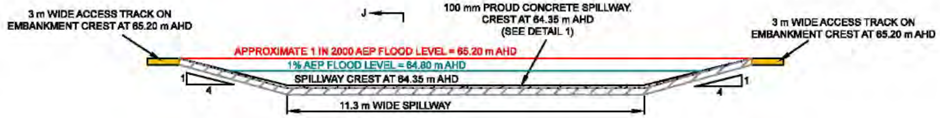
MACARTHUR PARK CEMETERY
WSUD STRATEGY AND STORM WATER MANAGMENT PLAN
DAM 3 OUTLET STRUCTURE DETAILS

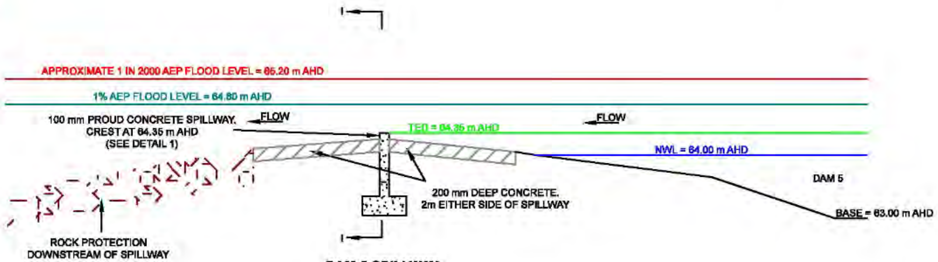
SCALE: AS SHOWN
SHEET 7 OF 10
DRAWING No. 1808/SWS/7



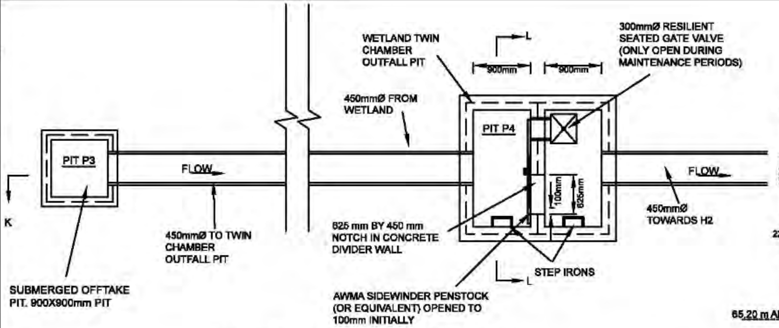
DAM 5 SPILLWAY
LOCALITY



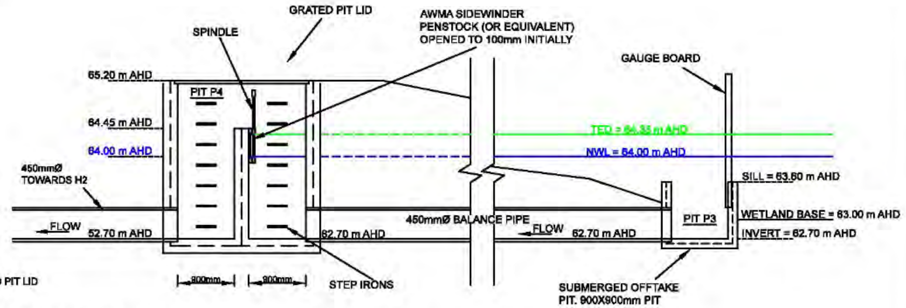
DAM 5 SPILLWAY
SECTION I-I



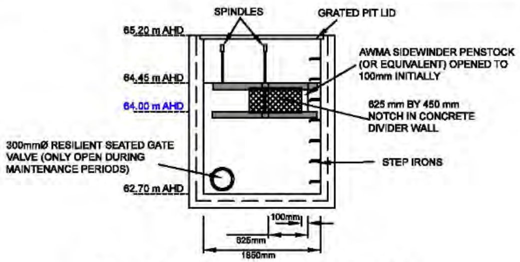
DAM 5 SPILLWAY
SECTION J-J



DAM 5 WETLAND TWIN CHAMBER OUTFALL PIT
PLAN VIEW



DAM 5 WETLAND TWIN CHAMBER OUTFALL PIT
SECTION K-K



DAM 5 WETLAND TWIN CHAMBER OUTFALL PIT
SECTION L-L

NOTE:
OUTLET PIPE SYSTEM TO BE:
- RUBBER RING JOINTED
- INCORPORATE NO LIFTING HOLES
- INCORPORATE CUT OFF WALLS AS SEEPAGE COLLARS AS REQUIRED.

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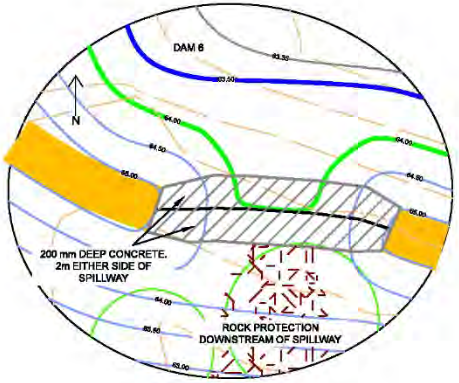
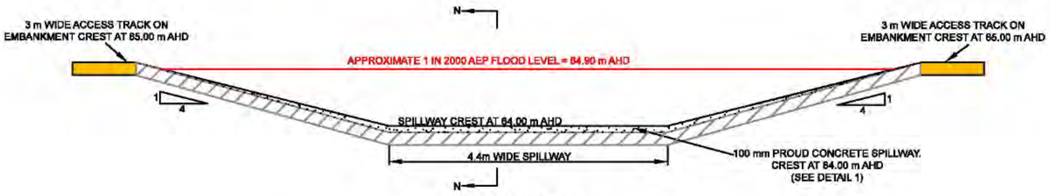
REVISION	DATE						ABN: 95 656 703 998

MACARTHUR PARK CEMETERY
WSUD STRATEGY AND STORM WATER MANAGMENT PLAN
DAM 5 OUTLET STRUCTURE DETAILS

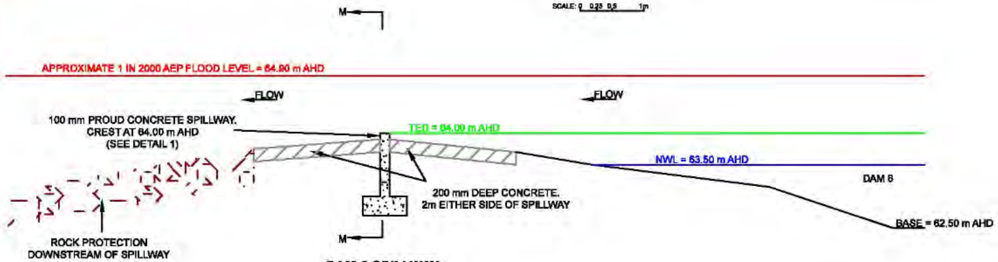
SCALE: AS SHOWN
SHEET 8 OF 10
DRAWING No. 1808/SWS/8



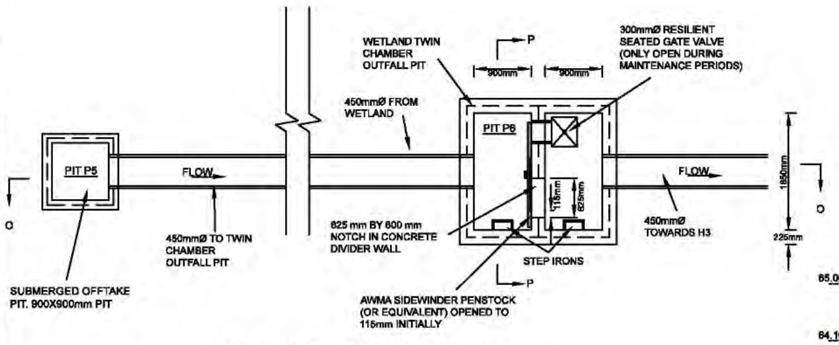
DAM 6 SPILLWAY
LOCALITY
SCALE: 0 1 2 4m



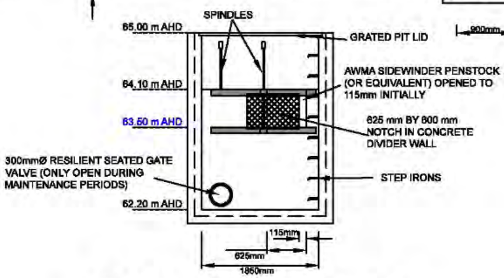
DAM 6 SPILLWAY
SECTION M-M
SCALE: 0 0.25 0.5 1m



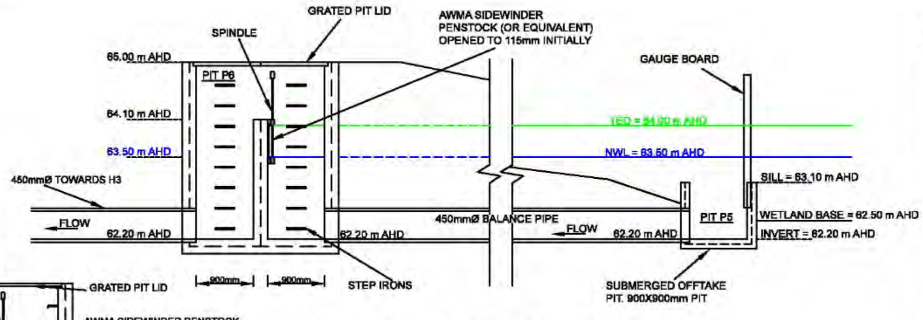
DAM 6 SPILLWAY
SECTION N-N
SCALE: 0 0.25 0.5 1m



DAM 6 WETLAND TWIN CHAMBER OUTFALL PIT
PLAN VIEW
SCALE: 0 0.25 0.5 1m



DAM 6 WETLAND TWIN CHAMBER OUTFALL PIT
SECTION P-P
SCALE: 0 0.25 0.5 1m



DAM 6 WETLAND TWIN CHAMBER OUTFALL PIT
SECTION O-O
SCALE: 0 0.25 0.5 1m

NOTE:
OUTLET PIPE SYSTEM TO BE:
- RUBBER RING JOINTED
- INCORPORATE NO LIFTING HOLES
- INCORPORATE CUT OFF WALLS AS SEEPAGE COLLARS AS REQUIRED.

DRAFT FOR COMMENT	16 OCTOBER 2018
REVISION	DATE

Stormy Water Solutions

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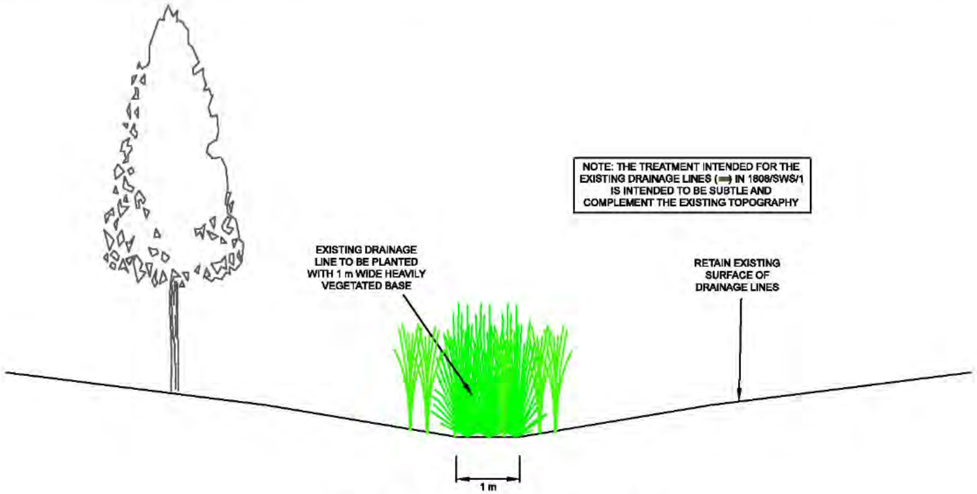
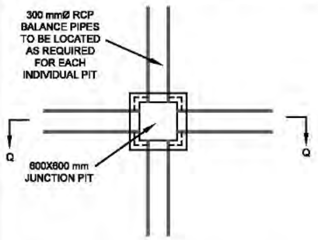
MACARTHUR PARK CEMETERY
WSUD STRATEGY AND STORM WATER MANAGMENT PLAN
DAM 6 OUTLET STRUCTURE DETAILS

SCALE: AS SHOWN
SHEET 9 OF 10
DRAWING No. 1808/SWS/9

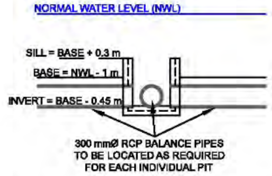


NOTE: THE TREATMENT INTENDED FOR THE EXISTING DRAINAGE LINES (⟹) IN 1808/SWS/1 IS INTENDED TO BE SUBTLE AND COMPLEMENT THE EXISTING TOPOGRAPHY

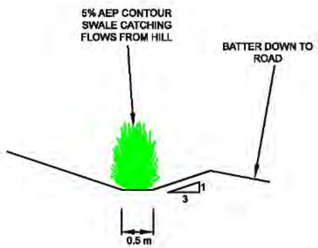
EXISTING DRAINAGE LINES SWALE TREATMENT
TYPICAL SECTION
SCALE: 0 0.25 0.5 1m



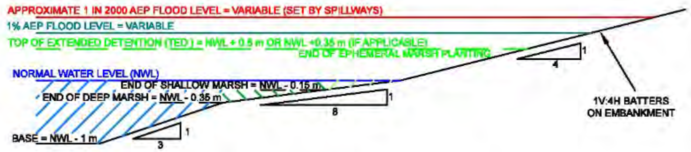
WETLAND BALANCE PIPE SYSTEM
TYPICAL PLAN
SCALE: 0 0.25 0.5 1m



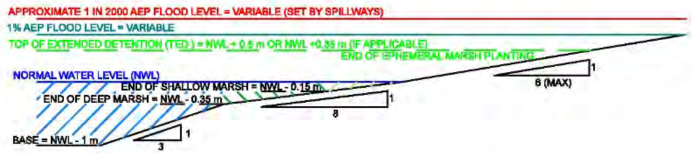
WETLAND BALANCE PIPE SYSTEM
TYPICAL SECTION Q-Q
SCALE: 0 0.25 0.5 1m



5% AEP SWALES DESIGNED BY WS&P
TYPICAL SECTION
SCALE: 0 0.25 0.5 1m



EMBANKMENT WATERBODY EDGE TREAMENT
TYPICAL SECTION
SCALE: 0 0.25 0.5 1m



WATERBODY EDGE TREAMENT (AWAY FROM FORMED EMBANKMENT)
TYPICAL SECTION
SCALE: 0 0.25 0.5 1m

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MACARTHUR PARK CEMETERY
WSUD STRATEGY AND
STORM WATER MANAGMENT PLAN
TYPICAL SECTIONS & DETAILS

SCALE: AS SHOWN
SHEET 10 OF 10
DRAWING No. 1808/SWS/10

Appendix B Hydrological Modelling

The RORB Runoff Routing Program – version 6.32, developed at Monash University by E. M. Laurenson and R. G. Mein, was used to determine the pre and post development design flows originating from the subject site. RORB is a general runoff and stream flow routing program used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to produce rainfall excess and routes this through catchment storage to produce the hydrograph.

Models have been produced to determine flows at locations as defined in Figure B.1 below.

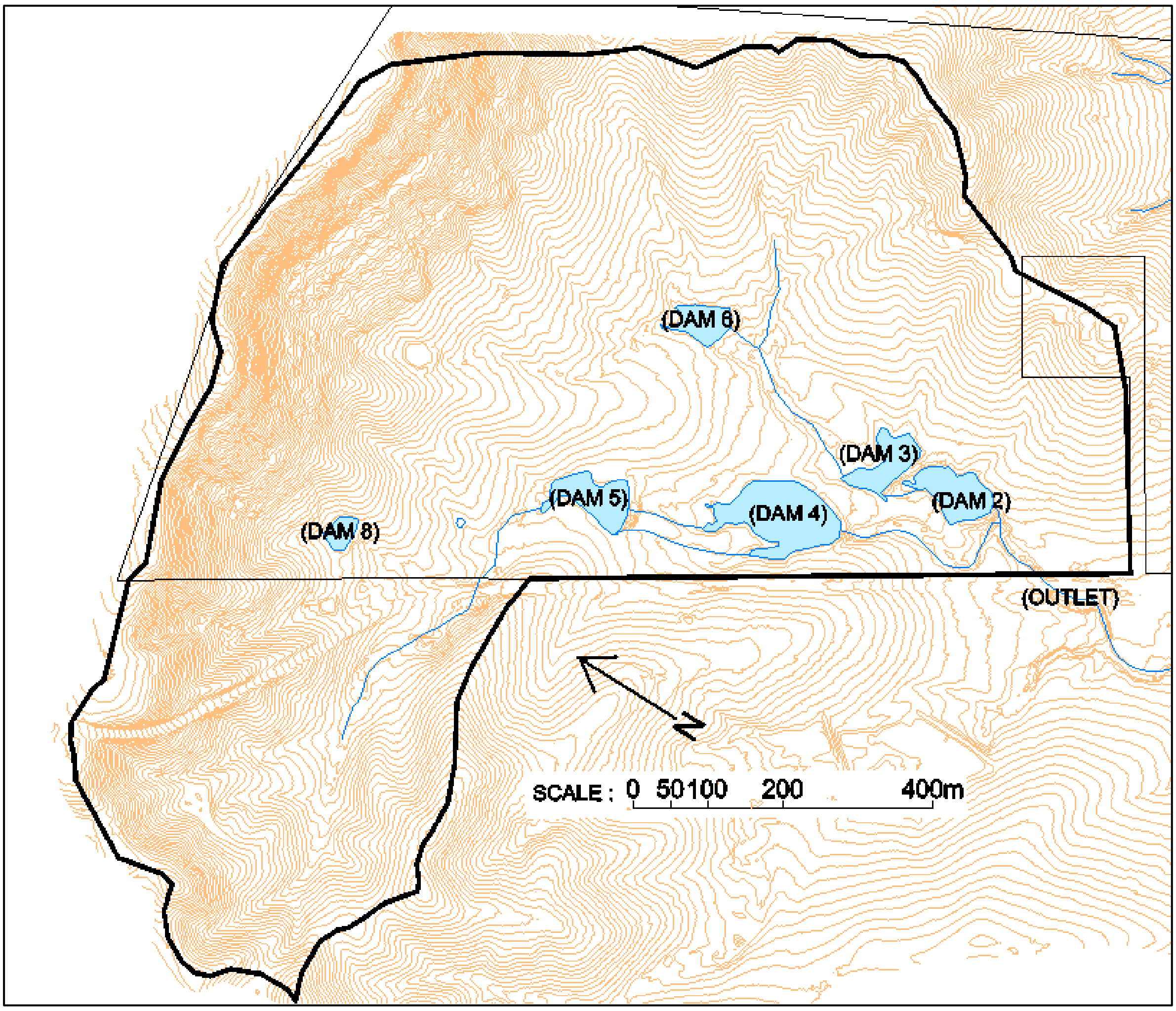

Figure B.1 Key locations of interest

B.1 Pre-development Hydrological Model

B.1.1 Model Description

Figure B.2 details the RORB model for the pre-development conditions model and Tables B.1 and B.2 detail the tabulation of the RORB model setup (i.e. catchment area, fraction imperviousness, reach lengths, etc). This catchment is based on existing site survey provided by the client. The external catchment input locations are as indicated by the WMA Water, August 2017 TUFLOW model.

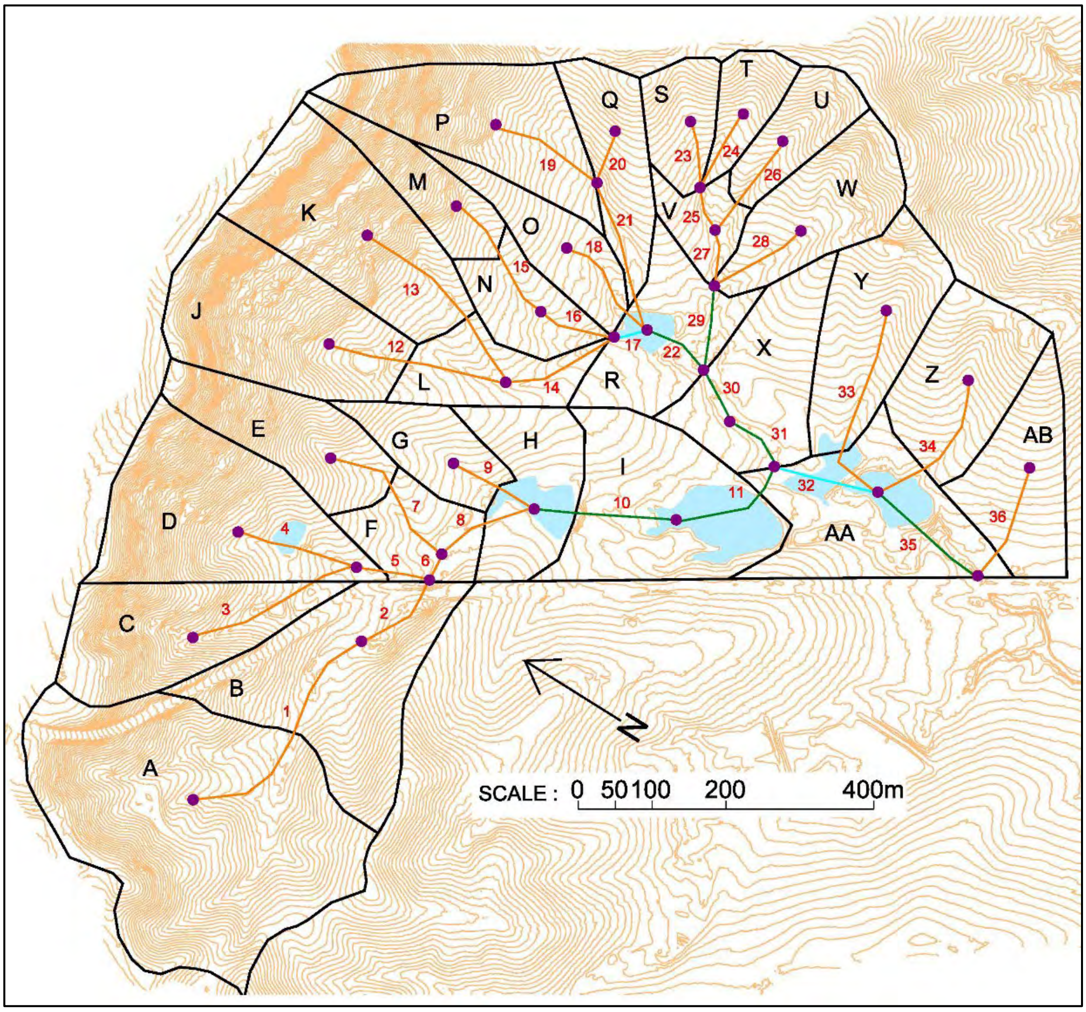

Figure B.2 Pre-Development RORB Model Layout

Table B.1 Pre-development RORB catchment details

Sub Area	Area (ha)	Area (km^2)	Fraction Imperviousness
A	12.31	0.123	0.05
B	6.04	0.060	0.05
C	4.12	0.041	0.05
D	5.92	0.059	0.05
E	3.07	0.031	0.05
F	1.95	0.020	0.05
G	1.84	0.018	0.05
H	2.82	0.028	0.05
I	5.20	0.052	0.05
J	5.21	0.052	0.05
K	5.23	0.052	0.05
L	2.17	0.022	0.05
M	1.88	0.019	0.05
N	1.78	0.018	0.05
O	2.05	0.021	0.05
P	4.32	0.043	0.05
Q	2.35	0.023	0.05
R	3.04	0.030	0.05
S	1.53	0.015	0.05
T	1.22	0.012	0.05
U	1.70	0.017	0.05
V	1.13	0.011	0.05
W	3.18	0.032	0.05
X	3.66	0.037	0.05
Y	3.70	0.037	0.05
Z	3.53	0.035	0.05
AA	4.69	0.047	0.05
AB	3.02	0.030	0.05
TOTAL	**98.6**	**0.986**	**0.05**

Table B.2 Pre-development RORB reach details

Reach	Length (km)	Slope %	Reach Type (Pre)
1	0.336	4.5%	EX/UNLINED
2	0.128	2.3%	EX/UNLINED
3	0.246	6.1%	EX/UNLINED
4	0.168	7.7%	EX/UNLINED
5	0.103	3.9%	EX/UNLINED
6	0.038	2.6%	EX/UNLINED
7	0.209	5.7%	EX/UNLINED
8	0.141	2.5%	EX/UNLINED
9	0.125	3.6%	EX/UNLINED
10	0.194		NATURAL
11	0.169		NATURAL
12	0.248	5.6%	EX/UNLINED
13	0.276	10.7%	EX/UNLINED
14	0.158	3.8%	EX/UNLINED
15	0.188	8.2%	EX/UNLINED
16	0.109	5.5%	EX/UNLINED
17	0.046		DROWNED
18	0.161	5.3%	EX/UNLINED
19	0.155	10.0%	EX/UNLINED
20	0.074	12.2%	EX/UNLINED
21	0.211	3.6%	EX/UNLINED
22	0.094		NATURAL
23	0.096	9.4%	EX/UNLINED
24	0.117	7.7%	EX/UNLINED
25	0.061	5.7%	EX/UNLINED
26	0.143	7.3%	EX/UNLINED
27	0.075	2.7%	EX/UNLINED
28	0.136	4.0%	EX/UNLINED
29	0.120		NATURAL
30	0.081		NATURAL
31	0.091		NATURAL
32	0.148		DROWNED
33	0.284	4.6%	EX/UNLINED
34	0.214	4.7%	EX/UNLINED
35	0.174		NATURAL
36	0.165	6.1%	EX/UNLINED

Note: "Excavated /unlined" defines the relatively clear earthen drainage lines on site. "Natural" defines more densely vegetated existing watercourses on site

B.1.2 Model Parameters

RORB is based on the following equation relating storage (S) and discharge (Q) of a watercourse:

$S = k \times Q^m$ where $k = K_c \times K_r$

The values of K_c and m are parameters that can be obtained by calibration of the model using corresponding sets of data on rainfall for selected historical flows. If historical flows are unknown, values can be estimated from regional analysis or by values suggested by Australian Rainfall & Runoff (AR&R). The value of k_r is a physical parameter related to the reach type chosen by the modeller which is automatically calculated by RORB.

In this case, flow gauging information was not available. However, a regional parameter set (recommended by AR&R 2016) is applicable. The K_c parameter used is as detailed in ARR 2016, Book 7, Chapter 6, Equation 7.6.9 for New South Wales catchments in the Newcastle-Sydney-Wollongong region.

$K_c = 1.09 \times A^{0.45} = 1.08$

$m = 0.8$

Other parameters of RORB are the initial loss (IL) and the continuing loss (CL). IL is the amount of rainfall needed before runoff occurs. As the current catchment is largely pervious, the use of a CL rather than a pervious area runoff coefficient is appropriate. IL and CL values have been obtained from the ARR 2016 datahub for the location 34.0011221 S, 150.822312 E as shown below:

$IL = 37\ mm,$

$CL = 2.3\ mm/hr,$

Australian Rainfall and Runoff (ARR) 2016 Data hub (Lat: 34.001121 S, Lon: 150.822312 E, accessed: 26th September 2018) rainfall depths, rainfall temporal patterns (for 24 durations ranging from 10-minutes to 168-hours) and areal reduction factors have been used in the model.

It should be noted, unlike pervious area runoff coefficients, CL values are independent of AEP and <u>should not</u> be varied with AEP (ARR 2016, Book 5, Chapter 3.7.1). As such, the parameters quoted above apply for all AEP with only the rainfall depths and temporal patterns changing with AEP.

B.1.3 Model Verification

It is required to check the estimated flows against other flow calculation methods to ensure the RORB model developed is valid for application. To achieve this check design flows are compared against other flow computational methods.

GRC Hydro Pty Ltd and WMA have formulated a combined 1D/2D hydrologic model with the WBNM model and the TUFLOW model that incorporates "Rain on Grid" to model the existing conditions. This modelling has been used to verify the 1% AEP RORB flows obtained at:

- Dam 5 (RORB) = 6.9 m^3/s
- Dam 6 (RORB) = 5.2 m^3/s

It should be noted that the WMA modelling used to verify has been conducted using ARR1987 practices.

The ARR1987 1% AEP flows from the WMA model are:

- Dam 5 (WMA model) = 5.1 m^3/s
- Dam 6 (WMA Model) = 5.8 m^3/s

The results from both modelling practices are comparable and as such the RORB modelling is appropriate for use.

B.1.4 Model Results

ARR 2016 recommends the use of ensemble simulation. 10 temporal patterns have been simulated for each duration and AEP. As recommended in ARR 2016, it is appropriate to take the peak flow from the hydrograph associated with the temporal pattern that is closest to the average flow produced from all ten hydrographs in the ensemble simulation. Appendix F details the required ARR 2016 methodology. Figure B.3 shows the results from all 240 simulations at the catchment outlet for the 1% AEP rainfall events.

The 1, 2, 5 and 10% AEP pre-development design flows are reported in Table B.3. The flows reported along with the temporal pattern number as obtained from the ARR 2016 data hub which results in the hydrograph closest to the peak average.

Table B.3 Catchment Outlet, Pre-development Results

Annual Exceedance Probability	Peak Average		Appropriate Design Hydrograph		
	Q (m^3/s)	Critical Duration	Q (m^3/s)	Critical Duration	Temporal Pattern ID
1% AEP	11.1	2-hour	11.4	2-hour	26
2% AEP	8.9	2-hour	8.7	2-hour	25
5% AEP	6.5	2-hour	6.3	2-hour	12
10% AEP	4.6	4.5-hour	4.6	4.5 hour	15

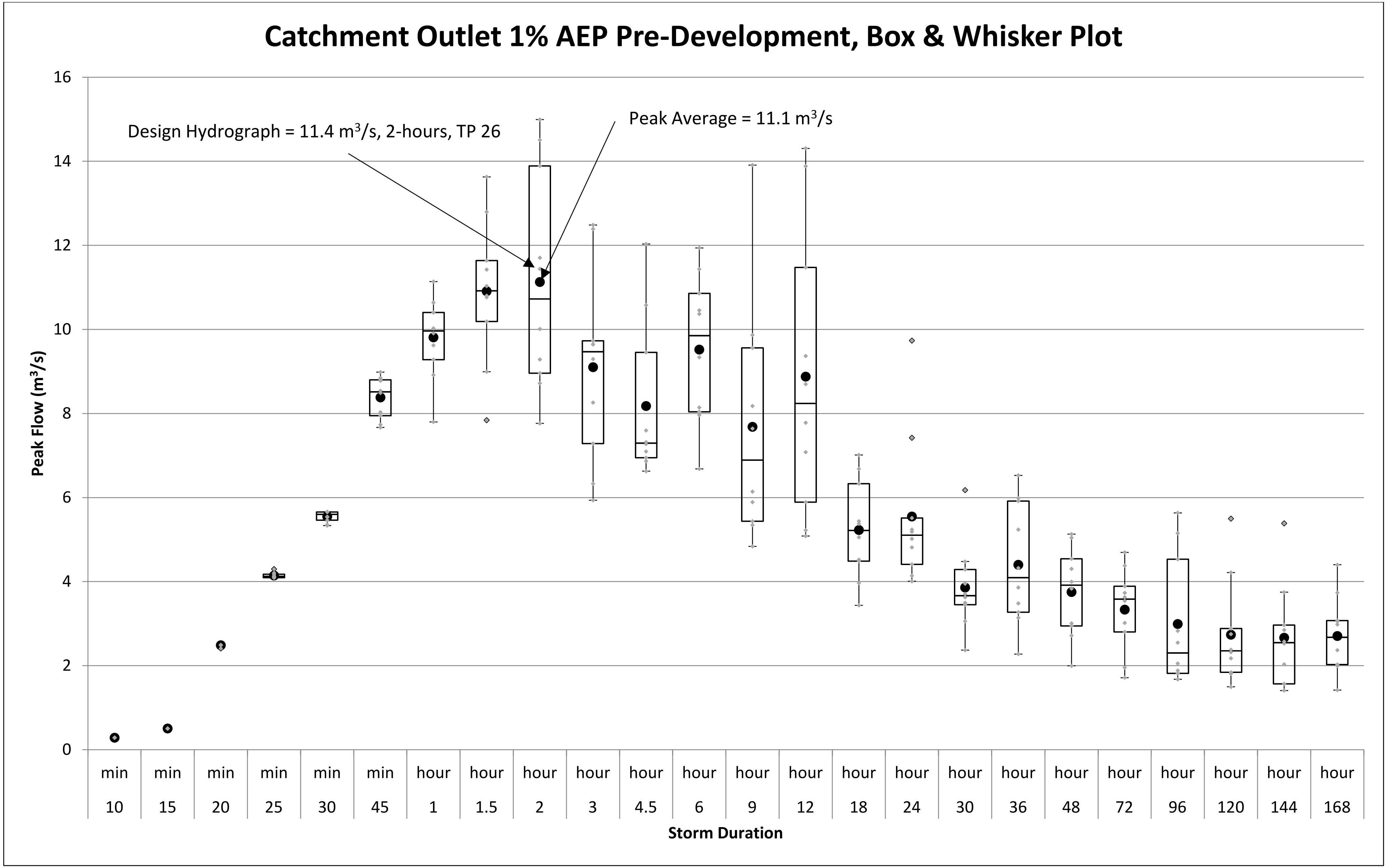

Figure B.3 Results from all 240 1% AEP RORB Post-Development simulations at the catchment outlet. *Note: See Appendix E for details on Box and Whisker plots*

B.2 Post-Develepment Hydrologic Modelling

B.2.1 Model Description

Figure B.4 details the RORB model for the pre-development conditions model and Tables B.4 and B.5 detail the tabulation of the RORB model setup (i.e. catchment area, fraction imperviousness, reach lengths, etc). This catchment is based on the proposed site development layout provided by the client. The external catchment input locations are as indicated by the WMA Water, August 2017 TUFLOW model.

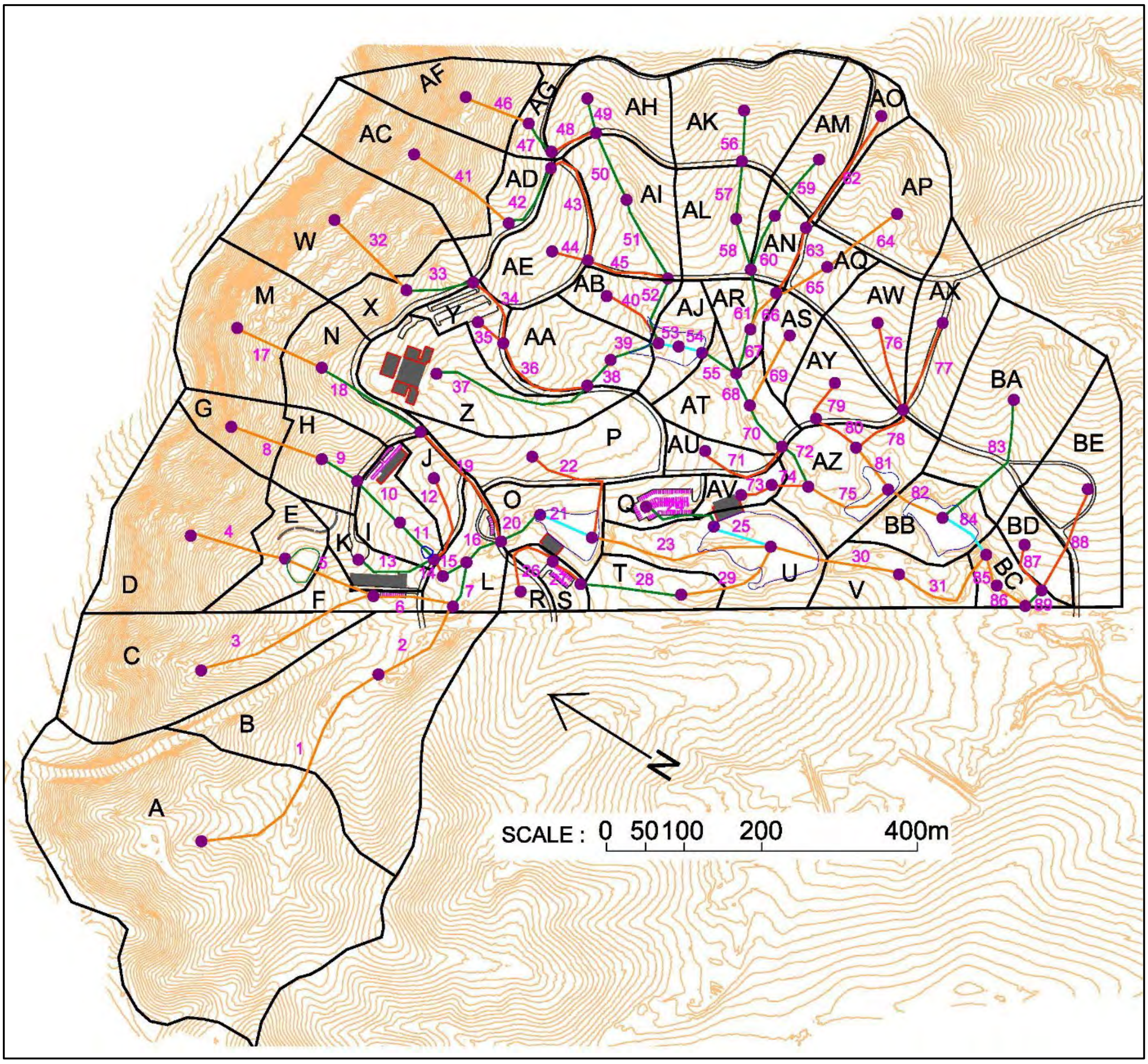

Figure B.4 Post-Development RORB Model Layout

Table B.4 Post-development RORB catchment details

Sub Area	Area (ha)	Area (km^2)	Fraction Imperviousness
A	12.31	0.123	0.05
B	6.04	0.060	0.05
C	4.12	0.041	0.05
D	4.03	0.040	0.00
E	1.21	0.012	0.25
F	0.69	0.007	0.10
G	0.94	0.009	0.00
H	1.29	0.013	0.25
I	1.07	0.011	0.22
J	0.74	0.007	0.10
K	0.74	0.007	0.42
L	1.14	0.011	0.10
M	2.62	0.026	0.00
N	1.59	0.016	0.10
O	1.29	0.013	0.20
P	2.22	0.022	0.25
Q	0.74	0.007	0.47
R	0.34	0.003	0.10
S	0.43	0.004	0.80
T	0.98	0.010	0.10
U	2.42	0.024	0.20
V	1.42	0.014	0.10
W	3.47	0.035	0.00
X	1.00	0.010	0.15
Y	0.82	0.008	0.80
Z	2.62	0.026	0.33
AA	1.69	0.017	0.25
AB	0.82	0.008	0.25
AC	2.47	0.025	0.00
AD	0.81	0.008	0.25
AE	1.56	0.016	0.25
AF	1.75	0.017	0.00
AG	0.47	0.005	0.25
AH	1.58	0.016	0.25
AI	1.98	0.020	0.25
AJ	0.88	0.009	0.25
AK	2.56	0.026	0.10
AL	1.52	0.015	0.10
AM	1.23	0.012	0.25
AN	0.64	0.006	0.10
AO	0.47	0.005	0.25
AP	2.24	0.022	0.15
AQ	0.76	0.008	0.25
AR	0.71	0.007	0.10
AS	0.65	0.007	0.25
AT	1.54	0.015	0.10
AU	0.62	0.006	0.25
AV	0.45	0.004	0.25
AW	1.29	0.013	0.25
AX	1.87	0.019	0.25
AY	1.19	0.012	0.25
AZ	2.24	0.022	0.15
BA	3.43	0.034	0.25
BB	1.30	0.013	0.10
BC	0.66	0.007	0.10
BD	0.72	0.007	0.25
BE	2.41	0.024	0.25
TOTAL	**98.8**	**0.988**	**0.14**

Table B.5 Post-development RORB reach details

Reach	Length (km)	Slope %	Reach Type (Post)
1	0.336	4.5%	EX/UNLINED
2	0.128	2.3%	EX/UNLINED
3	0.246	6.1%	EX/UNLINED
4	0.103	22.3%	EX/UNLINED
5	0.120	6.3%	EX/UNLINED
6	0.106	3.3%	EX/UNLINED
7	0.066		NATURAL
8	0.123	24.4%	EX/UNLINED
9	0.053		NATURAL
10	0.078		NATURAL
11	0.064		NATURAL
12	0.117	3.8%	PIPED
13	0.097		NATURAL
14	0.021	7.1%	PIPED
15	0.035		NATURAL
16	0.051		NATURAL
17	0.120	15.8%	EX/UNLINED
18	0.152		NATURAL
19	0.181	6.9%	PIPED
20	0.058		NATURAL
21	0.075		DROWNED
22	0.171	3.5%	PIPED
23	0.233	2.1%	EX/UNLINED
24	0.103		NATURAL
25	0.081		DROWNED
26	0.112	2.7%	PIPED
27	0.048	1.0%	PIPED
28	0.128		NATURAL
29	0.147	0.7%	EX/UNLINED
30	0.390	1.2%	EX/UNLINED
31	0.152	2.6%	EX/UNLINED
32	0.127	19.7%	EX/UNLINED
33	0.081		NATURAL
34	0.097	7.2%	PIPED
35	0.045	5.6%	PIPED
36	0.138	5.4%	PIPED
37	0.211		NATURAL
38	0.044		NATURAL
39	0.066		NATURAL
40	0.093	6.5%	PIPED
41	0.146	22.6%	EX/UNLINED

Reach	Length (km)	Slope %	Reach Type (Post)
42	0.095		NATURAL
43	0.154	4.9%	PIPED
44	0.051	7.8%	PIPED
45	0.109	5.5%	PIPED
46	0.083	24.1%	EX/UNLINED
47	0.050		NATURAL
48	0.061	6.6%	PIPED
49	0.045		NATURAL
50	0.097		NATURAL
51	0.116		NATURAL
52	0.092		NATURAL
53	0.030		DROWNED
54	0.033		DROWNED
55	0.042		NATURAL
56	0.066		NATURAL
57	0.084		NATURAL
58	0.066		NATURAL
59	0.178		NATURAL
60	0.076		NATURAL
61	0.080		NATURAL
62	0.174	10.1%	PIPED
63	0.093	7.0%	PIPED
64	0.113	8.0%	EX/UNLINED
65	0.076	5.3%	EX/UNLINED
66	0.061	4.1%	EX/UNLINED
67	0.057		NATURAL
68	0.040		NATURAL
69	0.105	4.8%	EX/UNLINED
70	0.067		NATURAL
71	0.123	3.7%	PIPED
72	0.063		NATURAL
73	0.042	2.4%	PIPED
74	0.046	5.4%	PIPED
75	0.117	0.4%	EX/UNLINED
76	0.122	6.6%	PIPED
77	0.126	6.7%	PIPED
78	0.083	3.0%	PIPED
79	0.056	6.3%	PIPED
80	0.057	3.5%	PIPED
81	0.068	1.5%	EX/UNLINED
82	0.081	1.9%	EX/UNLINED

Reach	Length (km)	Slope %	Reach Type (Post)
83	0.186		NATURAL
84	0.076		DROWNED
85	0.041	2.4%	EX/UNLINED
86	0.046	1.1%	EX/UNLINED
87	0.069	2.9%	PIPED
88	0.145	5.9%	PIPED
89	0.029		NATURAL

Note: "Excavated /Unlined" defines the relatively clear earthen drainage lines on site. "Natural" defines revegetated drainage lines with dense sedges and rushes over a 1 metre base width.

B.2.2 Model Parameters

The same RORB model parameters as detailed in Appendix B.1.2 have been used.

CL loss values have been used for the post-development scenario has the catchment is still largely pervious in the post-development scenario.

B.2.3 Model Retarding Basins

RORB allows the modelling of retarding basins by entering a stage/storage/discharge (SSD) relationship representative of the proposed retarding basins. Two retarding basins,

- In the void space above the Dam 4 normal water level, and
- In the void space above the Dam 2 normal water level,

At this conceptual stage of the project, the SSD's modelled are simplified. The SSD's represent simply:

- No outflow at normal water level (NWL), and
- Outflow above NWL controlled by the spillway's shown in Appendix A and C.

The spillway lengths determined have been sized iteratively so that at both at the catchment outlet, for each of the 1, 2, 5 and 10% AEP storm events, the post-development flow is less than the pre-development flow.

The relationships modelled are detailed in Tables B.6 and B.7.

Table B.6 Conceptual SSD for Dam 4 (5 m long spillway)

Level (m AHD)	Storage (m^3)	Q(m^3/s)
58.50	0	0.0
58.75	2929	1.3
59.00	5858	3.5
59.25	9226	6.5
59.40	11248	8.5
59.50	12595	10.0
59.75	16458	14.0
60.00	20320	18.4

Table B.7 Conceptual SSD for Dam 2 (4 m long spillway)

Level (m AHD)	Storage (m^3)	Q(m^3/s)
53.50	0	0.0
53.75	1061	1.0
54.00	2121	2.8
54.25	3409	5.2
54.50	4696	8.0
54.75	6239	11.2
55.00	7781	14.7

B.2.4 Model Results

ARR 2016 recommends the use of ensemble simulation. 10 temporal patterns have been simulated for each duration and AEP. As recommended in ARR 2016, it is appropriate to take the peak flow from the hydrograph associated with the temporal pattern that is closest to the average flow produced from all ten hydrographs in the ensemble simulation. Appendix E details the required ARR 2016 methodology. Figure B.5 shows the results from all 240 simulations at the catchment outlet for the 1% AEP rainfall events.

The 1, 2, 5 and 10% AEP post-development design flows are reported in Tables B.8, B.9 and B.10. The flows reported along with the temporal pattern number as obtained from the ARR 2016 data hub which results in the hydrograph closest to the peak average. Table B.8 also reports the post-development flows alongside the pre-development flows calculated in Appendix B.1 for ease of comparison of pre and post-development flows at the catchment outlet.

The analysis shows that the proposed retarding basin provisions for Dam 4 and 2 can reduce the peak average post-development flows to less than the peak average pre-development flows for the 1, 2, 5 and 10% AEP events.

The 1 in 2000 (0.05%) AEP event inflows into these storages has also been produced to ensure that the spillways have at lease 1 in 2000 AEP capacity (before flow over the embankment occurs).

Tables B.11, B.12 and B.13 have also been produced which detail the expected inflows into each of Dam 3, Dam 5 and Dam 6. These flows have been produced so that the wetland overflow spillways for each of these dams could be adequately designed as shown in Appendix C.

Table B.8 Results at the Catchment Outlet

Annual Exceedance Probability	Pre-Development					Post-Development				
	Peak Average		Appropriate Design Hydrograph			Peak Average		Appropriate Design Hydrograph		
	Q (m^3/s)	Critical Duration	Q (m^3/s)	Critical Duration	Temporal Pattern ID	Q (m^3/s)	Critical Duration	Q (m^3/s)	Critical Duration	Temporal Pattern ID
1% AEP	11.1	2-hour	11.4	2-hour	26	9.4	2-hour	9.8	2-hour	26
2% AEP	8.9	2-hour	8.7	2-hour	25	7.6	2-hour	7.8	2-hour	26
5% AEP	6.5	2-hour	6.3	2-hour	12	5.4	2-hour	5.6	2-hour	17
10% AEP	4.6	4.5-hour	4.6	4.5 hour	15	4.1	4.5-hour	3.9	4.5-hour	16

Table B.9 Storage Modelling Results at Dam 4

Annual Exceedance Probability	Inflow into the Storage					Outflow from the Storage						
	Peak Average		Appropriate Design Hydrograph			Peak Average		Appropriate Design Hydrograph				
	Q (m^3/s)	Critical Duration	Q (m^3/s)	Critical Duration	Temporal Pattern ID	Q (m^3/s)	Critical Duration	Q (m^3/s)	Critical Duration	Temporal Pattern ID	Water Level (m AHD)	Storage (m^3)
0.05% AEP	17.4	45-minute	17.4	45-minute	28	N/A		N/A				
1% AEP	6.9	1-hour	7.1	1-hour	27	4.4	2-hour	4.4	2-hour	26	59.10	6,890
2% AEP	5.5	1.5-hour	5.3	1.5-hour	28	3.5	2-hour	3.5	2-hour	25	59.00	5,840
5% AEP	4.4	2-hour	4.3	2-hour	17	2.5	2-hour	2.6	2-hour	17	58.90	4,660
10% AEP	3.2	2-hour	3.2	2-hour	17	1.8	6-hour	1.8	6-hour	18	58.80	3,590

Table B.10 Storage Modelling Results at Dam 2

Annual Exceedance Probability	Inflow into the Storage					Outflow from the Storage						
	Peak Average		Appropriate Design Hydrograph			Peak Average		Appropriate Design Hydrograph				
	Q (m^3/s)	Critical Duration	Q (m^3/s)	Critical Duration	Temporal Pattern ID	Q (m^3/s)	Critical Duration	Q (m^3/s)	Critical Duration	Temporal Pattern ID	Water Level (m AHD)	Storage (m^3)
0.05% AEP	13.6	45-minute	13.4	45-minute	22	N/A		N/A				
1% AEP	5.7	2-hour	6.0	2-hour	29	5.2	2-hour	5.4	2-hour	26	54.30	3,490
2% AEP	4.7	2-hour	4.7	2-hour	26	4.2	2-hour	4.1	2-hour	26	54.15	2,820
5% AEP	3.6	2-hour	3.6	2-hour	17	3.1	2-hour	3.3	2-hour	17	54.05	2,380
10% AEP	2.6	2-hour	2.6	2-hour	17	2.2	2-hour	2.1	2-hour	12	53.90	1,720

Table B.11 Inflows into Dam 3

Annual Exceedance Probability	Post Development Inflow into Dam 3				
	Peak Average		Appropriate Design Hydrograph		
	Q (m^3/s)	Critical Duration	Q (m^3/s)	Critical Duration	Temporal Pattern ID
0.05% AEP	12.2	45-minute	12.3	45-minute	28
1% AEP	5.2	2-hour	5.3	2-hour	29
2% AEP	4.2	2-hour	4.2	2-hour	26
5% AEP	3.2	2-hour	3.2	2-hour	17
10% AEP	2.3	2-hour	2.4	2-hour	17

Table B.12 Inflows into Dam 5

Annual Exceedance Probability	Post Development Inflow into Dam 5				
	Peak Average		Appropriate Design Hydrograph		
	Q (m^3/s)	Critical Duration	Q (m^3/s)	Critical Duration	Temporal Pattern ID
0.05% AEP	17.1	30-minute	17.2	30-minute	25
1% AEP	6.8	1-hour	6.6	1-hour	22
2% AEP	5.4	1-hour	5.4	1-hour	27
5% AEP	4.2	2-hour	4.2	2-hour	17
10% AEP	3.1	2-hour	3.2	2-hour	15

Table B.13 Inflows into Dam 6

Annual Exceedance Probability	Post Development Inflow into Dam 6				
	Peak Average		Appropriate Design Hydrograph		
	Q (m^3/s)	Critical Duration	Q (m^3/s)	Critical Duration	Temporal Pattern ID
0.05% AEP	7.7	45-minute	8.1	45-minute	26
1% AEP	3.1	1-hour	3.1	1-hour	22
2% AEP	2.5	2-hour	2.6	2-hour	29
5% AEP	2.0	2-hour	2.0	2-hour	17
10% AEP	1.4	2-hour	1.4	2-hour	17

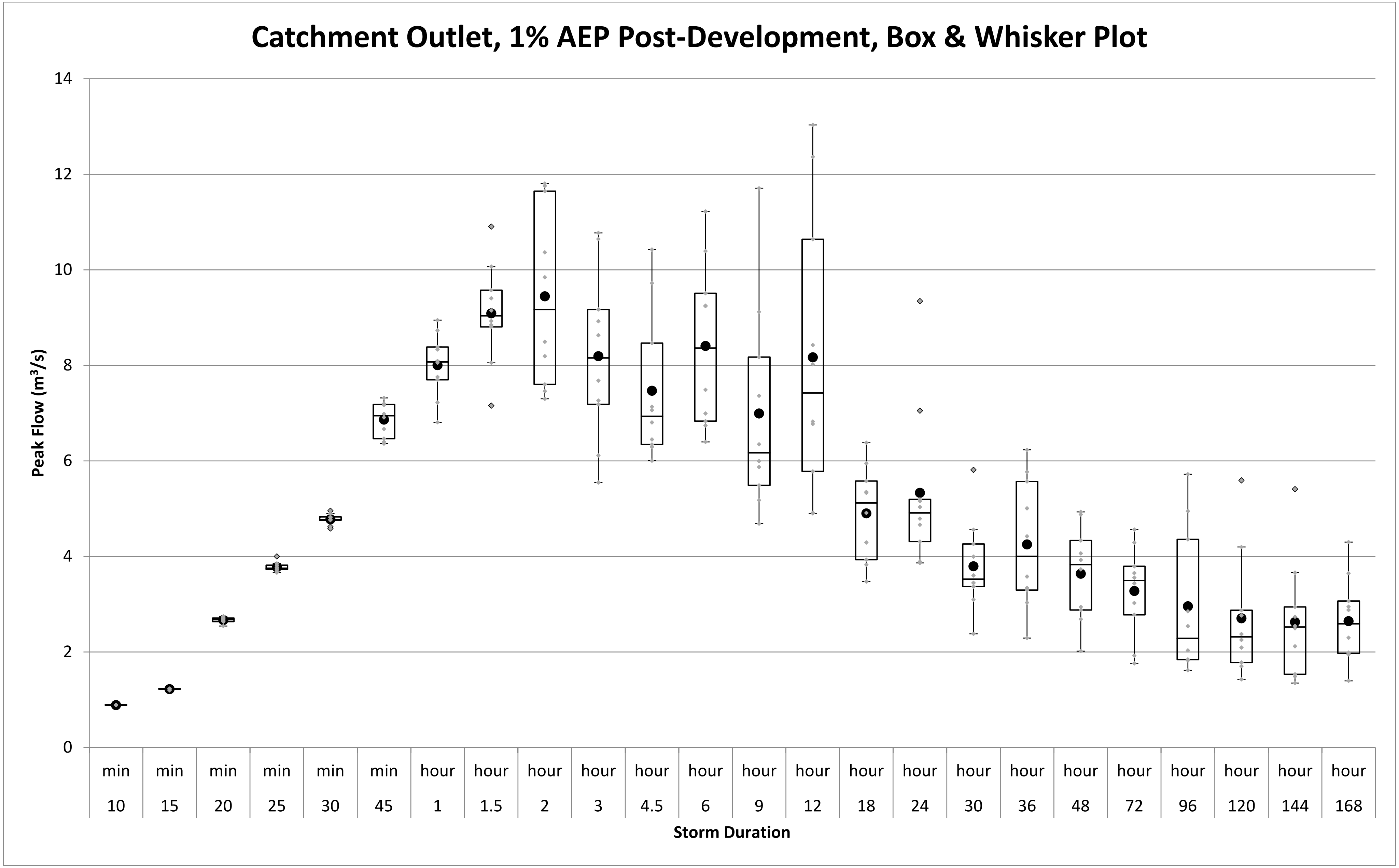

Figure B.5 **Results from all 240 1% AEP RORB Post-Development simulations at the catchment outlet.** ***Note: See Appendix E for details on Box and Whisker plots***

Appendix C Structural Calculations

C.1 Wetland Outlet Structures

C.1.1 Dam 3

ED Control							
$Q = B \times C \times L_e \times h^{1.5}$							
where							
Q = flow rate (m^3/s)							
h = head on the weir (m)							
B = blockage factor (assume no blockage as in manual)							
C = weir coefficient =	1.74	sharp crested weir					
L = Actual Weir Length =	0.112	m					
Area at NWL =	2240	m^2					
Area at TED =	3035	m^2					
Volume of water stored for treatment over Ed range						0.5	m
				.=		1319	m^3
L_e = effective length = L - 0.2h, where L = Actual Weir Length							
WL (m AHD)	h (m)	Le (m)	Q (m^3/s)	ED Volume (m^3)	ED Detention Time (hrs)		
55.00	0.00	0.112	0.000	0			
55.50	0.50	0.012	0.007	1319	50		

ED Balance Sizing			
head loss = $(Ke+Kex)\times V^2/2g + S_f \times L$			
$S_f = Q^2n^2/A^2R^{4/3}$			
pipe dia =		0.450	m
RCP pipe radius =		0.225	m
Design flow =		0.007	m^3/s
Wetted perimeter =		1.41	m
Area =		0.16	m^2
Hyd radius =		0.1125	m
V =		0.05	m/s
Ke =		0.5	
Kex =		1	
n =		0.013	
L=		35	
S_f =		0.0000	
Head loss =		0.0004	m

C.1.2 Dam 5

ED Control							
Q = B × C × L_e × $h^{1.5}$							
where							
Q = flow rate (m^3/s)							
h = head on the weir (m)							
B = blockage factor (assume no blockage as in manual)							
C = weir coefficient =	1.74	sharp crested weir					
L = Actual Weir Length =	0.10	m					
Area at NWL =	4850	m^2					
Area at TED =	5630	m^2					
Volume of water stored for treatment over Ed range						0.35	m
				.=		1834	m^3
L_e = effective length = L - 0.2h, where L = Actual Weir Length							
WL (m AHD)	h (m)	Le (m)	Q (m^3/s)	ED Volume (m^3)	ED Detention Time (hrs)		
64.00	0.00	0.1	0.000	0			
64.35	0.35	0.03	0.011	1834	47		

ED Balance Sizing		
head loss = (Ke+Kex)×V^2/2g+ S_f×L		
S_f = $Q^2n^2/A^2R^{4/3}$		
pipe dia =	0.450	m
RCP pipe radius =	0.225	m
Design flow =	0.011	m^3/s
Wetted perimeter =	1.41	m
Area =	0.16	m^2
Hyd radius =	0.1125	m
V =	0.07	m/s
Ke =	0.5	
Kex =	1	
n =	0.013	
L=	35	
S_f =	0.0000	
Head loss =	0.0009	m

C.1.3 Dam 6

ED Control							
$Q = B \times C \times L_e \times h^{1.5}$							
where							
Q = flow rate (m^3/s)							
h = head on the weir (m)							
B = blockage factor (assume no blockage as in manual)							
C = weir coefficient =	1.74	sharp crested weir					
L = Actual Weir Length =	0.115	m					
Area at NWL =	2820	m^2					
Area at TED =	3670	m^2					
Volume of water stored for treatment over Ed range						0.5	m
				.=		1623	m^3
L_e = effective length = L - 0.2h, where L = Actual Weir Length							
WL (m AHD)	h (m)	Le (m)	Q (m^3/s)	ED Volume (m^3)	ED Detention Time (hrs)		
63.50	0.00	0.115	0.000	0			
64.00	0.50	0.015	0.009	1623	49		

ED Balance Sizing			
head loss = $(Ke+Kex) \times V^2/2g + S_f \times L$			
$S_f = Q^2n^2/A^2R^{4/3}$			
pipe dia =		0.450	m
RCP pipe radius =		0.225	m
Design flow =		0.009	m^3/s
Wetted perimeter =		1.41	m
Area =		0.16	m^2
Hyd radius =		0.1125	m
V =		0.06	m/s
Ke =		0.5	
Kex =		1	
n =		0.013	
L=		30	
S_f =		0.0000	
Head loss =		0.0006	m

C.2 Dam Spillway Sizing's

C.2.1 Dam 2 Spillway

Dam 2			
NWL =	53.50	m AHD	
TED =	N/A	m AHD	
Weir Overflow			
$Q = (1\text{-}B) \times C \times L \times h^{1.5}$			
where			
Q = flow rate (m^3/s)			
h = head on the weir (m)			
B = 0			
C = weir coefficient =	2	broad crest slope	
L = Actual Weir Length =	4	m	
B = blockage factor	0		
WL (m AHD)	h (m)	Q (m^3/s)	
53.50	0.00	0.0	
53.75	0.25	1.0	
54.00	0.50	2.8	
54.25	0.75	5.2	
54.50	1.00	8.0	
54.75	1.25	11.2	
55.00	1.50	14.7	
55.25	1.75	18.5	
Weir Equation For Level			
$WL = Crest + (Q/[(1\text{-}B) \times C \times L])^{2/3}$			
Weir Crest =	53.50	m AHD	
$Q_{1in2000AEP}$ =	13.6	m^3/s	
Thus			
h =	1.42	m	
and 1in2000 AEP WL in Dam =	54.92	m AHD	
Embankment Crest =	55.00	m AHD	
1 in 2000 AEP containted?	YES		

C.2.2 Dam 3 Spillway

Dam 3		
NWL =	55.00	m AHD
TED =	55.50	m AHD
1% AEP Flow =	5.2	m^3/s
Allowable head for spillway =	0.50	m
Weir Equation For Length		
$L = Q/[(1-B) \times C \times h^{1.5}]$		
Q =	5.2	m^3/s
B =	0	assumed
C =	2	Broad Crest, slope approach
Thus		
Weir Length =	7.4	m
Weir Equation For Level		
$WL = Crest + (Q/[(1-B) \times C \times L])^{2/3}$		
Weir Crest =	55.50	m AHD
$Q_{1in2000AEP}$ =	12.2	m^3/s
Thus		
h =	0.88	m
and 1in2000 AEP WL in Dam =	56.38	m AHD
Embankment Crest =	56.50	m AHD
1 in 2000 AEP containted?	YES	

As such, 7.4 m spillway is required to produce a 1% AEP flood level of 56.0 m AHD (0.5 m head over spillway crest = TED) and a 1 in 2000 AEP flood level less than the embankment level of 56.5 m AHD.

C.2.3 Dam 4 Spillway

Dam 4			
NWL =	58.50	m AHD	
TED =	N/A	m AHD	
Required Floor Level =	59.90	m AHD	
Freeboard Required =	0.50	m	
Required 1% AEP Level =	59.40	m AHD	
Weir Overflow			
$Q = (1\text{-}B) \times C \times L \times h^{1.5}$			
where			
Q = flow rate (m^3/s)			
h = head on the weir (m)			
B = 0			
C = weir coefficient =	2	broad crest slope	
L = Actual Weir Length =	5	m	
B = blockage factor	0		
WL (m AHD)	h (m)	Q (m^3/s)	
58.50	0.00	0.0	
58.75	0.25	1.3	
59.00	0.50	3.5	
59.25	0.75	6.5	
59.40	0.90	8.5	
59.50	1.00	10.0	
59.75	1.25	14.0	
60.00	1.50	18.4	
Weir Equation For Level			
$WL = Crest + (Q/[(1\text{-}B) \times C \times L])^{2/3}$			
Weir Crest =	58.50	m AHD	
$Q_{1in2000AEP}$ =	17.4	m^3/s	
Thus			
h =	1.45	m	
and 1in2000 AEP WL in Dam =	59.95	m AHD	
Embankment Crest =	60.00	m AHD	
1 in 2000 AEP containted?	YES		

RORB confirms a 5 m spillway produces a 1% flood level of 59.1 m AHD < 59.4 m AHD. This results in an adequate flood level for the adjacent building flood level as specified in the DA. In addition, the 1 in 2000 AEP flood level less than the embankment level of 60.0 m AHD.

C.2.4 Dam 5 Spillway

Dam 5		
NWL =	64.00	m AHD
TED =	64.35	m AHD
1% AEP Flow =	6.8	m^3/s
Required Floor Level =	65.3	m AHD
Freeboard Required =	0.5	m
Required 1% AEP Level =	64.8	m AHD
Allowable head for spillway =	0.45	m
Weir Equation For Length		
$L = Q/[(1-B) \times C \times h^{1.5}]$		
Q =	6.8	m^3/s
B =	0	assumed
C =	2	Broad Crest, slope approach
Thus		
Weir Length =	11.3	m
Weir Equation For Level		
$WL = Crest + (Q/[(1-B) \times C \times L])^{2/3}$		
Weir Crest =	64.35	m AHD
$Q_{1in2000AEP}$ =	17.1	m^3/s
Thus		
h =	0.83	m
and 1in2000 AEP WL in Dam =	65.18	m AHD
Embankment Crest =	65.20	m AHD
1 in 2000 AEP containted?	YES	

RORB confirms an 11.3 m spillway produces a 1% flood level of 64.8 m AHD. This results in an adequate flood level for the adjacent building flood level as specified in the DA. In addition, the 1 in 2000 AEP flood level less than the embankment level of 65.2 m AHD.

C.2.5 Dam 6 Spillway

Dam 6		
NWL =	63.50	m AHD
TED =	64.00	m AHD
1% AEP Flow =	3.1	m^3/s
Allowable head for spillway =	0.50	m
Weir Equation For Length		
$L = Q/[(1-B) \times C \times h^{1.5}]$		
Q =	3.1	m^3/s
B =	0	assumed
C =	2	Broad Crest, slope approach
Thus		
Weir Length =	4.4	m
Weir Equation For Level		
$WL = Crest + (Q/[(1-B) \times C \times L])^{2/3}$		
Weir Crest =	64.00	m AHD
$Q_{1in2000AEP}$ =	7.7	m^3/s
Thus		
h =	0.92	m
and 1in2000 AEP WL in Dam =	64.92	m AHD
Embankment Crest =	65.00	m AHD
1 in 2000 AEP containted?	YES	

As such, 4.4 m spillway is required to produce a 1% AEP flood level of 64.5 m AHD (0.5 m head over spillway crest = TED) and a 1 in 2000 AEP flood level less than the embankment level of 65.0 m AHD.

Appendix D Continuous Rainfall Simulation Modelling

The Model for Urban Stormwater Improvement Conceptualisation (MUSIC, Version 6.3.0) has been used to assess the proposed SWMP detailed in Section 4

MUSIC has been used to perform:

- The stormwater pollutant retention benefits of the proposed treatment train,
- An inundation frequency analysis on the proposed waterbodies, and
- A turnover analysis on the waterbodies to access their long-term sustainability.

D.1 Model Description

The draft NSW MUSIC Modelling Guidelines (Sydney Metropolitan CMA, August 2010) have been used as the bases for the MUSIC model development.

D.1.1 Catchments

Subareas and fraction imperviousness used in the MUSIC modelling are as detailed in the RORB model. Sub areas are subject to change given the final development layout, however, provided the criteria of directing as much catchment as possible to (or close to) the defined WSUD element inlet locations are adhered to, the final MUSIC results are not expected to change significantly.

In line with draft NSW MUSIC Modelling Guidelines, Table 3.2:

- Burial and cemetery areas ("Urban Parks") have been modelled with "Residential" nodes,
- The external catchment has been modelled with "Agricultural" nodes, and
- The protected vegetation along the ridgeline has been modelled with "Revegetated Land" nodes.

Following the previous water cycle management plan, Rainfall-Runoff parameters for "Sandy Clay" have been used as shown in Table D.1 (from Tables 3.6, 3.7 and 3.8 of the draft NSW MUSIC Modelling Guidelines).

Table D.1 MUSIC Rainfall-Runoff Parameters Adopted

Parameter	Adopted Value	Unit
Impervious Area Parameters		
Rainfall Threshold	1	mm
Pervious Area Properties		
Soil Storage Capacity	142	mm
Initial Storage	25	% of Capacity
Field Capacity	94	mm
Infiltration Capacity Coefficient – a	180	
Infiltration Capacity Exponent – b	3	
Groundwater Properties		
Initial Depth	10	mm
Daily Recharge Rate	25	%
Daily Baseflow Rate	25	%
Daily Deep Seepage Rate	0	%

D.1.2 Climate Data

Bureau of Meteorology rainfall and evaporation data for Liverpool (Whitlam Centre) (gauge: 67035) from January 1967 until December 1976 at 6-minute intervals was utilised. This is gauge is located approximately 12.5 km from the subject site and is one of the closest pluviograph rainfall gauges from the site. The period chosen exhibits a mean annual rainfall of 857 mm.

Two nearby rainfall gauges (from the site) have been inspected to investigate if a mean annual rainfall of 857 mm is appropriate for use at the subject site. The gauges inspected are detailed in Table D.2. Standard practice is to assume if the mean annual rainfall of the metrological data used in MUSIC is within 10% of the expected rainfall at the site, it is appropriate for use at the site. From Table D.2, the expected mean annual rainfall at the site is approximately 780 mm/yr. As the metrological data is within +10% of 780 mm/yr, it is appropriate for use.

Table D.2 MUSIC Reference Gauges

Gauge Number	Gauge Name	Mean Annual Rainfall	Approximate Distance from Subject Site
68043	Minto Surrey Street	785 mm/yr	3.2 km
66190	Ingleburn (Sackville Street)	737 mm/yr	3.5 km

D.1.3 Treatment Elements

Four types of treatment elements have been modelled:

- Buffers,
- Swales,
- Wetlands, and
- Ponds.

D.1.3.1 Buffers

In catchments where there are some roads, roofs etc. up to 20% of the catchment may be directly connected to a pipe. In this situation it is assumed that 80% of the catchment is buffered and the buffer area is assumed to be 50% of the upstream impervious area. In catchments only exhibiting burials, none of the catchment will be directly connected to a pipe system, and as such 100% of the catchment is assumed to be buffered and the buffer area is assumed to be 50% of the upstream impervious area. Both of these assumptions are considered conservative in this application in regard to accounting for burial areas being disconnected from the drainage system. Where an existing riparian corridor was present through a catchment, no buffer was modelled for this part of the catchment.

Buffers have only been modelled on catchments exhibiting burials and are detailed in Table D.2 below.

Table D.3 Buffers Modelled

Sub Area	Area (m^2)	Area not Burials (m^2)	Percentage not Burials (i.e. % of Area Not Buffered)	Buffer Percentage
P	22158	2313	10%	90%
Z	26184	2910	11%	89%
AA	16933	0	0%	100%
AB	8222	0	0%	100%
AD	8127	1260	16%	84%
AE	15585	1795	12%	88%
AG	4666	700	15%	85%
AH	15827	1350	9%	91%
AI	19828	985	5%	95%
AJ	8809	3809	43%	57%
AK	25558	1141	4%	96%
AM	12270	710	6%	94%
AO	4694	1340	29%	71%
AP	22436	1320	6%	94%
AQ	7598	1035	14%	86%
AS	6520	3297	51%	49%
AT	15424	11451	74%	26%
AU	6181	1650	27%	73%
AW	12946	1110	9%	91%
AX	18733	2120	11%	89%
AY	11931	1000	8%	92%
BA	34300	1215	4%	96%
BD	7200	0	0%	100%
BE	24085	1745	7%	93%

D.1.3.2 Swales

Swales have been modelled in the locations shown in 1808/SWS/1 and Figure D.1. Where an existing riparian corridor is present, and no offset is proposed, conservatively **no** swale has been modelled (i.e. orange swales in 1808/SWS/1.

Two types of swales are proposed within the site:

- 5% AEP swales around the northern boundary road designed by WS&P, which have been modelled as:
  - A 0.5 m vegetated base,
  - A depth of 0.3m
  - 1V:3H side batters,
  - A vegetation height of 0.25 m,
  - Lengths and bed slopes as shown in Table D.4, and
- vegetation of the base of existing drainage lines (outside the riparian corridor), which have been modelled as:
  - A 1m vegetated base,
  - A depth of 0.5m
  - 1V:6H side batters,
  - A vegetation height of 0.25 m, and
  - Lengths and bed slopes as shown in Table D.4.

Table D.4 Swales Modelled

Swale ID	Swale Type	Length (m)	Bed Slope (1 in …)	Bed Slope (%)
S1 & S2	5% AEP swale by WS&P	60	19	5.3%
S3	1m vegetated base of drainage line	84	18	5.6%
S4	1m vegetated base of drainage line	35	68	1.5%
S5 & S6	5% AEP swale by WS&P	115	30	3.3%
S7	1m vegetated base of drainage line	100	40	2.5%
S8	1m vegetated base of drainage line	170	25	4.0%
S9	1m vegetated base of drainage line	45	20	5.0%
S10	5% AEP swale by WS&P	90	20	5.0%
S11	5% AEP swale by WS&P	150	100	1.0%
S12	5% AEP swale by WS&P	95	20	5.0%
S13A	1m vegetated base of drainage line	80	12	8.3%
S13	1m vegetated base of drainage line	190	25	4.0%
S14	1m vegetated base of drainage line	35	35	2.9%
S14 A&B	1m vegetated base of drainage line	110	10	10.0%
S15	1m vegetated base of drainage line	85	12	8.3%
S16	1m vegetated base of drainage line	80	20	5.0%
S17	1m vegetated base of drainage line	90	15	6.7%
S18	1m vegetated base of drainage line	60	80	1.3%
S19	1m vegetated base of drainage line	100	25	4.0%

D.1.3.3 Wetlands

Three Wetlands as shown and detailed in Section 4 have been modelled. Table D.5 summaries the waterbodies modelled as wetlands. Each of these waterbodies is designed to be vegetated for at least 60% of the NWL area.

Table D.5 Wetlands Modelled

Water body	NWL (m AHD)	TED (m AHD)	NWL Area (m^2)	Permanent Pool Volume (m^3)	Detention Time (Hrs)	Overflow Weir Size (m)
Dam 3	55.00	55.50	2240	1010	48	7.4
Dam 5	64.00	64.35	4850	2161	48	11.3
Dam 6	63.50	64.00	2820	1295	48	4.4

D.1.3.4 Ponds

Two Ponds as shown and detailed in Section 4 have been modelled. Table D.6 summaries the waterbodies modelled as ponds.

Table D.6 Ponds Modelled

Waterbody	NWL (m AHD)	NWL Area (m^2)	Permanent Pool Volume (m^3)	Overflow Weir Size (m)
Dam 2	53.50	3905	3135	4
Dam 4	58.50	11050	9402	5

D.1.4 Model Schematic

Figure D.1 details the model schematic.

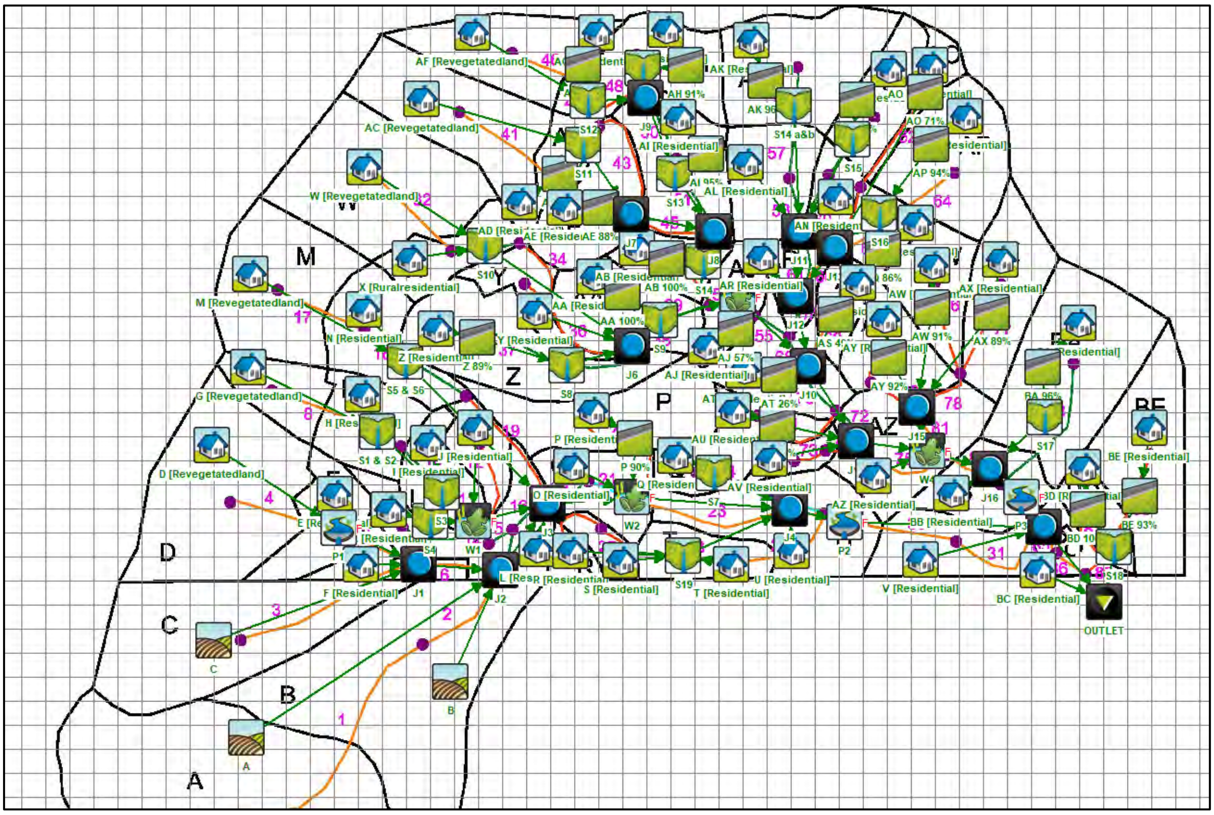

Figure D.1 Model Schematic

D.2 Stormwater Pollutant Retention

As discussed in Section 3.5, the Landcom pollutant targets shown below are required to be met from the treatment train located within the proposed development.

Total Suspended solids (TSS)	85% retention of the typical urban annual load
Total Phosphorus (TP)	65% retention of the typical urban annual load
Total Nitrogen (TN)	45% retention of the typical urban annual load
Gross Pollutants	95% reduction of the typical urban annual load

The treatment elements detail in Appendix D.1.3 and described above form the treatment train for the proposed development. The proposed treatment train pollutant retention effectiveness, in regards to pollutants generated from the proposed development (as catchments A, B and C) and external catchment inputs are shown in Table D.7 below.

Table D.7 MUSIC treatment train effectiveness

Pollutant	Pollutants generated from whole catchment (kg/yr)	Pollutants generated by external catchments (kg/yr)	Pollutants generated by internal catchments (kg/yr)	Amount withheld in treatment systems (kg/yr)	% Pollutants withheld relative to pollutant generation within proposed development (%)
Total Suspended Solids	34700.0	6880.0	27820.0	26270.0	94.4%
Total Phosphorus	67.3	18.6	48.7	38.5	79.1%
Total Nitrogen	501.0	130.2	370.8	191.0	51.5%
Gross Pollutants	4840.0	343.1	4496.9	4762.8	100.0%

As can be seen in Table D.7, the proposed treatment train is able in retain the pollutants to in excess of the Landcom pollutant targets.

D.3 Inundation Frequency Analysis

An inundation frequency analysis is performed by exporting the waterbodies daily average water level relative to the design NWL. This data is then statistically interrogated to determine (say for example) the 20% percentile water level. The 20% percentile water level would be the water level, relative the design NWL level, that is expected to be exceeded 20% of the time.

This type of analysis gives an indication of waterbody long term sustainability and aesthetics. The analysis and can be used in plant specification at the functional design stage of the project.

Inundation frequency analysis's have been completed on the three wetlands and two ponds detailed in Section 4.

D.3.1 Wetlands

Table D.8 and Figure D.2 below detail the results from the inundation frequency analysis on the wetlands. The results indicate that the current wetland proposals for Dam 3 is adequate with 40% of the time, the water level being at or above the NWL. However, Dams 5 and 6 appear to be oversized for their catchments as only 25% of the time, is the water level at or above NWL. For all systems, the ED of is very unlikely to be exceeded. Further, if the drawdown is to occur in any of the wetlands, it should be well hidden with appropriate batter planting to 350mm below NWL.

Table D.8 Wetland Inundation Frequency Results

Percentile	Depth Relative to NWL (mm)		
	Dam 3	Dam 5	Dam 6
99%	-211	-295	-221
95%	-118	-171	-126
90%	-70	-102	-76
85%	-52	-78	-57
80%	-40	-64	-45
75%	-31	-53	-36
70%	-24	-44	-29
65%	-18	-35	-23
60%	-13	-28	-18
55%	-9	-23	-14
50%	-5	-17	-10
45%	-2	-13	-6
40%	1	-8	-4
35%	11	-4	-1
30%	37	-1	2
25%	88	1	7
20%	162	7	27
15%	281	28	65
10%	432	81	145
5%	502	246	314
1%	510	354	501

The results confirm site observations that significant drawdown causing unsightly dam edges should not be an issue in these "upper" water bodies.

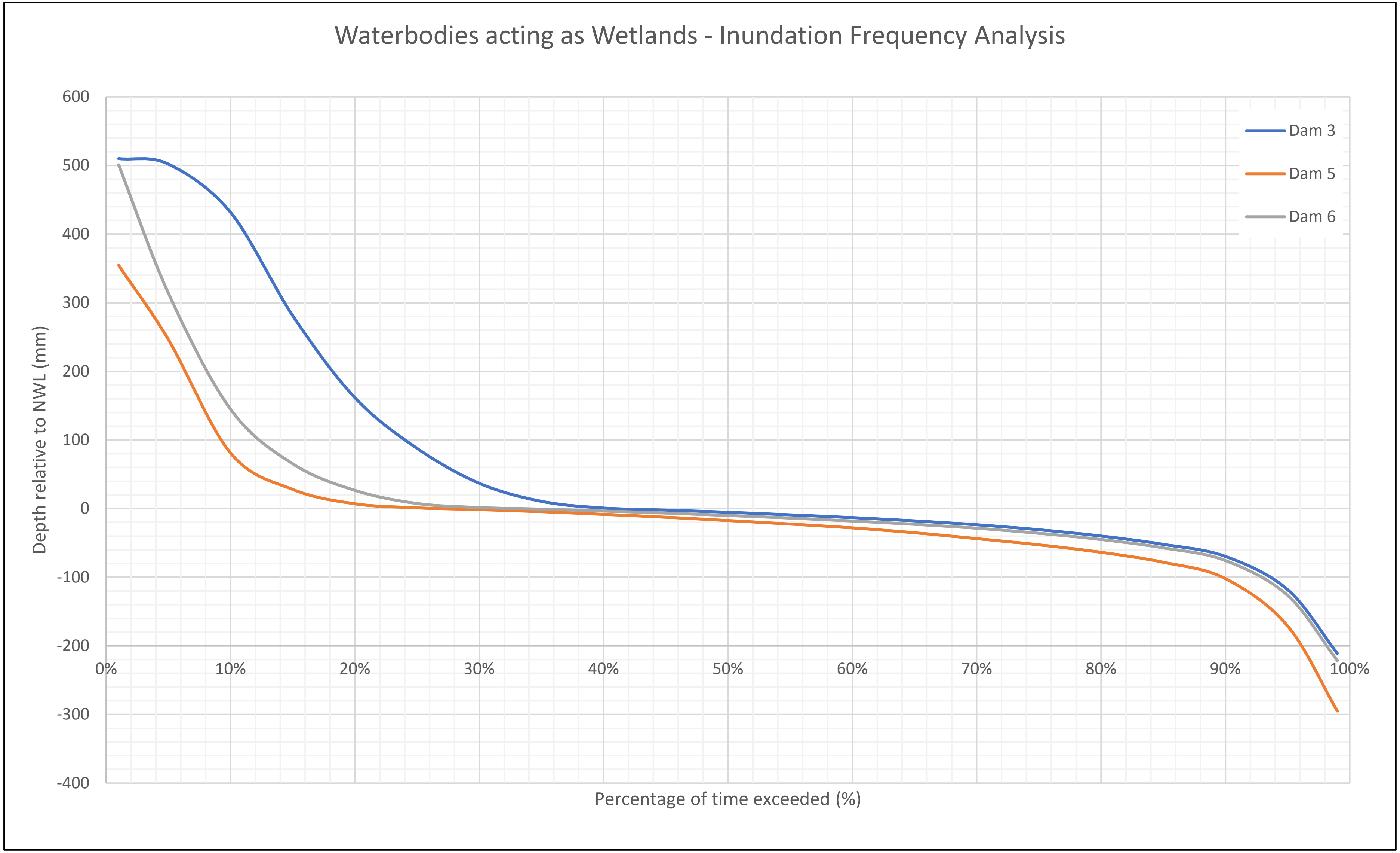

Figure D.2 Wetland Inundation Frequency Analysis Results

D.3.2 Dams with open Ponds

Table D.9 and Figure D.3 below detail the results from the inundation frequency analysis on the dams acting as ponds. The ponds are always at or below their NWL as to the outlet configurations on the ponds have no ED range.

Of note is that Dam 4 is expected to be significantly (more than 100mm) drawn down below NWL 25% of the time. The vegetated safety edge treatment of this water body should ensure minimal landscape impact during drawdown periods.

Table D.9 Pond Inundation Frequency Results

Percentile	Depth Relative to NWL (mm)	
	Dam 2	Dam 4
99%	-276	-435
95%	-143	-278
90%	-79	-201
85%	-58	-161
80%	-46	-133
75%	-36	-109
70%	-29	-91
65%	-23	-78
60%	-17	-66
55%	-13	-56
50%	-9	-47
45%	-6	-38
40%	-3	-29
35%	-1	-22
30%	0	-15
25%	0	-8
20%	0	-2
15%	1	0
10%	1	0
5%	1	0
1%	5	1

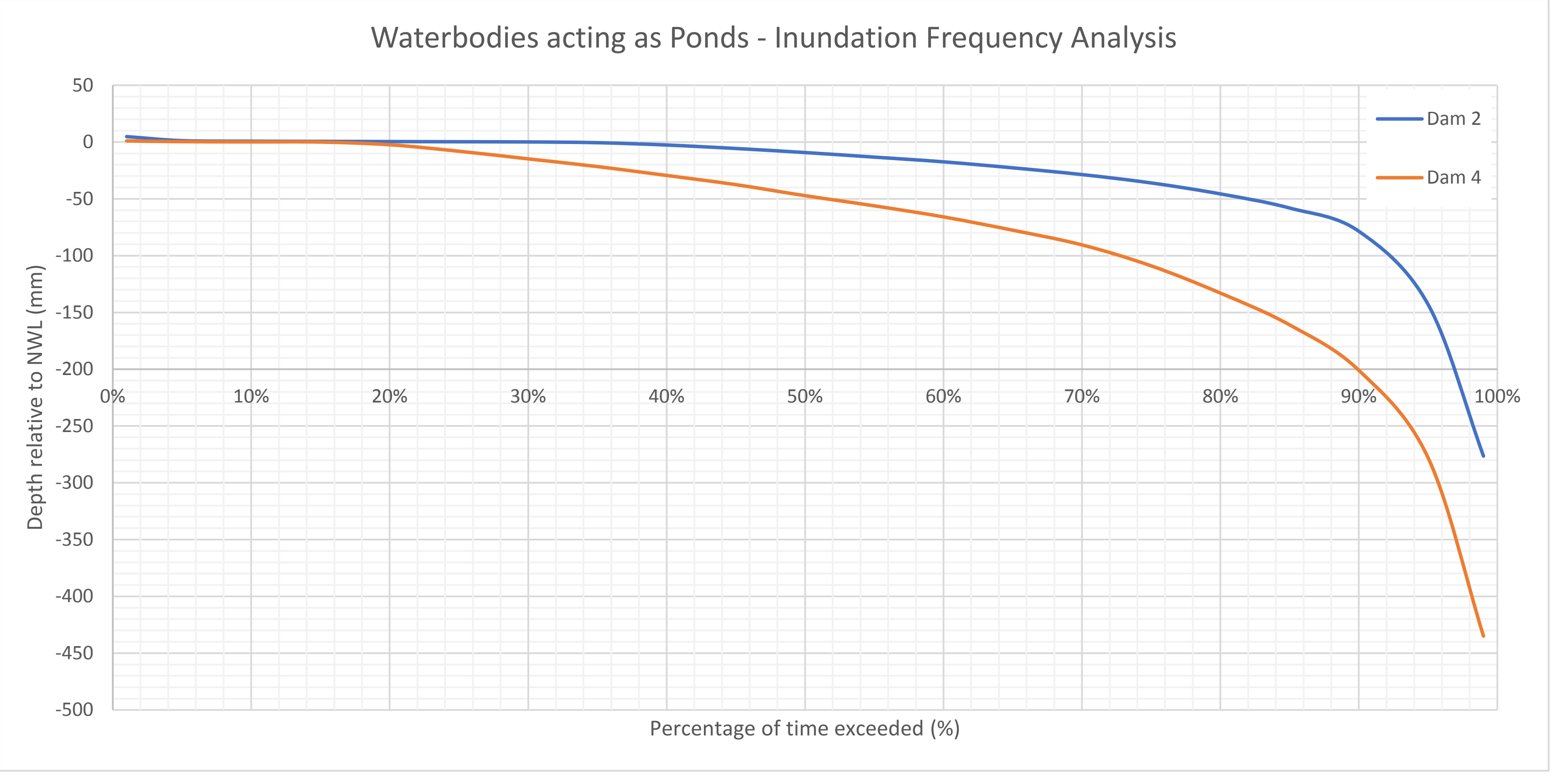

Figure D.3 Pond Inundation Frequency Analysis Results

D.4 Waterbody Long Term Sustainability

SWS has assessed the waterbodies long term sustainability using a turnover analysis on the waterbodies proposed.

A turnover analysis involves calculating a time series of the outflows from the waterbody. Then for each time step in the series (days in the analysis performed) calculating how long it has taken for the outflows from the waterbody to exceed the waterbodies permeant pool volume (the volume of water stored below NWL). This data can then be statistically interrogated.

In general, for deep open water bodies with little vegetation, residence times over 30 days (especially in summer) can be an indication of an increased risk of algal blooms over time.

D.4.1 Wetlands

Figure D.4 details the results of the turnover analysis for all four wetlands. As expected the wetlands that have a lower volume compared to their catchment areas (Dam 3 and Dam 6) perform better.

At least 50% of the time, the residence time is 30 days or more in Dams 3 and 6. This increases to a residence time of 50 days or more 50% of the time in Dam 5.

However, given the “wetland systems” are at least 60% vegetated, very shallow, and quite large (enabling good wind aeration), it is expected that the ongoing risk of algal blooms in these systems is relatively low.

Notwithstanding the above, a dam management plan incorporating adequate water quality monitoring and required actions should be formulated to manage ongoing risks regarding this issue.

These issues are further discussed in Section 4.3.6 of this report.

Figure D.4 Wetland Turnover Analysis Results

D.4.2 Ponds

Figure D.5 details the results of the turnover analysis for all dams acting as open water ponds.

Both ponds have an apparently high risk of algal blooms as residence times are well over 30 days more than 50% of the time. Dam 4 in particular has long water residence times.

Dam 4 is of concern. This asset is to be used as a major aesthetic site for the development. There is a relatively high risk of algal blooms which could impact the appeal of the asset and also cause a health and safety risk. These risks can be reduced by incorporating mitigation techniques as per section 4.3.6 of this report.

The 1 metre depth proposed combined with the option to vegetate this system with submerged wetland plants and/or to recirculate water though upstream wetlands should result in a relatively low ongoing risk of algal blooms in the two dam pond systems.

Notwithstanding the above, a dam management plan incorporating adequate water quality monitoring and required actions should be formulated to manage ongoing risks regarding this issue.

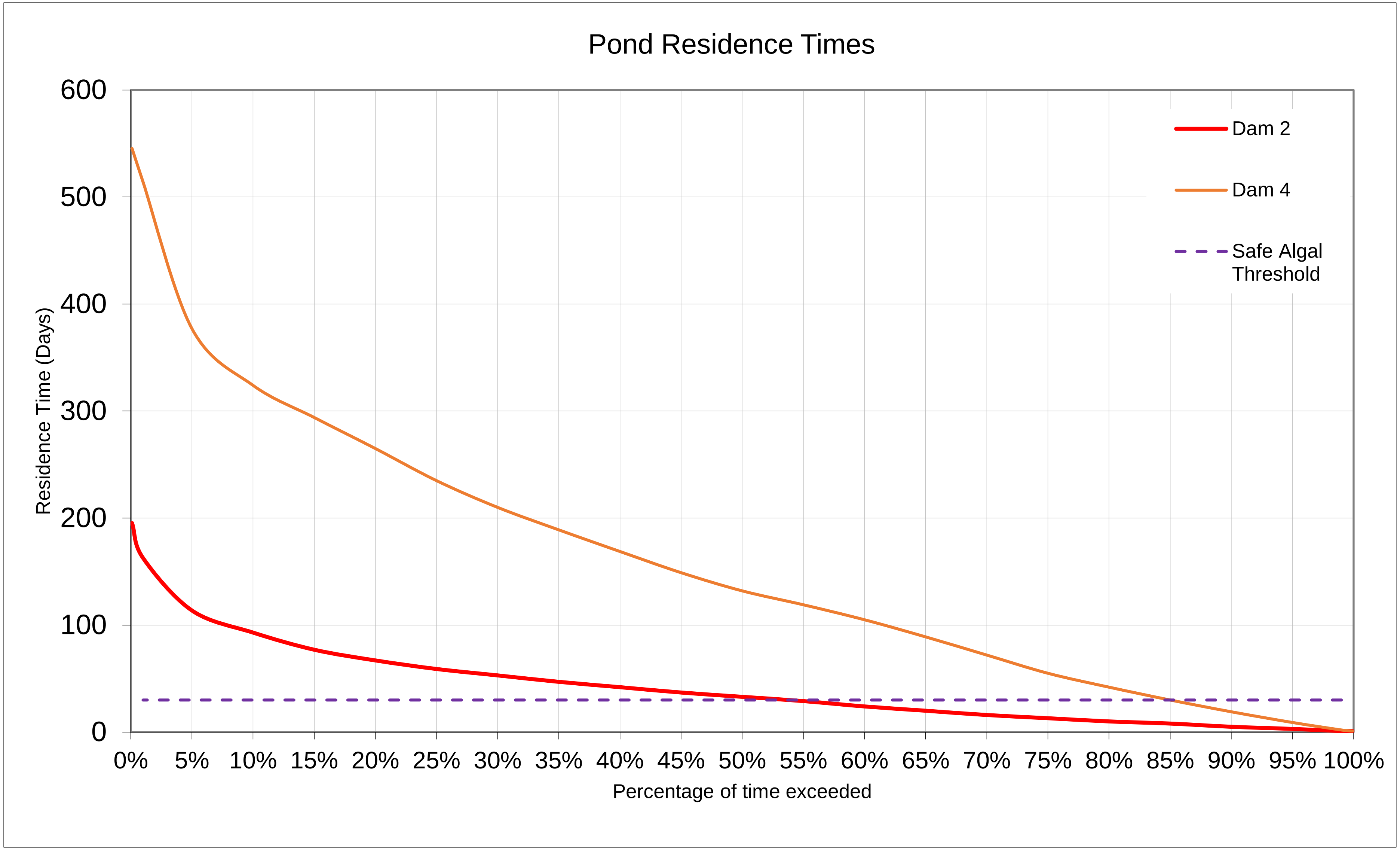

Figure D.5 **Pond Turnover Analysis Results**

D.5 Supplementary Water Source Recommendations

Given the results presented in Appendix D.4, SWS did investigate the possibility of sourcing supplementary water feeds into both Dam 2 and Dam 4. After discussion with the project team, treated bore water was deemed a feasible option to provide this extra water.

The MUSIC model was modified to allow for various amounts of bore water feed into the two drainage lines. The bore water feed was entered at 6-minute timesteps into the model. The feed was also scaled by the PET of the model. As such, more feed is assumed to be provided than winter.

The analysis indicated that significant bore water feeds (50 ML/yr or more) would be required to enable 30 residence times at least 50% of the time in Dams 2 and 4. As this source of water is not deemed the more sustainable environmental solution, SWS recommends that bore water only be used for dam top up in extremely dry times. This may require in the order of 17 ML in rare instances.

Business as usual should rely more on monitoring, maximising wetland plants in the system, aeration though fountains and recirculation of water though upstream wetland systems.

The functional design as specified in drawing set 1808/SWS/1 – 10 builds in all the design aspects to accommodate “businesses as usual” monitoring and remediation actions.

Appendix E ARR 2016 Design Flow Determination Methodology

Note the example below is hypothetical only. It is provided to show the ARR 2016 methodology. Design flows for this project are as detailed in the body of the report.

ARR 2016 requires the modeller/designer to simulate two hundred and forty (240) individual storms as part of selecting the one critical design event for each Annual Exceedance Probability (AEP) analysed.

There are 24 separate durations of events required to be analysed ranging from 10 minutes through to 168 hours (7 days). For each of these durations ten distinct temporal patterns (storms) can be downloaded from the ARR 2016 datahub for the location of interest (and the relevant AEP). All of these temporal patterns are then simulated within the hydrologic modelling software of choice. 240 hydrographs are subsequently produced.

In order to determine which of these 240 hydrographs should be used as the design hydrograph, a statistical analysis is performed. For each 10 hydrographs in the 24 durations analysed a separate box and whisker plot is developed. Box and whisker plots are a way of easily comparing large amounts of data.

The following example is provided for a sample ensemble of hydrographs as shown in Figure E.1 for a 18.13% AEP (5-year ARI) 1-hour storm duration.

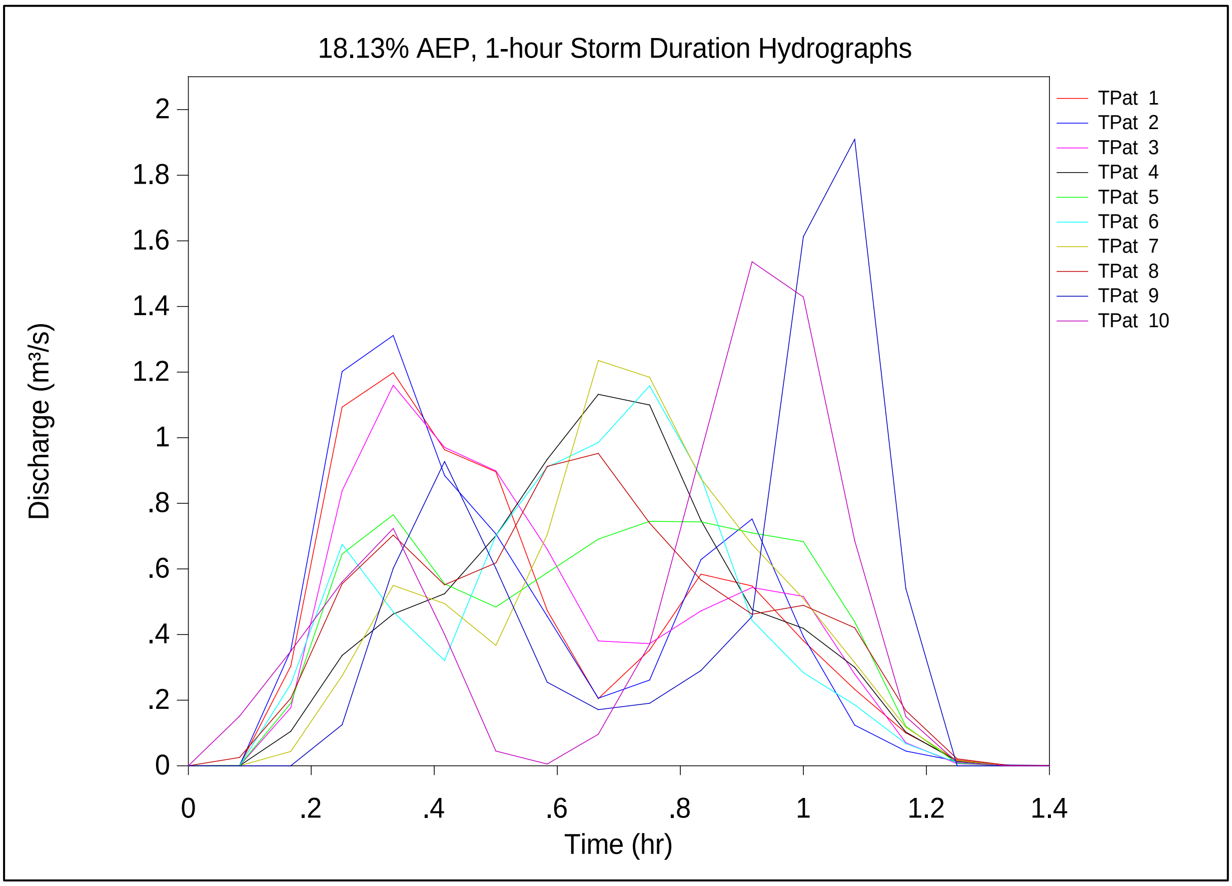

Figure E.1 Example ensemble of 10 hydrographs produced by hydrological modelling

The peak of each hydrograph is then sorted from smallest to largest as shown in Table E.1.

Table E.1 Example ordered peaks from Figure E.1

Local Order	1	2	3	4	5	6	7	8	9	10
Temporal Pattern ID	5	8	4	6	3	1	7	2	10	9
Peak Flow (m^3/s)	0.77	0.95	1.13	1.16	1.16	1.20	1.24	1.31	1.54	1.91

From the data set shown in Table E.1, the following can be calculated:

- The median = Q_2 = the middle value of the data set = 1.18
- The lower quartile = Q_1 = the median of the lower half of the data set = 1.13
- The upper quartile = Q_3 = the median of the upper half of the data set = 1.31
- The interquartile range = IQR = $Q_3 - Q_1$ = 0.18
- The lower outlier value = $Q_1 - 1.5 \times IQR$ = 0.86

  If a hydrograph has a peak lower than this value it is considered and outlier. The hydrograph with a peak flow of 0.77 (TP 5) is an outlier in this example.
- The upper outlier value = Q3 + 1.5 x IQR = 1.58

  If a hydrograph has a peak higher than this value it is considered and outlier. The hydrograph with a peak flow of 1.91 (TP 9) is an outlier in this example.
- **The average flow = 1.24**

The 10 hydrographs for the 18.13% AEP, 1-hour storm are then plotted on a box and whisker plot as shown below in Figure E.2.

The hydrograph that exhibits a peak flow closest to the average peak flow is then selected as the design hydrograph for this duration. In this example, for this 1- hour duration, it is the hydrograph produced by temporal pattern 7 that produces a peak flow of 1.24 m³/s.

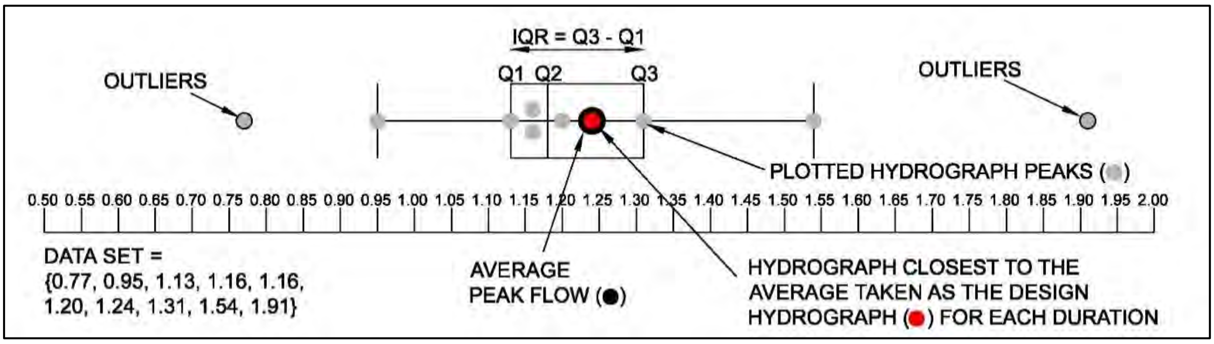

Figure E.2 Box and whisker plot example for a 1-hour storm duration

This analyis is then completed for each of the 23 remaining storm durations and the resultant 24 box and wisker plots are plotted as shown in Figure E.3.

The final design hydrograph for in this case, the 18.13% AEP storm, is then selected as the hydrograph that exhibits a peak flow closest to the peak average peak flow from each duration shown in Figure E.3. This design hydrogaph is then reported as shown in Table E.2.

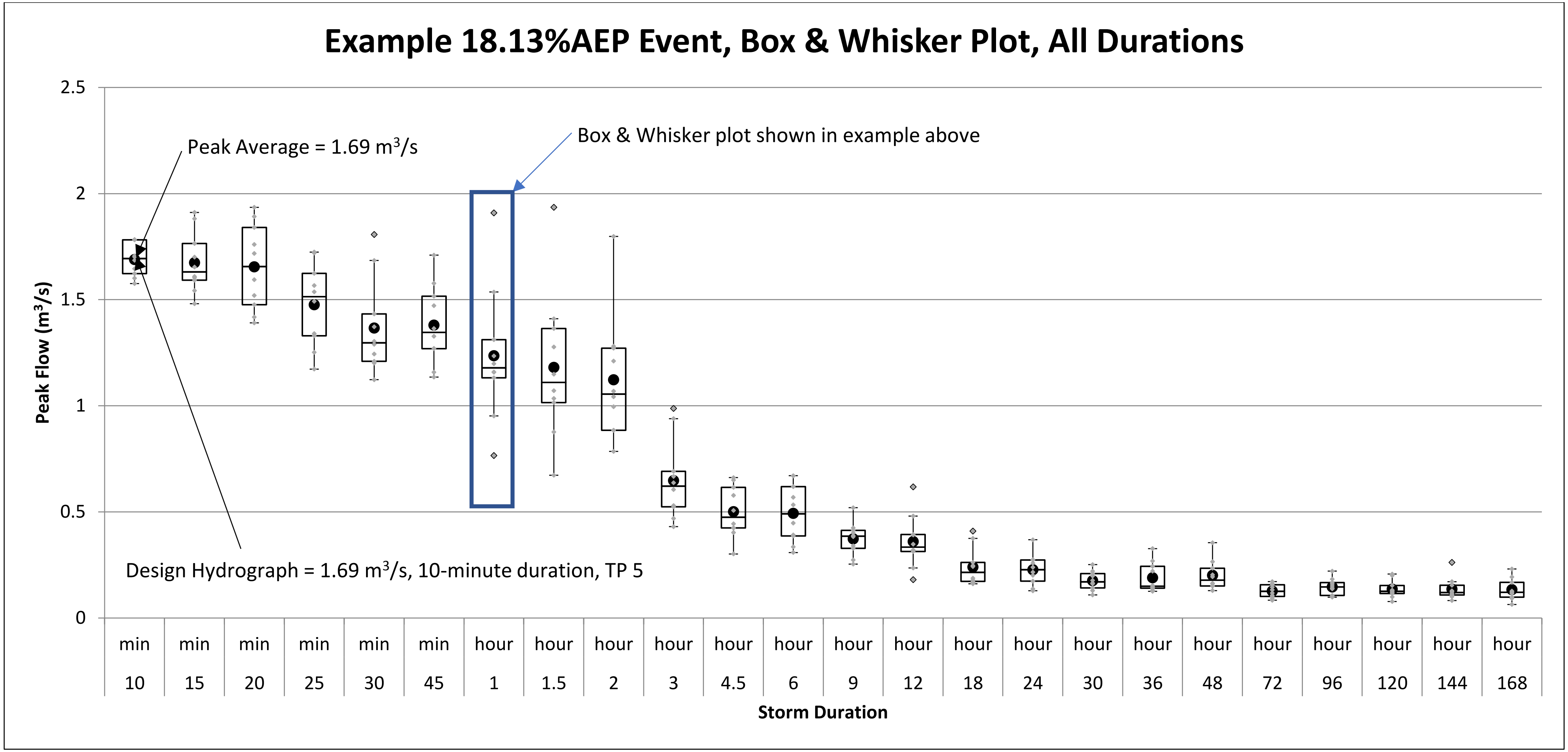

Figure E.3 **Box and whisker plot example for all durations.**

Table E.2 **Design hydrograph for the example**

Location	Example Peak Average		Example Appropriate Design Hydrograph		
	$Q_{18.13\%AEP}$ (m^3/s)	Critical Duration	$Q_{18.13\%AEP}$ (m^3/s)	Critical Duration	Temporal Pattern ID
Example	1.69	10-minute	1.69	10-minute	5

The example 18.13% AEP hydrograph to be used in designs, is the hydrograph defined by a 10-minute storm with temporal pattern 5, producing a peak flow of 1.69 m^3/s.