Innovation in Infrastructure

Wyndham North Stormwater Management Strategy

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Prepared by Spiire Australia

This report has been prepared by the office of Spiire

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List of Abbreviations

AAV	Aboriginal Affairs Victoria
AR&R	Australian Rainfall and Runoff: A Guide to Flood Estimation (reference)
ARI	Average Recurrence Interval
BP	Best Practice
BPEMG	Best Practice Environmental Management Guidelines
CHMP	Cultural Heritage Management Plan
CWW	City West Water
DSE	Department of Sustainability and Environment
DSS	Development Services Scheme
EPBC Act	Environmental Protection and Biodiversity Act
EVC	Ecological Vegetation Class
GAA	Growth Areas Authority
GGF	Growling Grass Frog
HEC-RAS	Hydraulic Engineering Center: River Analysis System (software program)
IWM	Integrated Water Management
MWC	Melbourne Water Corporation
MUSIC	Model for Urban Stormwater Improvement Conceptualisation (software program)
NOI	Notice of Intent
OLV	Office of Living Victoria
PSP	Precinct Structure Plan
RORB	Runoff Routing (software program)
RRL	Regional Rail Link
SEPP	State Environmental Protection Policy
SWMS	Stormwater Management Strategy
WSUD	Water Sensitive Urban Design
UGZ	Urban Growth Zone
VPP	Victorian Planning Provisions

1. Introduction

Spiire has been engaged by the Growth Areas Authority (GAA) in order to confirm the allocation of space in the draft Wyndham North Precinct Structure Plan for stormwater management infrastructure and assess stormwater harvesting options to assist City West Water (CWW) and Melbourne Water Corporation (MWC).

Integrated Water Management is an important part of the planning process and its incorporation into the Precinct Structure Plan (PSP) will ensure efficient and sustainable water management within the development framework.

This report will outline a number of strategies to achieve a desirable outcome as depicted in Figure 1 below.

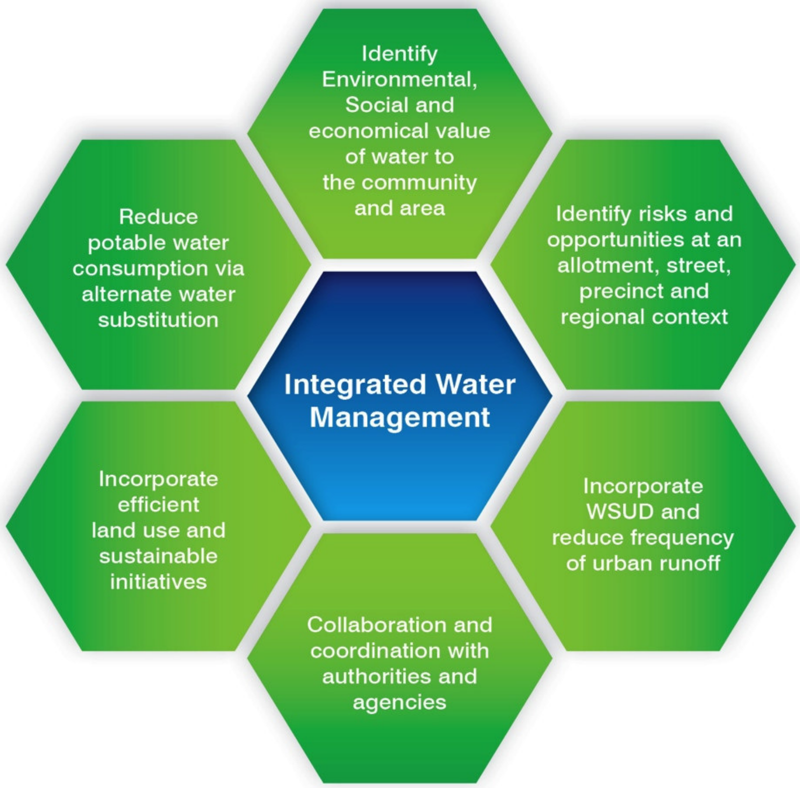


Figure 1 – IWM Outcomes

Ultimately this report is aimed at improving Victoria’s local environments and urban liveability, while providing resilient and flexible water services.

1.1 Background

The Wyndham North study area is made up of land within the Tarneit and Truganina districts which is within the Urban Growth Zone (UGZ).

The Wyndham North area will cover an area of over 4,500 hectares and when fully developed is expected to deliver over 35,000 dwellings and be home to over 93,000 people. The investigation area is made up of four PSP's which included PSP 88, 89, 90 & 91. Refer to Figure 2 for the indicative locations of these PSP's.

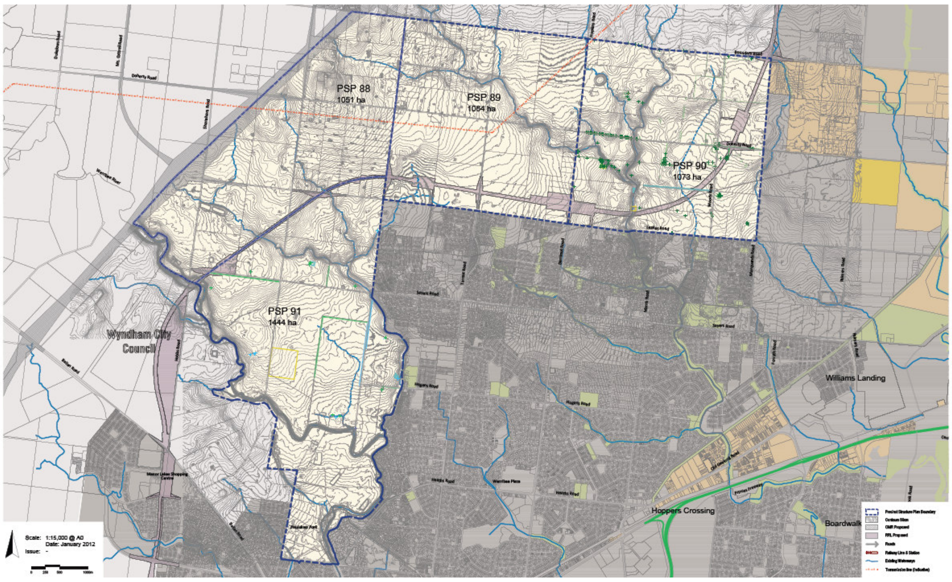


Figure 2 - PSP boundaries

The GAA is in the process of preparing a draft urban structure for all four PSPs that make up the Wyndham North area.

The Wyndham North study area in terms of stormwater drainage is broken into two major catchments and four major waterways: Davis Creek, Skeleton Creek, Dry Creek and Forsyth Road Drain, with Dry Creek and Forsyth Road Drain being tributaries of Skeleton Creek. The Department of Sustainability and Environment (DSE) 2011 Draft Report on the Sub-regional Species Strategy for the Growling Grass Frog (GGF) indicates that Davis Creek, Skeleton Creek and Dry Creek comprise of habitat that will need to be protected and managed for the conservation of the Growling Grass Frog. Since the onset of this project the DSE have now removed the GGF requirements on the Skeleton Creek and Dry Creek tributaries.

The Regional Rail Link (RRL) traverses through the site as per Figure 2. MWC has acknowledged that planning and construction of the RRL project will ensure capacities and depths of infrastructure to allow for the Wyndham North Growth area. A summary of the background reports that have been made available during this engagement are as follows:

1. Preliminary Geomorphic, Flora, Fauna and Socio-economic Assessment, SKM.
2. Development Services Scheme for Davis Creek & Doherty's Drain, MWC.
3. Davis Creek & Skeleton Creek RORB Models, MWC.
4. IWM Strategy for Tarneit PSP 88-91, Arup.

5. Wyndham Vale – Sundry IWMP Tasks, Alluvium.
6. Utilities Capacity Assessment - Tarneit PSP Area, GHD.
7. Strategic Drainage Proposals and the GGF Corridor Requirements, N Craigie.
8. Sub Regional Species Strategy for the GGF, DSE
9. Aboriginal Cultural Heritage report for Wyndham North area, AHMS.
10. Regional Rail Link Tender drawings, GAA & MWC.

A site visit was carried out with the GAA on the 20th February 2012 to review the potential locations for stormwater infrastructure. Refer to Appendix 1 for the site visit photo log.

These reports and their findings have been used to guide the Storm Water Management Strategy (SWMS) that will be adopted for the Wyndham North study area.

2. SWMS Strategy Approach

The Wyndham North SWMS aims to achieve best practice stormwater quantity, quality and reuse objectives, these are discussed separately below.

2.1 Stormwater Quantity

Quantity is the reference to stormwater discharge magnitude and volume. The SWMS achieves best practise objectives by:

- Recommending under ground drainage systems for storm events up to the 1 in 5 or 1 in 10 year ARI recurrence interval subject to land zoning.
- Recommending the safe conveyance of storm events exceeding the capacity of the under ground system up to the 1 in 100 year ARI recurrence interval via overland flow paths, constructed waterways and or existing waterways.
- And recommending the use of retarding / detention systems through out the catchment to limit the magnitude of developed discharge to acceptable levels to be conveyed via the existing downstream infrastructure without causing detrimental impact.
- Achieve equitable land take for all key drainage infrastructure within the WNPSP.

2.2 Stormwater Quality

Quality is the reference to pollutant levels carried by stormwater discharge. The SWMS aims to achieve State Environment Protection Policy objectives by recommending solutions in accordance with the urban stormwater best practise environmental management guidelines (BPMEG), these are:

- Recommending the incorporation of MWC Development Services Scheme (DSS) wetland systems into retarding basin locations.
- Treat every catchment to BPMEG prior to discharging stormwater into receiving waterways.

This recommendation provides a balanced approach in which:

- The delivery of assets is shared between the public and private sector, allowing for innovation in design and also promoting the objective of Clause 56.07 incorporating water sensitive urban design (WSUD) at a neighbourhood level.
- The ownership/maintenance responsibilities for water quality treatment assets are intended to be shared between MWC and Council.

2.3 Stormwater Reuse

Reuse is the reference to harvesting stormwater and reusing it to supplement either recycled and or potable water supply where appropriate. The SWMS aims to achieve objectives as set out the Office of Living Victoria Business Strategy 2012 “A new era in urban water cycle management”. The SWMS achieves these objectives by recommending:

- Stormwater harvesting and reuse within the Wyndham North study area for sporting facilities and public open space to supplement recycled water supply.
- Stormwater harvesting and reuse within the Wyndham North study area for toilet flushing
- Treated sewer water and stormwater for use in greening the Wyndham North study area.
- Capture and store rainwater at allotment level, in line with the residential sustainability measure provision under the current building code.

City West Water has acknowledged that they are considering stormwater as a source of non-potable water supply for the Wyndham North study area. This has been reinforced by the Integrated Water Management strategy for the Wyndham Vale Area prepared by GHD which discusses the stormwater and rainwater harvesting in the area.

Recent projects at Point Cook have demonstrated that stormwater and recycled water can be integrated to supplement the potable water supply. The strategy aims to ensure stormwater and recycled water are working in synergy rather than opposing each other. The stormwater harvesting analysis will assume the following:

- Typical rain tank provision on each allotment is to reflect the level of rain tank provision under the current Building Code of Australia requirements, which will identify the maximum land take requirements for meeting non-potable supply.

3. Cultural Heritage Objectives

A Cultural Heritage Assessment has been carried out by AHMS for PSPs 88-91 dated 7th November 2011. A notice of intent (NOI) to prepare the CHMP was lodged by AHMS with Aboriginal Affairs Victoria (AAV) on the 13th July 2011. There is currently no Registered Aboriginal Party relevant to the activity area and therefore AAV are the statutory authority responsible for evaluating the CHMP.

AHMS found that thirty-five Aboriginal sites were recorded on the Victorian Aboriginal Heritage Register, one of which was described as an earth feature where as all others were recording of scattered artefacts.

The report recommended the PSP design responses and legal requirements that would apply to the planning and development within the study area.

4. Flora & Fauna and Geomorphic Objectives

A preliminary geomorphic, flora, fauna and socio-economic assessment was carried out by SKM dated 3rd February 2012. The report concluded the following:

Geomorphic

The Werribee River and sections of Skeleton Creek were considered to have stream forms of regional significance and were in relatively good condition. The remainder of waterways have relatively common stream forms in poor to moderate condition.

Flora and Fauna

The assessment did not identify any endangered flora and fauna species within the investigation area under the EPBC Act. However SKM did identify various threatened species such as the Growling Grass Frog (GGF). Furthermore a number of environmentally significant areas were identified with plains grasslands (e.g. spiny rice flower, flax lily and plains joyweed).

Sensitivity of reaches to hydrological change

The change in imperviousness within the catchment will alter the magnitude and frequency of flows entering the waterways. Some reaches have been identified as having intact or incised alluvial fills which will be highly sensitivity to erosion as a result of hydrological changes. Flora and fauna values are identified as having a very high sensitivity to hydrological change, due to the presence of mapped EVC and threatened species adjacent to the waterways.

Management intervention measures to protect waterways

A series of management intervention measures were recommended to reduce the magnitude of hydrological changes. These include:

- Buffers or setbacks: To provide the waterway and its associated ecosystems with sufficient area within which it can adjust to anticipated hydrological changes and other future uses.
- Infiltration and retention in contributing catchment areas: These works are important in reducing overland flow and reducing runoff close to the source areas where greatest changes in catchment imperviousness occur.

- Retarding basins with different outlet controls to release different ARI flows: These provide temporary runoff storages but aim to release flows in such a way to minimise the occurrence of a constant discharge and create a more variable flow regime.

It was noted that standard WSUD measures may not provide sufficient hydrological protection for stream values and it may be necessary to consider alternative techniques. This could include off-stream reserves or wetlands that function as refuges for species identified as having a high sensitivity to hydrological change, such as the Growling Grass Frog.

The Department of Sustainability and Environment (DSE) have also released the Sub regional species Strategy for Growling Grass Frog for public consultation. The strategy will be used as the key mechanism to deliver conservation outcomes such as:

- Functioning sustainable populations of Growling Grass Frog with connectivity between populations
- Protection and enhancement of important populations.

The Strategy identifies land in the growth areas that is suitable for the Growling Grass Frog, and places this land into two categories as follows:

- Habitat that will be protected and managed for the conservation of the Growling Grass Frog (Category 1 habitat)
- Habitat that can be cleared for urban development, but for which compensatory habitat is required (Category 2 habitat).

Figure 3 below depicts the presence of the two categories within the Wyndham North growth area.

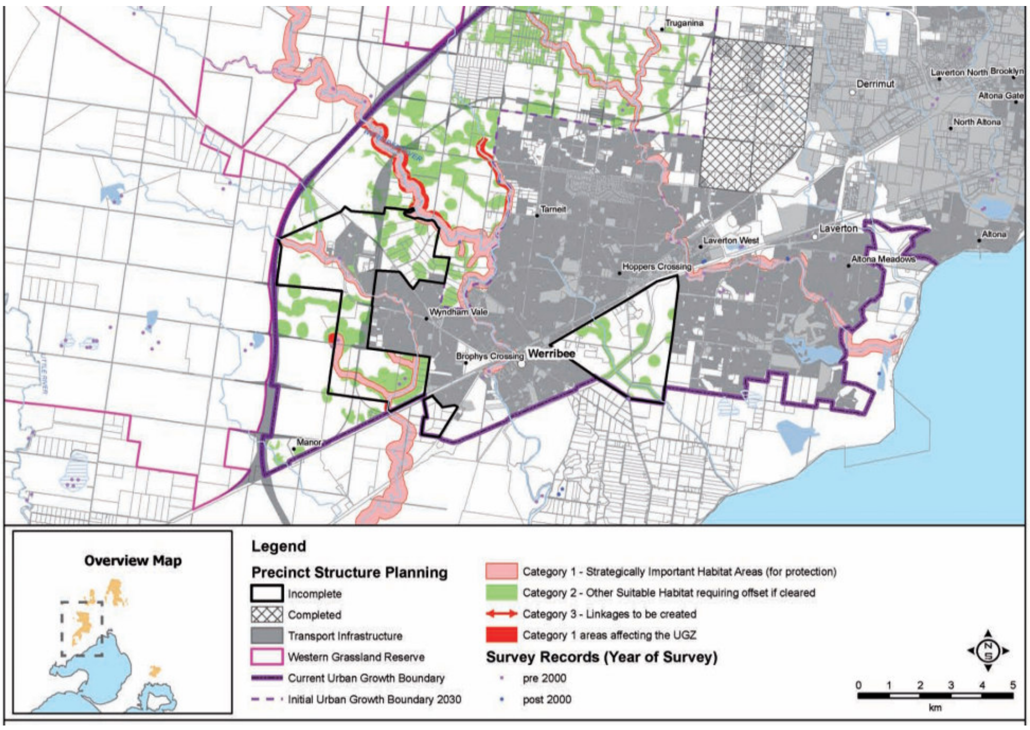


Figure 3 - DSE Sub-Regional Species Strategy for the GGF

It can be seen in Figure 3 that both Skeleton Creek and Dry Creek have category 1 habitats overlaid on these tributaries, however the DSE have confirmed with the GAA that the requirements of category 1 habitats over these areas has now been removed.

Both the SKM report and DSE GGF strategy have been used throughout the project to ensure environmentally significant areas are protected. Through a collaborative approach we have identified that Dry Creek, Davis Creek and Skeleton Creek are to remain intact to preserve existing geomorphic, flora and fauna conditions.

A consultative approach was carried out to ensure the process of identifying stormwater infrastructure locations held high regard to environmentally significant areas. To protect the geomorphic, flora and fauna conditions from hydrological changes due to urban development a number of measures as mentioned above will be used throughout the Wyndham North area. The management techniques adopted will be discussed within the following sections.

5. Regional Rail Link Considerations

The Regional Rail Link (RRL) is a Victorian and Australian Government initiative to create dedicated regional tracks from West Werribee Junction to Deer Park, then along the existing rail corridor from Sunshine to Southern Cross Station. The RRL goes through each of the four PSP’s and crosses the major tributaries within the Davis Creek, Skeleton Creek and Forsyth Road Drain catchment areas.

Spiire obtained all the RRL drawings from the GAA and also liaised with MWC with respect to rail bridge hydraulic capacities. The RRL has made allowance for the future urban growth by allowing for the deepening of existing creeks where appropriate under the RRL. Based on our review of this information we believe the RRL has provided sufficient capacity for future urban flow regimes.

The RRL has caused modifications to the existing catchment boundaries as no allowance of upstream flow conveyance infrastructure has been catered for other than the key creek crossing points. The RRL is reflected in our RORB catchment boundaries.

6. Waterway Health Objectives

Melbourne is a city expanding – it is estimated that by 2030 Melbourne's population will increase by one million, and with that expansion comes urban development that encompasses formerly untouched rural landscapes.

Where previously stormwater would permeate the soil and produce a steady sub-surface flow, traditional development methods produce new impervious surfaces such as housing roofs, concrete footpaths and asphalt roads that convey the water through a drainage scheme into the delicate ecosystem that is our creeks and rivers, increasing the frequency of surface runoff. As such measures need to be incorporated into the Wyndham North study area to protect receiving waterways where possible.

Urban stormwater best practice environmental management guidelines (BPEMG) endeavour to protect waterway health objectives by complying with State Environmental Protection Policies (SEPPs). Likewise, the Melbourne Water Healthy Waterways strategy also outlines how to protect and manage waterway health. The objectives of the BPEMG are discussed below.

6.1 Stormwater Quantity

The Best Practice Guidelines objective is to:

- Maintain discharges for the 1.5 year ARI event at pre-development levels.

Based on the analysis undertaken within this strategy it has been determined that flows should be detained to at least the 1 year ARI event rather than the 1.5 year ARI. This detention provides the optimum balance for infrastructure sizing (retarding basins, pipes, floodways) within the Wyndham North study area.

6.2 Stormwater Quality

The Best Practice Guidelines objectives are:

- 70% reduction of total Gross Pollutant loads
- 80% reduction of total Suspended Solids
- 45% reduction of total Nitrogen
- 45% reduction of total Phosphorus

The objectives are implemented to protect the natural environment by implementing measures that not only treat the water but retard the flow to counter the problem of increased impervious surfaces runoff through construction of new roads, housing and other development.

6.3 Water Sensitive Urban Design

Water Sensitive Urban Design (WSUD) aims to improve the quality of stormwater prior to its entering our rivers and waterways. This is done by implementing one or a number of treatment measures to remove pollutants from the stormwater discharge. Developers are now required to use WSUD practices to achieve best practice guidelines in any new development in accordance with the requirements of Clause 56.07 of the Victorian Planning Provisions.

The aims of WSUD are outlined below:

- Protect natural waterways within urban development

- Protect water quality of natural waterways through removal of pollutants at proximity to the source
- Reduce stormwater quantity and frequency of flows.
- Integrate stormwater treatments with the surrounding landscape
- Treat stormwater locally as it flows to reduce the requirement for large infrastructure ventures downstream
- Add value while reducing development costs and the overall cost of drainage infrastructure.

Water Sensitive Urban Design seeks to integrate Best Planning Practice (ie. design) with Best Management Practice (ie. technology) to identify the constraints and conditions of the proposed site and recommend a multi-faceted drainage system that will achieve the objectives required to comply with the SEPPs. This system will include measures across a spectrum of scale from allotment through to streetscape and precinct.

Initiatives at the allotment level have been and are still being investigated within the water industry, however there is still some uncertainty as to how to best factor in allotment level treatment into scheme based approaches. The current Building Code of Australia with respect to green star ratings provides the opportunity for new building to either install solar panel systems or rain water tanks. From a water perspective rain tanks at allotment level are beneficial to the receiving waterways in reducing volume and peaks. That being said to understand how these tanks will be operated and maintained in the long term is lacking authority control and assumptions made at the PSP stage could be unrealistic without the support of home owners, developers and government agencies.

The Bureau of Statistics 2010 results relating to rainwater tanks shows Australia has an approximate proportion of 30% rainwater tanks for every suitable dwelling. For property's less than 1 year old, 56.7% of dwellings have rainwater tanks. These statistics show that there is a huge potential to include rainwater tanks as part of a scheme, however due to lack of controls it is hard to define the benefits and end uses. For this reason rainwater tanks have not been included in the overall water quality assessment, however we believe there is potential for the industry to challenge this as the PSP progresses and greater levels of certainty can be assured.

WSUD is versatile and can be scaled to suit any situation to ensure our waterways remain healthy. As such we need to define the objectives of WSUD and further discuss the two major attributes, stormwater quantity and stormwater quality.

6.3.1 Stormwater Quality Approach

As previously discussed the recommendation for storm water quality treatment comprises:

- The incorporation of MWC Development Services Scheme (DSS) wetland systems into retarding basin locations.
- Treat every catchment to BPMEG prior to discharging stormwater into receiving waterways.

As such the Wyndham North study area water quality modelling was undertaken by Spiire with respect to determining the benefit of wetlands co-located within the retardation basins. Melbourne Water carried out an independent review of the water quality objectives to determine the obligations to MWC and the development community for the entire Wyndham North Study area.

Spiire's water quality analysis was determined through the use of a computer modelling software package MUSIC (Model for Urban Stormwater Improvement Conceptualisation). The model establishes a result based on parameters entered into the system and catchments created. The output indicates the performance of the treatment measures and the residual pollutant load generated once the catchment is treated.

This software enables us to optimise the size and arrangement of the selected treatment measures to produce an output that achieves the most favourable results in terms of stormwater quality, flow frequency reduction, flow rate minimisation, reliability, cost and size. These results have been achieved through utilisation of Water Sensitive Urban Design methods across a range of spatial scales.

The incorporation of sediment basins/wetlands along all creek corridors in order for developers to meet their obligations under Clause 56 creates the opportunity to integrate stormwater wetlands and frog ponds (along Davis Creek south of Sayers Road and along Werribee River) to supply habitat, treatment, alternative water supplies and visual amenity.

Whilst sediment basins/wetland in the corridor provides one element of the stormwater quality strategy, other distributed WSUD treatments will be required (eg rainwater tanks, rain gardens, swales, stormwater reuse etc). The corridor sediment ponds and wetlands will aim to deliver in the water quality objectives as per the BEMPG for the PSP in both Skeleton Creek and Davis Creek catchments.

All final infrastructure sizes and locations will need to be resolved through discussions with Council, MWC and the developers in the preparation of the DSS. There may also be the need for interaction with other authorities during the approval process.

The potential impacts (eg additional pollutant loads) of recycled water entering the stormwater system and downstream environs due to third pipe use for irrigation or other runoff-generating activities will need to be considered in accordance with the Victorian EPA and DHS "Dual Pipe Guidelines". It is expected that this issue will be addressed as part of the "Health and Environmental Management Plan (HEMP)" that will be prepared for the recycled water system for each scheme. Depending on the setup of the scheme different organisations may be responsible for managing components of the HEMP. The HEMP is a statutory process for obtaining approvals for dual pipe schemes throughout Victoria.

7. Flood Mitigation and Conveyance Considerations

As described earlier the Wyndham North precinct encompasses two major catchments:

- Skeleton Creek
- Davis Creek

A high level schematic design for conveyance and flood mitigation requirements will be developed, including the alignment and location of drainage reserves and location and size of retarding basins.

This analysis then forms part of the holistic Stormwater Management Strategy, which will bring the conveyance and flood mitigation requirements together with such elements as waterway health objectives.

7.1 Initial Criteria

At the onset of the project the GAA provided Spiire with indicative retarding basin/wetland locations within the four PSP areas as shown in Figure 4. The sizes were based on a rule of thumb of 500m^3/ha and as such no detailed modelling had been carried out to verify these locations. Our initial site visit reviewed each of the potential locations, while obtaining a good feel for the site topography and characteristics. Refer to Appendix A for all site photos.

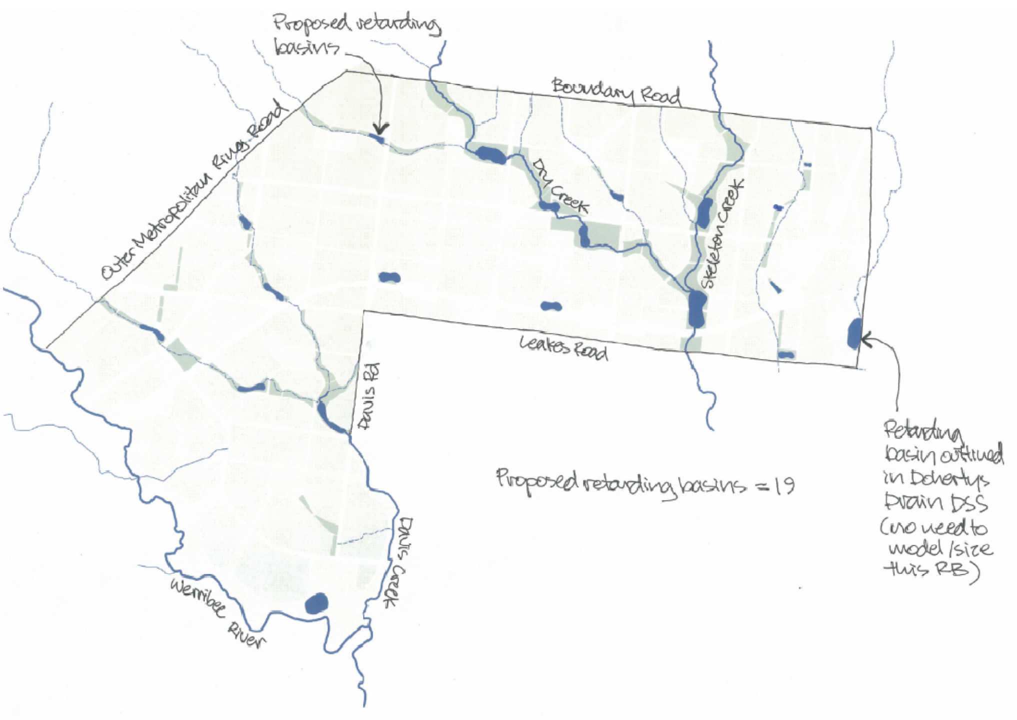


Figure 4 - Indicative retarding basin locations

As shown below in Figure 5 the approximate catchment areas for Skeleton and Davis Creek are 6900ha and 3350ha respectively. Melbourne Water provided Spiire with the current RORB models for both catchments. It was noted that the Skeleton Creek RORB model had been calibrated to a gauge station located on Skeleton Creek.

The far eastern part of the site has a small amount of catchment associated with the Doherty's Drain Development Services Scheme. An end of line retarding basin has been identified for this area within the existing scheme and therefore no further work is required to assess this basin location as shown in the Figure 4 above.

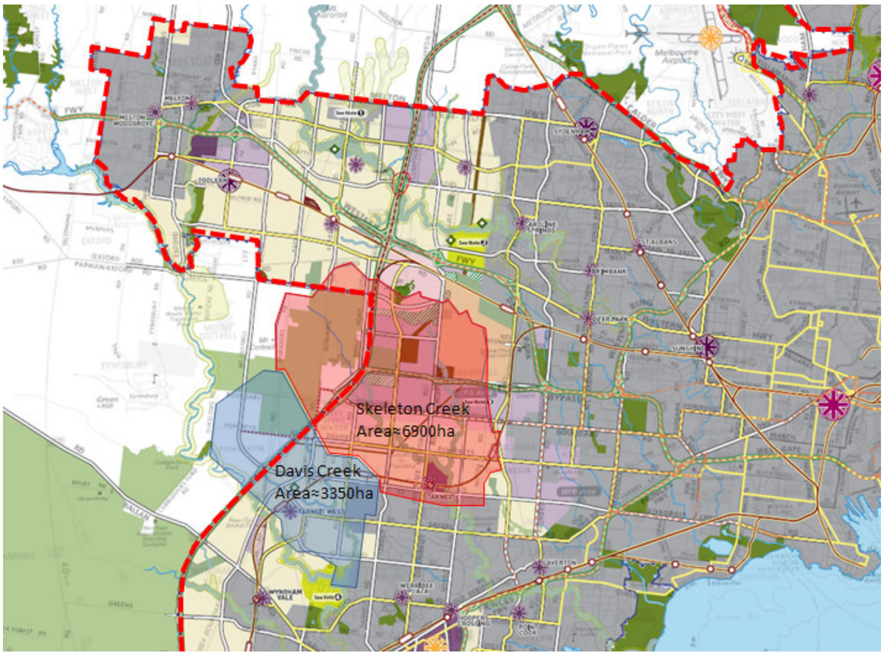


Figure 5 -Skeleton Creek and Davis Creek Catchment areas

Melbourne's growth area boundary is depicted by the red dashed line shown in Figure 5 above. The hydrologic modelling has allowed for all areas within this growth boundary to be considered as developed. Figure 6 below was also supplied to Spiire as the basis of underpinning where retarding basins could be located outside the Wyndham North area.

Typically urban development is set up on the basis that external land outside an investigation area will be delivered through the subject site as pre-developed flows. This was the initial intent of Figure 6 in that the flows from the upstream tributaries of Skeleton Creek flowing through Wyndham North would be retarded back to pre-developed levels. Davis Creek flows from outside the Wyndham North area where outside Melbourne's Growth area boundary and therefore this flow could be treated as rural without retardation.

As mentioned in section 1.1 multiple background reports were provided for this project. In particular a report prepared by Neil M Craigie dated 19th December 2011 for the Davis Creek Catchment and Arup's IWMS for Tarneit PSP 88-91 dated September 2011 provided a good insight to the previous work carried out within our investigation area.

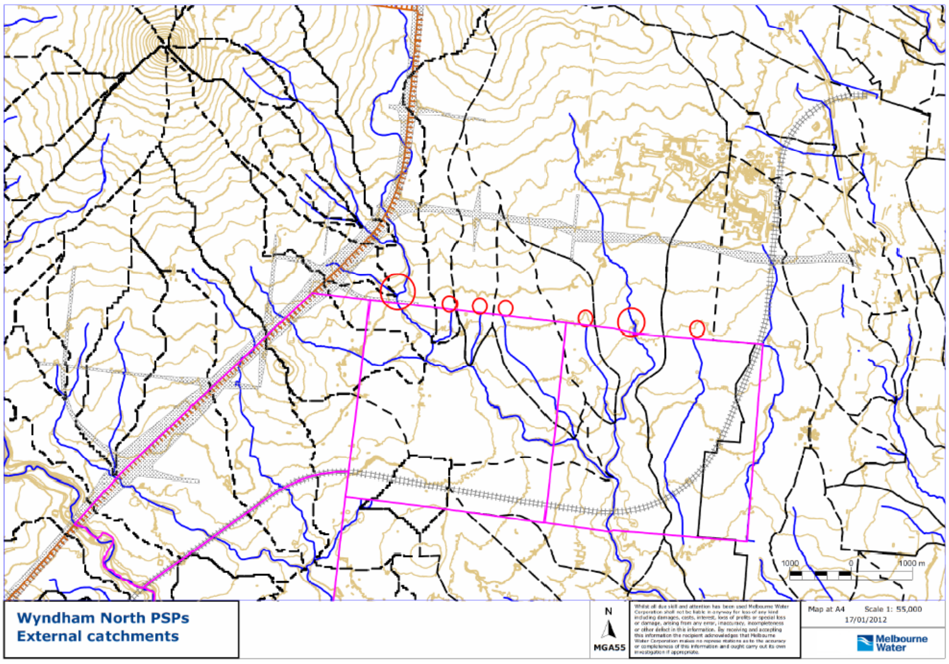


Figure 6 - Conceptual locations of retarding basins outside the Wyndham North Area

The information above formed the basis of the initial setup of this project however the refinement of the modelling process will be discussed in the following sections.

7.2 Retarding basin embankments

The basis of the embankments is to utilise major roads to provide embankments for the RB's as preferred by GAA. It is understood that ANCOLD requirements will need to be considered when designing these embankments.

7.3 Online vs. Offline constructed wetland systems

It is MWC's preference to construct offline wetland systems for stormwater treatment, however online wetlands may be permitted if they meet a number of criteria as per Figure 7 below. To enable a truly integrated system this criteria needs to be considered whilst determining the flood mitigation and conveyance measures as the required surface area of wetlands (where located within a floodway or retardation measure) may be a controlling factor based on meeting MWC criteria.

Due the environmental significance of Dry Creek, Davis Creek and Skeleton Creek, online systems are unlikely to be accepted as the works involved to incorporate such treatment infrastructure. The upper reaches of the above mentioned creeks will require modification as they are less defined creek lines and therefore online systems could be considered providing they meet MWC wetland guidelines.

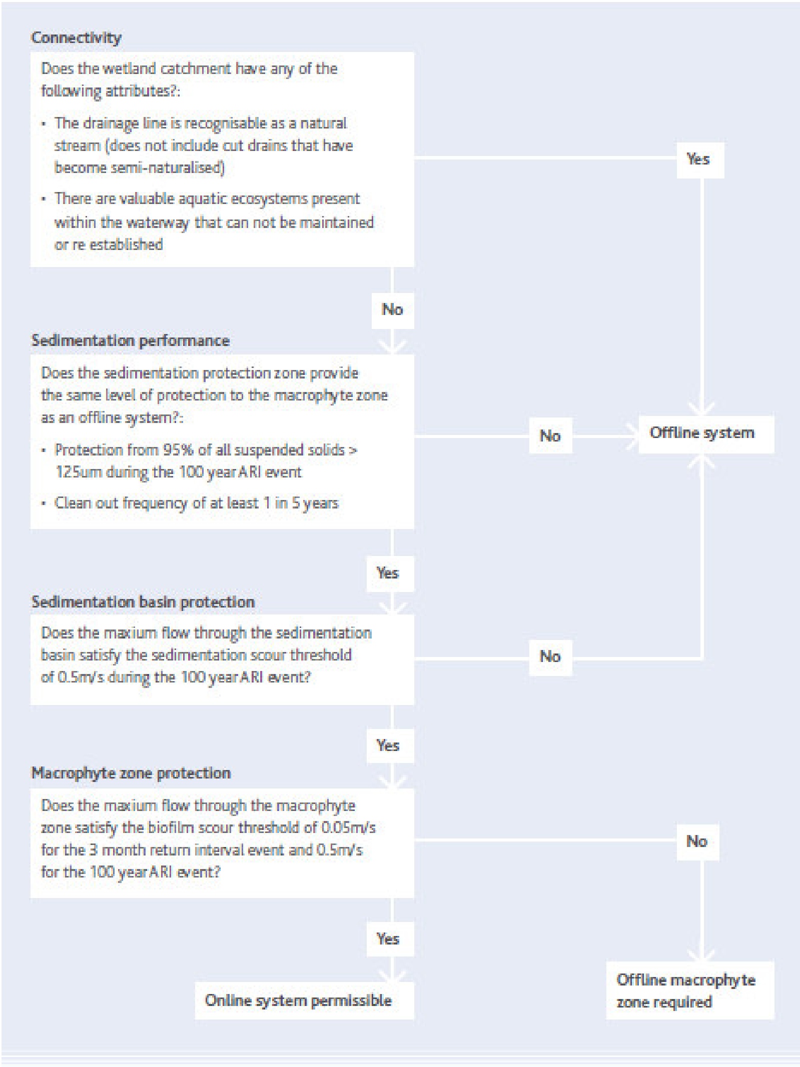


Figure 7 - MWC Decision making framework for locating wetland system offline or online

Wetlands are typically designed to treat the 3 month ARI peak storm events over a 72 hour detention time. More and more urban development areas are constructing ‘online’ wetlands which effectively treat flows from the whole of catchment.

These ‘online’ wetlands are sometimes placed downstream of other water quality treatments, retarding basins and within large catchments, all of which contribute to longer durations of base flows. This coupled with successive rainfall events causes increased frequency of inundation and detention times in excess of 72 hours. Additional stress on vegetation and reduced treatment performance therefore occurs, especially during establishment periods.

Spiire recognise this as an issue, however to mitigate this ‘online’ wetlands should be designed with consideration to the whole of the catchment, including the flow regime and critical storm durations. Inundation frequency analysis should be undertaken to ensure water depth in the wetland does not consistently remain at levels detrimental to the health of the vegetation. To deal with this at a strategy level Spiire will design all online wetlands with a reduced extended detention (300mm ED) depth to ensure plants are not drowned in their normal operating period.

Discussions to date with MWC and other stakeholders have discussed the performance requirements for sedimentation capture with respect to 100 year ARI storm flow capture efficiency as per Figure 7 above. Spiire believe this criteria is un-realistic and needs additional thought and/or modification. To capture 95% of sediments in a 100 year event is unwarranted; the key consideration is to ensure no re-suspension of sediments during these events. This should be dealt with by ensuring the sedimentation basin velocities are kept below 0.5m/s which is the next criteria in the MWC flow chart. For the purposes of this SWMS we will adopt 95% capture efficiency for 1 year ARI events and ensure the 100-year velocities do not exceed 0.5m/s over the sedimentation basin.

MWC's macrophyte zone protection criteria require a scour threshold of 0.05m/s and generally this criteria controls the wetlands width rather than the 100-year velocity criteria. The objective will be to meet this criterion for all online wetland systems.

7.4 Planning Waterway Corridor Widths

The purpose of the hydraulic analysis is to understand the necessary hydraulic widths for the various drainage reserves that are required to convey the 100-year ARI flows whilst understanding the various setback requirements. These reserves will be designed in accordance with MWC's Waterway Corridors Guidelines and Constructed Waterways Guidelines.

Depending on the existing landform, the waterway corridor may take the form of the existing natural waterway or be formed as a constructed waterway. MWC's Draft Waterway Corridors Guidelines discuss the natural (Figure 8) and constructed (Figure 9) waterways and the determination of the reference points and setback widths.

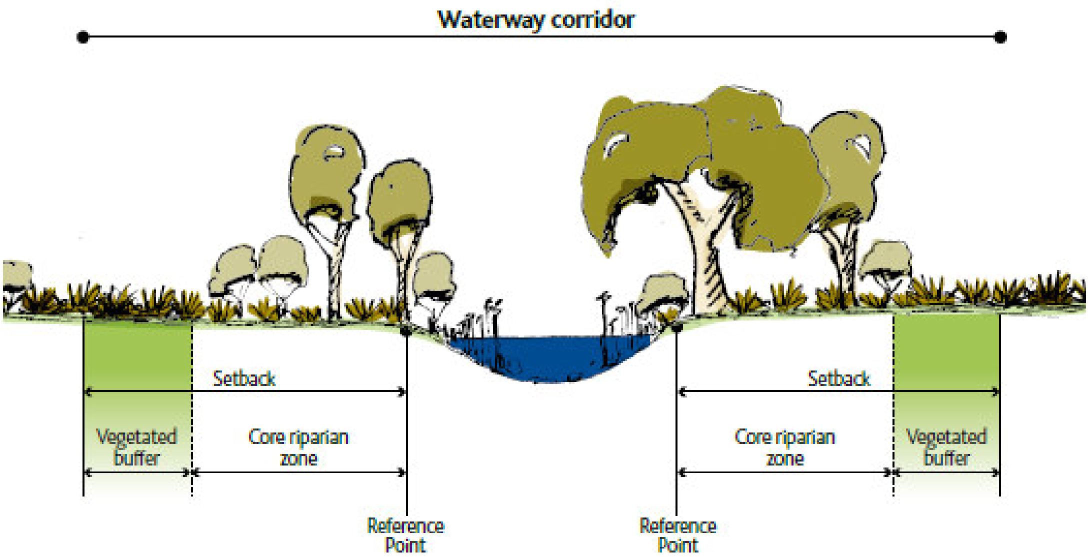


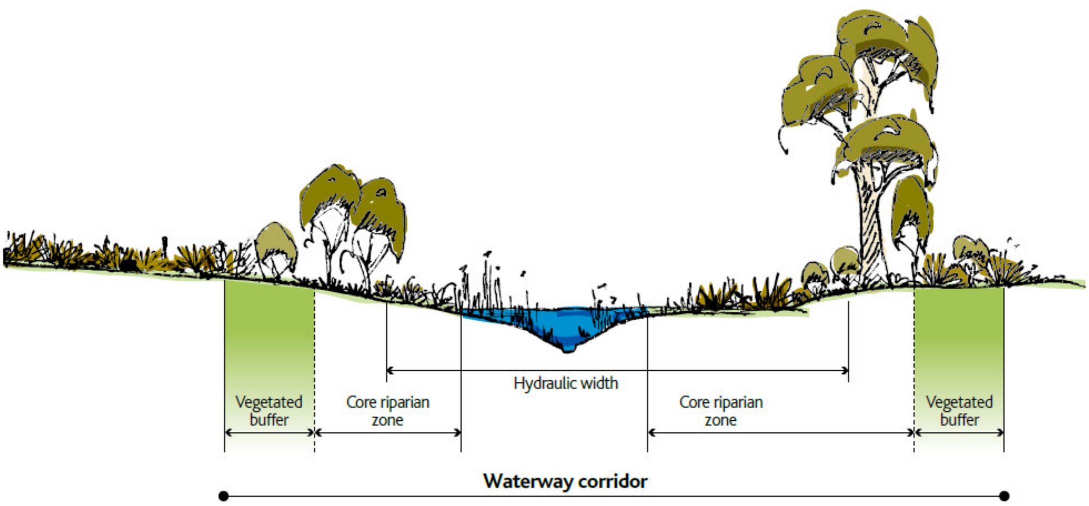
Figure 8 - Typical Natural Waterway Corridor based on MWC Waterway guidelines

In the case of the Wyndham North study area, Melbourne Water has defined a setback line for the natural waterways, which encompasses the additional setback requirements. These setback lines were derived by the hydraulic analysis carried out by Spiire, reviewing the natural topography and setback as per the MWC Corridor width guidelines.

Where retarding basins may elevate flood levels beyond existing conditions the reference points have been taken on the basis of pre-developed conditions and hence all setbacks are then from the un-retarded flood line. The waterway corridor width is then determined by which ever is greater i.e. the retarding basin extents or the waterway extents.

Where the natural waterways do not have sufficient capacity to contain the 100year ARI flows at appropriate depths, constructed waterway corridors will be necessary. These widths have been determined on the basis of determining the hydraulic width and then using the MWC draft waterway corridor guidelines to determine the overall waterway corridor width. As shown in Figure 9 the constructed waterway corridors are made of three different elements, vegetated buffer, Core riparian zone and hydraulic width.

Figure 9 - Typical Constructed Waterway Corridor based on MWC waterway guidelines



To determine the hydraulic widths a combination of HEC-RAS and manning's channel flow calculations (PC Convey) have been carried out based on existing channel grades and a formed channel to determine a suitable flow width. The Draft Waterway Corridors Guidelines give guidance on relating the hydraulic width to the corridor width.

The HEC-RAS model was developed for all the main reaches to understand their existing capacities based on conveying developed flows. A 100 year inundation line could then be produced to understand the hydraulics widths and whether it would be possible to retain the existing waterway or provide a constructed waterway on a hydraulic basis only. Factors such as flora and fauna have been considered above and beyond hydraulic requirements. All constructed waterway corridor widths have been sized based on section 9.2, Table 4 within the MWC Waterway corridor widths.

8. Skeleton Creek Catchment

Objectives and design approaches detailed in Sections 0-7 have been applied to the Skeleton Creek catchment. The following technical investigations have been undertaken to develop the SWMS recommendations:

- Hydrologic Modelling
- Hydraulic Modelling
- Water Quality Modelling
- Analysis of Results

This analysis then forms part of the holistic stormwater management strategy for the Wyndham North study area, which will bring the conveyance, flood mitigation and water quality requirements together with such elements as waterway health objectives.

8.1 Hydrologic Modelling

8.1.1 Design Rainfall

The design rainfall data used for Skeleton Creek RORB model was derived from AR&R Volume 2 as follows:

Table 1 - AR&R Design Rainfall Parameters

AR&R Parameter	Value
1hr 2yr	17.87
12hr 2yr	3.42
72hr 2yr	0.89
1hr 50yr	39.40
12hr 50yr	6.98
72hr 50yr	1.77
Skew	0.37
F2	4.29
F50	14.92
Zone	1

8.1.2 Skeleton Creek/Dry Creek Hydrologic Modelling

MWC have advised Spiire that the model has been calibrated to gauged stations on Skeleton Creek and therefore the RORB parameters should not be altered. The RORB parameters used within this catchment are as per Table 2 below.

Table 2 - Adopted RORB parameters

RORB Parameter	Value
m	0.8
Kc	25
IL	10
Roc	0.75

Refer to Appendix 2 for the RORB catchment plan for Skeleton Creek.

The Skeleton Creek catchment has 3 main waterways within it; Skeleton Creek, Dry Creek and Forsyth Road Drain.

Skeleton and Dry Creek meet at a confluence near the southern boundary of the PSP study area to form Skeleton Creek. The Forsyth Road Drain discharge point is further east than the Skeleton Creek discharge point, however it combines with Skeleton Creek further south of the PSP investigation area via a manmade diversion. Figure 10 below depicts the two discharge points on Leakes Road and the Forsyth Road Drain diversion confluence point.

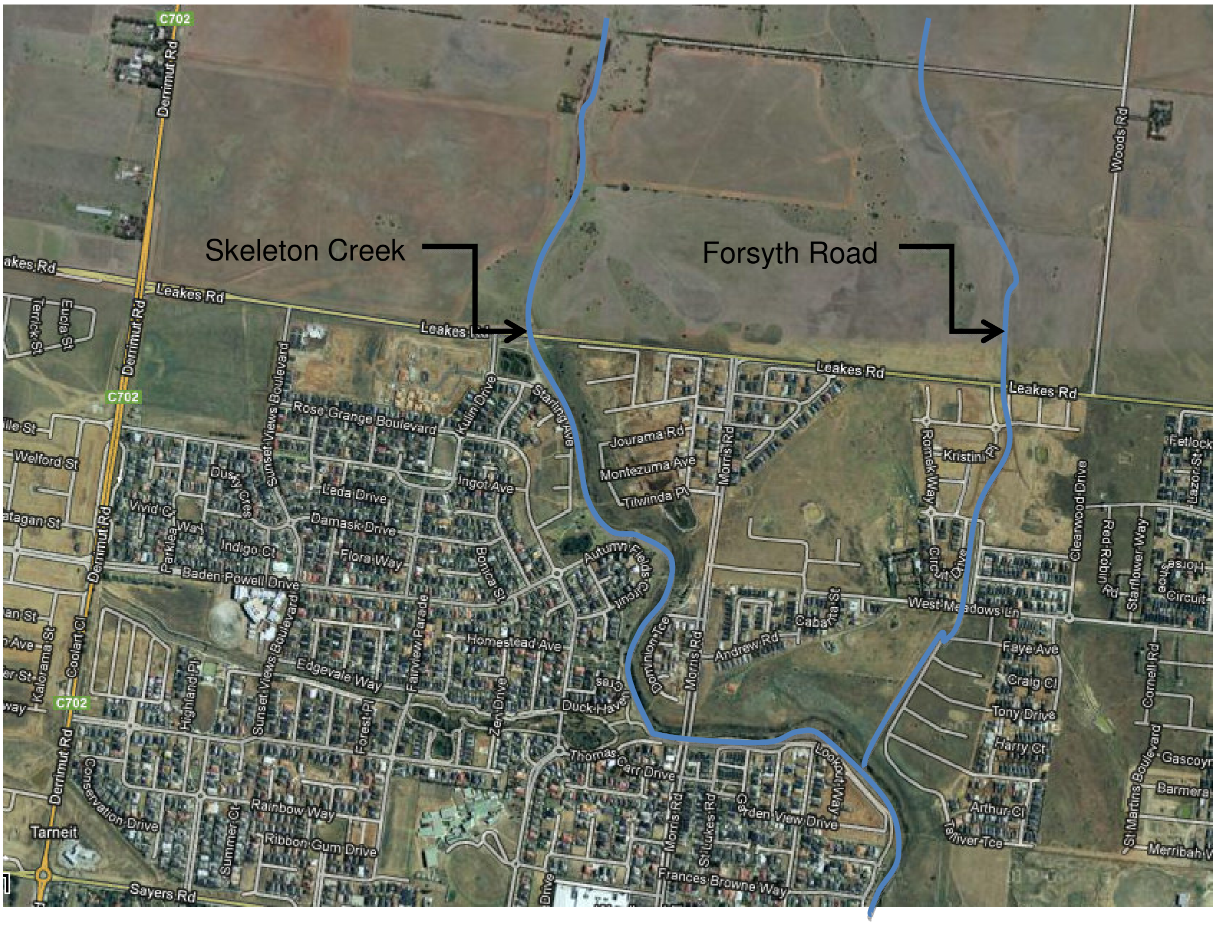


Figure 10 - Forsyth Road Drain Diversion South of PSP

8.1.3 Pre-Developed Flows

The pre-developed flows have been determined from the RORB model with all sub-areas and reaches set to pre-developed conditions in accordance with MWC requirements. The model was run to the parameters as above to give the following flow constraints at the PSP study area downstream boundary:

- Skeleton Creek: 139.3m^3/sec
- Forsyth Road Drain: 17.6m^3/sec

The Forsyth Road Drain is to be considered separately, so all further analysis and recommendation considers Skeleton Creek only. In developing the retardation options for the Skeleton Creek Catchment, Spiire also reviewed the flow changes downstream of the PSP boundary to understand if any significant increases were occurring as a result of the retardation provided upstream of the PSP study area boundary. Flows were taken from the pre-developed model at major road crossing locations for comparison with the flows in the developed conditions RORB model. The flows were identified at critical road overpasses downstream of the PSP area.

The existing (pre-developed) flows and their locations are as follows:

- Skeleton Creek at Sayers Road: 138.41m^3/sec
- Skeleton Creek at Old Geelong Road: 145.21m^3/sec
- Skeleton Creek at Princess Hwy: 145.95m^3/sec
- Skeleton Creek at Boardwalk: 147.44m^3/sec
- Skeleton Creek at Palmers Road: 162.97m^3/sec
- Skeleton Creek at Point Cook Road: 164.88m^3/sec

The HECRAS model for Skeleton Creek has been used to assess the impact of the changes in flood levels in the lower reaches of Skeleton Creek for all the options identified through during this project.

8.1.4 Calibration Review

Spiire reviewed the calibration of the RORB model and presented to MWC pre-developed flows for Skeleton Creek at the boundary of the PSP study area calculated by Rational Method and the DNRE method. These indicated pre-developed flows in the range of 69m^3/s - 104 m^3/s.

Spiire presented this to MWC to verify the calibration of the RORB model and expressed that the current calibration resulted in high pre-developed flows being produced. However, Spiire were instructed to adopt the original parameters as given by MWC as these were derived from gauged river stations and any revision to the calibration was not to be undertaken.

8.1.5 Retarding Basin Locations and Preferences

As mentioned in section 7.1 the original objective was to retard the upstream developed flows back to pre-developed levels at the northern boundary of the Wyndham North PSP area. This approach was deemed unacceptable as the results showed that minimal retardation was then required within the Wyndham North study area, with the majority of retardation structures required outside the PSP.

This approach did not consider the impacts of timing on flows as retardation without consideration of the whole of catchment leads to inefficient and ineffective infrastructure. This was demonstrated through the earlier options analysis Spiire carried out.

The proposed retarding basins have been located to be off the major waterways of Skeleton Creek and Dry Creek. The intention is to retard the catchment primarily at the tributaries upstream of the major waterways. The retarding basins have been located in such a way where possible to prevent retarding basins in series as this creates inefficiencies in retarding basin sizes. Similarly, locating retarding basins on tributaries protects mains waterways from excessive disturbance.

MWC also requested that the RBs be dispersed across the catchment to allow for water quality assets such as wetlands to be incorporated into the RBs where possible.

In consultation with the stakeholder group, Spiire was advised to consider avoiding RB locations across multiple allotments. This is primarily evident in the north-west area of the PSP, where the existing lots are relatively small when compared to other areas of the PSP area.

8.1.6 Stage-Storage and Land take

Where suitable the retarding basins have been sized based on the existing LiDAR contours. Stage-storage relationships have been provided in RORB.

Where the retarding basin is to be located on a constructed waterway and the existing landform could be modified to maximise the performance of a retarding basin, constructed stage-storage relationships have been adopted in RORB. The stage-storage relationships for constructed retarding basins have been calculated in 12D.

The land take areas include freeboard requirements above the 100year water surface of the RB. Where the stage-storage relationship for the RB is based on a constructed RB, the area is taken based on the area required to provide the corresponding stage-storage relationship.

The land take required for each RB is subjective and dependent on the final earthworks to be undertaken. For each retarding basin there is opportunity for refinement and should include maintenance access and landscaped amenity. The basins where wetlands have been proposed all have buffers of between 30-40m and have grades varying between 10 - 16% where functional and aesthetic treatments can be provided. This is to be rationalised during the detailed design of each RB progresses.

It is expected that storage-discharge relationships will be refined in the RORB model upon functional and detailed design of the retarding basins to ensure the outlet structures are designed appropriately whilst still meeting the overall catchments flow objectives.

8.1.7 10 year ARI Retarding Basin Analysis

In addition to the traditional 100 year ARI retarding basins, MWC requested Spiire to investigate the possibility of including 10 year ARI retarding basins. The purpose of the 10 year ARI basins was to enhance waterway health by mimicking the pre-developed flows in smaller ARI events. Figure 11 depicts the locations identified for 10 year ARI retarding basins.

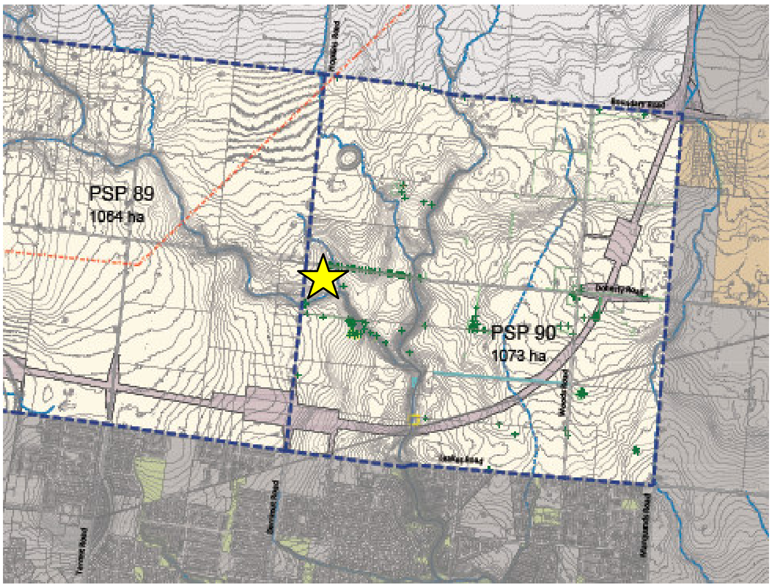


Figure 11 - 10 year ARI Retarding Basin Location

Table 3 below shows the 10 yr ARI retarding basin results from the RORB modelling undertaken. The basin was set with a spillway embankment 2m high and 30m long as it was felt that if the embankment was any higher it would excessively raise the 100 year flood level passing over the embankment. As per the results the maximum height of water over the spillway was kept in the order of 0.3m.

Table 3 - 10 year ARI basin performance

Performance of 10 year RB during 10 year ARI event					
Pre –Development (m3/s)	**Post-Developed Inflow (m3/s)**	**Outflow (m3/s)**	**Critical Duration (hr)**	**Elevation (m) Spillway set to 2m**	**Storage (m3)**
2.88	11.80	3.13	9	1.8	53,200
Performance of 10 year RB during 100 year ARI event					
8.09	26.67	12.32	9	2.3	*n/a*

The results showed that the 10 year retarding basins mimicked the pre-developed flow rates rather well for different duration as further shown by the output hydrographs shown in Figure 12 below. The downside to these basins was that they still required large storage volumes and in some cases were similar to the 100 year volumes.

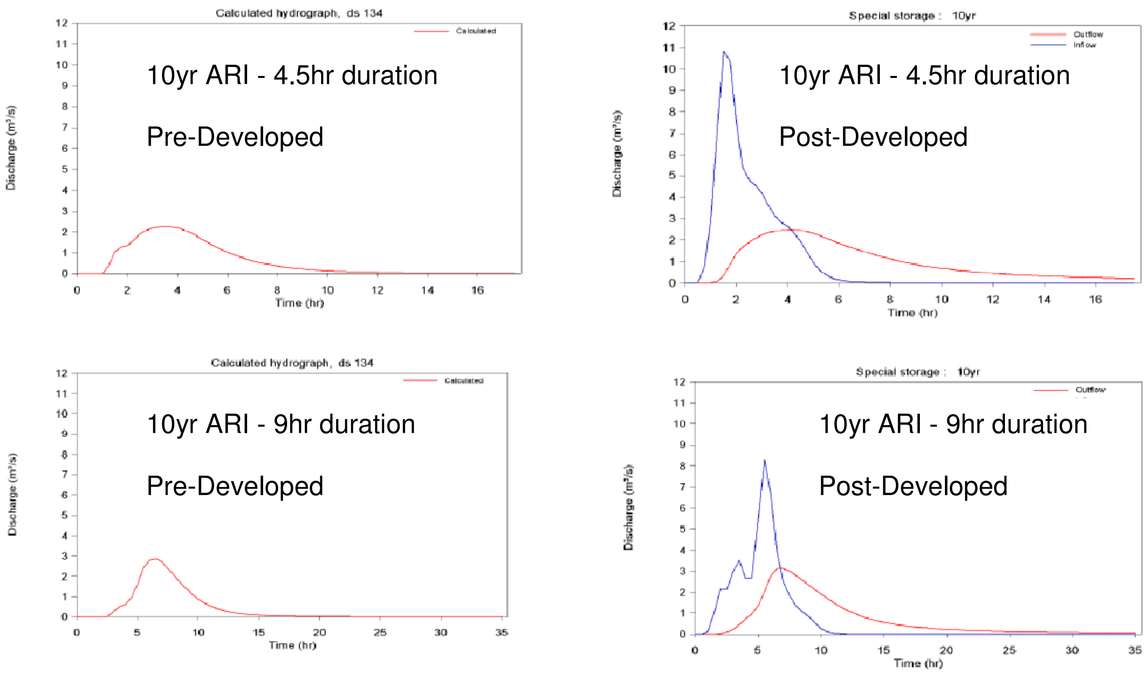


Figure 12 – Pre-developed Hydrograph vs. Post Developed Hydrograph

A further sensitivity was carried out to understand the volumes for varying ARI events as shown in Table 4 below. This demonstrated that the volume would reduce however they were still reasonably large. Refer to Appendix 4 for the 10 year basin sensitivity analysis.

Table 4 - Sensitivity Analysis

ARI	Required Storage (m3)	Base Size (ha)
10	53,200	2.8
5	46,400	2.3
2	34,500	2.0
1	30,000	1.6

We believe there is merit in the 10 year basins; however based on the size of assets required it was agreed with MWC that a different approach should be taken that may further reduce the developable land take up. This will be discussed further in water quality section 8.5.

8.2 Dry Creek/Doherty Road Re-alignment

The existing landform of Dry Creek means that it crosses Dohertys Road three times between Tarneit and Derrimut Road. In conjunction with Council, MWC and GAA a preferred option has been coordinated which considers the re-alignment of Doherty's Road and re-instating the existing creek alignment. The key factors in selecting this option were as follows:

- Achieve a balance in encumbrance and land required for drainage and road infrastructure between properties on the north and south side of Dohertys Road.
- Avoids significant earthworks to Dry Creek embankments by avoiding waterway channel realignment and therefore retains landscape and biodiversity values and avoids impact on aboriginal cultural heritage values.
- The creek re-alignment option primarily requires a bridge/culvert over Dry Creek, and otherwise generally follows existing landform/levels. This should require less fill to achieve a road above flood levels in comparison to upgrading the existing road which sits quite low in places given it moves in and out of the creek valley.
- Avoids significant impact on existing homes on the north and south side of the existing Dohertys Road reserve.
- Avoids impact on the two properties with heritage significance, being north of Dohertys Rd/west side of Dry Creek and south of Dohertys Rd/east side of Dry Creek. This option retains the current relationship of the buildings north of Dohertys Rd/west side of Dry Creek with the creek.

Other options did consider the re-alignment of the road on the north side of the creek, however this had two main draw-backs:

- This would impact on the existing home on the north side of Dohertys Road, east of the creek reserve.
- This would change the relationship of the historic buildings on the north side of Dohertys Rd/west side of Dry Creek with the creek.

It is understood that all stakeholders including Vicroads have accepted the proposal to re-align Dohertys Road and re-instate Dry Creek as shown in Figure 13.

Figure 13 – Dohertys Road Re-alignment

8.3 Derrimut Road Consideration

The catchment areas west of Derrimut Road have little topographical definition with respect to defining valleys for drainage conveyance infrastructure. Figure 14 below has further defined catchment boundaries to alleviate the quantum of flows generated at Derrimut Road.

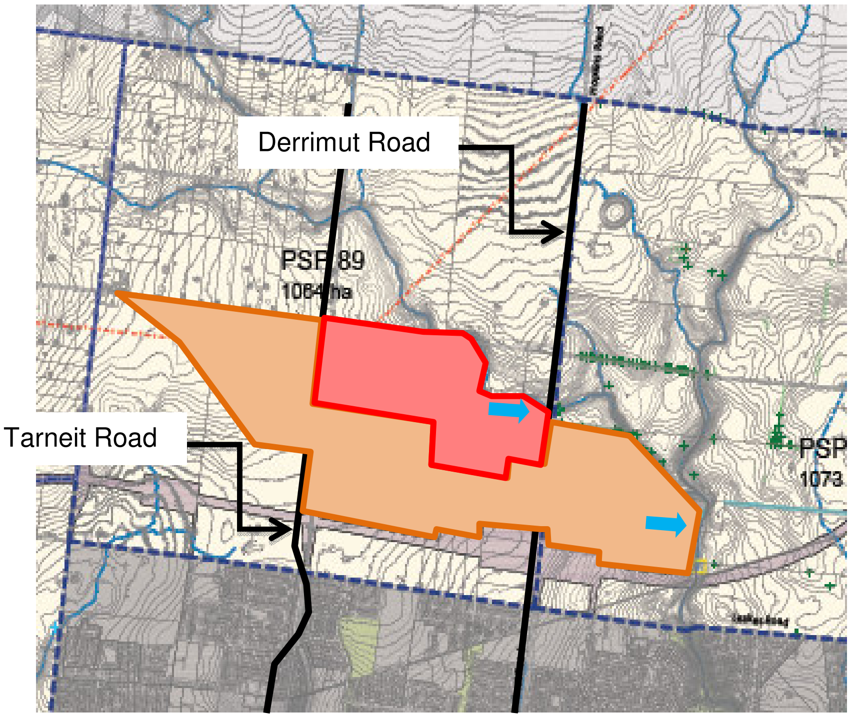


Figure 14 - Derrimut Road Catchment

A typical 16m wide local road with a constant grade of 0.5% has an overland flow capacity limitation of about 1.5m^3/s. A connector road with a reserve width of 26m has a capacity of 2.7m^3/s at the same grade. Where overland flow volumes are greater than the capacity of a single road, a well-designed road layout can allow for multiple roads to convey flows or an increased road reserve width to cater for additional flows.

We are relying on good urban design and road layouts to be sympathetic to overland flow requirements to enable this scheme to function. In the event of urban design layouts that do not take into consideration flood conveyance, the scheme infrastructure such as a waterway or large pipes may need to be extended upstream, west of Derrimut Road.

Wyndham City Council do not allow 1 in 100 year ARI flow events to cross major road infrastructure and in the case of this area, Derrimut and Tarneit Road would be considered major roads. At these crossings, overland flow would need to transition into a single collection point and pass under the road using appropriate infrastructure, before being redistributed into an overland flow/piped system.

The red catchment area should be able to discharge to Skeleton Creek prior to Derrimut Road. It is expected that road layouts will be sympathetic to the flow conveyance requirements of this area to move overland flows to the creek via multiple road reserves.

The orange catchment crosses both Tarneit and Derrimut Roads. The total catchment area is approximately 240ha, and initial calculations demonstrate overland flows of a magnitude too large to be conveyed in road reserves with a 5 year underground piped system through to the creek. To convey flows safely from Derrimut Road through the nominated town centre area and into Skeleton Creek, three options were considered:

1. Retardation upstream of Derrimut Road to reduce downstream conveyance infrastructure.
2. 100 year pipe from Derrimut Road to outfall location on Skeleton Creek
3. Drainage Reserve from Derrimut Road to Skeleton Creek.

For flow, storage, and pipe sizing calculations, see Appendix 5.

Additionally, it is possible that approximately 20ha across the north-western corner of the catchment could be directed to the red catchment and reach Skeleton Creek west of Derrimut Road. This would reduce the final peak flow rate, but the general concepts of the options will remain the same. This alteration of the catchment would need to be negotiated with all stakeholders.

Option 1

Option 1 proposes retarding storm flows west of Derrimut Road back to the pre-developed 100 year flow and then providing a 100 year pipe through to Skeleton Creek.

Due to the magnitude of the developed 100 year flow, this option would require:

- Gap flow conveyed in multiple road reserves west of Tarneit Road transitioning to a single collection point for the road crossing
- Gap flow conveyed in multiple road reserves west of Derrimut Road and transitioning to a single collection for retardation storage
- 43,000m^3 of retardation storage
- 100 year pipe from Derrimut Road to Skeleton Creek transitioning from 1350mm to 2100mm at the outlet
- Flow dissipation structure.

This option is not considered appropriate for a number of reasons.

The catchment west of Derrimut Road is relatively narrow and the potential to convey flow along multiple road reserves is limited. This may result in a requirement to upsize drains to a higher level of service (for example, 20 year ARI) than is standard. This type of design is not encouraged by Council.

The required retardation storage would extend across multiple lots on the western side of Derrimut Road, which has been discouraged through this project by all stakeholders. This would also add a significant cost to the total system. Additionally, retardation is not required in this catchment based on hydrologic modelling of Skeleton Creek meaning that this would be a redundant structure built purely to decrease downstream pipe size.

The pipe velocities (for a single 100 year pipe) approach 4m/s by the outfall to Skeleton Creek, and either a large dissipation structure or multiple pipes will be required to ensure that velocities into Skeleton Creek are not detrimental to the health of the waterway.

Option 2

Option 2 proposes conveying the 100 year flow in piped drainage from Derrimut Road to Skeleton Creek. This option will also require 100 year pipes upstream in order to collect overland flows throughout the catchment and mitigate the risks and logistical issues associated with a single collection point at Derrimut Road.

This option would require:

- Gap flow conveyed in multiple road reserves west of Tarneit Road transitioning to a single collection point for the road crossing.
- Gap flow conveyed in multiple road reserves west of Derrimut Road and transitioning to a single collection for the road crossing
- 100 year pipe from Derrimut Road to Skeleton Creek transitioning from 2400mm to 2700mm at the outlet
- Flow dissipation structure.

This option is not considered appropriate for several reasons.

The catchment west of Derrimut Road is relatively narrow and the potential to convey flow along multiple road reserves is limited. This may result in a requirement to upsize drains to a higher level of service (for example, 20 year ARI) than is standard. This type of design is not encouraged by Council.

Piping the full 100 year flow is not considered best practice from an environmental, flora, fauna or water quality perspective, and encourages excessive flow velocities. The pipe velocities (for a single 100 year pipe) approach 4m/s, and either a large dissipation structure or multiple pipes will be required to ensure that velocities into Skeleton Creek are not detrimental to the health of the waterway.

The pipe costs are also increased by this upsized infrastructure. These large pipes will be excessively deep and will attract greater construction costs.

Option 3

Option 3 proposes directing overland and piped flows into a waterway from Derrimut Road to Skeleton Creek. Due to the magnitude of the developed overland flow, this option would require:

- Gap flow conveyed in multiple road reserves west of Tarneit Road transitioning to a single collection point for the road crossing.
- Gap flow conveyed in multiple road reserves west of Derrimut Road and transitioning to a single collection for the road crossing
- 50m waterway between Derrimut Road and Skeleton Creek.

This option is considered the most appropriate possibility. A waterway provides the most natural conveyance path for storm flows and maintains an average velocity of under 1.5m/s at the outlet, resulting in a lower potential risk of damage to the health of Skeleton Creek.

The catchment west of Derrimut Road is relatively narrow and the potential to convey flow along multiple road reserves is limited. This may result in a requirement to upsize drains to a higher level of service (for example, 20 year ARI) than is standard. This type of design is not encouraged by Council. Another possibility would be to extend the waterway west of Derrimut Road for the distance required to allow a transition of overland flow from road reserves into it while maintaining a 5 year minor drainage system. It is for this reason that we are relying on good urban design and road layouts to be sympathetic to overland flow requirements to enable this scheme to function.

Although each option is possible, we believe that Option 3 will deliver better waterway health objectives by naturalising the conveyance of stormwater flows from Derrimut Road. On this basis we have recommended setting aside land for drainage corridors.

8.4 Hydraulic modelling

HEC-RAS modelling has been conducted for the Skeleton Creek catchment waterways and the required corridor widths based on this (see Appendix 6). It is possible that the waterway corridor widths along Dry Creek and Skeleton Creek can be reduced, however, this should be determined on a case by case scenario in consultation with Melbourne Water. HEC-RAS modelling was conducted and corridor widths have been obtained from these results. The results of the hydraulic modelling for the Skeleton Creek catchment have been presented in Appendix 6 – Skeleton Creek SWMS. Forsyth Road Drain has been excluded from this analysis as it will be considered separately.

8.5 Water Quality Modelling

Further to section 6, the Skeleton Creek water quality analysis identified a number of retarding basins which could co-locate a sediment basin/wetland. These basins effectively create online wetland systems therefore the criteria mentioned in section 7.3 regarding online wetlands need to be adopted.

Further to this MWC have adopted to treat all stormwater to Clause 56.07 requirements prior to discharging to the receiving waterway. The basis of this approach is as follows:

- Sub-catchments and their treatment systems have a mix catchment scales (less than or greater than 60ha) to enable clear ownership and equity between Melbourne Water and Council.
- It is assumed the water quality treatments will be sediment pond and wetland systems.
- MWC have carried out a preliminary sizing based on a percentage of catchment, therefore may require some additional buffer areas.
- The treatments have considered the title property boundaries and drainage pipes, as to be consistent with typical scheme methodologies.
- In a number of locations, additional diversion pipes will be shown to convey only low flows to the treatment systems.
- Melbourne Water has aimed to combine small catchments as much as possible to avoid many small treatment systems.

There is limited scope to modify the Skeleton Creek and Dry Creek drainage corridors due to their environmental significance. Although the 10-year retarding basin described in section 8.1.7 was deemed an inappropriate size, it was still deemed important that some level of flow management techniques were utilised.

8.6 Skeleton Creek SWMS Results

Based on the above analysis all the results have been depicted in the tables below and in Appendix 6 for the final SWMS for Skeleton Creek.

Table 5 - RORB results for Skeleton Creek

RB Location (Refer to plan)	Treatment Type	Stage (m)	Storage (m^3)	Area (m^2)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	IL (m)	Pipe No.	Size (mm)
R	Wetland	77.59	126,000	58,000	50.9	9.1	74	2	1200 & 1050
T	Wetland	63.77	78,100	40,000	29.2	5.8	61	2	900 & 1050
U	Wetland	63.03	44,500	27,500	18.6	3.9	60.5	2	825
V	Wetland	61.69	78,100	44,000	22.4	4.5	59	2	825 & 900
WLRB1	Wetland	59.33	255,000	105,9 00	36.6	13.2	56	2	900
WLRB2	Wetland	52.35	90,000	27,000	26.4	5.9	48	3	600 & 750(2)
WLRB3	Wetland	41.44	72,300	59,300	20.6	4.5	37	1	1050
Total				361,700					

Table 6 - MUSIC results for Skeleton Creek

Location	Treatment Type	Wetland Area	NWL	EDD
WLRB1	Wetland	25,000	56.0	0.3
WLRB2	Wetland	7,800	47.5	0.3
WLRB3	Wetland	29,000	37.0	0.3
Remaining Catchments	Wetland	TBC	TBC	TBC
Total				

The proposed option has the following flows downstream of the Wyndham North study area at the key locations as discussed previously in this report. The table shows the flows from the pre-developed model and the flows generated as part of the proposed option.

Table 7 - Skeleton Creek Downstream flow changes

Location	Existing Conditions	Proposed Strategy	% from Existing
Skeleton at Leakes Road	139.3	133.8	**-3.9%**
Skeleton at Sayers Road	138.4	136.5	**-1.4%**
Skeleton at Old Geelong Road	145.2	146.7	1.0%
Skeleton at Princes Hwy	145.9	147.4	1.0%
Skeleton at Boardwalk	147.4	149.0	1.1%
Skeleton at Palmers	163.0	163.0	0.0%
Skeleton at Point Cook Road	164.9	165.8	0.6%

As shown above, the discharge limit at the PSP study area boundary is met.

It is evident that the developed flows in some lower reaches of Skeleton Creek do slightly exceed flows from the pre-developed model. The HECRAS analysis indicates that the increase in flood levels is a maximum of 20mm. MWC has also reviewed the flow estimates against internal data and have deemed the results acceptable.

9. Davis Creek Catchment

Objectives and design approaches detailed in Sections 0-7 have been applied to the Davis Creek catchment. The following technical investigations have been undertaken to develop the SWMS recommendations:

- Hydrologic Modelling
- Hydraulic Modelling
- Water Quality Modelling
- Analysis of Results

This analysis then forms part of the holistic stormwater management strategy for the Wyndham North study area, which will bring the conveyance, flood mitigation and water quality requirements together with such elements as waterway health objectives.

9.1 Hydrologic Modelling

9.1.1 Design Rainfall

The design rainfall data used for Davis Creek RORB model was derived from AR&R Volume 2 as follows:

Table 8 - AR&R Design Rainfall Parameters

AR&R Parameter	Value
1hr 2yr	17.87
12hr 2yr	3.42
72hr 2yr	0.89
1hr 50yr	39.40
12hr 50yr	6.98
72hr 50yr	1.77
Skew	0.37
F2	4.29
F50	14.92
Zone	1

9.1.2 Davis Creek Hydrologic Modelling

A RORB model was supplied by MWC, the supplied RORB model had the following parameters m=0.8, kc=13.05, IL=15mm, RoC=0.6. This model was used by Neil M Craigie in his report Davis Creek Catchment “Strategic Drainage Proposals and Proposed Growling Grass Frog Corridor Requirements”.

To better represent the development within the Wyndham North study area, Spiire altered this RORB model by further refining the RORB catchment plan to better represent the proposed major roads and regional rail link. The model was therefore recalibrated and will be discussed in section 9.1.3, whilst the adopted parameters are shown in Table 9 below.

Table 9 - Adopted RORB parameters

RORB Parameter	Value
m	0.80
Kc	12.70
IL	15.00
Roc	0.60

Davis Creek catchment consists of two main tributaries, Davis Creek and an unnamed tributary. Refer to Appendix 7 for the RORB catchment plan.

9.1.3 Calibration Review

As mentioned above, the Spiire recalibration of the existing RORB model was achieved by:

- Using the rational method to estimate existing peak flow rates at Sayers Road, Hogans Road and the Davis Creek Outfall Location.
- Altering the supplied MWC model to represent existing conditions, and running the model with known parameters.
- Comparing flows from points 1 and 2 above.
- Altering the Spiire model to represent existing conditions and running the model with a kc/dav shift.
- Comparing flows from points 1,2 and 3 above and then altering kc to suit.
- Comparing to flow estimates in the Neil M Craiagie in his report Davis Creek Catchment “Strategic Drainage Proposals and Proposed Growling Grass Frog Corridor Requirements” (19 December 2011).

Table 10 shows the rational flow estimates, the existing MWC RORB model results and the calibrated Spiire model results for existing conditions.

Table 10 - Re-calibration comparisons

Location	Rational Estimates	"MWC Ex." Kc 13.05	Spiire Kc 12.7
OMR	16.32	15.8254	17.1
Cnr Davis and Sayers Rd	37.90	41.3601	41.4
Hogans Road	41.77	40.5585	39.95
Davis Creek Outfall	42.37	40.1723	40.17

During the calibration it was found that the kc/dav was not appropriate. Further analysis of the existing supplied RORB model showed that reach lengths in the upper catchment were different than that in other sections of the model. As such simply shifting the kc based on the kc/dav ratio resulted in flow estimates at Sayers Road being slightly low.

Neil M Craigie in his report Davis Creek Catchment “Strategic Drainage Proposals and Proposed Growling Grass Frog Corridor Requirements” (19 December 2011) suggested that the existing flow prior to urbanisation was in the order of 40 m^3/s.

Spiire believe the revised existing RORB model and calibration is reflective of the existing catchment conditions.

9.1.4 Pre-Developed Flows/Discharge Limits

The traditional approach for drainage strategies is to control discharge flows to pre-developed levels, unless it can be demonstrated that the increase in peak discharge from the catchment has no detrimental impacts downstream.

Davis Creek is within the Werribee River catchment. It is understood that extensive 2D modelling has been undertaken for the Werribee River and shows the impacts at the Davis Creek confluence point with respect to flood levels. We also understand that the Werribee River Lower DSS (Scheme 8001) has not incorporated retardation within the scheme as the aim is to ensure the peak flow events of the Werribee and the Werribee Lower DSS do not combine to increase flood levels at the critical duration.

Whilst the Davis Creek scheme will retard some flows the primary aim is to avoid increasing the downstream flood levels within Davis Creek itself and within the Werribee River. Melbourne Water have had a preliminary review of the Werribee River RORB model for the Werribee River Lower scheme and Davis Creek scheme and believe there is no impact on downstream flood levels.

Spiire created a HEC-RAS model for the Davis Creek catchment to establish the existing capacities south of Sayers Road. This model was created using LiDAR information provided by the GAA. Refer to Appendix 8 for the HEC-RAS results.

The Neil M Craigie report “Davis Creek Catchment, Strategic drainage proposals and the proposed growling grass frog corridor requirements” (19 December 2011) suggested the Davis Creek downstream flood level during a 1 in 100 year ARI storm event is 32.30. This level has also been used as a tail water control for the Spiire model. Neil’s report also concluded that peak flows up to 55m^3/s could be carried without raising flood levels at Hogans Road.

Spiire’s key findings from the HEC-RAS model are as follows:

- Spiire concur with Neil’s findings that flows up to 55 m^3/s can be carried within Davis Creek without raising flood levels at Hogan’s Road. (*This statement is prefaced in suggesting the existing flood level at Hogans Road would have been 32.33).*
- That at chainage 919.88 the freeboard to allotment levels appears to be only 80mm for 40m^3/s, this is decreased to 70mm for 55m^3/s. This is however controlled by the Werribee River downstream flood level.
- If the catchments do not coincide and the tail water can be regarded as normal depth, Davis Creek appears able to accommodate events up to and possibly exceeding 60m^3/s.

Spiire presented the above information to MWC with a recommendation to increase the discharge limit of up to 50m^3/s at the Sayers Road. MWC have since accepted this recommendation on the basis of the following benefits:

- Reduction in flood mitigation infrastructure required within the Davis Creek Catchment.
- Reduction in embankment wall height and hence risk to population.
- Does not have a detrimental impact on downstream properties.

9.1.5 Retarding Basin Locations and Preferences

Spiire considered multiple retardation location options within the Davis Creek catchment. Some of these retarding basins utilised the existing land features to provide storage, where as some of the basins were constructed basins requiring bulk earthworks.

It is not envisaged that the BPEMG will be met at the drainage corridor level, as such stormwater treatment would have to be provided within the development sites. Therefore further land take will be required to meet BPEMG.

In consultation with the stakeholder group, Spiire was advised to consider avoiding RB locations across multiple allotments. This is primarily evident in the north-west area of the PSP, where the existing lots are relatively small when compared to other areas of the PSP area.

9.1.6 Stage-Storage and Land take

Where suitable the retarding basins have been sized based on the existing Lidar contours. Stage-storage relationships have been provided in RORB.

Where the retarding basin is to be located on a constructed waterway and the existing landform could be modified to maximise the performance of a retarding basin, constructed stage-storage relationships have been adopted in RORB. The stage-storage relationships for constructed retarding basins have been calculated in 12D.

The land take areas include freeboard requirements above the 100year water surface of the RB. Where the stage-storage relationship for the RB is based on a constructed RB, the area is taken based on what area was required to provide the corresponding stage-storage relationship.

The land take required for each RB is subjective and dependent on the final earthworks to be undertaken. For each retarding basin there is opportunity for refinement and should include maintenance access and landscaped amenity. This is to be rationalised during the detailed design of each RB progresses.

It is expected that storage-discharge relationships will be refined in the RORB model upon functional and detailed design of the retarding basins to ensure the outlet structures are designed appropriately whilst still meeting the overall catchments flow objectives.

9.1.7 10 year ARI Retarding Basin Analysis

In addition to the traditional 100 year ARI retarding basins, MWC requested Spiire to investigate the possibility to include 10 year ARI retarding basins. The purpose of the 10 year ARI basins was to enhance the waterway health by mimicking the pre-developed flows in smaller ARI events. Figure 15 depicts the locations identified for 10 year ARI retarding basins.

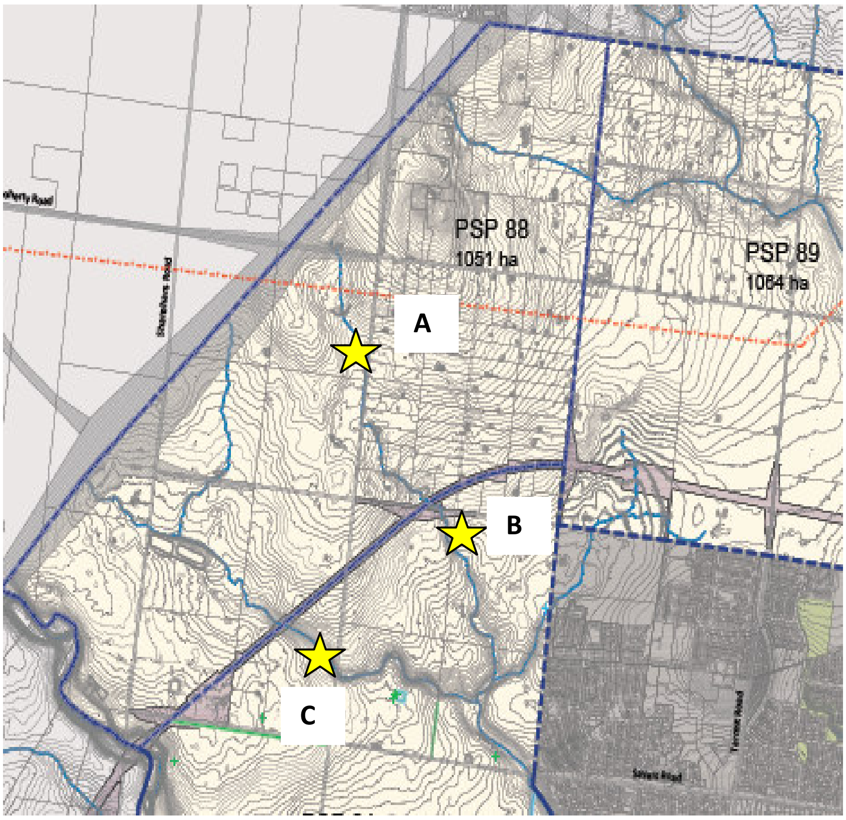


Figure 15 - 10 year ARI Retarding Basin Location

Table 11 below shows the 10 yr ARI retarding basins results from the RORB modelling carried out. The basin was set with a spillway embankment of 2m high and 30m long as it was felt that if the embankment was any higher it would excessively raise the 100 year flood level passing over the embankment. As per the results the maximum height over the spillway was kept to 0.3m.

Table 11 - 10 year ARI basin performance

Performance of 10 year RB during 10 year ARI event						
Location	**Pre – Development (m^3/s)**	**Post-Developed Inflow (m^3/s)**	**Outflow (m^3/s)**	**Critical Duration (hr)**	**Elevation (m) Spillway set to 2m**	**Storage (m^3)**
A	2.34	6.02	2.2	36	1.8	28,400
B	3.65	11.07	3.4	12		36,900
C	10.62	12.88	11.07	36		112,000
Performance of 10 year RB during 100 year ARI event						
A	5.72	12.52	7.055	9	2.2	n/a
B	8.44	23.44	12.22	9	2.35	n/a
C	25.30	30.46	29.77	9	2.32	n/a

The results showed that the 10 year retarding basins mimicked the pre-developed flow rates rather well for different duration as further shown by the output hydrographs shown in Figure 16 below. The downside to these basins was that they still required large storage volumes and in some cases were similar to the 100 year volumes.

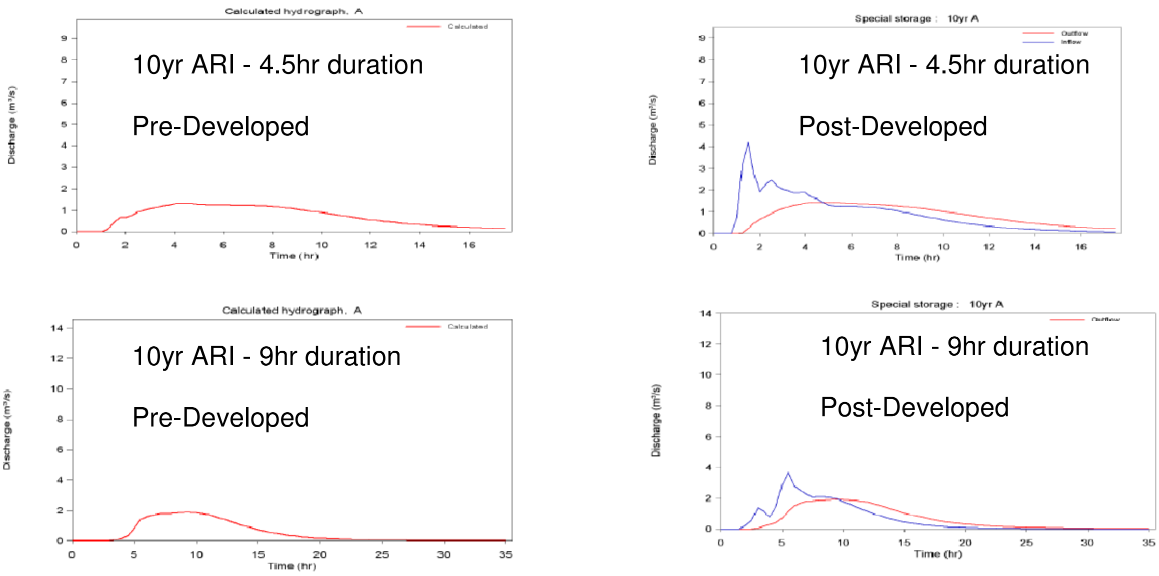


Figure 16 – Pre-developed Hydrograph vs. Post Developed Hydrograph

A further sensitivity was carried out to understand the volumes for a 1 year ARI event as for basin C as shown in Table 4 below. This demonstrated that the volume for 'C' only reduced slightly from the 10-year to the 1-year event. This basin had a considerable upstream catchment which played a major role in the in-efficient area/volume required. Spiire believe that 1-year retardation basins are best suited to small offline catchments rather than being placed online.

Table 12 - Sensitivity Analysis

Location	ARI	Required Storage (m^3)	10yr Storage (m^3)
C	1	90,400	112,000

As with Skeleton Creek we believe there was merit in the 10 year basins; however based on the size of assets required it was agreed with MWC that a different approach should be taken that may further reduce the developable land take up. This will be discussed further in water quality section 9.3.

9.2 Hydraulic modelling

The corridor width within the Davis Creek catchment has been determined as outlined within section 7.4. It is possible that the waterway corridor widths can be reduced where natural waterways occur, however this should be determined on a case by case scenario in consultation with Melbourne Water. The results of the hydraulic modelling for the Davis Creek catchment have been presented in Appendix 10 – Davis Creek SWMS.

9.3 Water Quality Modelling

Further to section 6, the Davis Creek water quality analysis identified no retarding basins which could co-locate a sediment basin/wetland. Although many options were investigated only one retarding basin was identified which will be present below. This basin is a category 1 habitat area for the Growling Grass Frog and MWC did not want to construct a wetland in the base of the natural channel due to its environmental values.

Further to this MWC have adopted to treat all stormwater to Clause 56.07 requirements prior to discharging to the receiving waterway. The basis of this approach is as follows:

- Sub-catchments and their treatment systems have a mix catchment scales (less than or greater than 60ha) to enable clear ownership and equity between Melbourne Water and Council.
- It is assumed the water quality treatments will be sediment pond and wetland systems.
- MWC have carried out a preliminary sizing based on a percentage of catchment, therefore may require some additional buffer areas.
- The treatments have considered the title property boundaries and drainage pipes, as to be consistent with typical scheme methodologies.
- In a number of locations, additional diversion pipes will be shown to convey only low flows to the treatment systems.
- Melbourne Water has aimed to combine small catchments as much as possible to avoid many small treatment systems.

The 10year basin in section 9.1.7 were deemed an inappropriate size, however it is still recommended to ensure that some flow management techniques are still utilised.

Therefore the objective is to construct sediment/wetlands basins with flow management techniques to reduce flow events for all key outlets to the Davis Creek. As mentioned previously the aim is to ensure treatment occurs prior to entering all waterways.

9.4 Davis Creek SWMS Results

Based on the above analysis all the results have been depicted in the tables below and in Appendix 10 for final strategy for Davis Creek.

Table 13 - RORB results for Davis Creek

Location	Treatment Type	Stage (m)	Storage (m^3)	Area (m^2)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	IL (m)	Pipe No.	Size (mm)
RB 1	RB	38.56	85,700	58,000	56.60	49.97	35	2	1500
							35	8	1050
Total				361,700					

Table 14 - MUSIC results for Davis Creek

Location	Treatment Type	Wetland Area	Sed Basin Area	Storage Volume	NWL	EDD
WL1	Sed Pond/ Wetland	27,900	3,100		NA	0.5
Ex WL*	Sed Pond/ Wetland	5,500	2,000		NA	0.5
Remaining Catchments	Sed Pond/ Wetland	TBC	TBC			
Total						

* Ex WL is the wetland east of Davis Rd and South of Leakes Rd

10. Integration of Growling Grass Frog Habitat

The hydrologic regime of the western plains provides relatively low rainfall and this, together with the geological constraints (ie rock), requires the careful consideration of water availability within the catchment. As a result it is essential to consider the synergies and potential integration of stormwater treatment measures (eg wetlands) and Growling Grass Frog wetlands / ponds.

Figure 17 illustrates a possible technique for integrating constructed stormwater treatment wetlands and the frog wetlands (along Davis Creek south of Sayers Road and along the Werribee River). The proposed stormwater wetlands will desirably be located outside the 1:10 to 1:100 year flood extent to ensure the GGF are protected from large storm events.

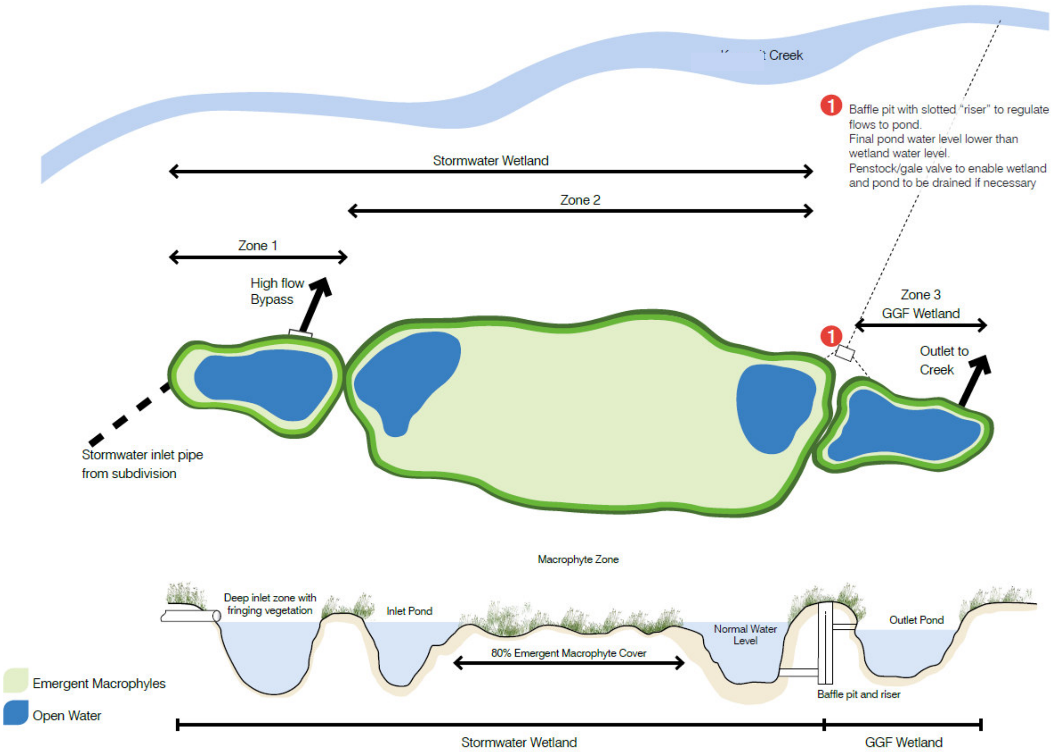


Figure 17 – Possible technique of integrating the GGF with Stormwater Treatments

11. Stormwater Harvesting

11.1 Climate Change

There has been an abundance of speculation as to whether we are in the midst of a climate change that threatens our way of life. While nothing is for certain, over the past decade we have certainly seen the pressures placed on our community when we are ill-prepared for an extended drought. Even climate change sceptics can agree that Australia's climate and rainfall are highly variable, with Australia having the lowest rainfall of the seven continents (besides Antarctica).

When looking at Victoria's rainfall over the latest 10 year period as outlined in the report "Our Water Our Future (2007)" prepared by the Victorian State Government, we see an alarming observation of a decline in our average rainfall. Among the other trends specified in the report there was a:

- Reduction in rainfall
- Reduction in river inflows
- Increase in temperatures
- Reduction in soil moisture content

To dismiss the facts or delay action against the effects of a climate shift could prove disastrous. Looking below (see Figure 18) at one of our driest winter seasons to date in 2006, it is imperative that we plan and take action towards securing our water's future.

Melbourne's Autumn/Winter rainfall over the past 10 years has also seen a steep decline from the Capital's average since recording began over 100 years ago. It is important to safeguard our community from these negative impacts as a result of the climate shift should it become an irreversible trend. Figure 19 indicates the reduction in flow into Melbourne's storages over the past decade as a result of reduced rainfall.

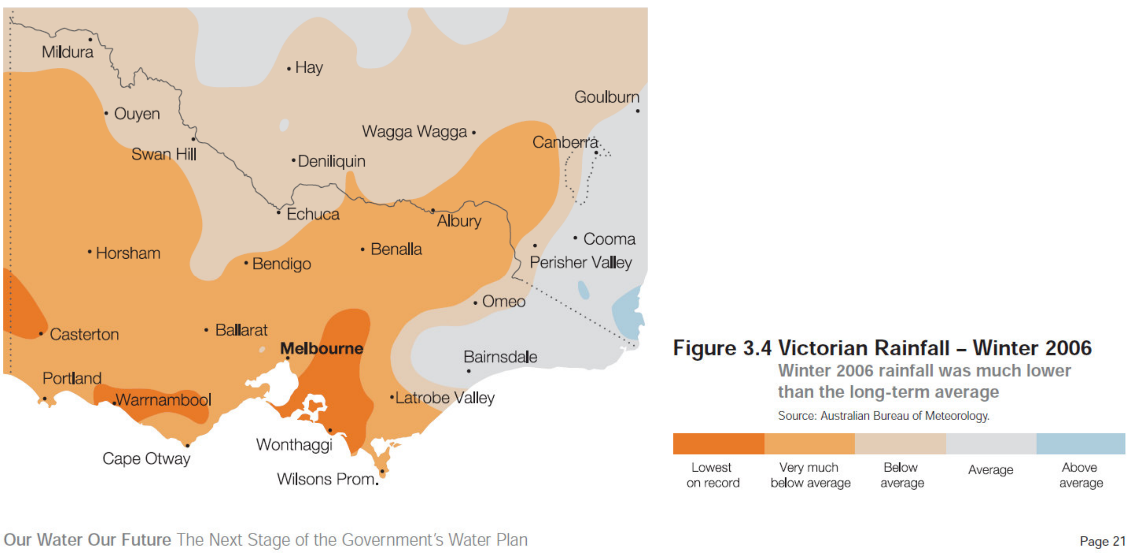


Figure 18 - Victorian Rainfall, Winter 2006

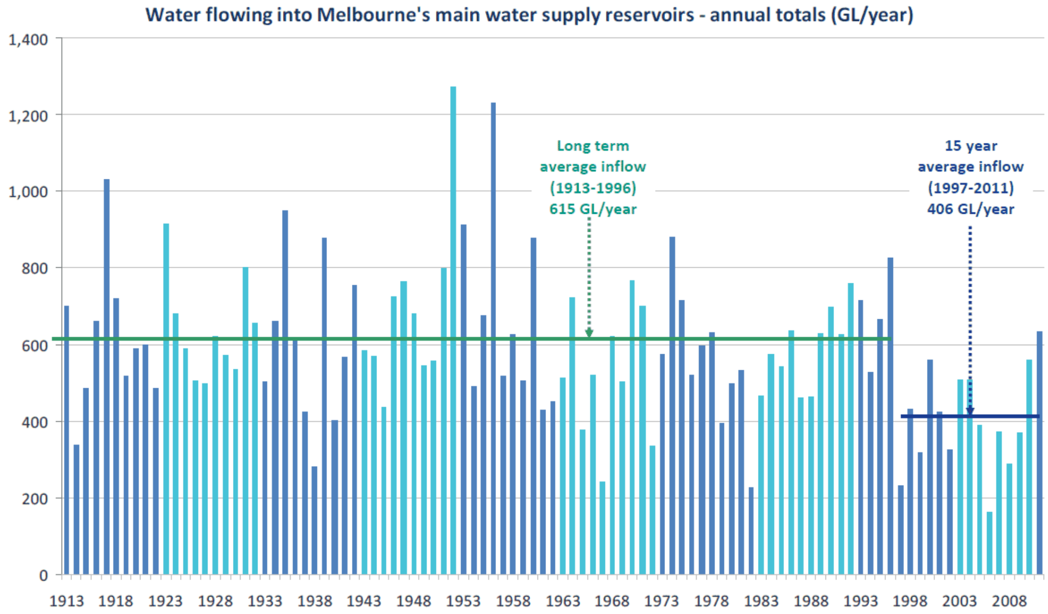


Figure 19 - Melbourne Storage Inflows, 1913-2011

As with any new development, there comes an increased demand for water and this reduction in our potable source combined with a surge in usage requires strategic water management planning and action to ensure a viable, sustainable development. The SWMS will ensure a flexible and adaptive stormwater system that is resilient to these rainfall trends by using rainfall data during the drought period between 1997 and 2011 for all stormwater harvesting modelling.

11.2 Rainfall Data

Due to the high natural variability associated with rainfall and its distribution there is really no such thing as an 'average year'. As a result it is important that the stormwater harvesting investigation be based on historical data over a period of time (ie years). As a result the stormwater harvesting balance requires continuous simulation modelling that will provide a closer representation of the range of conditions (e.g. dry periods) that is likely to occur into the future as discussed above.

The Bureau of Meteorology has a rain gauging site at Melbourne Airport, so to ensure the stormwater harvesting modelling is both representative of the location and anticipated climatic trends, Melbourne Airport rainfall data at a daily time step from 01/01/2000 to 01/01/2009 was used and typically have 6-7% lower rainfall than the long term average.

11.3 Evapotranspiration

Evapotranspiration (ET) is the other climatic data set that is needed to understand your water balance as it defines the amount of water that is lost from a surface. It therefore influences the potential supply of stormwater available as well as the demand for irrigation water.

The water balance model MUSIC (Model for Stormwater Improvement Conceptualisation) uses average areal potential evapotranspiration data. This refers to the evapotranspiration that would take place, under the condition of unlimited water supply, from an area so large that the effects of any up-wind boundary transitions are negligible and local variations are integrated to an areal average. The average areal potential Evapotranspiration (ET) data used for the MUSIC modelling were supplied by the Bureau of Meteorology for the Melbourne Airport. The data is used in the form of daily averages for each month and is as follows:

Table 15: Average Areal Potential ET (Melbourne Airport)

Calendar Month	Evapotranspiration (mm/day)
January	**5.0**
February	**4.46**
March	**3.23**
April	**2.17**
May	**1.29**
June	**1.0**
July	**1.13**
August	**1.45**
September	**2.17**
October	**3.55**
November	**4.5**
December	**4.52**

11.4 Demands

The demands identified for this investigation are as follows:

Table 16 - Stormwater Harvesting Demands

Demand Type	Demand
Active Open Space	**5ML/ha/yr**
Passive Open Space	**2ML/ha/yr**
Household Toilet Flushing	**35kL/hh/yr**

For all annual irrigation demands Figure 20 represents the monthly irrigation distribution.

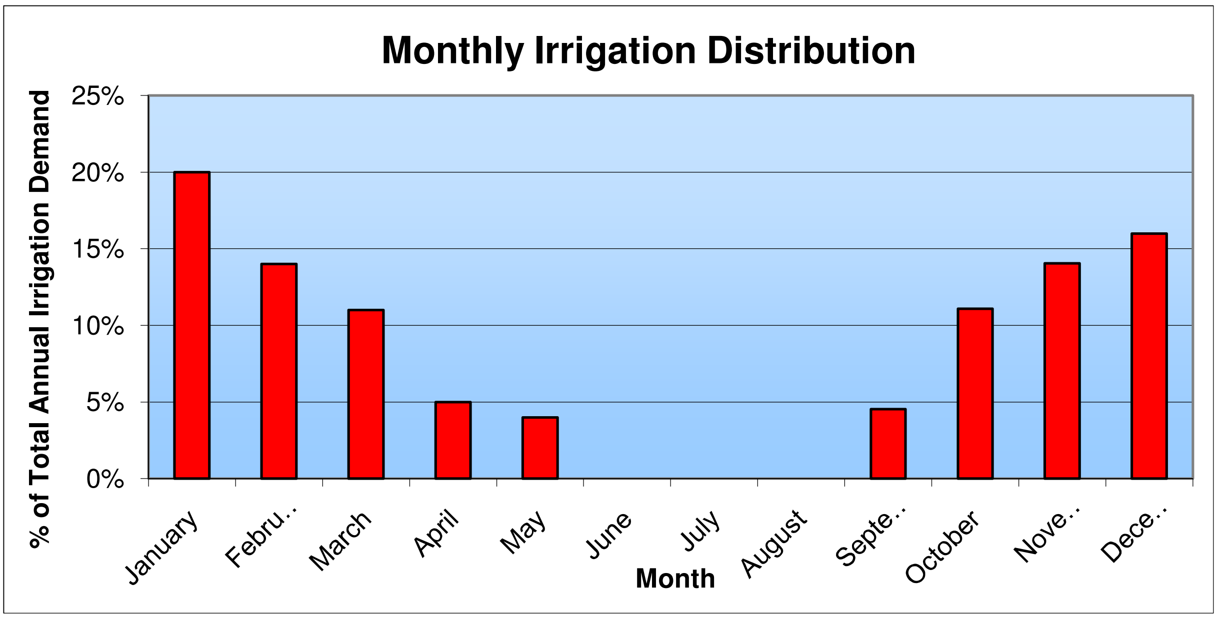


Figure 20 - Monthly Irrigation Distribution for Seasonal Demands

11.5 Water Balance

The Wyndham North study area site will produce an increased pressure on local water resources. The aim of this Stormwater Management Strategy is to recommend a reliable system where individual treatments recommended complement each other's strengths while minimising their weaknesses. By utilising multiple alternate water sources we are able to significantly reduce reliance on potable water and at the same time treat rainfall runoff to within best practice guidelines and control the flow frequency and flow rate at which it enters the waterways.

Stormwater runoff vastly surpasses Wyndham North's overall demand, however reliability of stormwater and rainwater harvesting systems can be still problematic due to vulnerability with respect to rainfall.

For example Figure 21 below illustrates that with increased tank size comes an increased reliability that the tank can service the irrigation requirements of the public open space within a section of the Wyndham North study area. The relationship between tank size increase and tank reliability is not linear, creating a point at which the levelised cost of increasing the size of the tank is not justified. For this Stormwater Management Strategy a reliability of 60% for the open space stormwater tanks is recommended with the additional 40% of demand to be supplied by treated sewer water (third pipe system). This will avoid constructing unnecessarily large and costly infrastructure that provides only a modest return, and optimise the use of the available alternative water sources.

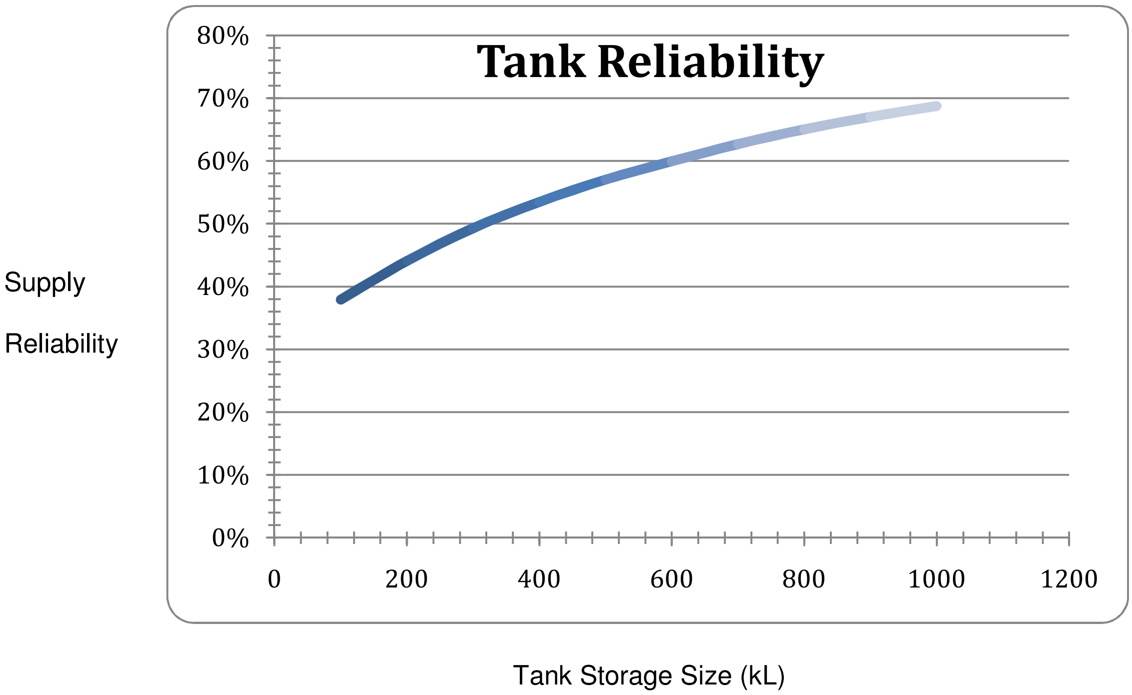


Figure 21 - Typical Stormwater Tank Size vs. Reliability Relationship

The combination of stormwater harvesting with the use of treated water from the third pipe system will seek to improve the reliability of supply such that the entire irrigation demand can be met by alternate sources to potable water. This scenario is achieved through supplying the base demand from stormwater harvested into large underground tanks that when storage levels are close to depleted, engage a recycled water irrigation system as a replacement until such time that its stormwater stocks are replenished. The tanks can be located in Public Open Spaces and thereby reduce the need to take up further developable land. This arrangement is illustrated below Figure 22 and has been modelled using the MUSIC software to identify suitable tank sizes to match the requested reliability based on location and irrigation demands.

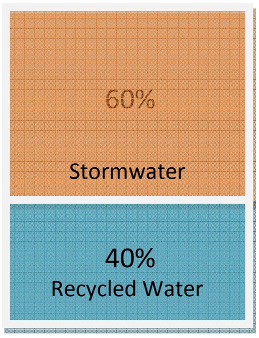


Figure 22 - Stormwater / Recycled Water Open Space Irrigation Balance

In addition to stormwater tanks harvesting runoff for the purpose of open space irrigation there also lies an opportunity to reduce potable demand at an allotment scale. Rainwater tanks could be located within the PSP as a potential alternative solution to potable water for internal and external allotment uses. A 2kL tank installed on an average sized home has the potential to supply hot water for a residential use at a reliability of 90% when this scenario is modelled in MUSIC. This seeks to further reduce potable demand and also aid waterway health objectives. For the purposes of this strategy we have excluded the analysis of rainwater tanks due to the limitations in controlling their use.

11.6 Stormwater Harvesting Options

Three concepts have been produced for the Wyndham North area. The three concepts can be referred to in Appendix 11.

The three concepts are:

11.6.1 Option 1

Option 1 involves a large end of line storage system for each of the Davis Creek, Skeleton Creek and Forsyth Road Drain catchments. These storages will be most effective where flows are diverted from the adjacent watercourse. However, flows can also be diverted into these tanks from the local catchment, or adjacent stormwater quality treatment asset. Diversion from the local catchment/WSUD treatment will involve greater infrastructure costs in order to connect a large enough catchment to the storage to meet the calculated demand.

The content of the end of line storages will be pumped to the top of catchment storage at Greek Hill. At Greek Hill, the stored water will be treated to Class A standard and distributed to the PSP area via the third pipe system.

The Greek Hill storage has been sized to supply the daily summer demand for irrigation of all active open spaces and daily toilet demand for the entire PSP area. The winter demand is much lower, meaning the storage will hold four days' worth of water without needing to be topped up across this season. If the Greek Hill storage is completely empty, it is expected to take around 20 hours to be filled by the supplementary tanks. The combined system of four storages, pump stations and connecting infrastructure provides a reliability of supply of 67% for active open space irrigation and toilet demand for the PSP area, or 1600ML/yr (see Table 17).

Table 17 - Option 1 Stormwater Harvesting Results

Tank Name	Volume (kL)	Reuse supplied (kL)	Reliability
RWT1	35000	1198000	
RWT2	10000	936000	
RWT3	10000	409000	
RWT4	12000	1616000	67%
		Total yield:	**1616000 kL**

11.6.2 Option 2

Option 2 involves the construction of 18 storages across the PSP area. These storages will be largely fed by local catchments and located adjacent to the active open space they are sized to irrigate, although in three areas pumping will be used to transfer captured water to an upstream location unable to sustain an appropriate harvesting scheme.

Stored water will be subjected to a fit for purpose treatment, and will be used only to satisfy irrigation demand in the specified area. This system will remain separate to any third pipe system.

The combined system of 18 storages, pump stations and connecting infrastructure provides a reliability of supply of 64% for active open space irrigation for the PSP area, or 872ML/yr (see Table 18).

Table 18 - Option 2 Stormwater Harvesting Results

Tank Name	Volume (kL)	Reuse supplied (kL)	Reliability
RWT1-1	7500	139000	65%
RWT1-2	410	-	
RWT1-3	410	-	
RWT2	4000	132000	72%
RWT2-1	410	-	
RWT2-2	410	-	
RWT2-3	310	-	
RWT3	2000	42000	68%

RWT4	1400	38000	67%
RWT5	950	25000	71%
RWT6	1500	30500	67%
RWT7	2000	38000	62%
RWT8	7000	53000	60%
RWT9	1500	28000	72%
RWT10	1500	52000	84%
RWT11	1600	36000	67%
RWT12-1	1500	79000	65%
RWT12-2	800	-	
		Total yield:	**872000 kL**

11.6.3 Option 3

Option 3 involves two buffer tanks and pump stations located on Skeleton and Davis Creeks transferring water via rising main to a single large storage on Greek Hill.

The 45ML storage has been sized to hold four days' worth of water during January, the month of highest demand. The winter demand is much lower, meaning the storage will hold twelve days' worth of water across this season. Water is treated to a Class A standard at this location and held in the 12ML storage prior to being distributed via the third pipe network throughout the PSP area for the irrigation of all active open spaces and to meet toilet flushing demand in all dwellings.

This option is based on opportunistic water collection at the location of the pump stations – it is expected that water will be transferred to the Greek Hill storage whenever there is flow available and the storage is not full.

The combined system of pump stations, storage and connecting infrastructure provides a reliability of supply of 60% for active open space irrigation and toilet demand for the PSP area, or 1450ML/yr (see Table 19).

Table 19 - Option 3 Stormwater Harvesting Results

Tank Name	Volume (kL)	Reuse supplied (kL)	Reliability
RWT1	45000	1450000	60%

11.6.4 Option cost

A high level cost estimate has been conducted for each of the three options presented. For a detailed breakdown of the cost estimate see Appendix 11. The total cost of each option is estimated as:

- Option 1 - $30,000,000
- Option 2 - $34,000,000
- Option 3 - $28,000,000.

12. Conclusion

The Wyndham North SWMS has been prepared in collaboration with all stakeholders, which considers the various objectives outlined within this strategy. The strategy ensures there is no detrimental impact to downstream environments and integrates the objectives of flood mitigation with stormwater quality and quantities.

The Davis Creek and Skeleton Creek will meet the Clause 56 requirements from an overall perspective, however the strategy still lends itself for developers and Council to come up with innovative solutions to enable various solutions at different spatial scales.

The stormwater harvesting concepts will provide City West Water with high level concepts to determine the potential of using stormwater as an alternative water source at a regional scale. The report still recommends the use of Rainwater tanks at the allotment level.

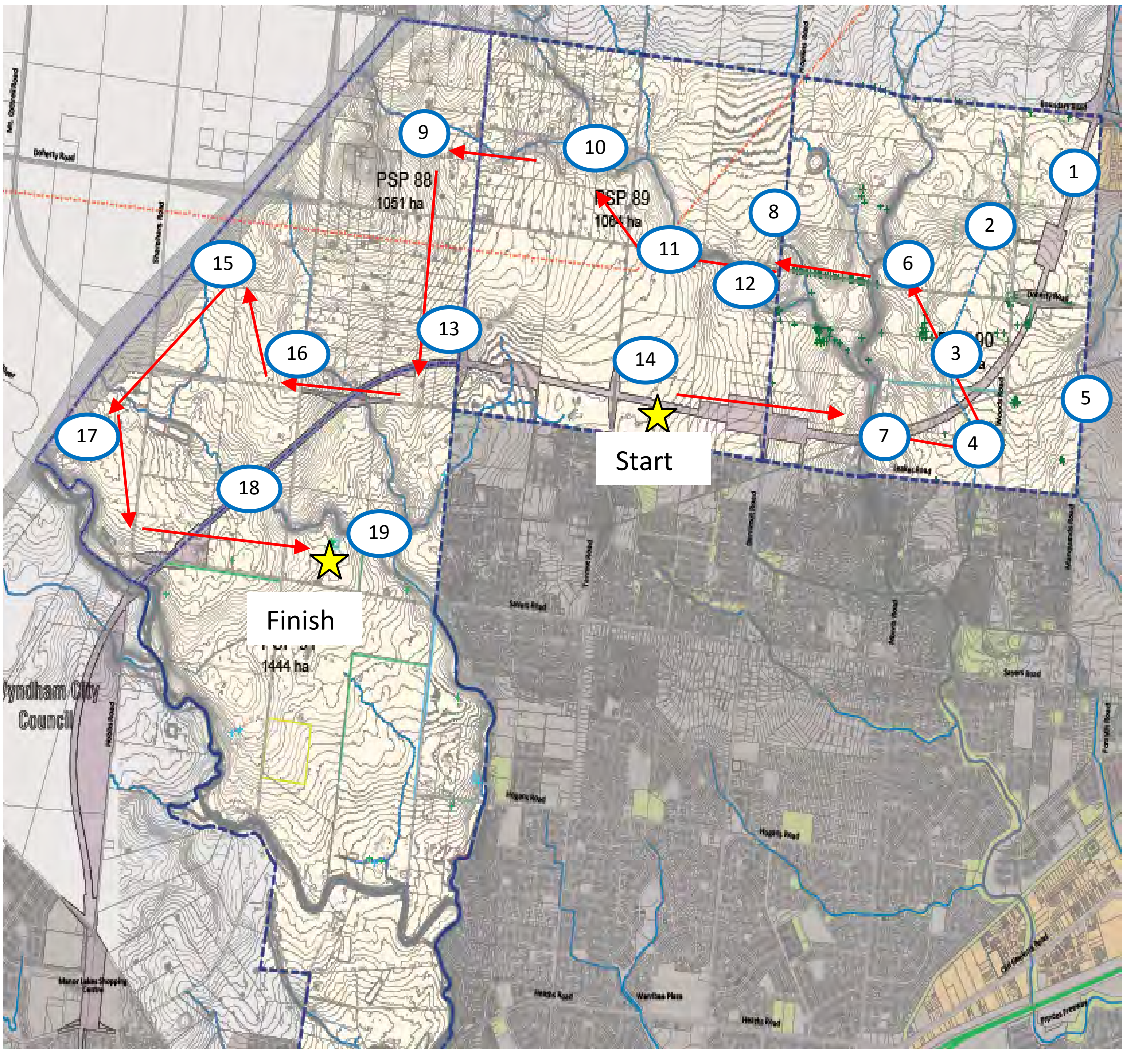
Appendix 1

Site Visit Photo Log

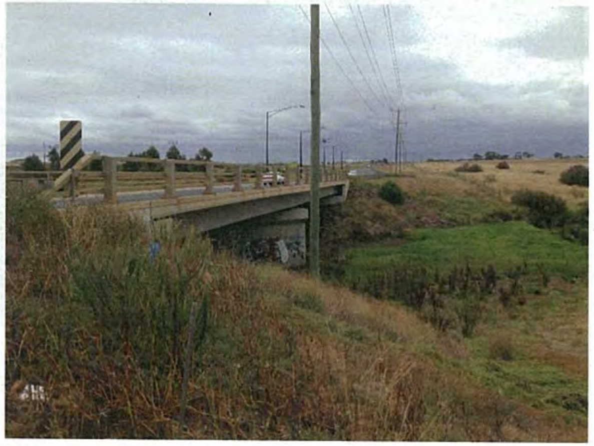
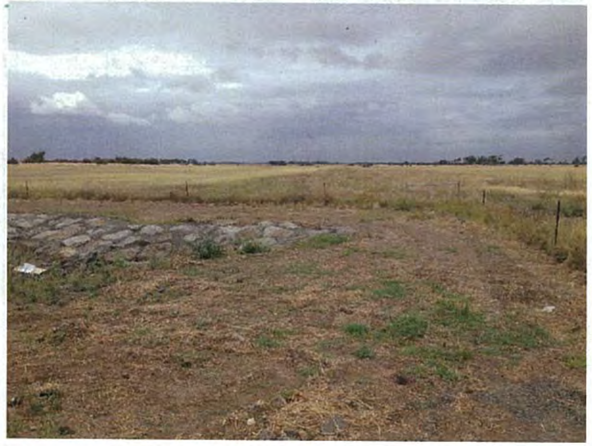
spiire

Title / Subject	Wyndham North PSP Site Visit
Site Meeting Date	20/2/2012
Attendees	Mark Brennan, Dane Logan, Leigh Holmes, Jonathon McLean

The site photos are in order of the path we took during our site visit. See below the site visit path.





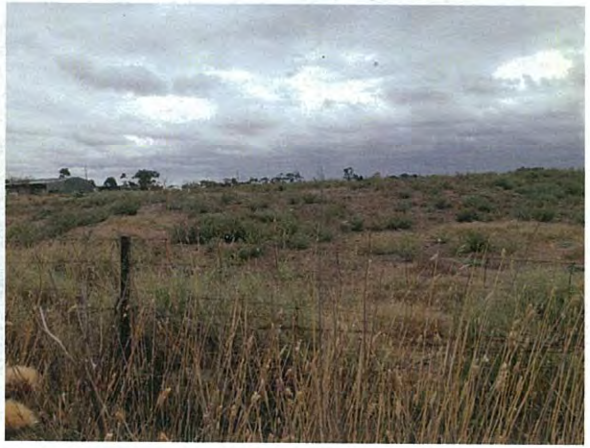
























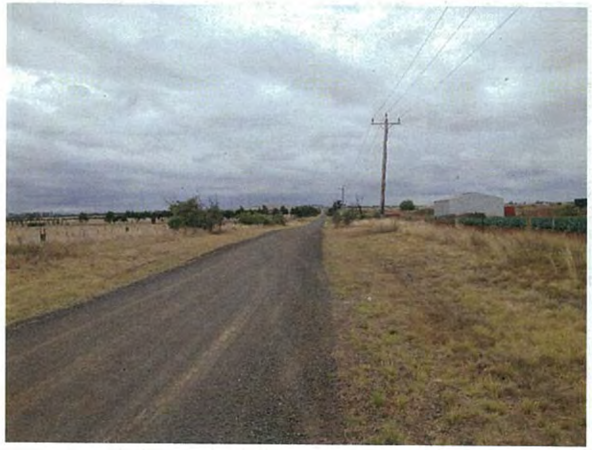








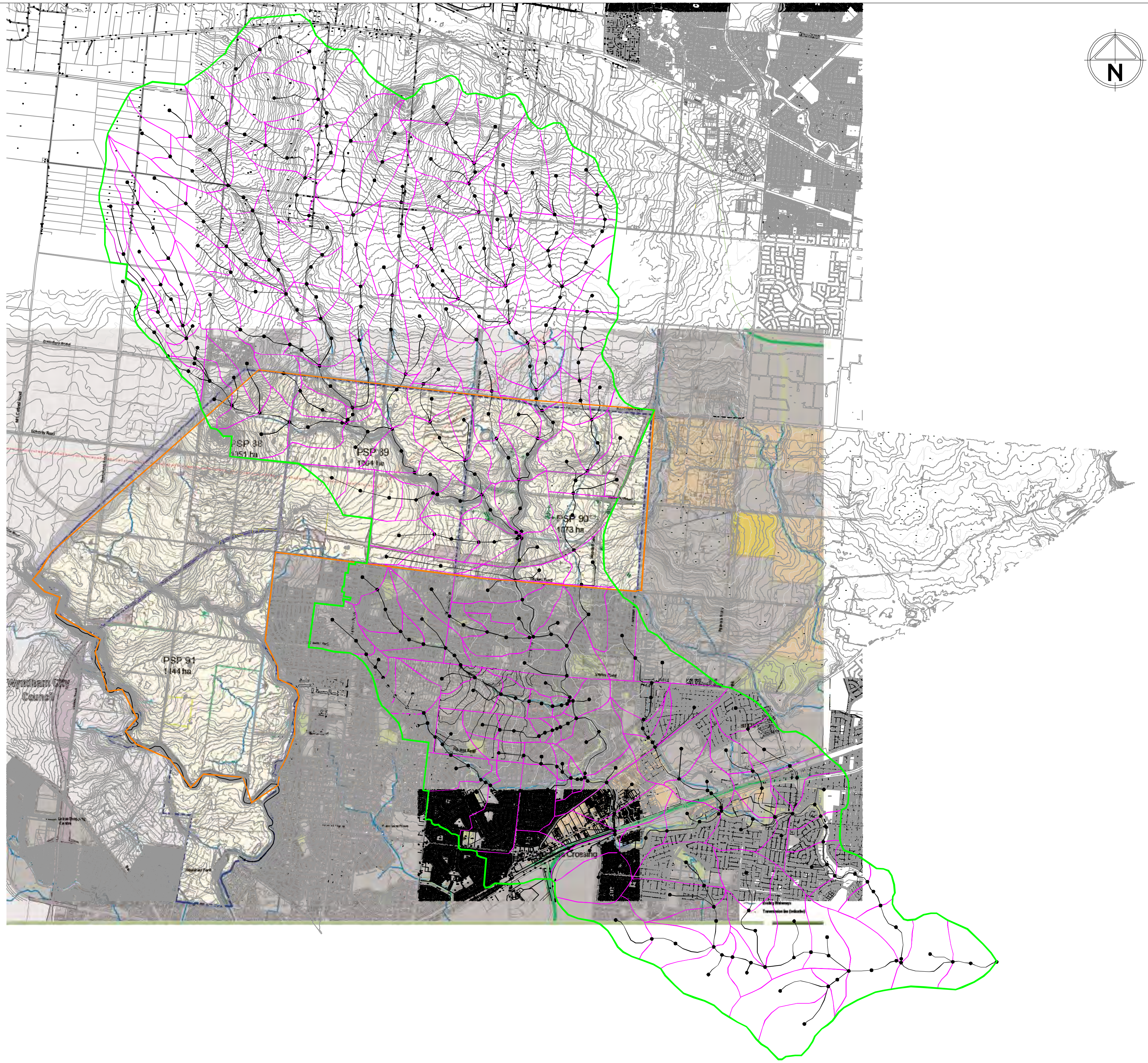




Appendix 2

Skeleton Creek RORB Catchment Plan



Rev	Amendments	App'd	Date
B	AMEND BACKGROUND IMAGE AS PER GAA COMMENTS	LH	22/05/13
A	ISSUED FOR DISCUSSION	LH	12/09/12

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file location G:\13\137654\ACAD
plotted by Mitch Johnson plot date 24/5/2013 3:51 PM

Standard Drawing RDA1 - Version 20120911

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Vertical 0 400 800 1200 1600 2000

Designed PENDING
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Checked PENDING
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Authorised PENDING
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Map Reference MELWAY 359 J8
Sheet Number F01
Drg Status PRELIMINARY

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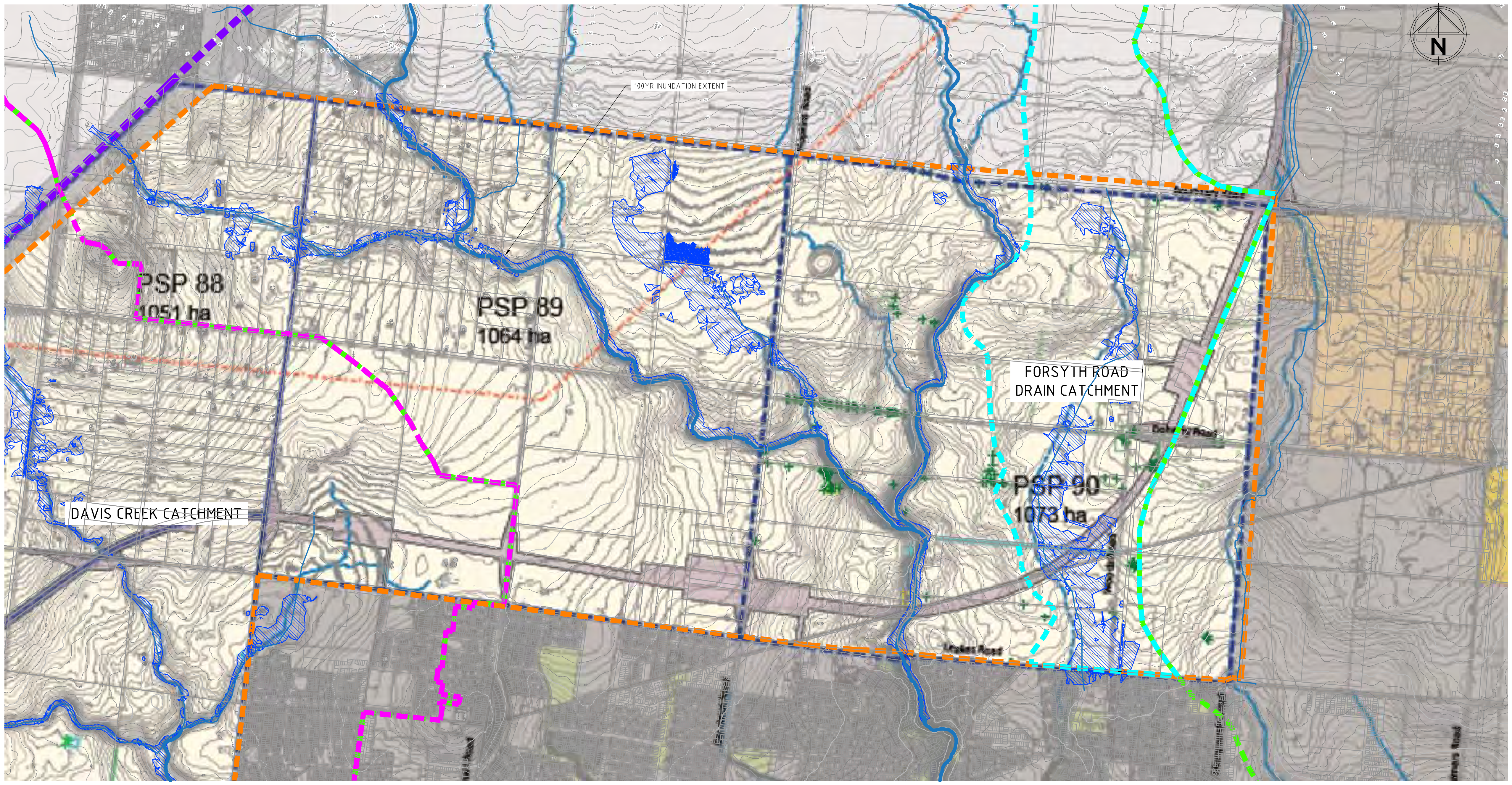
Innovation in Infrastructure

WYNDHAM NORTH
PRECINCT STRUCTURE PLAN
RORB CATCHMENT PLAN
SKELETON CREEK
GROWTH AREAS AUTHORITY
MUNICIPALITY

Rev **B**
Drg No **137654F01**

Appendix 3

Skeleton Creek Inundation Mapping



SKELETON CREEK INUNDATION LAYOUT PLAN

(SCALE 1:15000, A1)

LEGEND

100YR ARI INUNDATION AREA

WYNDHAM NORTH PSP BOUNDARY

SKELETON CREEK CATCHMENT BOUNDARY

DAVIS CREEK CATCHMENT BOUNDARY

FORSYTH ROAD DRAIN CATCHMENT BOUNDARY

LIMIT OF URBAN GROWTH BOUNDARY

Rev	Amendments	App'd	Date
B	AMEND BACKGROUND IMAGE AS PER GAA COMMENTS	L.H.	22/05/13
A	ISSUED FOR DISCUSSION	L.H.	29/10/12

file name 137654M00.dwg layout name S1-1
file location G:\13\137654\ACAD
plotted by Mitch Johnson plot date 24/5/2013 9:56 AM
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Standard Drawing RDA1 - Version 20120911

Scale 1:15000,A1
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Vertical 0 150 300 450 600 750

Designed PENDING
PENDING
Checked PENDING
PENDING
Authorised PENDING
PENDING

Map Reference MELWAY 359 J8
Sheet Number 1-2
Drg Status PRELIMINARY

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WYNDHAM NORTH PRECINCT STRUCTURE PLAN
SKELETON CREEK INUNDATION PLAN
GROWTH AREAS AUTHORITY
MUNICIPALITY

Rev B
Drg No 137654S1-2

Appendix 4

Skeleton Creek 10yr RB Sensitivity Analysis

Skeleton Creek Semi Final

10 Year ARI

Catchment	Pre Development	Post Developed Inflow	Outflow	Critical Duration	Required Storage Approx	Elevation	Pipes	Comments
	(m^3/s)	(m^3/s)	(m^3/s)	(hr)	(m^3)	(m)	No x (m)	
	2.88	11.8	3.13	9	53200	1.8	1 x 1.2	Arbitary at present

100 Year ARI

Catchment	Pre Development	Post Developed Inflow	Outflow	Spillway Width	Spillway Height	Depth over spillway
	(m^3/s)	(m^3/s)	(m^3/s)	(m)	(m)	(m)
	8.09	26.67	12.32	30	2	0.3

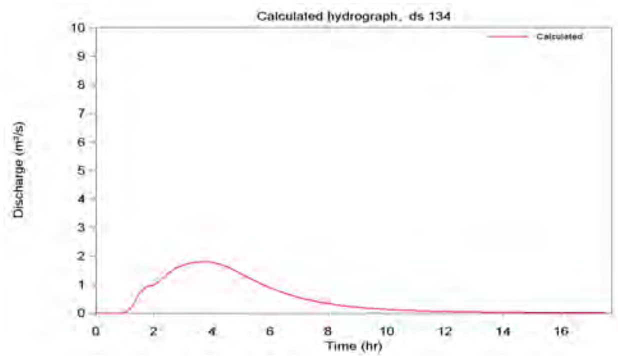
ARI	Required Storage (m^3)	Base Size (Ha)
10	53200	2.8
5	46400	2.3
2	34500	2.0
1	30000	1.6

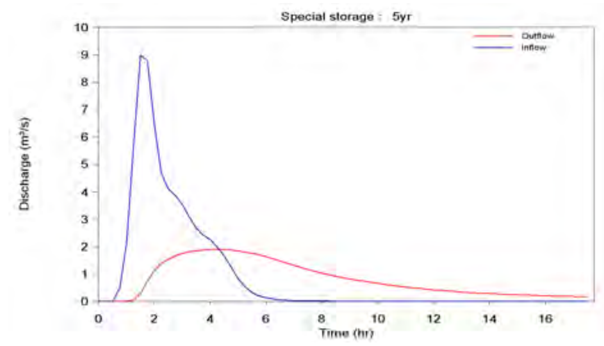
5 Year ARI

Catchment	Pre Development	Post Developed Inflow	Outflow	Critical Duration	Required Storage Approx	Elevation	Pipes	Comments
	(m^3/s)	(m^3/s)	(m^3/s)	(hr)	(m^3)	(m)	No x (m)	
	2.35	9.52	2.45	9	46400	2	1 x 0.9	Arbitary at present

100 Year ARI

Catchment	Pre Development	Post Developed Inflow	Outflow	Spillway Width	Spillway Height	Depth over spillway
	(m^3/s)	(m^3/s)	(m^3/s)	(m)	(m)	(m)
	8.09	26.67	13.9	30	2	0.37



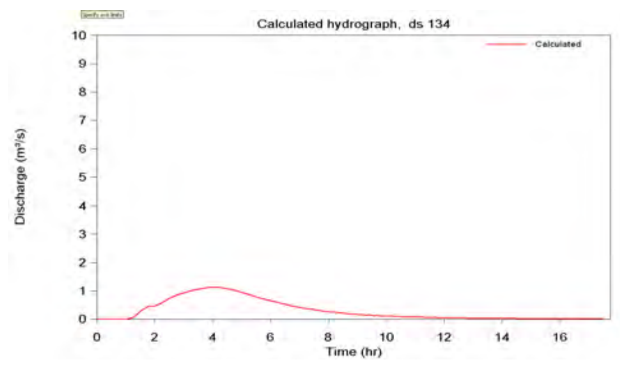


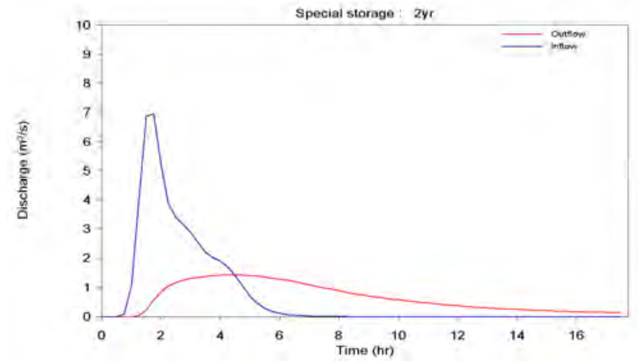
2 Year ARI

Catchment	Pre Development	Post Developed Inflow	Outflow	Critical Duration	Required Storage Approx	Elevation	Pipes	Comments
	(m^3/s)	(m^3/s)	(m^3/s)	(hr)	(m^3)	(m)	No x (m)	
	1.56	6.38	1.55	2	34500	1.8	1 x 0.75, 1 x 0.225	Arbitary at present

100 Year ARI

Catchment	Pre Development	Post Developed Inflow	Outflow	Spillway Width	Spillway Height	Depth over spillway
	(m^3/s)	(m^3/s)	(m^3/s)	(m)	(m)	(m)
	8.09	26.67	16.2	30	2	0.43



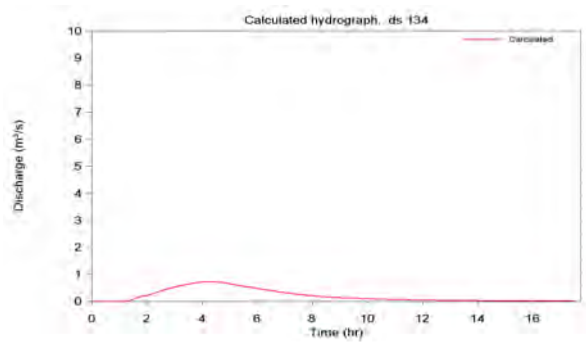


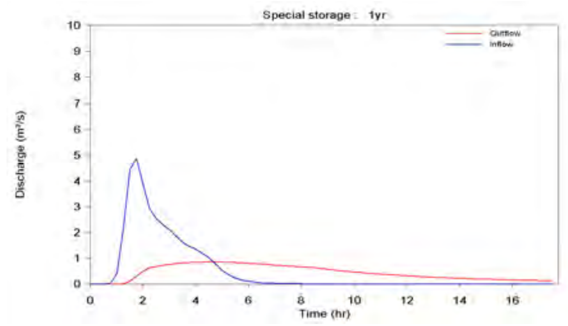
1 Year ARI

Catchment	Pre Development	Post Developed Inflow	Outflow	Critical Duration	Required Storage Approx	Elevation	Pipes	Comments
	(m^3/s)	(m^3/s)	(m^3/s)	(hr)	(m^3)	(m)	No x (m)	
	1.11	5.3	1.01	36	30000	1.9	1 x 0.6, 1 x 0.225	Arbitary at present

100 Year ARI

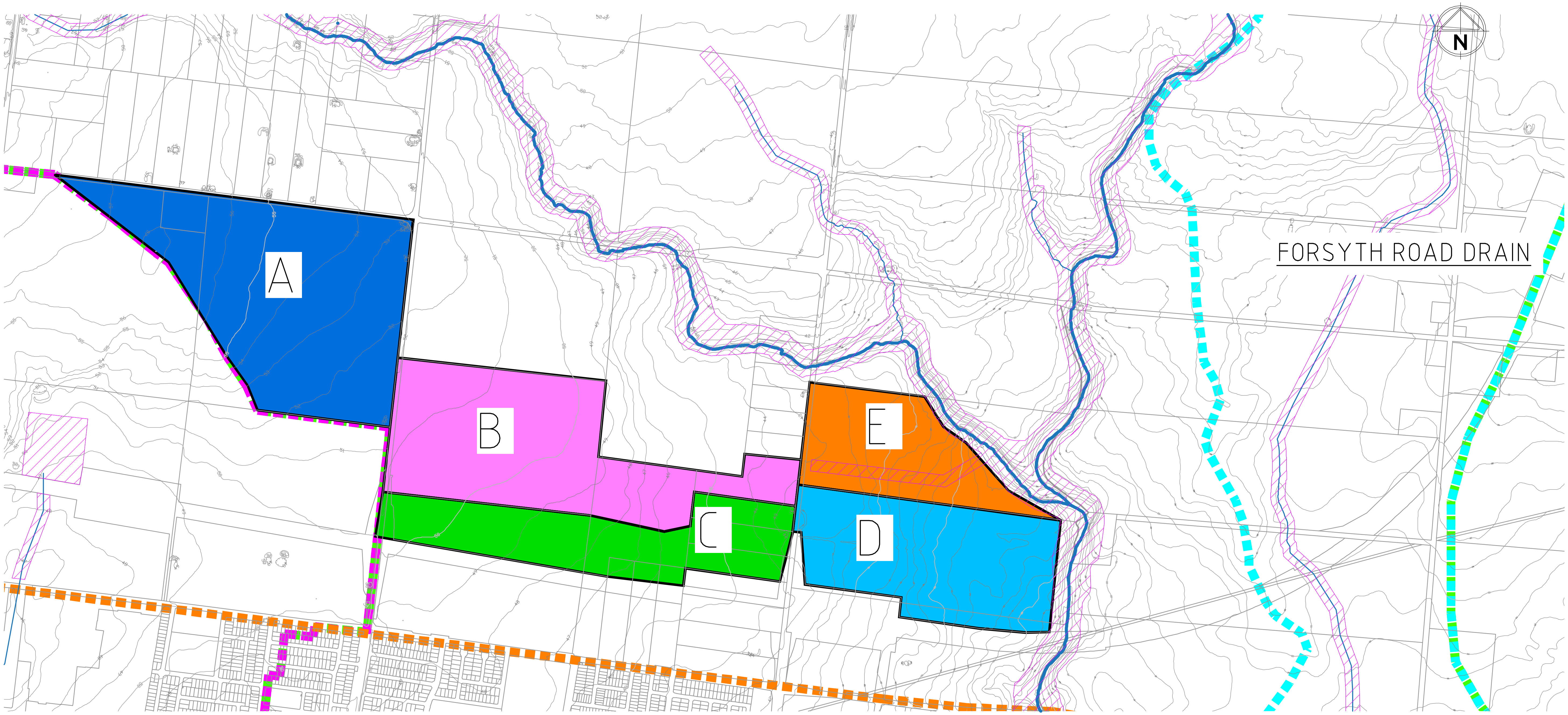
Catchment	Pre Development	Post Developed Inflow	Outflow	Spillway Width	Spillway Height	Depth over spillway
	(m^3/s)	(m^3/s)	(m^3/s)	(m)	(m)	(m)
	8.09	26.67	20.64	40	2	0.43





Appendix 5

Derrimut Road Overland Flow Analysis



SKELETON CREEK FLOODWAY CATCHMENT PLAN
(SCALE 1:15000, A1)

LEGEND

- NATURAL WATERWAY CORRIDOR
- WYNDHAM NORTH PSP BOUNDARY
- SKELETON CREEK CATCHMENT BOUNDARY
- DAVIS CREEK CATCHMENT BOUNDARY
- LIMIT OF URBAN GROWTH BOUNDARY

Rev	Amendments	App'd	Date
B	AMENDMENTS AS PER GAA COMMENTS	L.H.	22/05/13
A	ISSUED FOR DISCUSSION	L.H.	29/10/12

file name 137654M00 LS.dwg layout name AREAS
file location G:\13\137654\ACAD
plotted by Mitch Johnson plot date 24/5/2013 2:33 PM

Standard Drawing RDA1 - Version 20120911

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Vertical 0 125 250 375 500 625

Designed PENDING
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Checked ----

Authorised PENDING
PENDING

Map Reference MELWAY 359 J8
Sheet Number 02
Drg Status PRELIMINARY

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WYNDHAM NORTH
PRECINCT STRUCTURE PLAN
SKELETON CREEK
SWMS PLAN
GROWTH AREA AUTHORITY
MELBOURNE WATER

Rev B
Drg No 137654S02

100 Year ARI Calculations

Project: Wyndham North **Prepared By:** L.Strauch NB: Q5 taken as Q100/2.5

Spiire Ref: 137654 **Checked By:**

Client: GAA

Date: 22/05/2013

Polynomial Coefficients

ARI	a	b	c	d	e	f	g	F_y
1	2.562769	-6.18E-01	-3.39E-02	9.86E-03	-1.11E-04	-5.10E-04	4.82E-05	0.8
2	2.851214	-6.23E-01	-3.48E-02	9.41E-03	1.25E-04	-4.37E-04	2.75E-05	0.85
5	3.168822	-6.36E-01	-3.58E-02	9.47E-03	2.70E-04	-4.23E-04	2.11E-05	0.95
10	3.340628	-6.44E-01	-3.65E-02	9.51E-03	3.74E-04	-4.08E-04	1.41E-05	1
20	3.531127	-6.50E-01	-3.66E-02	9.37E-03	4.12E-04	-3.77E-04	8.17E-06	1.05
50	3.755543	-6.57E-01	-3.74E-02	9.10E-03	5.64E-04	-3.33E-04	-3.85E-06	1.15
100	3.910359	-6.63E-01	-3.74E-02	9.44E-03	5.19E-04	-3.59E-04	2.12E-06	1.2

Rural Drainage Calculations - Wyndham North

									Adams t_c approximation $t_c = 0.76A^{0.38}$					
Catchment	**Area (ha)**	**∑A (ha)**	**C_{10}**	**Ae (ha)**	**∑Ae (ha)**	**Flow Length (m)**	**∑Flow Length (m)**	**Velocity (m/s)**	**t_{c100} (mins)**	**$^{100}I_{tc}$ (mm/hr)**	**Q_{100} m^3/s**	**Q_5 m^3/s**	**Q_{gap} m^3/s**	**Comments**
Derrimut Road		167.87			37.84				50.44		**4.588**			Total retarded flow at Derrimut Road
Derrimut Road	47.180	47.18	0.52	24.61	24.61				50.44	55.94	4.588	1.835	3.120	Equivalent area for pre-developed flow at Derrimut Road, developed tc
D	28.346	196.22	0.52	14.78	52.62	1040	1040	1.5	62.00	48.84	8.568	3.427	5.826	
E	43.604	43.60	0.52	22.74	22.74	1120	1120	1.5	19.44	99.28	7.526	3.010	5.118	
Skeleton Creek		239.82			75.36				62.00	48.84	**12.270**	4.908	8.344	Total flow at outlet to creek

100 Year ARI Calculations

Project:	Wyndham North	**Prepared By:**	L.Strauch	NB: Q5 taken as Q100/2.5
Spiire Ref:	137654	**Checked By:**		
Client:	GAA			
Date:	22/05/2013			

Polynomial Coefficients

ARI	a	b	c	d	e	f	g	F_y
1	2.562769	-6.18E-01	-3.39E-02	9.86E-03	-1.11E-04	-5.10E-04	4.82E-05	0.8
2	2.851214	-6.23E-01	-3.48E-02	9.41E-03	1.25E-04	-4.37E-04	2.75E-05	0.85
5	3.168822	-6.36E-01	-3.58E-02	9.47E-03	2.70E-04	-4.23E-04	2.11E-05	0.95
10	3.340628	-6.44E-01	-3.65E-02	9.51E-03	3.74E-04	-4.08E-04	1.41E-05	1
20	3.531127	-6.50E-01	-3.66E-02	9.37E-03	4.12E-04	-3.77E-04	8.17E-06	1.05
50	3.755543	-6.57E-01	-3.74E-02	9.10E-03	5.64E-04	-3.33E-04	-3.85E-06	1.15
100	3.910359	-6.63E-01	-3.74E-02	9.44E-03	5.19E-04	-3.59E-04	2.12E-06	1.2

Urban Drainage Calculations - Wyndham North

									Lot + pipe t_c = D/V + 6min					
Catchment	**Area (ha)**	**∑A (ha)**	C_{10}	**Ae (ha)**	**∑Ae (ha)**	**Flow Length (m)**	**∑Flow Length (m)**	**Velocity (m/s)**	t_{c100} **(mins)**	$^{100}I_{tc}$ **(mm/hr)**	Q_{100} m^3/s	Q_5 m^3/s	Q_{gap} m^3/s	**Comments**
A	72.385	72.39	0.52	37.75	37.75	2240	2240	1.5	31.89	74.61	**9.388**	3.755	6.384	Total flow at Tarneit Road
B	58.957	131.34	0.52	30.75	68.50	1670	3910	1.5	50.44	55.94	12.772	5.109	8.685	
C	38.081	38.08	0.52	19.86	19.86	1890	1890	1.5	28.00	80.65	5.339	2.136	3.630	
Derrimut Road		169.42			88.36		3910		50.44	55.94	**16.475**	6.590	11.203	Total flow at Derrimut Road
D	28.346	197.77	0.52	14.78	103.14	1040	4950	1.5	62.00	48.84	16.792	6.717	11.419	
E	43.604	43.60	0.52	22.74	22.74	1120	1120	1.5	19.44	99.28	7.526	3.010	5.118	
Skeleton Creek		241.37			125.88				62.00	48.84	**20.494**	8.198	13.936	Total flow at outlet to creek

100 Year ARI Calculations

Project:	Wyndham North	**Prepared By:**	L.Strauch	NB: Q5 taken as Q100/2.5
Spiire Ref:	137654	**Checked By:**		
Client:	GAA			
Date:	22/05/2013			

Polynomial Coefficients

ARI	a	b	c	d	e	f	g	F_y (Rural)
1	2.562769	-6.18E-01	-3.39E-02	9.86E-03	-1.11E-04	-5.10E-04	4.82E-05	
2	2.851214	-6.23E-01	-3.48E-02	9.41E-03	1.25E-04	-4.37E-04	2.75E-05	0.75
5	3.168822	-6.36E-01	-3.58E-02	9.47E-03	2.70E-04	-4.23E-04	2.11E-05	0.90
10	3.340628	-6.44E-01	-3.65E-02	9.51E-03	3.74E-04	-4.08E-04	1.41E-05	1.00
20	3.531127	-6.50E-01	-3.66E-02	9.37E-03	4.12E-04	-3.77E-04	8.17E-06	1.10
50	3.755543	-6.57E-01	-3.74E-02	9.10E-03	5.64E-04	-3.33E-04	-3.85E-06	1.20
100	3.910359	-6.63E-01	-3.74E-02	9.44E-03	5.19E-04	-3.59E-04	2.12E-06	1.30

Rural Drainage Calculations - Wyndham North

									Adams t_c approximation $t_c = 0.76A^{0.38}$					
Catchment	**Area (ha)**	**∑A (ha)**	**C_{10}**	**Ae (ha)**	**∑Ae (ha)**	**Flow Length (m)**	**∑Flow Length (m)**	**Velocity (m/s)**	**t_{c100} (mins)**	**$^{100}I_{tc}$ (mm/hr)**	**Q_{100} m^3/s**	**Q_5 m^3/s**	**Q_{gap} m^3/s**	**Comments**
A	72.385	72.39	0.14	10.35	10.35				40.33	64.54	**2.413**	0.965	1.641	Total flow at Tarneit Road
B	58.957	131.34	0.14	8.43	18.79				50.58	55.84	3.789	1.515	2.576	
C	38.081	38.08	0.14	5.45	5.45				31.60	75.03	1.476	0.590	1.004	
Derrimut Road		169.42			24.24				55.72	52.42	**4.588**	1.835	3.120	Total flow at Derrimut Road
D	28.346	197.77	0.14	4.05	28.29				59.09	50.43	5.152	2.061	3.503	
E	43.604	43.60	0.14	6.24	6.24				33.26	72.72	1.638	0.655	1.114	
Skeleton Creek		241.37			34.53				63.74	47.95	**5.979**	2.392	4.066	Total flow at outlet to creek

Mannings Pipe Calculations

spiire

Project: Wyndham North
Client: GAA
Reference: 137654

100 and 5 year pipe sizing based on rational calculations

Flow at Tarneit Road
100 year flow in pipe

Pipe Type	RCP	NS at D/S end	51	Q_{100}	9.388 m3/s
Diameter	2,100 mm	IL D/S	48		
Slope	1 in 300	NS at U/S end	58		
n	0.0130 mm	IL U/S	55		
OUTPUT DATA		Length	2000		
v	2.890 m/s	Max grade	1 in 286		
Qcap	10.011 m3/s				

Flow at Tarneit Road
5 year flow in pipe

Pipe Type	RCP	NS at D/S end	51	Q_5	3.755 m3/s
Diameter	1,500 mm	IL D/S	48.6		
Slope	1 in 300	NS at U/S end	58		
n	0.0130 mm	IL U/S	55.6		
OUTPUT DATA		Length	2000		
v	2.309 m/s	Max grade	1 in 286		
Qcap	4.081 m3/s				

Flow at Derrimut Road
100 year flow in pipe

Pipe Type	RCP	NS at D/S end	43	Q_{100}	16.475 m3/s
Diameter	2,400 mm	IL D/S	39.7		
Slope	1 in 220	NS at U/S end	51		
n	0.0130 mm	IL U/S	47.7		
OUTPUT DATA		Length	1670		
v	3.689 m/s	Max grade	1 in 209		
Qcap	16.690 m3/s				

Flow at Derrimut Road
5 year flow in pipe

Pipe Type	RCP	NS at D/S end	43	Q_5	6.590 m3/s
Diameter	1,800 mm	IL D/S	40.3		
Slope	1 in 220	NS at U/S end	51		
n	0.0130 mm	IL U/S	48.3		
OUTPUT DATA		Length	1670		
v	3.045 m/s	Max grade	1 in 209		
Qcap	7.750 m3/s				

Outfall to creek
100 year flow in pipe

Pipe Type	RCP	Outfall RL	30	Q_{100}	20.494 m3/s
Diameter	2,700 mm	NS at U/S end	43		
Slope	1 in 250	IL U/S	39.4		
n	0.0130 mm	Length	1050		
OUTPUT DATA		Max grade	1 in 112		
v	3.744 m/s				
Qcap	21.434 m3/s				

Outfall to creek
5 year flow in pipe

Pipe Type	RCP	Outfall RL	30	Q_5	8.198 m3/s
Diameter	1,650 mm	NS at U/S end	43		
Slope	1 in 115	IL U/S	40.45		
n	0.0130 mm	Length	1050		
OUTPUT DATA		Max grade	1 in 100		
v	3.975 m/s				
Qcap	8.499 m3/s				

Flow after Derrimut Road
Retarded 100 year flow in pipe

Pipe Type	RCP	Outfall RL	30	Q_{100}	4.588 m3/s
Diameter	1,350 mm	NS at U/S end	43		
Slope	1 in 135	IL U/S	40.75		
n	0.0130 mm	Length	1050		
OUTPUT DATA		Max grade	1 in 98		
v	3.209 m/s				
Qcap	4.594 m3/s				

Outfall to creek
100 year flow in pipe
with RB at Derrimut Road

Pipe Type	RCP	Outfall RL	30	Q_{100}	12.270 m3/s
Diameter	2,100 mm	NS at U/S end	43		
Slope	1 in 200	IL U/S	40		
n	0.0130 mm	Length	1050		
OUTPUT DATA		Max grade	1 in 105		
v	3.540 m/s				
Qcap	12.261 m3/s				

Outfall to creek
5 year flow in pipe
with RB at Derrimut Road

Pipe Type	RCP	Outfall RL	30	Q_5	4.908 m3/s
Diameter	1,350 mm	NS at U/S end	43		
Slope	1 in 100	IL U/S	40.75		
n	0.0130 mm	Length	1050		
OUTPUT DATA		Max grade	1 in 98		
v	3.729 m/s				
Qcap	5.337 m3/s				

Project: Wyndham North
Client: **GAA**
Reference: 137654

ON SITE STORAGE

Calculation in accordance with Swinburne Institute of Technology 1987

Onsite storage calculation - Derrimut Road

Qa Peak inflow for design storm. (L/s) (Calculated for a given td)
Qp1 Peak permitted controlled outflow to drainage system (L/s)
Qp2 Outflow to main drain at commencement of above ground storage or the capacity of the outfall drain running full.

Where Qp2 = 0.8 Qp1 approx

ts Site time of concentration
td Duration of critical storm (mins)
Vs Volume of on site storage needed (cubic metres)
A Area of catchment (ha)
c Coefficent of runoff
I Intensity (design yr corresponding to td)

I	Ae	Qa	Qp1	Qp2	ts	td	Vstored	Check
56	88.360	16475.9	4588	0	50.44	50.4	35977.5	
53	88.360	15489.6	4588	0	50.44	55.4	36951.2	More Storage
50	88.360	14631.1	4588	0	50.44	60.4	37796.7	More Storage
47	88.360	13876.2	4588	0	50.44	65.4	38533.9	More Storage
45	88.360	13206.5	4588	0	50.44	70.4	39178.2	More Storage
43	88.360	12607.9	4588	0	50.44	75.4	39742.1	More Storage
41	88.360	12069.1	4588	0	50.44	80.4	40235.8	More Storage
39	88.360	11581.2	4588	0	50.44	85.4	40667.6	More Storage
38	88.360	11137.2	4588	0	50.44	90.4	41044.4	More Storage
36	88.360	10731.2	4588	0	50.44	95.4	41372.2	More Storage
35	88.360	10358.3	4588	0	50.44	100	41655.9	More Storage
34	88.360	10014.4	4588	0	50.44	105	41899.7	More Storage
33	88.360	9696.2	4588	0	50.44	110	42107.3	More Storage
32	88.360	9400.8	4588	0	50.44	115	42281.9	More Storage
31	88.360	9125.7	4588	0	50.44	120	42426.3	More Storage
30	88.360	8868.9	4588	0	50.44	125	42542.9	More Storage
29	88.360	8628.5	4588	0	50.44	130	42633.9	More Storage
29	88.360	8402.9	4588	0	50.44	135	42701.2	More Storage
28	88.360	8190.8	4588	0	50.44	140	42746.5	More Storage
27	88.360	7990.9	4588	0	50.44	145	42771.2	More Storage
26	88.360	7802.2	4588	0	50.44	150	42776.8	More Storage
26	88.360	7623.7	4588	0	50.44	155	42764.6	Okay
25	88.360	7454.6	4588	0	50.44	160	42735.5	Okay
25	88.360	7294.1	4588	0	50.44	165	42690.7	Okay
24	88.360	7141.6	4588	0	50.44	170	42631.0	Okay
24	88.360	6996.5	4588	0	50.44	175	42557.4	Okay
23	88.360	6858.1	4588	0	50.44	180	42470.4	Okay

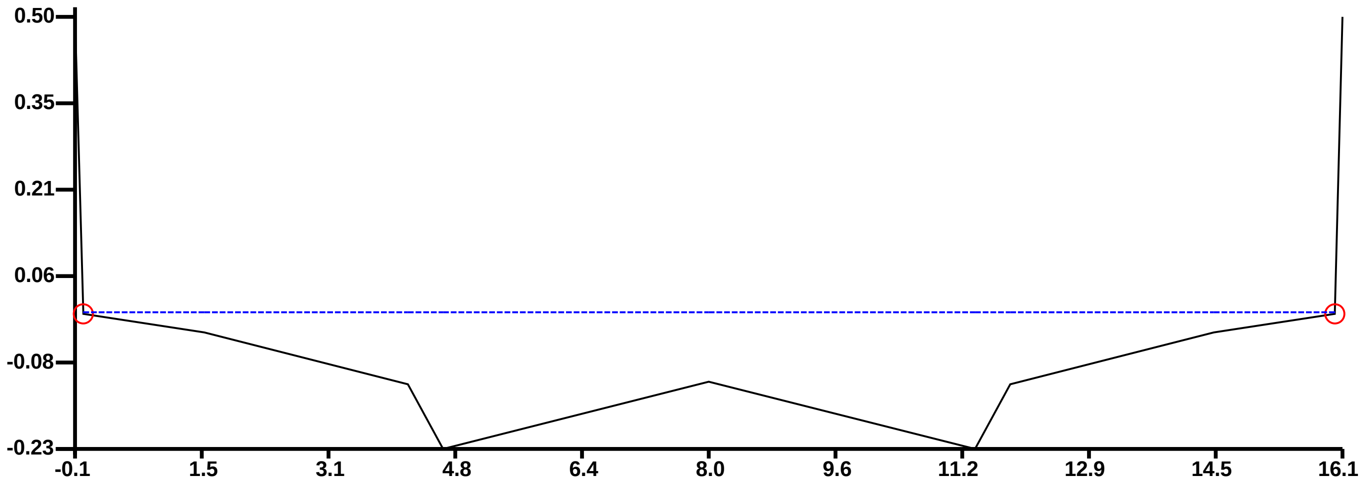
PROJECT: 16m Road Reserve

Comment

Print-out date: 22/05/2013 - Time: 16:29

Data File: G:\13\137654\Water Resources\PC CONVEY\Skeleton\16m Road Reserve.dat

1. CROSS-SECTION:



2. DISCHARGE INFORMATION:

100 year (1%) storm event

Total discharge = 1.5 cumecs

There is no pipe discharge
Overland/channel/watercourse discharge = 1.5 cumecs

3. RESULTS: Water surface elevation = 0.003m

Current Grade = 1 in 200

	LEFT OVERBANK	MAIN CHANNEL	RIGHT OVERBANK	TOTAL CROSS-SECTION
Discharge (cumecs):	0.00	1.48	0.00	1.48
D(Max) = Max. Depth (m):	0.00	0.23	0.00	0.23
D(Ave) = Ave. Depth (m):	0.00	0.11	0.00	0.11
V = Ave. Velocity (m/s):	0.02	0.82	0.02	0.82
D(Max) x V (cumecs/m):	0.00	0.19	0.00	0.19
D(Ave) x V (cumecs/m):	0.00	0.09	0.00	0.09
Froude Number:	0.13	0.78	0.13	0.78
Area (m^2):	0.00	1.80	0.00	1.80
Wetted Perimeter (m):	0.00	16.03	0.00	16.04
Flow Width (m):	0.00	16.00	0.00	16.00
Hydraulic Radius (m):	0.00	0.11	0.00	0.11
Composite Manning's n:	0.020	0.020	0.020	0.020
Split Flow?	-	-	-	No

4. CROSS-SECTION DATA:

	LEFT HAND POINT		RIGHT HAND POINT		
SEGMENT NO.	CHAINAGE (m)	R.L. (m)	CHAINAGE (m)	R.L. (m)	MANNING'S N
1	-0.100	0.500	0.000	0.000	0.020
2	0.000	0.000	1.550	-0.031	0.020
3	1.550	-0.031	4.150	-0.118	0.020
4	4.150	-0.118	4.600	-0.227	0.020
5	4.600	-0.227	8.000	-0.114	0.020
6	8.000	-0.114	11.400	-0.227	0.020
7	11.400	-0.227	11.850	-0.118	0.020
8	11.850	-0.118	14.450	-0.031	0.020
9	14.450	-0.031	16.000	0.000	0.020
10	16.000	0.000	16.100	0.500	0.020

PROJECT: 26m Road Reserve
Comment
Print-out date: 22/05/2013 - Time: 16:31
Data File: G:\13\137654\Water Resources\PC CONVEY\Skeleton\26m Road Reserve.dat

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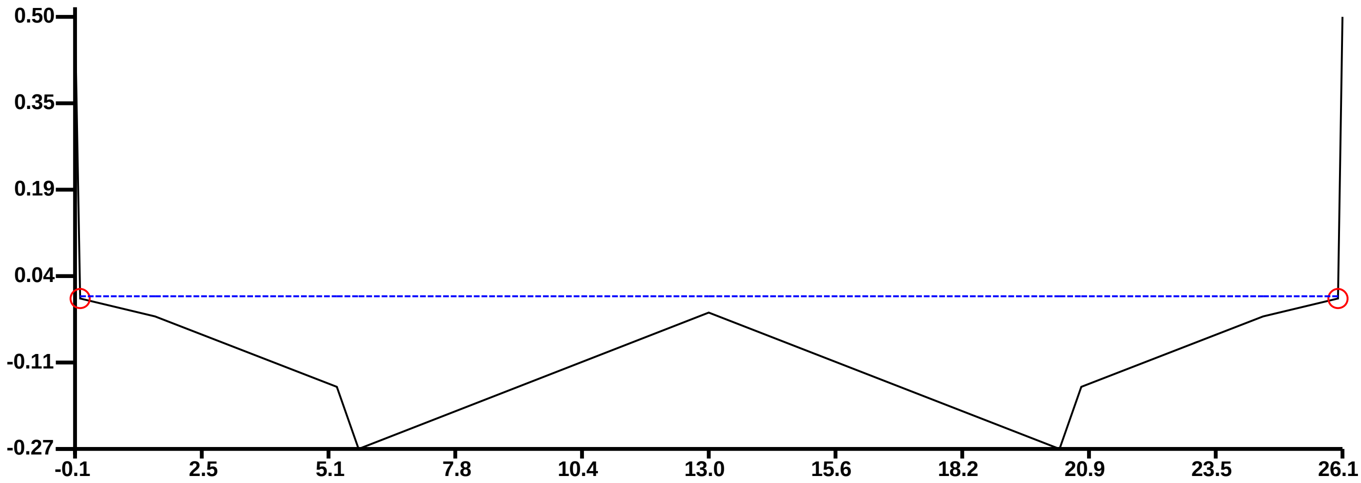


2. DISCHARGE INFORMATION:

100 year (1%) storm event

Total discharge = 2.7 cumecs

There is no pipe discharge
Overland/channel/watercourse discharge = 2.7 cumecs

3. RESULTS: Water surface elevation = 0.004m

Current Grade = 1 in 200

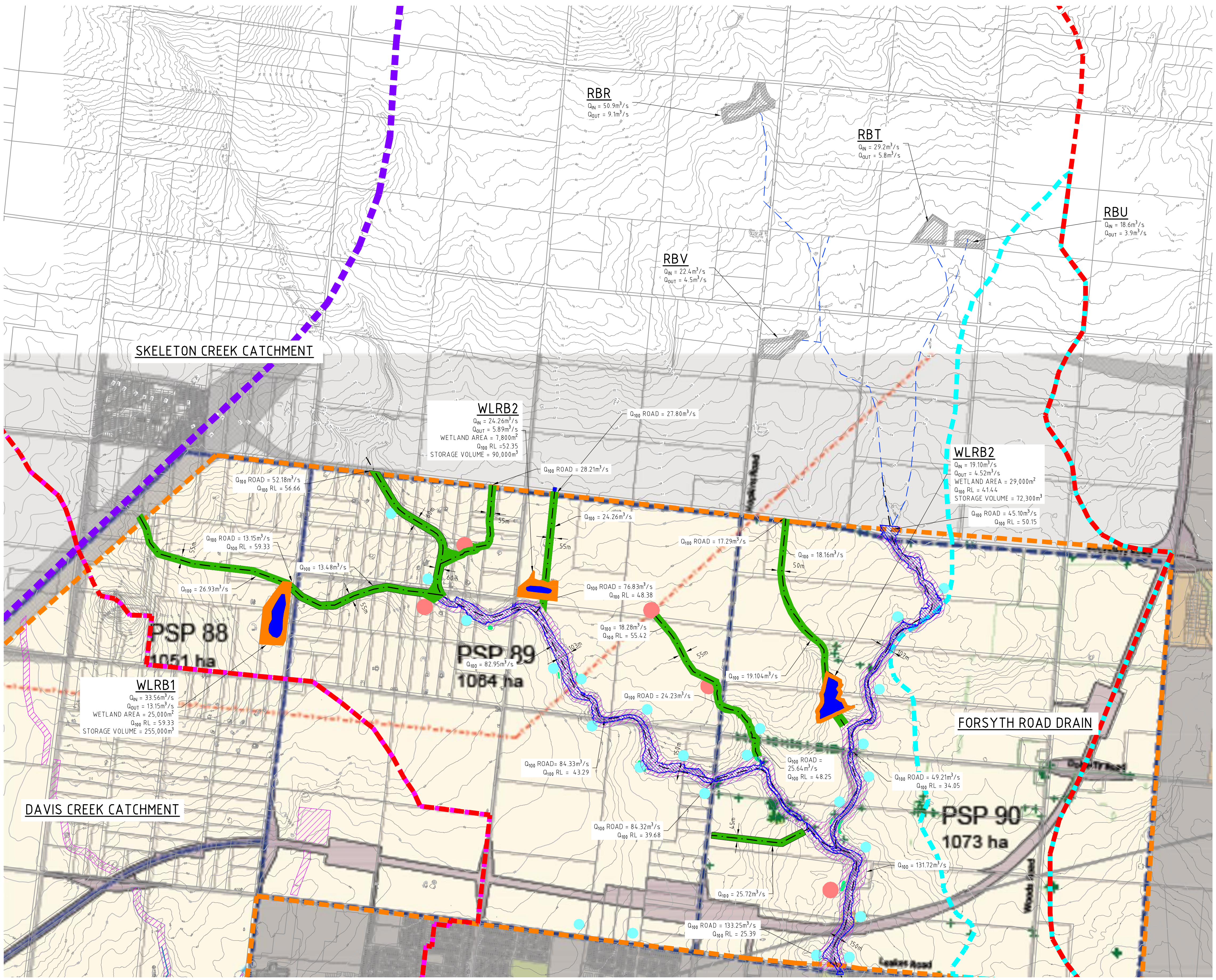
	LEFT OVERBANK	MAIN CHANNEL	RIGHT OVERBANK	TOTAL CROSS-SECTION
Discharge (cumecs):	0.00	2.72	0.00	2.72
D(Max) = Max. Depth (m):	0.00	0.27	0.00	0.27
D(Ave) = Ave. Depth (m):	0.00	0.12	0.00	0.12
V = Ave. Velocity (m/s):	0.02	0.86	0.02	0.86
D(Max) x V (cumecs/m):	0.00	0.23	0.00	0.23
D(Ave) x V (cumecs/m):	0.00	0.10	0.00	0.10
Froude Number:	0.14	0.79	0.14	0.79
Area (m^2):	0.00	3.15	0.00	3.15
Wetted Perimeter (m):	0.00	26.04	0.00	26.05
Flow Width (m):	0.00	26.00	0.00	26.00
Hydraulic Radius (m):	0.00	0.12	0.00	0.12
Composite Manning's n:	0.020	0.020	0.020	0.020
Split Flow?	-	-	-	No

4. CROSS-SECTION DATA:

	LEFT HAND POINT		RIGHT HAND POINT		
SEGMENT NO.	CHAINAGE (m)	R.L. (m)	CHAINAGE (m)	R.L. (m)	MANNING'S N
1	-0.100	0.500	0.000	0.000	0.020
2	0.000	0.000	1.550	-0.031	0.020
3	1.550	-0.031	5.300	-0.156	0.020
4	5.300	-0.156	5.750	-0.266	0.020
5	5.750	-0.266	13.000	-0.024	0.020
6	13.000	-0.024	20.250	-0.266	0.020
7	20.250	-0.266	20.700	-0.156	0.020
8	20.700	-0.156	24.450	-0.031	0.020
9	24.450	-0.031	26.000	0.000	0.020
10	26.000	0.000	26.100	0.500	0.020

Appendix 6

Skeleton Creek SWMS



SKELETON CREEK SWMS LAYOUT PLAN
(SCALE 1:15000, A1)

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LEGEND

- STORMWATER QUALITY TREATMENT ASSET (COUNCIL MAINTENANCE RESPONSIBILITY)
- STORMWATER QUALITY TREATMENT ASSET (MELBOURNE WATER MAINTENANCE RESPONSIBILITY)
- STORMWATER QUALITY TREATMENT ASSET (MELBOURNE WATER MAINTENANCE RESPONSIBILITY)
- 1 IN 100YR ARI RETARDING BASIN
- CONSTRUCTED WATERWAY
- 100YR ARI INUNDATION AREA
- NATURAL WATERWAY CORRIDOR
- WYNDHAM NORTH PSP BOUNDARY
- SKELETON CREEK CATCHMENT BOUNDARY
- DAVIS CREEK CATCHMENT BOUNDARY
- LIMIT OF URBAN GROWTH BOUNDARY
- FORSYTH ROAD DRAIN CATCHMENT BOUNDARY
- FUTURE 1 IN 100YR ARI RETARDING BASIN

Rev	Amendments	App'd	Date
C	AMEND BACKGROUND IMAGE AND PLAN DETAILS	L.H.	30/05/13
B	AMENDMENTS AS PER GAA COMMENTS	L.H.	22/05/13
A	ISSUED FOR DISCUSSION	L.H.	29/10/12

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plotted by Mitch Johnson plot date 30/5/2013 1:41 PM
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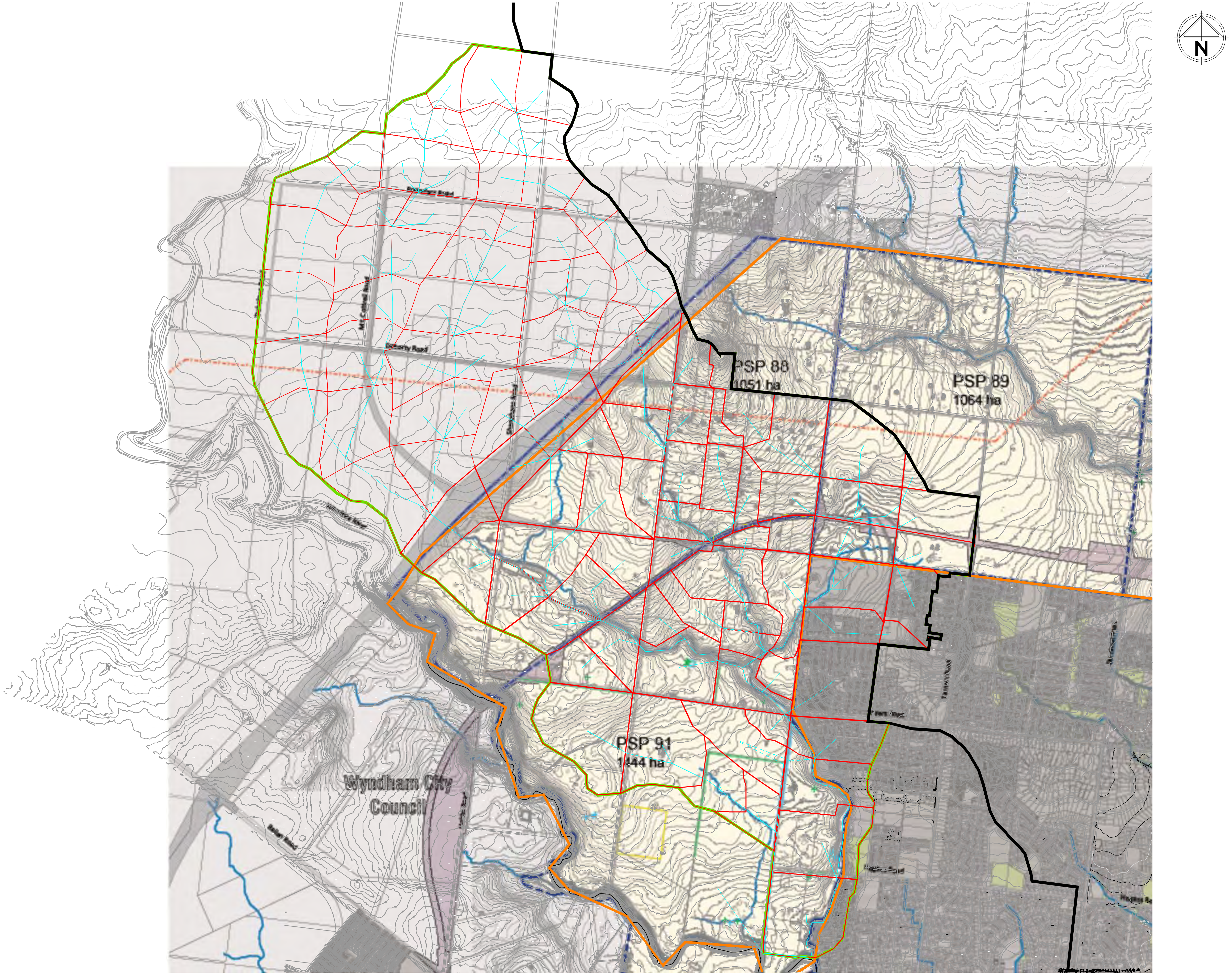
SKELETON CREEK SWMS PLAN
GROWTH AREA AUTHORITY
MELBOURNE WATER

Rev C
Drg No 137654S02

Appendix 7

Davis Creek RORB catchment Plan



Rev	Amendments	App'd	Date
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A	ISSUED FOR DISCUSSION	LH	12/09/12

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RORB CATCHMENT PLAN
DAVIS CREEK
GROWTH AREAS AUTHORITY
MUNICIPALITY

Rev B
Drg No 137654F01

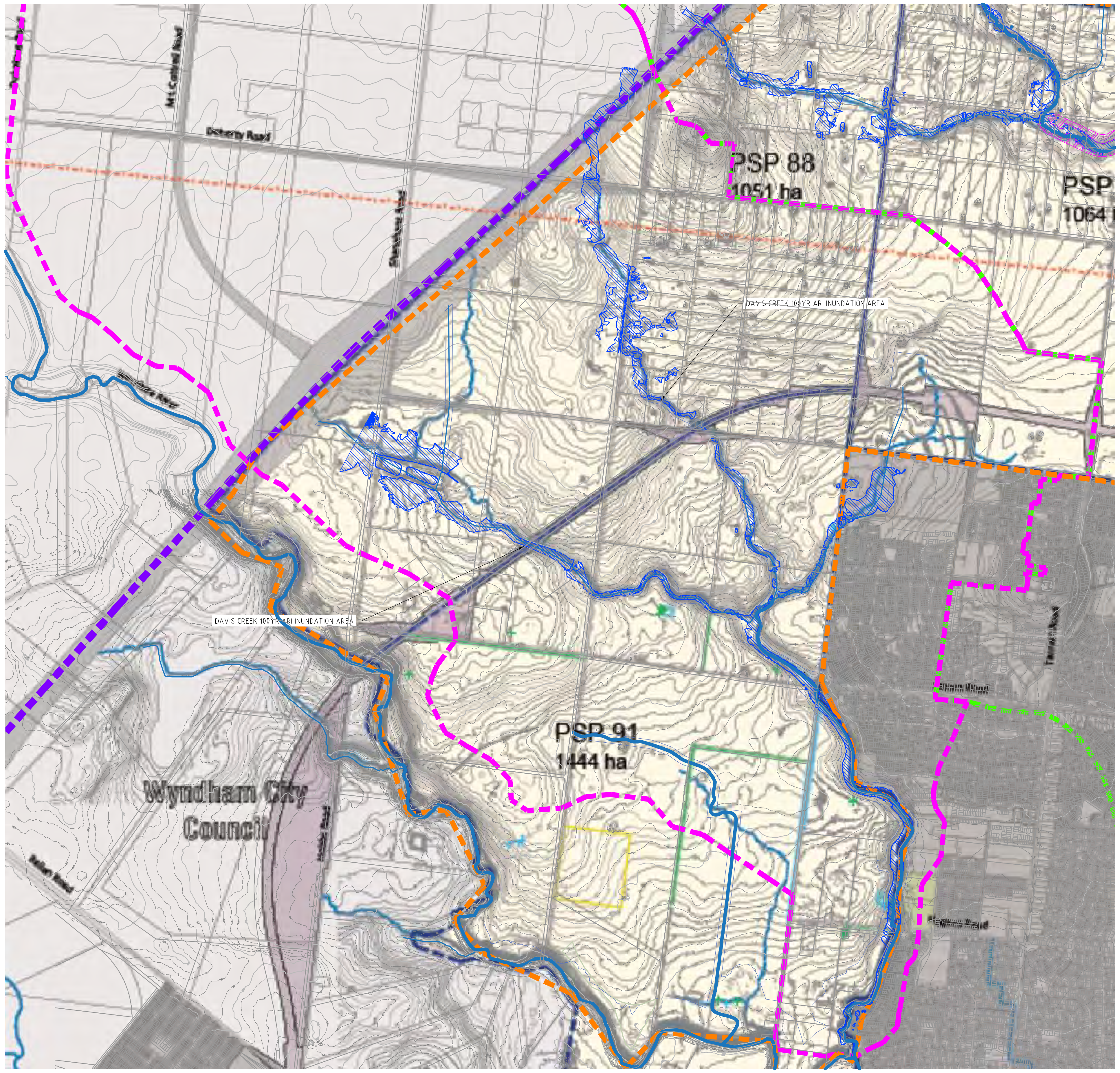
Appendix 8

Davis Creek Discharge Limit Hec-Ras Output

RS River Station (m)	LL Lot Level RL (m)	Flow (m3/s)	WS Water Surface RL (m)	Free Board (m)	Flow (m3/s)	WS Water Surface RL (m)	Free Board (m)	Flow (m3/s)	WS Water Surface RL (m)	Free Board (m)	Flow (m3/s)	WS Water Surface RL (m)	Free Board (m)	Flow (m3/s)	WS Water Surface RL (m)	Free Board (m)
3119.88	42.30	40	36.495	5.81	45	36.567	5.73	50	36.629	5.67	55	36.687	5.61	60	36.743	5.56
3019.88	42.00	40	36.373	5.63	45	36.439	5.56	50	36.498	5.50	55	36.555	5.45	60	36.609	5.39
2919.88	42.00	40	36.18	5.82	45	36.243	5.76	50	36.302	5.70	55	36.359	5.64	60	36.414	5.59
2819.88	40.40	40	35.871	4.53	45	35.938	4.46	50	36.001	4.40	55	36.061	4.34	60	36.118	4.28
2719.88	39.80	40	35.561	4.24	45	35.626	4.17	50	35.688	4.11	55	35.745	4.06	60	35.8	4.00
2619.88	40.60	40	35.234	5.37	45	35.291	5.31	50	35.345	5.26	55	35.395	5.21	60	35.443	5.16
2519.88	40.10	40	34.911	5.19	45	34.966	5.13	50	35.019	5.08	55	35.07	5.03	60	35.12	4.98
2419.88	38.40	40	34.612	3.79	45	34.674	3.73	50	34.732	3.67	55	34.788	3.61	60	34.843	3.56
2319.88	38.40	40	34.382	4.02	45	34.445	3.96	50	34.503	3.90	55	34.559	3.84	60	34.614	3.79
2219.88	38.40	40	34.178	4.22	45	34.241	4.16	50	34.299	4.10	55	34.356	4.04	60	34.41	3.99
2119.88	38.00	40	33.997	4.00	45	34.062	3.94	50	34.123	3.88	55	34.181	3.82	60	34.237	3.76
2019.88	38.00	40	33.711	4.29	45	33.774	4.23	50	33.832	4.17	55	33.889	4.11	60	33.942	4.06
1919.88	38.00	40	33.388	4.61	45	33.451	4.55	50	33.51	4.49	55	33.568	4.43	60	33.621	4.38
1819.88	38.20	40	33.211	4.99	45	33.27	4.93	50	33.326	4.87	55	33.38	4.82	60	33.428	4.77
1719.88	38.20	40	33.038	5.16	45	33.094	5.11	50	33.148	5.05	55	33.199	5.00	60	33.24	4.96
1619.88	36.70	40	32.84	3.86	45	32.897	3.80	50	32.951	3.75	55	33.003	3.70	60	33.033	3.67
1519.88	36.70	40	32.69	4.01	45	32.746	3.95	50	32.801	3.90	55	32.854	3.85	60	32.868	3.83
1419.88	36.10	40	32.597	3.50	45	32.645	3.46	50	32.693	3.41	55	32.739	3.36	60	32.732	3.37
1319.88	35.40	40	32.494	2.91	45	32.53	2.87	50	32.566	2.83	55	32.602	2.80	60	32.551	2.85
1219.88	34.60	40	32.393	2.21	45	32.414	2.19	50	32.436	2.16	55	32.46	2.14	60	32.265	2.34
1119.88	33.10	40	32.347	0.75	45	32.359	0.74	50	32.372	0.73	55	32.385	0.72	60	31.963	1.14
1019.88	33.10	40	32.326	0.77	45	32.332	0.77	50	32.339	0.76	55	32.347	0.75	60	31.53	1.57
919.88	32.40	40	32.318	**0.08**	45	32.323	**0.08**	50	32.328	**0.07**	55	32.334	**0.07**	60	31.04	1.36
819.88	33.20	40	32.315	0.89	45	32.318	0.88	50	32.323	0.88	55	32.327	0.87	60	30.799	2.40
719.88	32.80	40	32.312	0.49	45	32.315	0.48	50	32.318	0.48	55	32.322	0.48	60	30.438	2.36
619.88	32.90	40	32.308	0.59	45	32.31	0.59	50	32.312	0.59	55	32.315	0.59	60	30.047	2.85
519.88	33.15	40	32.307	0.84	45	32.308	0.84	50	32.31	0.84	55	32.313	0.84	60	29.876	3.27
419.88	33.10	40	32.306	0.79	45	32.308	0.79	50	32.309	0.79	55	32.311	0.79	60	29.793	3.31
319.88	33.10	40	32.303	0.80	45	32.304	0.80	50	32.305	0.80	55	32.306	0.79	60	29.451	3.65
219.88	33.10	40	32.301	0.80	45	32.301	0.80	50	32.301	0.80	55	32.302	0.80	60	28.953	4.15
119.88	33.00	40	32.301	0.70	45	32.302	0.70	50	32.302	0.70	55	32.303	0.70	60	28.059	4.94
19.88	33.00	40	32.3	0.70	45	32.3	0.70	50	32.3	0.70	55	32.3	0.70	60	**27.244**	5.76

Appendix 9

Davis Creek Inundation Mapping



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LEGEND

- 100YR ARI INUNDATION AREA
- WYNDHAM NORTH PSP BOUNDARY
- SKELETON CREEK CATCHMENT BOUNDARY
- DAVIS CREEK CATCHMENT BOUNDARY
- LIMIT OF URBAN GROWTH BOUNDARY

DAVIS CREEK INUNDATION LAYOUT PLAN
(SCALE 1:15000, A1)

Rev	Amendments	App'd	Date
B	AMEND BACKGROUND IMAGE AS PER GAA COMMENTS	L.H.	22/05/13
A	ISSUED FOR DISCUSSION	L.H.	29/10/12

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Map Reference MELWAY 359 J8
Sheet Number 1-1
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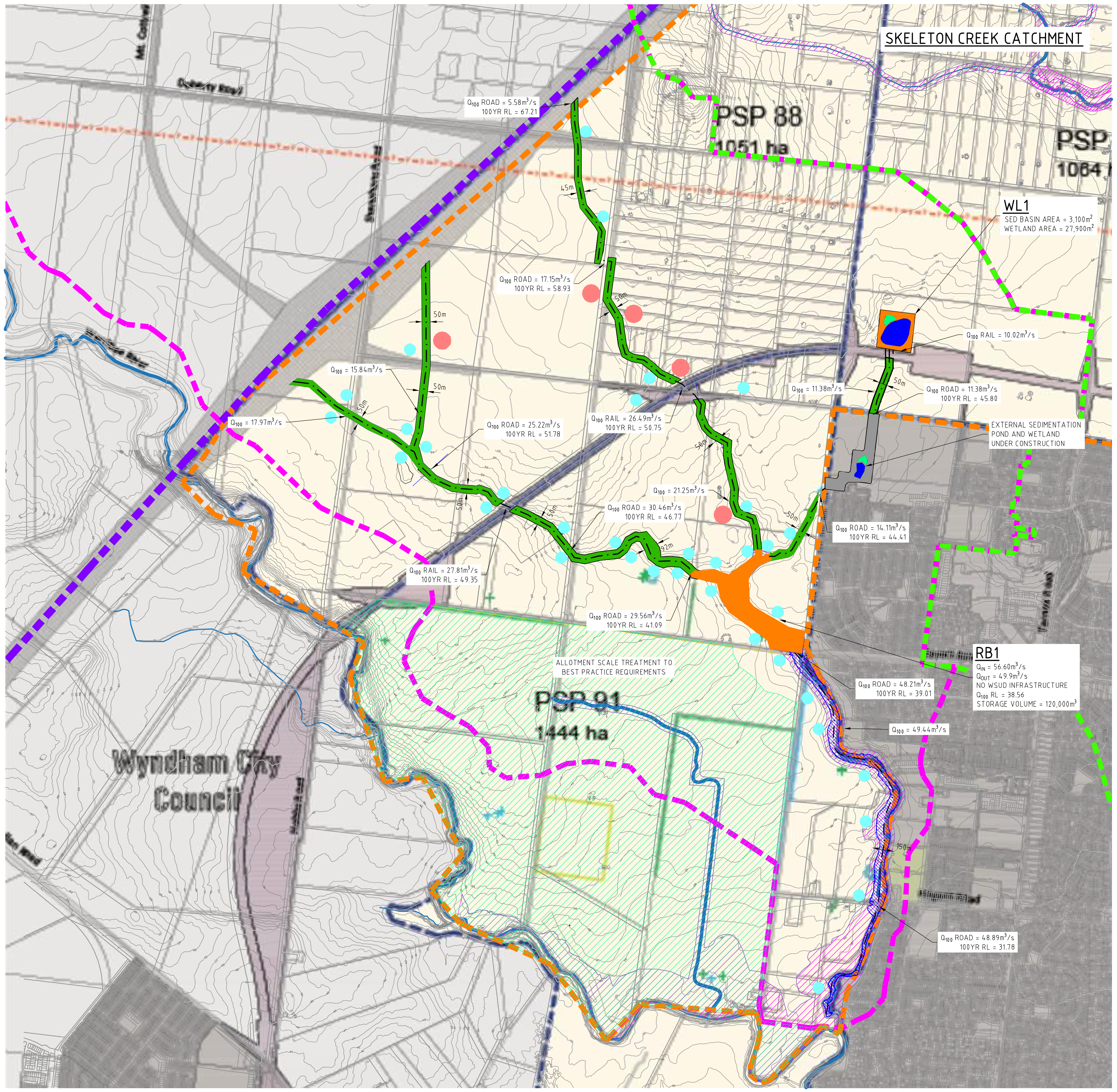
DAVIS CREEK
INUNDATION PLAN
GROWTH AREAS AUTHORITY
MUNICIPALITY

Rev **B**
Drg No **137654D1-1**

Appendix 10

Davis Creek SWMS



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LEGEND

- STORMWATER QUALITY TREATMENT ASSET (COUNCIL MAINTENANCE RESPONSIBILITY)
- STORMWATER QUALITY TREATMENT ASSET (MELBOURNE WATER MAINTENANCE RESPONSIBILITY)
- STORMWATER QUALITY TREATMENT ASSET (MELBOURNE WATER MAINTENANCE RESPONSIBILITY)
- 1 IN 100YR ARI RETARDING BASIN
- CONSTRUCTED FLOODWAY
- 100YR ARI INUNDATION AREA
- NATURAL WATERWAY CORRIDOR
- WYNDHAM NORTH PSP BOUNDARY
- SKELETON CREEK CATCHMENT BOUNDARY
- DAVIS CREEK CATCHMENT BOUNDARY
- LIMIT OF URBAN GROWTH BOUNDARY
- AREA TO BE COVERED BY WERRIBEE RIVER LOWER DSS 8001

DAVIS CREEK SWMS LAYOUT PLAN

(SCALE 1:15000, A1)

Rev	Amendments	App'd	Date
C	AMEND BACKGROUND IMAGE AND PLAN DETAILS	L.H.	30/05/13
B	AMENDMENTS AS PER GAA COMMENTS	L.H.	22/05/13
A	ISSUED FOR DISCUSSION	L.H.	29/10/12

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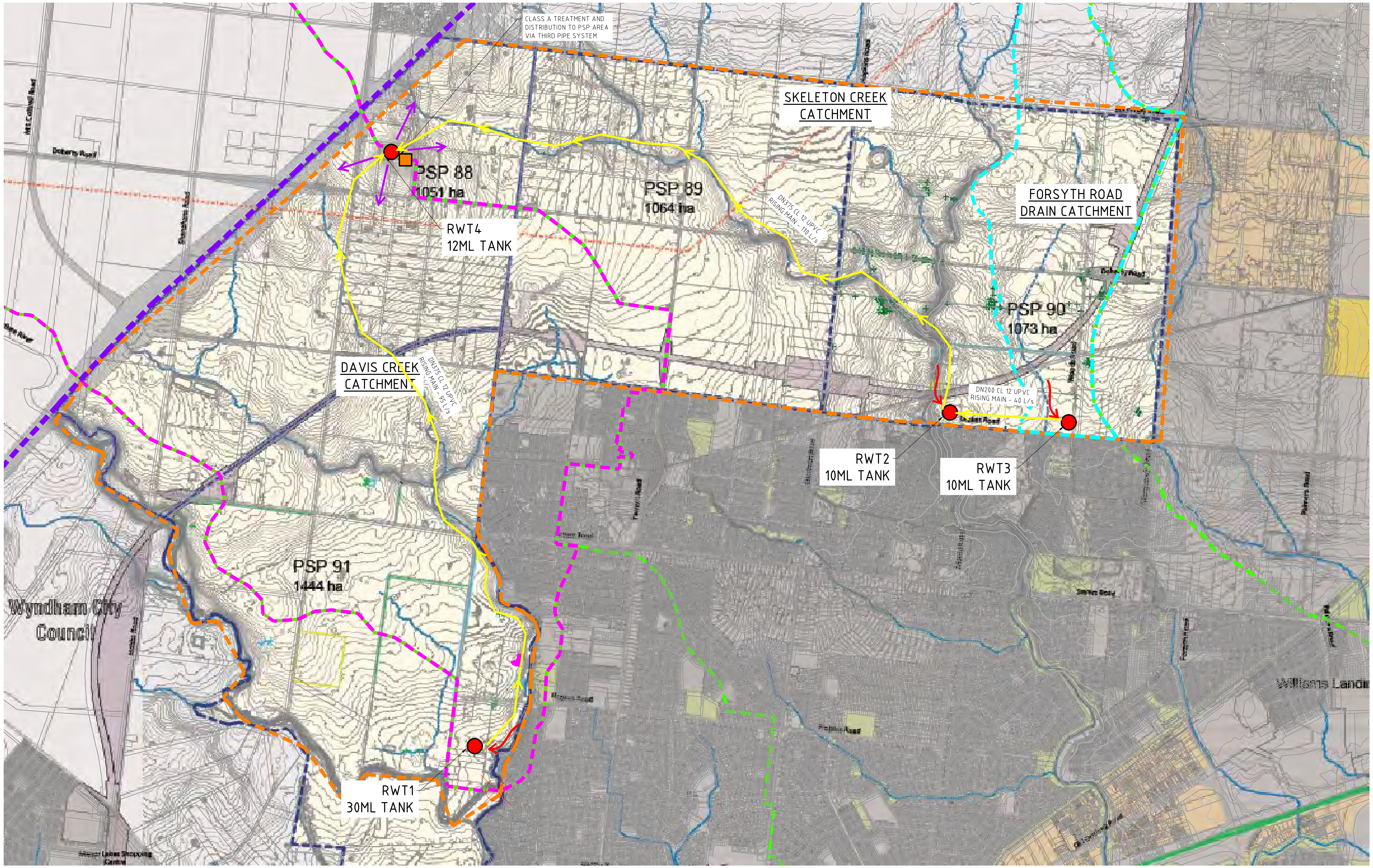
GROWTH AREA AUTHORITY
MELBOURNE WATER

Rev C
Drg No 137654D02

Appendix 11

Wyndham North PSP Stormwater Harvesting Concepts & Cost Estimates



WATER REUSE: OPTION 1

(SCALE 1:20000, A1)

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LEGEND

WYNDHAM NORTH PSP BOUNDARY

SKELETON CREEK CATCHMENT BOUNDARY

DAVIS CREEK CATCHMENT BOUNDARY

LIMIT OF URBAN GROWTH BOUNDARY

TANK

WATER TREATMENT FACILITY

DIVERSION FROM WATERCOURSE, CATCHMENT OR ADJACENT STORMWATER QUALITY TREATMENT ASSET

TO PSP AREA VIA THIRD PIPE DISTRIBUTION SYSTEM

PUMPED TO STORAGE

Tank Name	Volume (kL)	Reuse supplied (kL)	Reliability
RWT1	35000	1198000	63%
RWT2	10000	936000	61%
RWT3	10000	409000	63%
RWT4	12000	1616000	67%
Total yield:			**1616000 kL**

MINIMUM TANK SIZING BASED ON 7 DAY STORAGE AT 60% RELIABILITY DURING JANUARY.

Rev	Amendments	App'd	Date
C	AMENDMENTS AS PER GAA COMMENTS	L.H.	22/05/13
B	TANK VOLUMES AMENDED	L.H.	24/01/13
A	ISSUED FOR DISCUSSION	L.H.	29/10/12

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SWMS OPTIONS
OPTION 1
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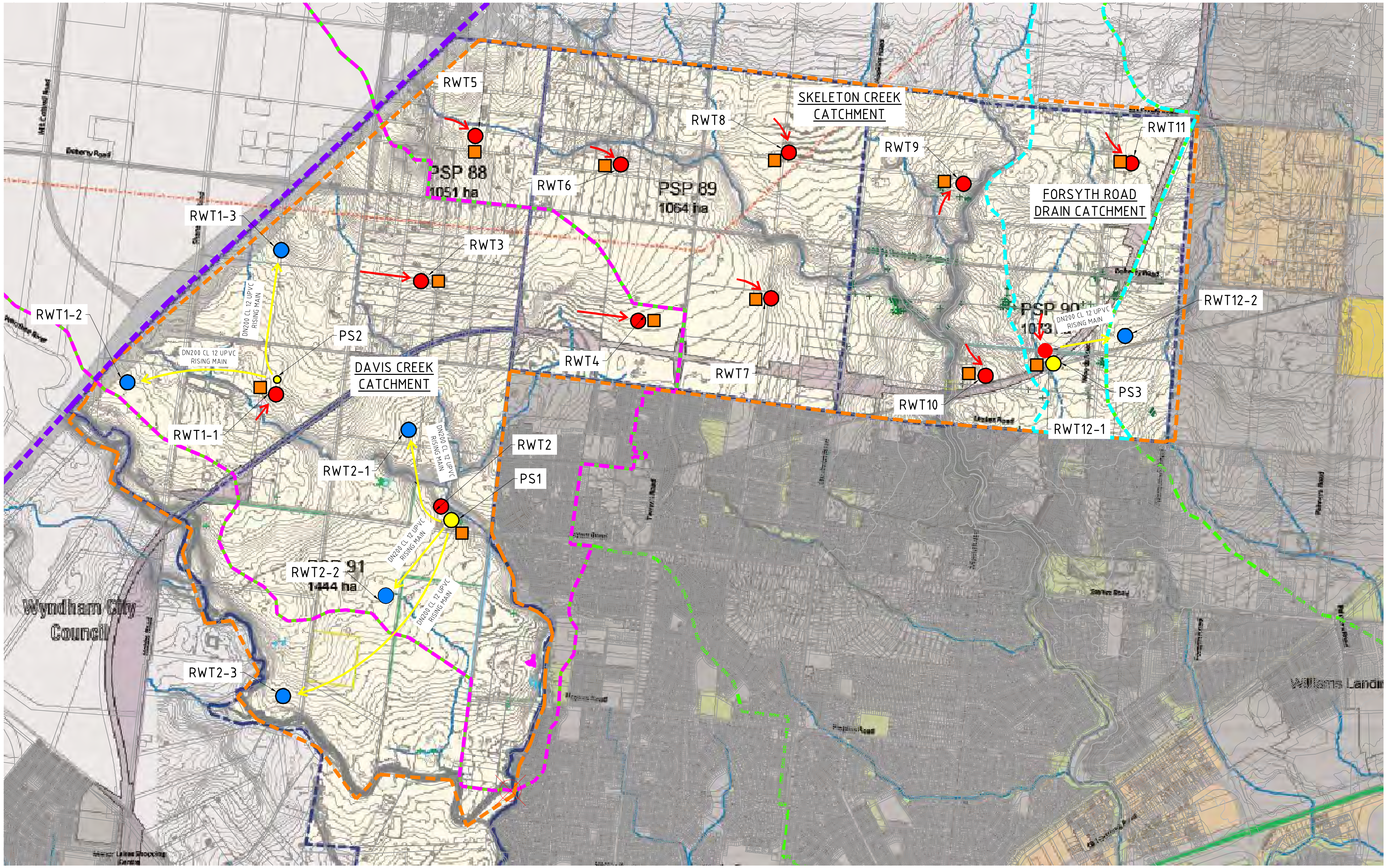

WATER REUSE: OPTION 2

(SCALE 1:20000, A1)

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LEGEND

- WYNDHAM NORTH PSP BOUNDARY
- SKELETON CREEK CATCHMENT BOUNDARY
- DAVIS CREEK CATCHMENT BOUNDARY
- LIMIT OF URBAN GROWTH BOUNDARY
- MAJOR STORAGE TANK
- DAILY STORAGE BUFFER TANK
- PUMP STATION
- WATER TREATMENT FACILITY
- DIVERSION FROM WATERCOURSE, CATCHMENT OR ADJACENT STORMWATER QUALITY TREATMENT ASSET
- PUMPED TO STORAGE

Name	Volume (kL)	Reuse supplied (kL)	Reliability
RWT1-1	7500	139000	65%
RWT1-2	410	-	-
RWT1-3	410	-	-
RWT2	4000	132000	72%
RWT2-1	410	-	72%
RWT2-2	410	-	72%
RWT2-3	310	-	72%
RWT3	2000	42000	68%
RWT4	1400	38000	67%
RWT5	950	25000	71%
RWT6	1500	30500	67%
RWT7	2000	38000	62%
RWT8	7000	53000	60%
RWT9	1500	28000	71%
RWT10	1500	52000	84%
RWT11	1600	36000	67%
RWT12-1	1500	79000	65%
RWT12-2	800	-	-

NB: PUMP STATIONS OPERATING AT 50 LITRES PER SECOND.
INFLOW TO TANKS CAPPED AT 100 LITRES PER SECOND.
MINIMUM TANK SIZING BASED ON 7 DAY STORAGE AT 60% RELIABILITY DURING JANUARY.

Rev	Amendments	App'd	Date
C	AMENDMENTS AS PER GAA COMMENTS	L.H.	22/05/13
B	TANK VOLUMES AMENDED	L.H.	24/01/13
A	ISSUED FOR DISCUSSION	L.H.	29/10/12

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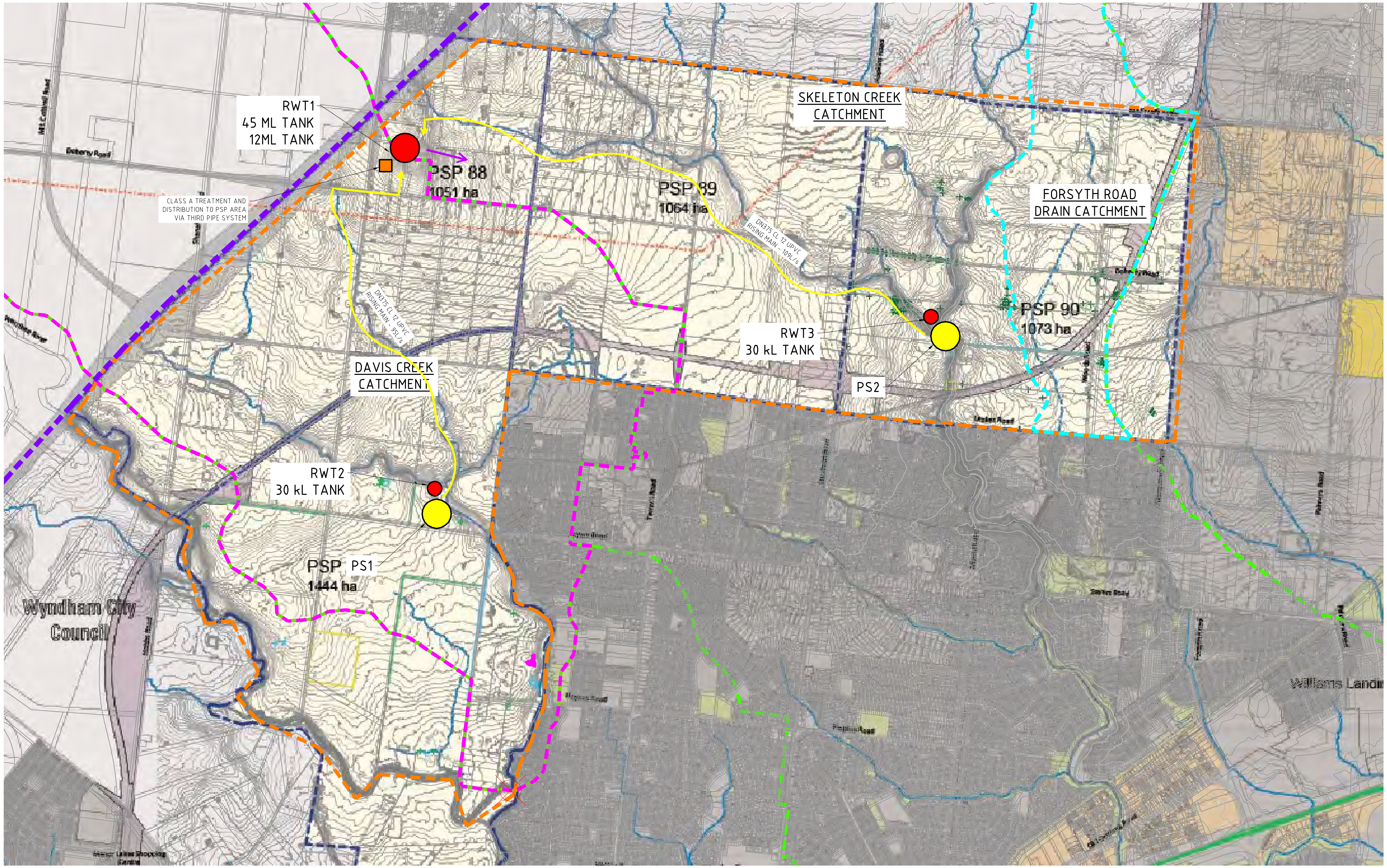
Drg No 137654RU02



WATER REUSE: OPTION 3

(SCALE 1:20000, A1)

LEGEND

- WYNDHAM NORTH PSP BOUNDARY
- SKELETON CREEK CATCHMENT BOUNDARY
- DAVIS CREEK CATCHMENT BOUNDARY
- LIMIT OF URBAN GROWTH BOUNDARY
- TANK
- PUMP STATION
- PUMPED TO STORAGE
- TO PSP AREA VIA THIRD PIPE DISTRIBUTION SYSTEM
- WATER TREATMENT FACILITY

Name	Volume (kL)	Reuse supplied (kL)	Reliability
RWT1	45000	1450000	60%

Rev	Amendments	App'd	Date
C	AMENDMENTS AS PER GAA COMMENTS	L.H.	22/05/13
B	TANK VOLUMES AMENDED	L.H.	24/01/13
A	ISSUED FOR DISCUSSION	L.H.	29/10/12

file name 137654M02.dwg layout name R1-3
file location G:\13\137654\ACAD
plotted by Mitch Johnson plot date 24/5/2013 4:11 PM

Standard Drawing RDA1 - Version 20120911

Scale 1:15000 A1
Horizontal 0 150 300 450 600 750
Vertical 0 150 300 450 600 750

Designed PENDING
PENDING
Checked PENDING
PENDING
Authorised PENDING
PENDING

Map Reference MELWAY 359 J8
Sheet Number 03
Drg Status PRELIMINARY

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Innovation in Infrastructure

WYNDHAM NORTH PRECINCT STRUCTURE PLAN
SWMS OPTIONS
OPTION 3
GROWTH AREAS AUTHORITY
MUNICIPALITY

Rev C
Drg No 137654RU03

Schedule of Prices - Wyndham North PSP
Note: Any Altered schedule will be deemed to be non-conforming
Contract No: 137654 - Water Reuse Option 1
* Denotes New Item/Change in Item from Previous Schedule

(To be submitted with Tender)
Schedule of Prices - Rev 21/01/13

Item	Description	Qty	Unit	Rate	Amount
1	**STORMWATER TANK** Supply and installation of Stormwater reuse tank including all associated pipes, fittings, foundations, electical cabinents, elecronic talepetry and level sensors				
1.1	30 ML Stormwater Tank (RWT1)	1	Item	$ 7,200,000.00	$ 7,200,000.00
1.2	10 ML Stormwater Tank (RWT2)	1	Item	$ 2,400,000.00	$ 2,400,000.00
1.3	10 ML Stormwater Tank (RWT3)	1	Item	$ 2,400,000.00	$ 2,400,000.00
1.4	12 ML Stormwater Tank (RWT4)	1	Item	$ 2,880,000.00	$ 2,880,000.00
2	**RISING MAIN** Supply and install rising main including all pipes, fittings, valves and anchor blocks				
2.1	375dia Class 12 UPVC Pipe (PS1)	4450	Lm	$ 450.00	$ 2,002,500.00
2.2	375dia Class 12 UPVC Pipe (PS2)	6000	Lm	$ 450.00	$ 2,700,000.00
2.3	200dia Class 12 UPVC Pipe (PS3)	1000	Lm	$ 250.00	$ 250,000.00
3	**PUMPING STATION** Supply and install Pumping station and associated infrastructure including pumps, pump wells, diversion weirs, pits, electrical cabinents and level sensors				
3.1	100L/s at 65m Head Stormwater PS (PS1)	1	Item	$ 1,500,000.00	$ 1,500,000.00
3.2	100L/s at 75m Head Stormwater PS (PS2)	1	Item	$ 1,500,000.00	$ 1,500,000.00
4	**WATER TREATMENT SYSTEM** Supply and install complete water treatment system including media filtration, microfiltration, ultrafiltration, UV treatment and chlorene dosing				
4.1	200 L/s Water Treatment Facilty	1	Item	$ 2,000,000.00	$ 2,000,000.00
	SUBTOTAL PROVISIONAL ITEMS (Ex GST)				**$ 24,832,500.00**
	20% CONTINGENCY				**$ 4,966,500.00**
	TOTAL FOR STORMWATER REUSE CONSTRUCTION (Ex GST)				**$ 29,799,000.00**

Schedule of Prices - Wyndham North PSP
Note: Any Altered schedule will be deemed to be non-conforming
Contract No: 137654 - Water Reuse Option 2
* Denotes New Item/Change in Item from Previous Schedule

(To be submitted with Tender)
Schedule of Prices - Rev 21/01/13

Item	Description	Qty	Unit	Rate	Amount
1	**STORMWATER TANK**				
	Supply and installation of Stormwater reuse tank including all associated pipes and fittings, foundations, pumping equipment and electical cabinents				
1.1	7.5 ML Stormwater Tank (RWT1-1)	1	Item	$ 1,800,000.00	$ 1,800,000.00
1.2	0.41 ML Stormwater Tank (RWT1-2)	1	Item	$ 98,400.00	$ 98,400.00
1.3	0.41 ML Stormwater Tank (RWT1-3)	1	Item	$ 98,400.00	$ 98,400.00
1.4	4 ML Stormwater Tank (RWT2)	1	Item	$ 960,000.00	$ 960,000.00
1.4	0.41 ML Stormwater Tank (RWT2-1)	1	Item	$ 98,400.00	$ 98,400.00
1.5	0.41 ML Stormwater Tank (RWT2-2)	1	Item	$ 98,400.00	$ 98,400.00
1.6	0.31 ML Stormwater Tank (RWT2-3)	1	Item	$ 74,400.00	$ 74,400.00
1.7	2 ML Stormwater Tank (RWT3)	1	Item	$ 480,000.00	$ 480,000.00
1.8	1.4 ML Stormwater Tank (RWT4)	1	Item	$ 336,000.00	$ 336,000.00
1.9	0.95 ML Stormwater Tank (RWT5)	1	Item	$ 228,000.00	$ 228,000.00
1.1	1.5 ML Stormwater Tank (RWT6)	1	Item	$ 360,000.00	$ 360,000.00
1.11	2 ML Stormwater Tank (RWT7)	1	Item	$ 480,000.00	$ 480,000.00
1.12	7 ML Stormwater Tank (RWT8)	1	Item	$ 1,680,000.00	$ 1,680,000.00
1.13	1.5 ML Stormwater Tank (RWT9)	1	Item	$ 360,000.00	$ 360,000.00
1.14	1.5 ML Stormwater Tank (RWT10)	1	Item	$ 360,000.00	$ 360,000.00
1.15	1.6 ML Stormwater Tank (RWT11)	1	Item	$ 384,000.00	$ 384,000.00
1.16	1.6 ML Stormwater Tank (RWT12-1)	1	Item	$ 384,000.00	$ 384,000.00
1.17	0.8 ML Stormwater Tank (RWT12-2)	1	Item	$ 192,000.00	$ 192,000.00
2	**PUMPING STATION**				
	Supply and install Pumping station and associated infrastructure including pumps, pump wells, diversion weirs, pits, electrical cabinents and level sensors				
2.1	50L/s Stormwater Pumping Station (PS1)	1	Item	$ 1,000,000.00	$ 1,000,000.00
2.2	50L/s Stormwater Pumping Station (PS2)	1	Item	$ 1,000,000.00	$ 1,000,000.00
2.3	20L/s Stormwater Pumping Station (PS3)	1	Item	$ 800,000.00	$ 800,000.00
3	**RISING MAIN**				
	Supply and install rising main including all pipes, fittings, valves and anchor blocks				
3.1	DN200 Class 12 UPVC Pipe (PS1, 2-1)	900	Lm	$ 250.00	$ 225,000.00
3.2	DN200 Class 12 UPVC Pipe (PS1, 2-2)	800	Lm	$ 250.00	$ 200,000.00
3.3	DN200 Class 12 UPVC Pipe (PS1, 2-3)	2280	Lm	$ 250.00	$ 570,000.00
3.4	DN200 Class 12 UPVC Pipe (PS2, 1-2)	1250	Lm	$ 250.00	$ 312,500.00
3.5	DN200 Class 12 UPVC Pipe (PS2, 1-3)	1150	Lm	$ 250.00	$ 287,500.00
4	**STORMWATER DIVERSION**				
	Supply and install stormwater diversion to meet site specific requirements including pipes, pits, connections, endwalls and rock outlets				
4.1	DN 600 RC Pipe (RWT3)	500	Lm	$ 500.00	$ 250,000.00
4.2	DN 600 RC Pipe (RWT4)	500	Lm	$ 500.00	$ 250,000.00
4.3	DN 600 RC Pipe (RWT5)	200	Lm	$ 500.00	$ 100,000.00
4.4	DN 600 RC Pipe (RWT6)	200	Lm	$ 500.00	$ 100,000.00
4.5	DN 600 RC Pipe (RWT7)	500	Lm	$ 500.00	$ 250,000.00
4.6	DN 600 RC Pipe (RWT8)	250	Lm	$ 500.00	$ 125,000.00
4.7	DN 600 RC Pipe (RWT9)	250	Lm	$ 500.00	$ 125,000.00
4.8	DN 600 RC Pipe (RWT10)	250	Lm	$ 500.00	$ 125,000.00
4.9	DN 600 RC Pipe (RWT11)	330	Lm	$ 500.00	$ 165,000.00

Item	Description	Qty	Unit	Rate	Amount
4.10	DN 600 RC Pipe (RWT12)	250	Lm	$ 500.00	$ 125,000.00
4	**WATER TREATMENT SYSTEM** Supply and install complete water treatment system including media filtration, microfiltration, ultrafiltration, UV treatment and chlorene dosing				
4.1	20 L/s Water Treatment Facilty	9	Item	$ 1,200,000.00	$ 10,800,000.00
4.2	50 L/s Water Treatment Facilty	2	Item	$ 1,500,000.00	$ 3,000,000.00
	SUBTOTAL PROVISIONAL ITEMS (Ex GST)				**$ 28,282,000.00**
	20% CONTINGENCY				**$ 5,656,400.00**
	TOTAL FOR STORMWATER REUSE CONSTRUCTION (Ex GST)				**$ 33,938,400.00**

Schedule of Prices - Wyndham North PSP
Note: Any Altered schedule will be deemed to be non-conforming
Contract No: 137654 - Water Reuse Option 3
* Denotes New Item/Change in Item from Previous Schedule

(To be submitted with Tender)
Schedule of Prices - Rev 21/01/13

Item	Description	Qty	Unit	Rate	Amount
1	**STORMWATER TANK**				
	Supply and installation of Stormwater reuse tank including all associated pipes and				
	fittings, foundations, pumping equipment and electical cabinents				
1.1	45 ML Stormwater Tank (RWT1-1)	1	Item	$ 10,800,000.00	$ 10,800,000.00
1.2	11 ML Stormwater Tank (RWT1-2)	1	Item	$ 2,640,000.00	$ 2,640,000.00
1.3	30 kL Stormwater Tank (RWT2)	1	Item	$ 7,200.00	$ 7,200.00
1.4	30 kL Stormwater Tank (RWT3)	1	Item	$ 7,200.00	$ 7,200.00
2	**PUMPING STATION**				
	Supply and install Pumping station and associated infrastructure including pumps, pump wells, diversion weirs, pits, electrical cabinents and level sensors				
2.1	95L/s at 65m Head Stormwater PS (PS1)	1	Item	$ 1,500,000.00	$ 1,500,000.00
2.2	109L/s at 100m Head Stormwater PS (PS2)	1	Item	$ 1,500,000.00	$ 1,500,000.00
3	**RISING MAIN**				
	Supply and install rising main including all pipes, fittings, valves and anchor blocks				
2.1	375dia Class 12 UPVC Pipe (PS1)	4450	Lm	$ 450.00	$ 2,002,500.00
2.2	375dia Class 12 UPVC Pipe (PS2)	6000	Lm	$ 450.00	$ 2,700,000.00
4	**WATER TREATMENT SYSTEM**				
	Supply and install complete water treatment system including media filtration, microfiltration, ultrafiltration, UV treatment and chlorene dosing				
4.1	200L/s Water Treatment Facilty	1	Item	$ 2,000,000.00	$ 2,000,000.00
	SUBTOTAL PROVISIONAL ITEMS (Ex GST)				**$ 23,156,900.00**
	20% CONTINGENCY				**$ 4,631,380.00**
	TOTAL FOR STORMWATER REUSE CONSTRUCTION (Ex GST)				**$ 27,788,280.00**