JL Murphy Reserve Integrated Water Management Assessment
City of Port Phillip
March 2010

# JL Murphy Reserve Integrated Water Management Assessment

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# JL Murphy Reserve Integrated Water Management Assessment

Prepared for

City of Port Phillip

Prepared by

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# Executive Summary

AECOM Design + Planning were engaged by the City of Port Phillip to undertake an integrated water management assessment for JL Murphy Reserve. The water management assessment will inform the broader masterplanning process that is currently being undertaken for the reserve.

**Purpose and objectives**

The City of Port Phillip recently prepared a Draft Water Plan setting out goals and objectives for integrated water management for the municipality as follows:

- Reducing council potable water use by 70% or 363 ML/year from 2000/2001 levels (resulting in a target water use of 155 ML/year, note that projected water use without restrictions is 212 ML/year resulting in a future target reduction of 57 ML/year)
- Supplying 50% of projected future outdoor water use from alternative sources (78 ML/year)
- Achieving reductions in annual stormwater pollutant loads to Port Phillip Bay by 2020 of:
  - 19%-24% for total suspended solids (TSS)
  - 15%-19% for total phosphorus (TP)
  - 10%-13% for total nitrogen (TN)

The overall purpose of the integrated water management strategy for Murphy Reserve is to sustainably manage water for the reserve. A number of objectives have been identified as follows:

- Provide a reliable alternative source of water for the reserve
- Improve access to sports fields
- Improve passive recreational areas and the aesthetics of the reserve
- Increase green space and minimise infrastructure including fencing
- Contribute towards the achievement of the City of Port Phillip's water management and stormwater pollutant reduction objectives

The major water demand at the reserve is irrigation for the sports fields with a volume of 27,800 kL/year. A range of potential options were considered to reduce the reliance on mains water supply and included:

- Demand management (irrigation systems, vegetation type)
- Stormwater harvesting
- Existing wastewater recycling system (Barry Brothers)
- Synthetic turf
- Neighbouring opportunities (Direct roof runoff capture)
- Groundwater
- Aquifer storage and recovery
- Sewage recycling

The outcomes of the options assessment have been briefly summarised as follows:

- Stormwater harvesting was identified as the option with the greatest potential to supply a substantial proportion of the demand. A range of options with respect to diversion location, pumps, treatments and storage and a recommended stormwater harvesting scheme is detailed in the report.
- Council has already achieved reductions in mains water use through effective demand management. There are limited opportunities remaining for further reductions through demand management strategies at the site.

- The existing wastewater recycling system was found to have significant supply and reliability constraints. While it has limited potential as a reliable supply it could be used to augment a broader system.
- Synthetic turf presents a broad range of questions and issues including access to playing surfaces, management, costs and urban heat island effects. The main benefits are increased accessibility while the costs are significant. The water management benefits are uncertain and depend on the type of system, its use and the watering requirements for temperature control and cleaning. Synthetic turf is presently being proposed for the soccer training field. Given the breadth of issues raised and the small part it may play in the water management strategy, it was considered that the issue of synthetic turf needs to be more broadly reviewed by council.
- The proximity of large roof areas on neighbouring properties presented a potential opportunity for direct capture of roof runoff for reuse on sports fields. While such a system has merit, there are significant difficulties with establishing a partnership with local industry and comprehensive distribution network to capture roof runoff within private property. Given the availability of water from the Plummer Street Drain and the likelihood that additional stormwater harvesting would be required there were limited benefits that could be realised from such a scheme.
- Groundwater in the area was found to have relatively low yields, high salinities and other chemical properties making it unfavourable for turf irrigation. There is also difficulty in quantifying sustainable yields if aquifer storage and recovery is not part of a groundwater scheme. On this basis the use of groundwater is not recommended as it is unlikely to be a viable long term source.
- The potential for aquifer storage and recovery was considered. Given the low groundwater yields it is likely that it would be costly. It would require supporting surface treatment and temporary storage infrastructure to hold stormwater prior to discharge into the aquifer and introduces significant technical challenges and risk to the project.
- The main benefit of sewage recycling over alternatives such as stormwater harvesting is the reliability of supply, reducing storage requirements. However treatment costs and energy requirements for recycled water are significantly higher than for stormwater harvesting.

**Stormwater harvesting**

Stormwater harvesting was identified as the most sustainable option. Further analysis was undertaken to evaluate a range of stormwater harvesting opportunities. These included the location and height of the diversion weir, minimising pumping (and therefore energy) requirements, treatment type and dimensions, and the required capacity of irrigation storage tanks.

Surrounding catchment areas were identified and modelled to estimate potential stormwater yields. The Plummer Street drain located adjacent to the reserve was identified as having stormwater discharges of 103,000 kL/year, of which is well in excess of the potential demand of 27,900 kL/year. This volume of stormwater is substantially reduced by a range of factors including tidal influences, pumping limitations, influences of treatment and storage system capacities and the mismatch in timing of stormwater flows and irrigation demands.

The proposed scheme provides a reliable water supply while simultaneously establishing a significant landmark wetland feature to improve the aesthetics of the reserve and create new opportunities for recreational activities. In choosing this option, consideration was given to cost as well as the broader objectives of the reserve's masterplan. It is proposed that the wetland treatment will be integrated within a broader landscape including facilities such as BBQ areas, shelter, mounds and walking paths.

The preferred stormwater harvesting scheme can be summarised as follows:

- Diversion weir in Plummer Street Drain at 0.6m AHD
- Diversion pump with capacity of 250 L/s
- Relief swale up to 10m wide
- Rising main to treatment system of approximately 600 m
- Wetland with 500 $m^2$ inlet sedimentation pond and macrophyte zone up to 3,500 $m^2$ with permanent pool volume of 750 $m^3$ and extended detention depth of 0.5 m.
- Irrigation storage tanks with a capacity of 1,750 kL

The proposed stormwater harvesting scheme, wetlands and additional landscaping and facilities will deliver a range of benefits to the community including:

- A reliable source of water providing an average of 21 ML/year for irrigating sports fields. Given a total demand of 28 ML/year this provides a reliability of 75%
- Increased access to sporting fields and higher participation rates for local sporting clubs
- A greatly enhanced passive recreational space
- A landmark entrance feature to the reserve greatly enhancing its appearance and amenity
- Substantial water savings of 21 ML/year achieving more than 37% of the City of Port Phillip's potable water use reduction and 25% of the alternative water source targets for 2020 (these equate to 370% and 250% of the works required annually)
- A significant contribution towards reducing stormwater pollutant loads to the bay, particularly nitrogen (TN) with the following load reductions and percentages of the works required to achieve the load reduction target each year:

**Table 0-1 Reductions in stormwater pollutant loads**

| Pollutant | Reduction in pollutant load (kg/year) | Percentage of catchment load | Percentage of reasonable and achievable target* | Percentage of aspirational target* |
|---|---|---|---|---|
| Total suspended solids (TSS) | 3,624 | 35% | 33% | 24% |
| Total phosphorus (TP) | 7.4 | 30% | 41% | 29% |
| Total nitrogen (TN) | 53 | 27% | 60% | 43% |

*Percentages expressed relative to pollutant load reductions to be achieved in each given year

The cost of the stormwater harvesting scheme including the wetland is estimated at approximately $1.94M including design and management.

# 1.0 Introduction

## 1.1 Purpose of this report

AECOM Design + Planning were engaged by the City of Port Phillip to undertake an integrated water management assessment for JL Murphy Reserve. A new management and masterplan is presently being prepared for the reserve. The Integrated Water Management Assessment will inform the master planning process by providing an understanding of likely future water demands at the reserve, the potential to reduce these and opportunities to use alternate sources of water to meet site demands including stormwater.

## 1.2 Project context

J.L. Murphy Reserve is located in Port Melbourne and is bounded by Williamstown Road on the south, Graham Street to the East and Plummer Street to the north. The reserve is approximately 12.2 ha including a council depot with an area of 0.8 ha. The reserve consists of two formal cricket ovals, a baseball field, a soccer field and training area and surrounding areas for passive recreation. Facilities at the reserve include a soccer pavilion, a sports pavilion containing a kiosk, toilet and change room facilities, a playground and a BBQ area.

The reserve is one of Port Phillip's largest and most highly used reserves providing for a range of sporting and passive recreational activities. These include football, cricket, soccer and baseball. The Hobson's Bay dog obedience school also uses open space areas on the reserve and the Dig-In Community Garden is located to the west of the soccer pavilion. The reserve is additionally used for passive recreation pursuits such as walking, local community and business BBQ's and ball games.

The recent drought and introduction of water restrictions have reduced direct rainfall and the availability of mains water for irrigation purposes. This has adversely impacted on the accessibility and quality of playing surfaces for sporting clubs and the amenity of the surrounding passive recreational areas. There is a need for a water management strategy to ensure that the City of Port Phillip is able to reliably provide suitable playing surfaces and recreational areas to meet the needs of sports clubs and broader community.

## 1.3 Objectives

This integrated water management strategy for Murphy Reserve aims to:

- Provide a reliable alternative source of water to meet irrigation demands for the reserve
- Improve access to sports fields through improved health of playing surfaces
- Improve passive recreational areas and the aesthetics of the reserve
- Increase green space and minimise infrastructure including fencing
- Contribute towards the achievement of the City of Port Phillip's water management and stormwater pollutant reduction objectives

# 2.0 Background Review

Background information for this project was obtained from a review of a range of documents and plans. A brief summary of each is provided below followed by a summary of the policy framework and existing infrastructure at the reserve.

**Draft City of Port Phillip Water Plan**

The City of Port Phillip Water Plan sets out the vision and objectives of the City of Port Phillip for Integrated Water Management including water supply, stormwater and sewage. A water and pollutant balance was undertaken for the municipality to quantify current water use and pollutant loads. Water management targets for 2020 were established for the municipality based on the long term goals and analysis of the City of Port Phillip's capital work program for 2009/2010 (these are described in Section 2.1).

**Draft Open Space Water Plan**

The Draft Open Space Water Plan summaries Council's policy and plans for the management of water throughout the City of Port Phillip's parks, open spaces and sports fields. It identifies a hierarchy of priority sites for further investigation based on water requirements, usage and heritage and cultural considerations. The report provides broad guidance on alternative water source options and makes a series of recommendations to improve water management of open spaces. Key recommendations include the following:

- Current volumes of potable water for open space irrigation is not adequate to ensure the long term sustainability of these landscapes and addressing the shortfall must become a high priority
- Stormwater harvesting should be investigated where there is potential
- Cool season grasses should be converted to warm season grasses to reduce irrigation demands
- Improved turf management practices should be used in conjunction with alternative water sources

**Murphy Reserve 1999 Master Plan [*ASR Research*, 1999]**

The 1999 Master Plan for Murphy Reserve is being revised and a new master plan is currently being prepared. The master plan reviews the current and anticipated future use of the site and existing conditions of infrastructure and amenity areas.

**Contamination Assessment (underway)**

A preliminary desktop study of contamination has been completed by Environmental & Safety Professionals (ESP). The history of the site was reviewed to identify potential sources of contamination. A list of sources and potential contamination risks was completed for the various locations within the reserve.

**Strategies for Managing Sports Surfaces in a Drier Climate [*GHD*, 2007]**

GHD undertook a broad review of the impacts of drier climate conditions and opportunities to manage sports surfaces in a drier climate. A range of potential strategies including demand and turf management, irrigation methods, synthetic surfaces and the use of alternative water sources were described and typical approaches and costs were summarised.

## 2.1 Policy, plans and water budget

The Draft City of Port Phillip Water Plan and Draft Open Space Water Plan set out the overarching policy framework for integrated water management within the City of Port Phillip. The Water Plan sets the following goals and objectives for management of water across the municipality:

- Council reduction in potable water use of 70% from 2000/2001 (reduction from 518 ML/year to 155 ML/year)
- Council to supply of 50% of projected future outdoor water use from alternative sources (78 ML/year)
- Total reductions in pollutant loads by 2020 of:
    - 19%-24% for total suspended solids (TSS)
    - 15%-19% for total phosphorus (TP)
    - 10%-13% for total nitrogen (TN)

Achieving these goals and objectives will require council to continue integrating sustainable water management initiatives across the buildings and the broader urban landscape including demand management strategies and stormwater treatment and harvesting. The following objectives would need to be achieved in a given year:

- Council reduction in potable water use of 6 ML (reduce to 155ML by 2020)*
- Council increase in alternate water sources of 8 ML (78 ML by 2020)
- Total reductions in pollutant loads of
    - 10,973-15,255 kg/year
    - 18-25 kg/year
    - 88-122 kg/year

*This is based on an assumption that irrigation demands will increase to an ideal estimated demand of 155 ML/year if water restrictions are lifted. This would result in total Council water use of 212 ML/year with a further 57 ML/year of savings needed to return to the target water use of 155 ML/year.

Figure 2-1. shows the breakdown of water use for different purposes for the City of Port Phillip. A total of 150 ML/year is used with an estimated 93 ML/year of this being used for irrigation of public open space. While Council's water use is only a small fraction of the 8,179 ML/yr used across the municipality, its role as a leader in sustainable water management is crucial. Council's irrigation use has declined significantly from 368 ML/yr in 2000/2001 due to water restrictions, demand management and efficiency gains [*City of Port Phillip*, 2009]. It was estimated that optimum water use for council's open space would be approximately 155 ML/year. This was based on assumed irrigation demands of up to 15 mm/week for areas of varying importance with 15mm/week (780mm/year) considered to be optimum [*Cardno Grogan Richards*, 2009]. Current levels of irrigation are well below these and the reductions in irrigation of public open spaces have adversely impacted on the amenity and accessibility of these facilities.

Council's current and projected water use and objectives are summarised in Figure 2-2. Projected water use assumes water restrictions are lifted and council should aim to supply this additional volume through alternative water sources.

The Draft City of Port Phillip Water Plan summarises the water budget for the City, providing a strategy for sustainable water management. The City imports approximately 8,180 ML/year of potable water, discharges 7,690 ML/year of sewage and generates and 5,100 ML/year of stormwater as summarised in Figure 2-3.

Council's water management objectives for reductions in potable water use and sourcing alternative water supplies are complementary and attainment of the 50% target for alternative water sources by 2020 (equivalent to 78 ML/year) would enable the potable water target to also be achieved. If the water is sourced from stormwater, then the treatment and use of the water would also contribute to achieving the stormwater pollutant load reduction targets.

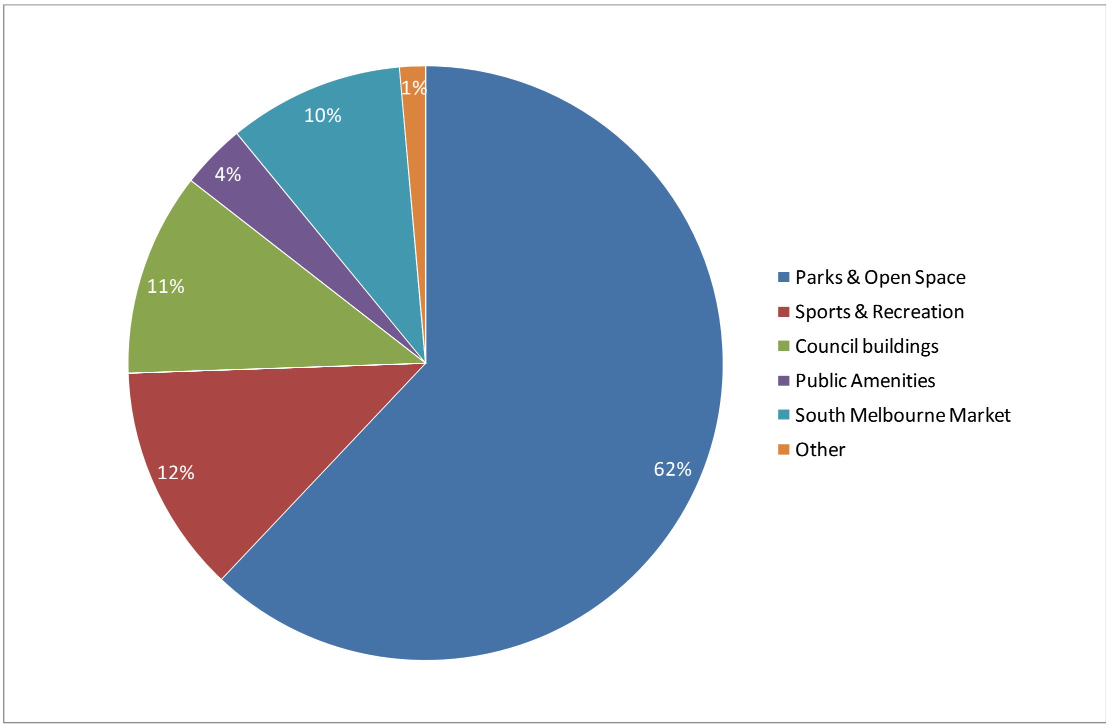


**Figure 2-1 Council water use 2008/09**

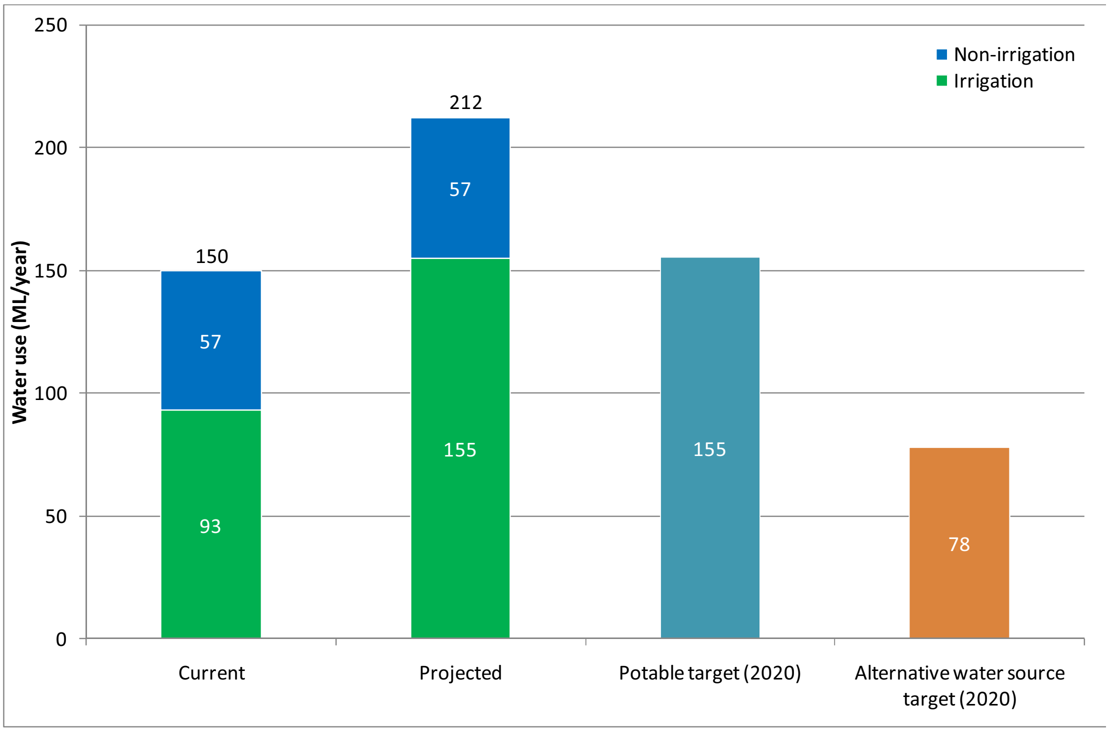


**Figure 2-2 Council water use and objectives**

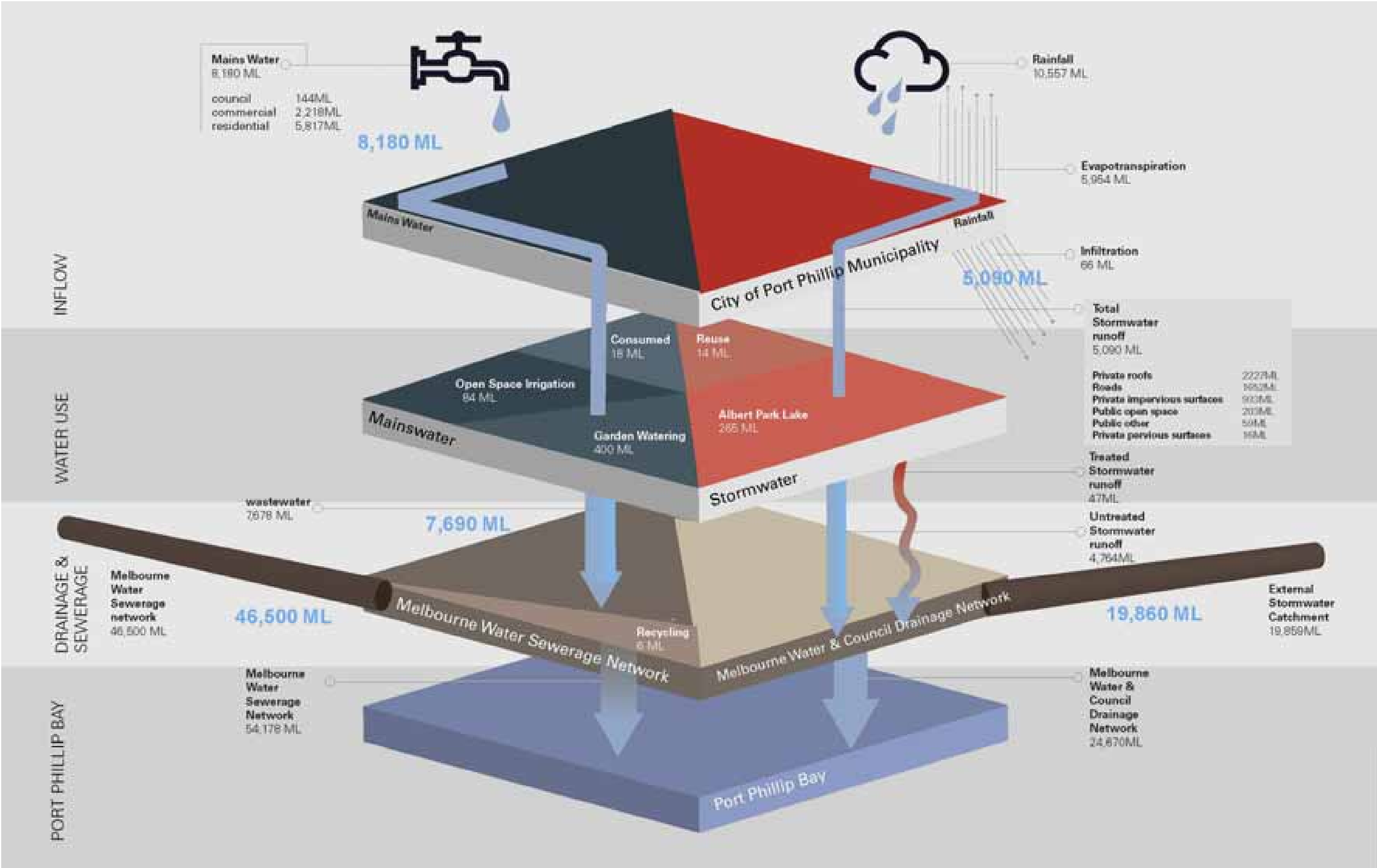


**Figure 2-3 City of Port Phillip water balance**

## 2.2 Existing conditions

Background information about current conditions, usage and infrastructure at Murphy Reserve was drawn together and summarised in this section to provide a basis for ongoing work.

### Description of sports fields

There are four sports ovals located within the reserve, see Figure 2-4. The Anderson Oval contains a soccer field for competition and a half soccer field used for training. The GS Williams Oval is used for baseball and parts of it for soccer fields at other times [*ASR Research*, 1999]. Ovals and usage are shown in Table 2-1.

**Table 2-1 Sports fields**

| Sports field | Use | Events per year* |
|---|---|---|
| Anderson Ovals 1&2 | Soccer | 261 |
| GS Williams Oval | Baseball and soccer | 105 |
| AT Aanenson Oval | Cricket and football | 135 |
| JM Woodruff Oval | Cricket and football | 129 |

*[*Cardno Grogan Richards*, 2009]


**Figure 2-4 Murphy Reserve sports fields and facilities**

Most of the ovals are vegetated with kikuyu. Anderson is vegetated with ryegrass and was recently line planted with couch grass, a warm season grass (Pers. Comm. Dwayne Cartwright). This will replace the rye grass over the next year. These grasses are suitable for their purpose provided irrigation levels are maintained. Kikuyu produces large amounts of thatch with a resultant spongy surface. A disadvantage is that it is dormant in winter when vigorous growth is needed to repair damage from football. Rye grass present in Woodruff Oval provides some winter grass growth [*ASR Research*, 1999]. The Aanenson Oval has a synthetic turf wicket. In 1999 Anderson and Woodruff ovals were in good condition and adequately irrigated while the grass surface of Williams and Aanenson were irregular and less vigorous due to lower irrigation levels [*ASR Research*, 1999]. The couch grass on Anderson oval will have a similar water use to kikuyu.

At present, the Anderson and Woodruff ovals are in reasonably good condition due to irrigation. The Williams oval has patchy, irregular and less vigorous growth, particularly further from the baseball diamond due to reduced irrigation. The Aanenson oval is not currently irrigated and the surface is patchy and in generally poor condition.


**Figure 2-5 (a) Woodruff oval, (b) Aanenson oval, (c) Williams oval**

The Williams oval is fitted with subsurface drip irrigation connected to a recycled water supply. The Anderson Oval, Aanenson Oval and Woodruff Oval are fitted with sprinkler irrigation. In 1999 the irrigation systems relied on manual (quick coupler) connections. Details of sports field vegetation and irrigation infrastructure are summarised in Table 2-2.

**Table 2-2 Sports field infrastructure**

| Sports field | Use | Dimensions* | Turf* | Irrigation** | Present Source** |
|---|---|---|---|---|---|
| Anderson Oval | Soccer | | Ryegrass/Couch | Sprinkler | Potable – Full exemption? |
| GS Williams Oval | Baseball | 130m x 80m | Kikuyu | Subsurface drip | Recycled water |
| AT Aanenson Oval | Cricket and football | 148m x 94m | Kikuyu | Sprinkler | None |
| JM Woodruff Oval | Cricket and football | 148m x 102m | Kikuyu with some ryegrass | Sprinkler | Potable – exemption |

*[*ASR Research*, 1999]. While the minimum requirements for high level play are 143m x 118m for football, 60m radius for cricket and 145m x 125m for baseball, the present fields are generally sufficient for current use.
**[*Cardno Grogan Richards*, 2009]

A water recycling plant is located in the Council Depot and operated by Barry Brothers. The plant recycles water from utility pits and is stored in the council tanks. Stormwater is treated to Class C standards and used for irrigating Williams oval, tree watering and street cleaning.

**Previous action**

Previous works to improve water management at the reserve include the following:

- Installation of a rainwater tank for toilet flushing
- Installation of storage at Council Depot on Graham Street with capacity of 410 kL.
- Water recycling plant operated by Barry Brothers (estimates indicate up to 4,000 ML/yr is available)
- Installation of subsurface drip irrigation system into GS Williams oval with recycled water sourced from council tanks
- Over-seeding of Anderson oval with couch grass

**Proposed upgrade works**

The following future works have also been proposed:

- Upgrade existing irrigation system to Best Practice Standard across Woodruff Oval, Aanenson Oval and Anderson Oval (Soccer 1).
- Upgrade existing irrigation system to Best Practice Standard – Soccer 2 scheduled 2009/2010.

**Other areas**

*BBQ Area*

The facilities are in good condition and functional although the setting has been noted to be unattractive and cramped [*ASR Research*, 1999].

*Council Depot*

The Council Depot area is used for storage and is not part of the reserve. Two council storage tanks and a privately operated water recycling plant are located within the depot. There is potential that this site could be used to locate tanks for storing treated stormwater or recycled water.

*Passive Recreational Areas*

There is generally no irrigation of passive recreational areas within the reserve given present water restrictions and council's approach to minimise irrigation of such areas. The turf area between the pavilion and Graham Street was historically automatically irrigated [*ASR Research*, 1999]. The area to the west of the pavilion is not irrigated and has little planting. The surface levels and drainage are adequate for non-sporting areas. There is an important historical vegetated area surrounding the pavilion with well established trees. At present, grassed passive recreational areas are in relatively poor condition due to limited rainfall in recent years and the absence of irrigation.

There are dense mature plantings of Grey Poplar, Lombardy Black Poplar and Radiata Pine along Plummer Street and near the pavilion [*ASR Research*, 1999]. Many of these trees along Plummer Street are reaching the end of their lifespan and additional plantings are required to maintain the woodland character into the future. There are mature Spotted Gums near the ovals and a line of Ash trees along the Williamstown Road frontage.

**Soil contamination**

The Preliminary Environmental Site (PES) Assessment investigated the history of the site and found that it has largely been used as a recreational area for the past 50 years. It is possible there may be residual contamination of the soils from the earlier land uses. There may be asbestos contamination across the site from building rubble in fill soils. The depot area is considered likely to be contaminated, with various historical uses potentially resulting in hydrocarbon and other contamination.

The Port Melbourne rifle range reportedly used part of the site along Williamstown Road up to the 1930's although the exact location is unknown. From 1942-1954 the reserve was occupied by an Army Ordnance Depot. The buildings to the west of the reserve have had a range of activities possibly including paint manufacturing and army storage. The sub-station is also likely to be a source of contaminants including poly chlorinated biphenyls (PCB's). The PES assessment recommends that targeted testing be undertaken at the site for any proposed works to identify any soil contamination risk issues.

# 3.0 Water and pollutant balance

## 3.1 Catchment

The catchment area upstream of Murphy Reserve drains to the Bay via the Rosny St Main Drain, a Melbourne Water drain that runs down Salmon Street close to the west side of the reserve. Council drains on Plummer Street, Salmon Street and Williamstown Road drain to the Rosny St Main Drain. The total catchment area is 105 ha and is predominantly zoned as business and industrial. The reserve is zoned public park and recreation, the council depot is zoned public use and the balance is road areas. Much of the catchment is highly impervious due to the large business/industrial buildings and surrounding paved areas. The significant pervious spaces are the reserve itself and the area surrounding the on-ramp to the freeway.

The catchment was divided into three subcatchment areas to allow for consideration of different diversion points. The catchments are shown in Figure 3-1 with a breakdown of pervious areas, impervious lot areas and impervious road areas. The catchments, areas and impervious fractions are summarised in Table 3-1. Mean annual flows were estimated at 103 ML/yr for Catchment A and 310 ML/yr for the whole catchment area to Williamstown Road.

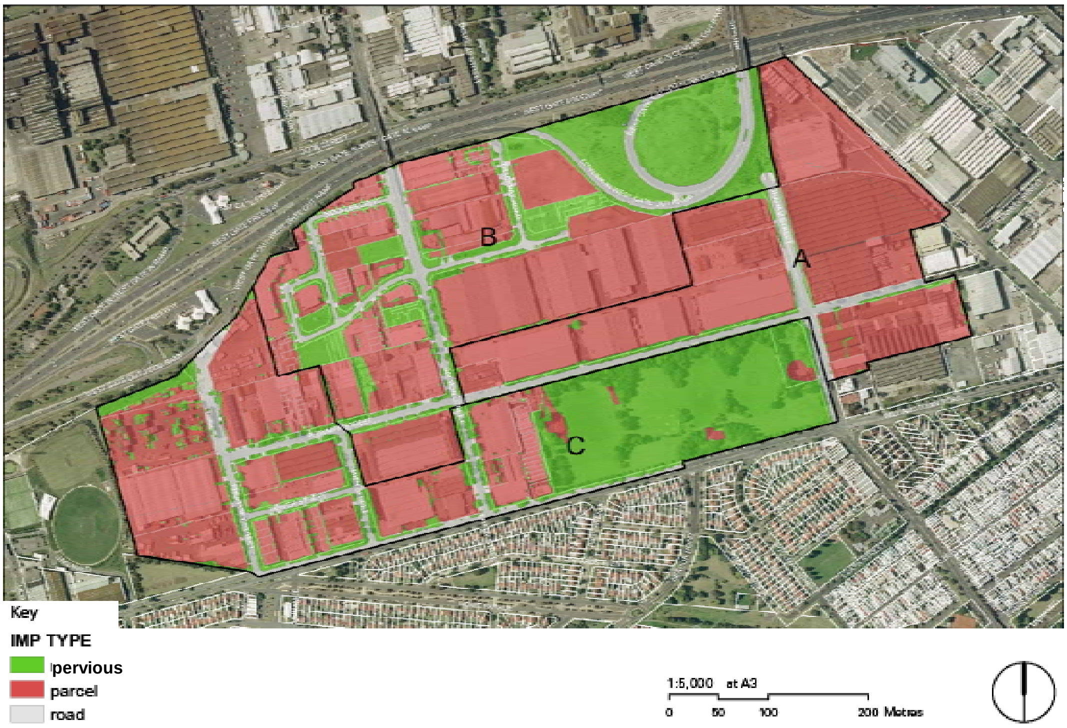


**Figure 3-1 Catchments and impervious breakdown**

**Table 3-1 Catchments**

| Catchment | Description | Area (ha) | Impervious Fraction | Mean annual flow (ML/yr) |
|---|---|---|---|---|
| A | Plummer St Drain to Salmon St | 25.89 | 95% | 103 |
| B | Rosny St Drain at Plummer St and Salmon St | 40.64 | 66% | 113 |
| C | Rosny St Drain at Williamstown Rd and Salmon St | 38.30 | 58% | 94 |
| Total | - | 104.84 | 70% | 310 |

**Figure 3-2 Surrounding catchment areas are mostly commercial and industrial**

## 3.2 Stormwater infrastructure

The catchment area surrounding Murphy Reserve drains entirely to Port Phillip Bay via Melbourne Water's Rosny Street Main Drain. Two drains were identified as potentially being useful for stormwater reuse, Melbourne Water's Rosny Street Main Drain and Council's Plummer Street drain. The layout of the stormwater infrastructure is shown in Figure 3-3.

The Rosny Street Main Drain is 1,575 mm in diameter at its upper end at the corner of Plummer Street and Salmon Street, just west of the northwest corner of Murphy Reserve. Its diameter increases to 1,675 mm as it runs south along Salmon Street. Manholes are located along Salmon Street at the corners of Plummer Street, Taver Street and Williamstown Road as well as in the reserve on Howe Parade. The Plummer St council drain runs along the north side of Plummer Street to the north of the reserve from east to west, increasing in diameter from 1,350 mm to 1,575 mm.

Pipe invert levels are summarised in Table 3-2 and illustrated in Figure 3-4. Sea levels were included for comparison and it can be seen that the invert levels are just above or below mean sea level. This means that water levels in the pipes will be tidally influenced. Therefore, a weir or tidal gate is required to allow stormwater to be extracted from the drains. It is also possible that groundwater levels will be close to the invert level and saline groundwater intrusion may also occur, particularly in the lower reaches below Williamstown Road.

The area is relatively low lying and some parts of the catchment are at risk of flooding in a 1 in 100 year storm event. Flooding risk areas are summarised in Figure 3-5 and Figure 3-6.

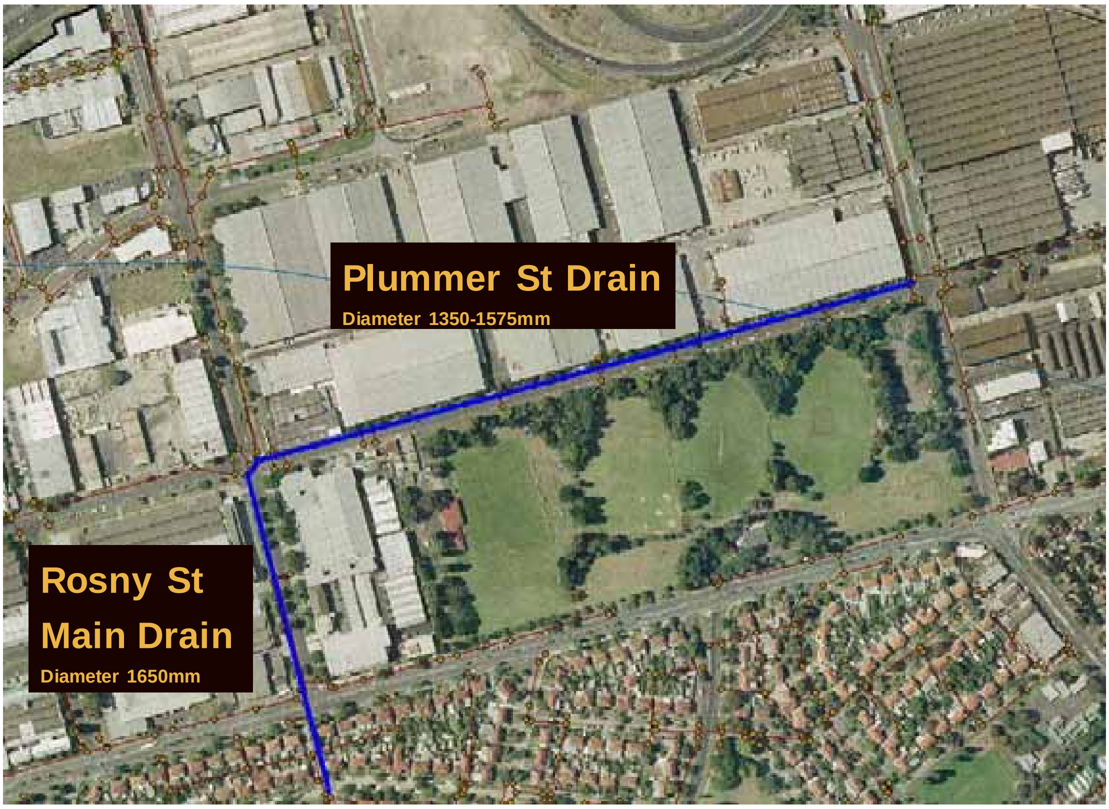


**Figure 3-3 Stormwater infrastructure**

**Table 3-2 Manhole locations and invert levels**

| Manhole | Drain invert level (m AHD) |
|---|---|
| Rosny St Main Drain at Plummer Street | 0.1920 |
| Rosny St Main Drain at Taver Street | 0.03962 |
| Rosny St Main Drain at Williamstown Road | -0.0427 |
| Rosny St Main Drain at Reserve between Edwards Avenue and Howe Parade | -0.1829 |
| Mean sea level | 0.059 |
| Median sea level | 0.057 |
| Maximum sea level 2009 | 1.14 |
| Minimum sea level 2009 | -0.62 |

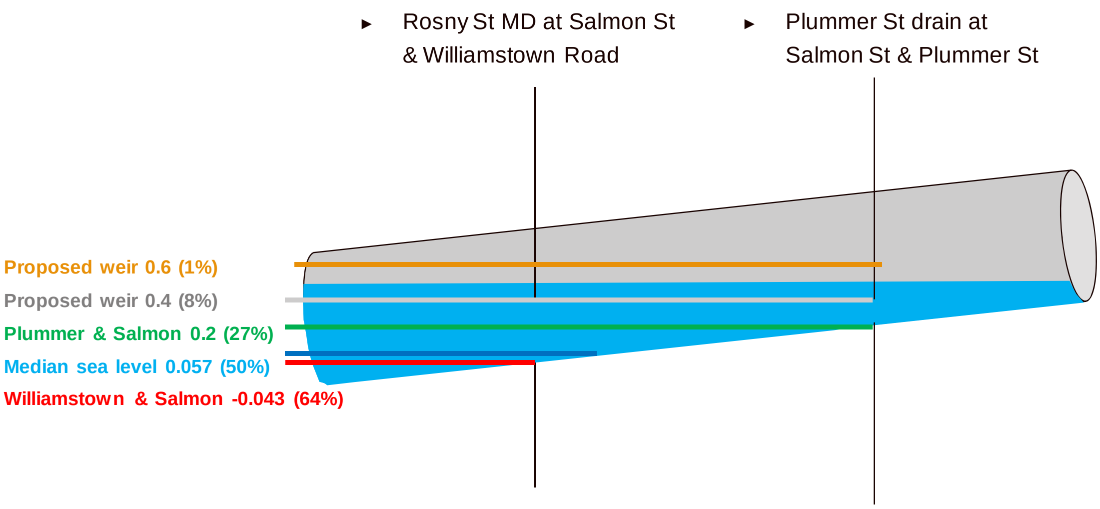


**Figure 3-4 Drain invert and tidal levels**

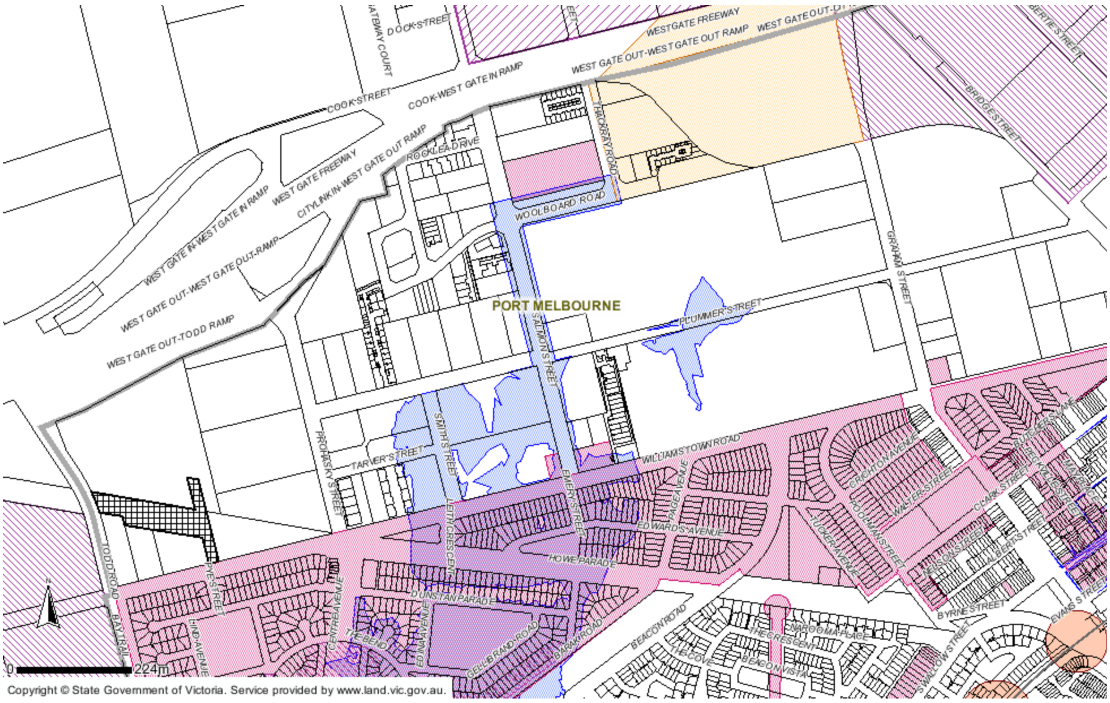


**Figure 3-5 Planning scheme floodplain overlay**

**Figure 3-6 Melbourne Water 1 in 100 year flood plain**

## 3.3 Water demand

Irrigation is the main water use within the reserve. There is also significant water use at the pavilions, particularly the soccer pavilion and reported groundwater use for the Dig In Community Garden.

The Anderson oval was planted with ryegrass, a cool season grass. It is estimated that the crop factor for this grass is approximately 0.8, equating to a water use of 780 mm/yr or 15 mm/week. This is consistent with the 15 mm/week identified as being required for optimal turf irrigation [*Cardno Grogan Richards*, 2009]. Anderson oval has recently been over-seeded with couch grass, a warm season grass and is expected to pre-dominantly couch grass within the next year. The Williams, Aanenson and Woodruff ovals are planted with kikuyu, a warm season grass. It is estimated that the crop factor for these warm season grasses is approximately 0.6, equating to a water use of 520 mm/yr or 10 mm/week. Based on these, estimated irrigation demands are shown in Table 3-3.

There is potential for the irrigation demands to vary depending on future management options. Changes in demands will result if different grasses are used, different watering regimes are adopted, additional areas are irrigated or if synthetic turf is used for one or more of the ovals. *It is assumed in these estimates that the irrigation systems are operating at optimum efficiency*. It is possible that the existing systems are operating at below this level and there is opportunity to improve the systems to reduce water demand.

The information available for water use for the pavilions is highly variable and shows strong seasonality which may indicate higher patronage use during summer. It is recommended that further investigation of the water supply system be conducted by council to confirm water usage patterns. This may require separate metering of indoor and outdoor demands. At this time, indoor demands have not been included in the analysis.

**Table 3-3 Estimated sports field irrigation demands**

| Oval | Area ($m^2$) | Estimated irrigation demand (kL/yr) |
|---|---|---|
| SS Anderson Oval | 16,900 | 8,812* |
| GS Williams Oval | 12,317 | 6,422 |
| AT Aanenson Oval | 11,911 | 6,211 |
| JM Woodruff Oval | 12,362 | 6,446 |
| Total | 53,490 | 27,891 |

*Estimated at 13,218 kL/year with ryegrass

## 3.4 Water quality

Stormwater contains a range of other pollutants including but not limited to litter (gross pollutants), suspended solids, nutrients, heavy metals, pesticides and other toxicants and pathogens. The catchment is highly industrial, consisting mostly of warehouses and there is potential for stormwater to be contaminated through spills. A summary of key stormwater pollutants is provided below. They include:

- Gross pollutants and sediment are of concern for system operation as they may potentially block pipes and pumps through the system. It is essential that these are removed early in the process through capture of gross pollutants and settling of sediments to protect downstream treatment components such as the wetland, UV treatment and irrigation system.
- Nutrients are of a lesser concern although excessive levels can adversely affect plant growth and may result in growth of algae and biofilms within irrigation infrastructure.
- It is likely that toxicants such as heavy metals will be present given the industrial areas of the catchment and some treatment will be needed prior to any likelihood of contact with the water. Treatment systems that effectively remove suspended solids and nutrients will generally be effective for removing most heavy metals as a significant proportion is closely associated with sediment.
- Prior to irrigation use, stormwater water should be treated using UV or similar treatment to reduce pathogen levels. It is important that suspended solids levels are low to ensure this is effective.

Groundwater intrusions into the stormwater systems and/or tidal influences may raise salinity levels which must be relatively low for irrigation use to ensure grasses are not adversely impacted. The salinity tolerance of different grasses is summarised in Table 3-4. The salinity of the surface aquifer in this area is anticipated to be in the range 3501-7000 mg/L. Groundwater levels are likely to comparable to sea levels, and hence will be in close proximity to the invert levels of the drains in the area. This means that baseflows into the drain are likely to be brackish. The ryegrasses present on Anderson Oval and Woodruff Oval are quite sensitive to salinity, while the kikuyu and couch have a greater tolerance.

**Table 3-4 Approximate salinity tolerance [*GHD*, 2007]***

| Kikuyu | Couch | Ryegrass | Tall Fescue |
|---|---|---|---|
| 600-3000 | 600-3000 | 200-500 | 200-500 |

- *Converted from EC to mg/L assuming 1000µS = 600 mg/L

# 4.0 Opportunity assessment

## 4.1 Demand management and efficiency

The first step in providing a sustainable water management system is the minimisation of irrigation demand through further improvements to the irrigation systems (demand management strategies).

### 4.1.1 Spray and subsurface irrigation

The Anderson, Aanenson and Woodruff ovals use spray irrigation systems consisting of a network of underground pipes and pop up sprinklers. These systems need careful selection of location, flow rate and outlet sizing to ensure that the whole surface is uniformly irrigated and to avoid over or under watering or irrigation of external areas. There is a need for regular checking of pressures, valve operation and also for blockages. Advantages of these systems are as follows:

- Cheaper than subsurface systems
- Well understood
- Failures can readily be detected
- Subsurface systems can result in an uneven surface less suitable for sports such as football or soccer

A subsurface irrigation system has been installed for the Williams oval. The system is connected to Council's storage tanks in the depot and supplied by the Barry Brothers plant. Subsurface irrigation systems have drippers integrated into the distribution network to deliver water directly to plant roots. This reduces evaporation and losses resulting in greater irrigation efficiency than spray systems. Advantages of these systems are as follows:

- Less sensitive to pressure, reducing the need for pressure pumps and storage tanks
- Reduce water use by up to 40% over spray irrigation through reduced evaporation, spray drift and runoff
- Encourages deeper rooting of grasses
- Avoids tripping risk of pop up sprinklers
- Reduces risk of vandalism
- Reduces the likelihood of direct human contact with the water allowing a lower quality of water to be used (Class C is used for irrigation of Williams oval)

The cost of subsurface irrigation is estimated at $75,000 to $100,000 for a 1.5 ha oval. However, installation of subsurface irrigation would also require resurfacing and planting of the ovals and may involve additional costs. To install subsurface irrigation in the remaining three ovals, costs are estimated in Table 4-1 (for subsurface irrigation), assuming an upper cost of $100,000 per 1.5 ha. The total costs would be higher once resurfacing and planting costs are included.

**Table 4-1 Subsurface irrigation costs**

| Oval | Area ($m^2$) | Cost ($) |
|---|---|---|
| Anderson oval | 16,900 | $112,667 |
| Aanenson oval | 11,911 | $79,407 |
| Woodruff oval | 12,362 | $82,413 |
| Total for subsurface irrigation | - | $274,487 |

### 4.1.2 Irrigation control

An irrigation audit conducted in 2008 [Rainlink Australia Pty Ltd and G&M Connellan Consultants, 2008] found that the irrigation system condition ranged from poor to fair and distribution uniformities ranging from 63% (poor) to 74% (acceptable). The distribution uniformity affects the efficiency of the irrigation system. There is potential to improve the efficiency of the systems through ongoing maintenance and upgrades. As a minimum, council should seek to achieve an irrigation efficiency of 75% for all sports fields and this level has been assumed in analysis of demands. It was estimated that if a poorer overall efficiency of 65% were achieved then the irrigation demand would increase by 4.3 ML/year. While the costs of upgrades and maintenance to maintain an overall efficiency of 75% have not been quantified, it is likely that they will be highly cost effective.

A further improvement could potentially be achieved with the installation of monitoring systems to monitor rainfall and evapotranspiration. The estimated cost of a weather station for ongoing climatic monitoring and control of irrigation is approximately $40 000 [GHD, 2007].

#### 4.1.3 Wetting Agents

Hydrophobic soil conditions are a significant source of water wastage, with extended dry periods and less frequent watering all contributing to water repellence. There are a number of wetting agents available on the market, with the majority being available in liquid form for spray application. These break down the water repellent nature of dry soils and assist with water penetration, maximising the effect of rainfall and irrigation on the soil and grass. The effective life of agents varies from one month to several months. Ionic wetting agent can be effective for 3-4 months and should therefore be applied in spring. The typical cost for applying a wetting agent, either in the irrigation water to improve penetration or with a herbicide to improve spread, is in the order of $1 200 per oval [GHD, 2007]. The application of wetting agents during spring prior to the resumption of irrigation would potentially improve the effectiveness of rainfall and irrigation and reduce losses to runoff and drainage.

## 4.2 Water recycling

There is water recycling infrastructure located at the Council Depot in the north-west corner of Murphy Reserve. This includes a proprietary treatment plant operated by Barry Brothers and 410 kL of storage tanks owned by council. Barry Brothers pump water from Citipower and Telstra utility pits and water that meets requirements for contamination and salinity levels is directed to the treatment plant for recycling. Water is treated to Class C and stored in Council's tanks for use for sub-surface irrigation of GS Williams Oval, tree watering and street cleaning.

The treatment system is reported to have a capacity of 192 kL/day. However, due to the low rainfall in recent years it is estimated that approximately 2,000 to 4,000 kL/year of water is recycled and this is limited by the supply available. It is not known how much water of lower quality than the recycling requirements is discharged. Given its supply and treatment limitations, the recycling system would seem to have limited capacity to provide a substantial component of the demand at Murphy Reserve and cannot be depended upon as a reliable source. It could be used as a supplementary source for up to 4,000 kL/year.

#### 4.2.1 Assessment of council water tanks in depot

Council own a series of above ground storage tanks located within the depot. These are currently used to store recycled water supplied by the Barry Brothers operation for irrigation of Williams oval. The storage capacity of these tanks is estimated to be 410 kL. The yield from the Barry Brothers operation is approximately 2,000 to 4,000 kL/year. The yield ratio for storage tanks is calculated by dividing the annual yield by the storage capacity and this provides an estimate of the effectiveness of use. The ratio depends on the catchment flows, tank size and the timing of use. Irregular storm event flows and variable irrigation demand can reduce the ratio while steady baseflows or constant demand such as toilet flushing can improve the ratio.

A typical stormwater harvesting scheme could be expected to provide a ratio in the range of 10 to 15 and as much as 20 to 30 for baseflow schemes or small household tanks for toilet flushing. The yield ratio for the present recycled scheme is 5-10. The recycling scheme has the potential to augment a primary stormwater harvesting scheme. The tanks do not have adequate capacity to be used as part of the primary stormwater harvesting scheme.

## 4.3 Warm season grasses

The Williams, Aanenson and Woodruff ovals are planted with kikuyu, a warm season grass with a relatively low annual irrigation demand of approximately 500 mm/year. Anderson Oval has recently been replanted with a warm season grass (couch) and this should reduce its irrigation demand by 30%-40%. This would result in a reduced irrigation demand for the oval of approximately 8,812 kL/year and a saving of 4,406 kL/year (refer to Table 4-2). Additional benefits are that the warm season grass has a greater tolerance of heat and drought conditions if water availability is limited and also a greater tolerance of salinity, see Table 4-3. Warm season grasses result in permanent water savings and are a highly cost effective means of reducing water use at $0.15 per kL/year.

**Table 4-2 Estimated irrigation demands with warm season grass on Anderson Oval**

| Manhole | Area ($m^2$) | Estimated irrigation demand (kL/yr) |
|---|---|---|
| Anderson Oval with rye grass | 16,900 | 13,218 |
| Anderson Oval with couch grass | 16,900 | 8,812 |
| Demand reduction | - | 4,406 |

**Table 4-3 Summer ratings for turf grass species [*GHD*, 2007]**

| Property | Kikuyu | Couch | Ryegrass | Tall Fescue |
|---|---|---|---|---|
| Irrigation requirement | Medium | Low | Very high | Very high |
| Drought resistance | Excellent | Excellent | Medium/Very Poor | Medium |
| Heat resistance | Excellent | Excellent | Medium/Poor | Medium |
| Salinity tolerance (mg/L)* | 600-3000 | 600-3000 | 200-500 | 200-500 |

*Converted from EC to mg/L assuming 1000µS = 600 mg/L

## 4.4 Synthetic Turf

Synthetic turf has a number of advantages and disadvantages. They are:
*Advantages*

- The main benefit is that it is highly durable allowing greater access to the surface
- Lower water use
- Potentially reduced maintenance costs

*Disadvantages*

- Synthetic turf experiences significantly higher temperatures than natural turf. The temperature of vegetated surfaces such as natural turf is reduced through evapotranspiration. In urban areas, vegetated spaces such as sports fields help to reduce urban heat island effects. Synthetic turf absorbs heat, is a poor conductor of heat to the ground and does not transpire. This results in the higher temperatures that have been observed. Synthetic turf is usually watered using water cannon just prior to and often at intervals during events to control temperatures. Watering and disinfectants are also needed to clean the surface and remove any bodily fluids or bacteria
- Security fencing and avoidance of overhanging vegetation is generally preferred to minimise risk of damage to surface

Three artificial pitches have been installed at the Football Federation Victoria headquarters at the Darebin International Sports Centre (DISC). Football Federation Victoria considers this to have been very successful [GHD, 2007]. Indicative prices to design and construct a vertically draining FIFA approved synthetic soccer pitch could range from $600,000 to $800,000 plus GST (2007 prices), inclusive of fencing and floodlighting. Indicative prices to maintain a synthetic soccer pitch could range from $7 500 to $10 000 per annum plus GST (2007 prices). In addition, there is likely to be significant management costs unless the facility can be incorporated into an existing leisure centre. [GHD, 2007]

Based on discussions with Council at the workshop, the soccer club wish to replace the training oval (half field) at the south of Anderson Oval. The field is approximately 6,400 $m^2$ in size. Assuming a cost of $700,000 per 1,500 $m^2$, the cost would be estimated at approximately $300,000.

*The merits of synthetic turf need to be more broadly considered as the benefits are primarily related to the opportunity to increase the accessibility and use of the surface. The potential water savings are uncertain as they depend on the water required for temperature control and cleaning.*

For the purposes of the water management analysis it will be conservatively assumed that synthetic turf has not been used. If at a later date it is implemented, then this would result in a net benefit by increasing the reliability of supply for the remaining natural turf fields.

## 4.5 Neighbouring opportunities

There is potential to directly capture and reuse roof runoff from neighbouring properties. This would reduce water quality issues and avoid the difficulties of accessing a tidally influenced drain. It has been identified that the block to the north of the reserve is 85% owned by one owner. The main challenge would be the need to construct and maintain a distribution system capturing all individual roof areas through private property. There would be a need to form a partnership agreement with the landowners to not use their roof water (ie no WSUD), to construct a distribution system on their land and provide for ongoing maintenance of the system. Potential difficulties that may arise with this kind of agreement are future changes of ownership, construction works and future subdivision.

Directly capturing roof runoff may be of limited benefit as it is possible to capture the water in the existing drain and there will still be a need to treat and store captured water. As it is likely a broader system will still be needed, the same treatment and storage train would be used to avoid duplication, which negates the benefits of direct roof capture.

*It is considered preferable to capture water from the existing drainage system*

## 4.6 Groundwater and aquifer storage and recovery (ASR)

It is recommended that any groundwater use by council should be conducted on a sustainable basis. This means that it should not exceed the ‘sustainable yield’ of the aquifer. Given the high levels or urbanisation within the municipality it is likely that recharge rates have diminished through reduced opportunity for infiltration. Groundwater aquifers in the area are not considered significant and there is little available information. It is known that there are numerous domestic bores within the municipality. It is difficult to meaningfully quantify a sustainable yield or determine whether or not it may already have been exceeded as the rates of groundwater recharge and extraction are uncertain.

Groundwater yields in the area are likely to be less than 1 L/s [*DSE & Smart Water Fund*, 2009]. This is in the lowest range and flow rates from groundwater bores are likely to be relatively low. As a consequence, implementation of aquifer storage and recovery would be expensive due to slow travel times and low recovery rates.

The City of Port Phillip is located in close proximity to the sea and any reduction in groundwater levels is likely to result in increased incursion of saline water into the groundwater aquifers. The salinity range for the surface aquifer at the reserve is expected to be in the range 3,500-7,000 mg/L [*DSE & Smart Water Fund*, 2009]. Bore measurements in the nearby area (Lot 1A, 69-119 Salmon St) were reported as 3,986-9,900 mg/L and 20,400 mg/L [*esp*, 2010] indicating salinities are likely to be in this range or higher. Conversely, anecdotal evidence of water use at the Dig-In community garden suggests that lower salinity water may be present in some areas. The expected salinity levels are higher than the preferred range of up to 3,000 mg/L for the warm season grasses on the sport fields and would require shandying with another source.

Previous research [*Cardno Grogan Richards*, 2009] indicates that the chemical properties of the groundwater including high sodium, chloride and bicarbonate levels and moderate alkalinity may adversely impact on the turf, soils and operation of the irrigation system. On the positive side no significant groundwater contamination was identified in the Port Melbourne Sand aquifer [*esp*, 2010].

*Given these factors and the considerable uncertainty surrounding groundwater it is recommended that council do not seek to extract groundwater without providing corresponding groundwater recharge.*

An aquifer storage and recovery (ASR) scheme injects clean water into an aquifer through a series of injection wells and then recovers the water using recovery wells. The main benefit of aquifer storage and recovery is the availability of a large storage without the usual associated storage costs and loss of land area. Unlike surface storages it also has no evaporation losses.

The schemes have a number of issues that need to be managed. These include the need for pre-treatment to minimise well clogging and the risk of pollution of the aquifer, temporary surface storage prior to injection, monitoring requirements, geotechnical changes and the quality of recovered water [*Cardno Grogan Richards*, 2009].

As identified above, the aquifer yields are relatively low and not well suited for aquifer storage and recovery. This means that such a scheme would likely be relatively costly and high risk. There would also be a need for significant surface facilities including treatment and temporary storage, which may be relatively large given constraints on injection rates. Given the water demands of the reserve and the availability of space for the construction of surface or underground tanks it is unlikely that the expense, technical complexity and risks of an aquifer storage and recovery scheme could be justified.

*Aquifer storage and recovery is not considered to be a practical storage option for Murphy Reserve*

## 4.7 Sewer mining

Sewer mining is broadly discussed in the Draft Open Space Water Management Plan. Essentially it involves the extraction of sewage from existing sewers and treatment of the water using a small scale sewage treatment plant. Typically a treatment train including screening, sedimentation, biological treatment, filtration and disinfection would be used.

The main benefit of sewage flows is that they are more constant and reliable than stormwater flows. This assumes that the sewer has a broad catchment with a diversity of land uses. The costs and technical difficulties are likely to be more substantial for sewer mining than stormwater harvesting. To date, few schemes have been implemented in Victoria although there is growing interest and the cost of the technology is reducing.

The nearest large sewer to the reserve is the Hobsons Bay Main which is located deep underground and has large flows associated with it. It is approximately 250 m from the nearest point of the reserve and more than 1,000 m to the depot, which would be the most likely location for a wastewater treatment plant.

*Sewer mining is a potential opportunity for Murphy Reserve although costs would be high and so should only be considered further in the absence of practical alternatives.*

## 4.8 Stormwater

A model of the catchments and potential treatment and storage measures was set up using MUSIC to determine potential flow volumes from the catchment and pollutant loads. Stormwater treatment measures and irrigation storage were then added to model the effectiveness of different systems to supply water to the sports fields based on the estimated demands.

### 4.8.1 Climate data

A 10 year period from 1996-2006 for the Melbourne rainfall gauge (#86071) with monthly average evapotranspiration was selected for modelling. This is a relatively dry sequence and can be considered representative of anticipated future conditions under climate change. The mean annual rainfall is 516 mm/year. Catchment areas and impervious fractions are summarised in Table 3-1.

### 4.8.2 Diversion locations

There are several possible diversion locations. They are:

- The City of Port Phillip's Plummer Street drain runs along Plummer Street to the north of Murphy Reserve. The drain runs along the north side of the street. A diversion could be located at the far western end of Murphy Reserve so that only a connection pipe running 15 m beneath the road would be required to transfer water to the reserve.
- Flows from a larger catchment could be captured by diverting from Melbourne Water's Rosny Street main drain at Plummer Street. This location has a higher invert level than downstream at Williamstown Road, minimising the weir height required. A connection would be required from the corner of Salmon Street to Murphy Reserve, a distance of approximately 150 m and 15 m beneath the road.
- A larger catchment area could be captured with a diversion from the Rosny Street Main Drain at the corner of Williamstown Road and Salmon Street. The invert level of the drain at this location is -0.0427m. A connection to the reserve would have a length of approximately 125 m and 20 m beneath the road. This would increase by 50 m if Williamstown Road were to be crossed for an access within Emery Street.
- An intermediate pit is available along Salmon St at Tarver Street. A piped connection from this location would have to pass an intersection, thereby increasing cost significantly.

The most likely diversion locations are considered to be the Plummer Street main drain or the Rosny Street Main Drain at either Plummer Street or Williamstown Road and further analysis was undertaken of the options for Plummer Street main drain at Salmon Street and Rosny Street Main Drain at Williamstown Road.

**Table 4-4 Likely diversion locations and invert levels**

| Manhole | Drain invert level (m AHD) |
|---|---|
| Rosny St Main Drain at Plummer Street | 0.1920 |
| Rosny St Main Drain at Williamstown Road | -0.0427 |

### 4.8.3 Tidal influences

Tidal information was supplied by the National Tidal Centre (NTC) for the Williamstown gauge. Data was corrected to AHD from chart datum using a correction of -0.524m [*Port Phillip Sea Pilots*, 2009]. Table 4-5 summarises tidal levels and frequency of exceedance of a diversion weir. The invert level of the Rosny Street Main Drain at Plummer Street is 0.192 m. Therefore, for catchment A for the Plummer Street drain a level of 0.2 m AHD was adopted. This level is exceeded approximately 29% of the time. At Williamstown Road, the drain invert level is slightly below the mean sea level. It was assumed that a low weir to a level of 0.057 m would be used, equating to the median sea level for the period considered (July 1996-June 2007). This level is exceeded 50% of the time.

Consideration of daily peak tide levels, as shown in Figure 4-1, indicate that levels will be exceeded on a higher daily frequency and that a level of 0.4 m AHD would be reached for 40% of days and a level of 0.6 m AHD for about 10% of days. These tidal influences need to be considered further during the functional and detailed design stages of the project.

**Table 4-5 Tidal levels and frequency of exceedance**

| Tidal level | Frequency of exceedance |
|---|---|
| -0.2 | 83% |
| -0.1 | 72% |
| 0 | 58% |
| 0.057 | 50% |
| 0.1 | 44% |
| 0.2 | 29% |
| 0.3 | 16% |
| 0.4 | 7.7% |
| 0.5 | 3.3% |
| 0.6 | 1.2% |
| 0.7 | 0.4% |

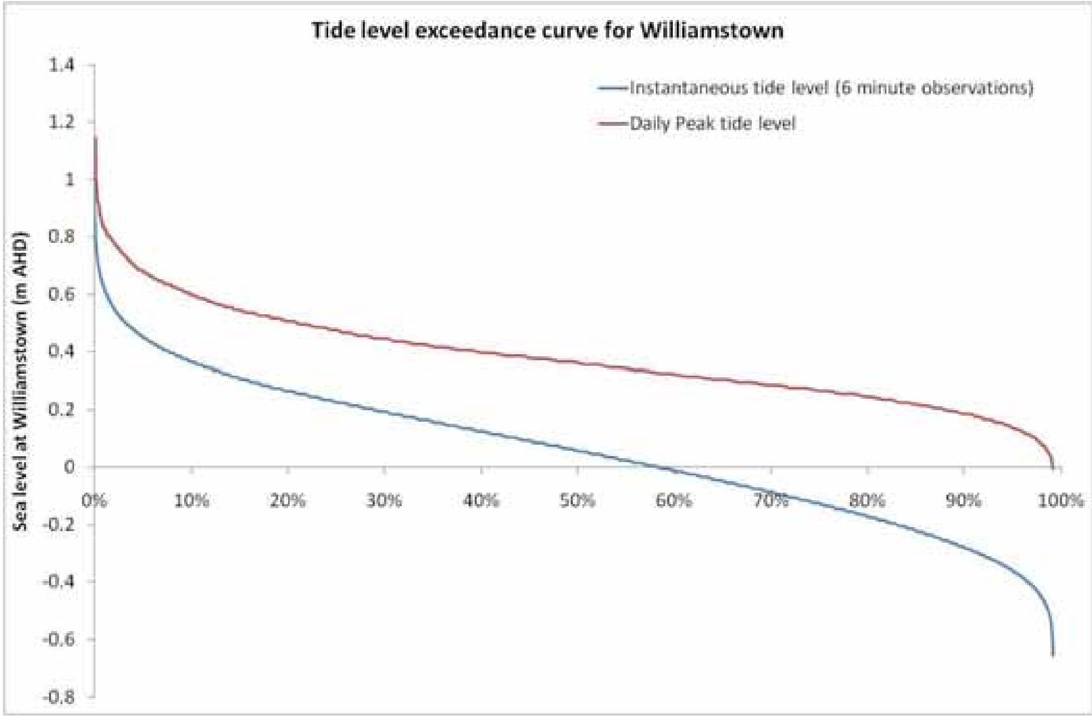


**Figure 4-1 Sea level exceedance curve for Williamstown gauge**

#### 4.8.4 Recommended diversion point

Modelling indicated that tidal levels have a significant influence on the behaviour of a stormwater harvesting system. As such, a weir is needed to obstruct the majority of tidal influxes with a depth of at least 0.4 m AHD and preferably 0.6 m AHD to minimise the frequency of daily tidal inundation. However, the height of the weir must be balanced with avoiding adverse impacts on flood levels. Further analysis was undertaken to determine the effects of different weir levels and this was included in Appendix 1 for reference.

***Plummer Street is considered to be the preferred diversion option for the following reasons:***

- A weir of at least 0.6m AHD at Williamstown Road is needed to provide comparable flows to Plummer St
- It is preferable to keep the weir height as low as possible to minimize flooding impacts. A weir to 0.6m AHD would be 650mm high at Williamstown Road or 450mm high at Plummer Street. The height could be further reduced by locating the weir further upstream along Plummer Street.
- There is limited opportunity for flood mitigation on Salmon Street whereas a swale within the reserve can cost effectively be used to mitigate any flood impacts of a weir within the Plummer St drain by carrying excess flows
- The Plummer St drain is more accessible as it is immediately adjacent to the reserve, minimizing the length and cost of the diversion and disruption to traffic
- Flows in Plummer St are more than sufficient to achieve a reasonable level of reliability
- Saline intrusion into the drain is likely to be more significant at Williamstown Road due to the lower invert level which is below sea level. This may necessitate pumping of saline flows.

***It is recommended that a weir to a level of 0.6m AHD is adopted. This will minimise daily ingress of saline water at high tide to 10% of days with inundation for 1% of the total time period considered. The height of the weir should be minimised by locating it further upstream if necessary to minimise flooding impacts.***

#### 4.8.5 Diversion infrastructure

Diversion infrastructure will detain stormwater to allow it to be pumped to the storage and limit tidal intrusion into the detention storage area. Submersible pumps will be used to pump water from the drain. The use of a pump behind a weir within the drain itself provides the important benefit of allowing diversions to be controlled according to salinity. This means that when tidal influxes occur or when the quality of baseflows is poor, no pumping occurs. When a storm event occurs and the quality improves, the pump can be activated. This approach avoids the need to pump saline water and minimises infrastructure associated with the drain.

The weir may result in an increase in upstream flood levels. These can be mitigated using a pipe or swale diversion. A minimum energy loss weir designed within an enlarged pit could reduce the diversion requirements. The structure and diversion must reduce energy losses such as friction losses by an amount equivalent to the head losses created by the weir itself and any additional losses imposed.

1. Swale diversion – Flows will be diverted from a selected upstream pit on the reserve side of Plummer Street. A swale will be excavated to the depth of the pit (likely to be 600-700 mm) and extended along Plummer Street to just downstream of the diversion point. An outlet pit and connecting pipe will then convey flows back into the drain downstream of the diversion.
2. Pipe diversion – Flows will be diverted from the drain and directed along a parallel pipe which is then reconnected to the drain downstream of the weir. The invert level of the pipe should be no lower than the weir. The pipe and pits will be sized such that the increased capacity reduces friction losses in the main by pipe by an amount equivalent to the weir losses plus any additional pit losses.
3. Minimum energy loss weir structure – A special pit containing the weir and transitions to the upstream and downstream sections of the drain is pre-cast. A section of the drain will be removed and replaced with the structure. This structure may be relatively large and expensive.

A potential opportunity is to locate the diversion at the point where the drain increases in size from a 1,350 mm to a 1,575 mm drain. *It is recommended that the pipe size transition is surveyed during functional design to determine whether a weir could be constructed at this location with a lesser effect on the drain capacity.*

Another option is to gravity drain water directly to a buffer storage located close to the drain. The main benefit of this approach is that it would allow much smaller pumps to be used while more water could be captured. However, this would require that a complete tidal gate is used to prevent tidal influxes above the weir or that saline water is pumped out of the storage.

*At this stage it is anticipated that the use of a relatively large pump located within a sump attached to the drain will be the preferred option.*

*Modelling of the diversion*

The diversion was modelled in MUSIC to determine the effect of different pump flow rates on the volume extracted. A significant challenge is that the source of stormwater is inherently flashy (unless significant baseflows can be accessed) and water needs to be rapidly pumped to ensure sufficient volumes are extracted to meet a target reliability for the irrigation demand. As shown in Figure 4-2 a pump with a capacity of at least 100 L/s would be required and preferably in the order of 250 L/s.

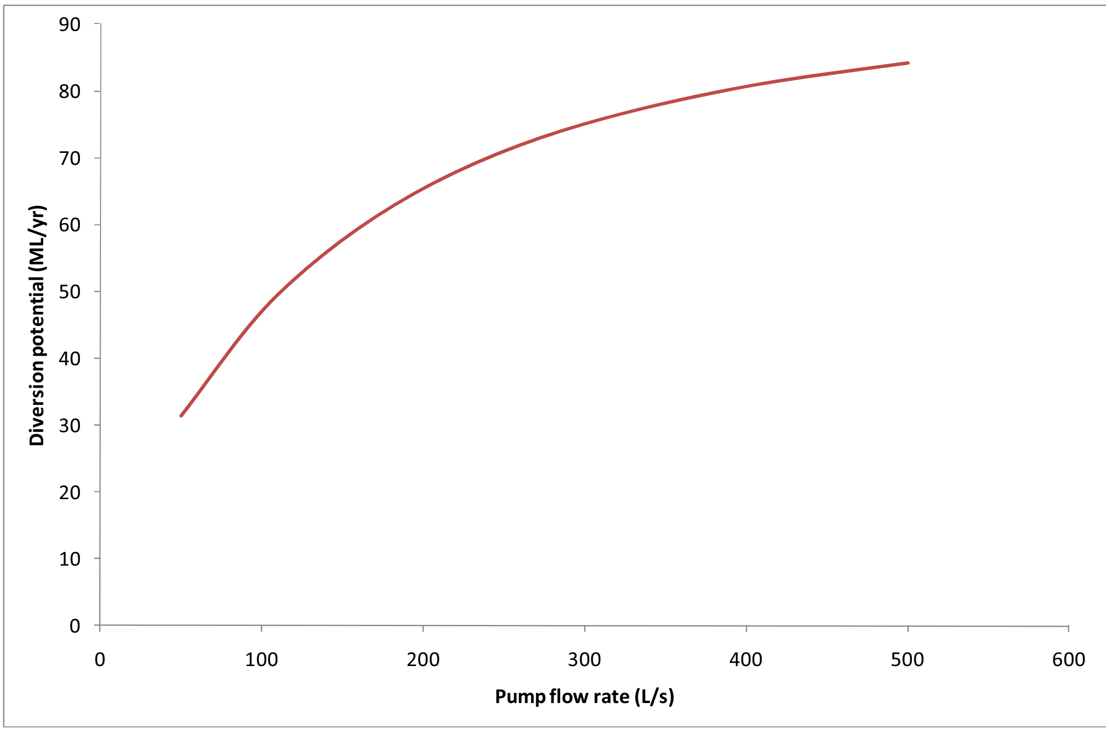


**Figure 4-2 Pump capacity vs. diversion potential**

An inline buffer storage connected to the drain was considered as it could potentially significantly reduce the pump size and the size of downstream infrastructure while allowing similar flow volumes to be captured. However, substantial dewatering would be required making it relatively costly to construct a large storage below the groundwater level. An additional issue is that the storage would be filled with saline water when tidal levels exceed the weir level. This would limit access to flows while salinity levels were high or require additional pumping.

### 4.8.6 Treatment

The treatment system provides treatment through sedimentation, filtering and biological treatment. A wetland or bioretention system could be used to provide treatment. The merits of these two approaches were discussed at the workshop with council. Based on preliminary analysis, estimated areas of 500 $m^2$ for bioretention with a 500 kL buffer tank and 2,000 $m^2$ - 3,000 $m^2$ for a wetland were used. Water is then stored until it is required. A secondary treatment will provide UV disinfection of irrigation water prior to use. Water needs to be treated to a sufficient standard for stormwater harvesting and the levels of suspended sediments and particles should be minimised to ensure the UV disinfection is effective.

**Wetland**

The wetlands assume an extended detention depth of 0.5 m to maximise the storage capacity of the system and allow fluctuations from the pumped diversion to be smoothed out. This was combined with a permanent pool depth of 0.3 m. An inlet pond of 500 $m^3$ was assumed and the outlet was sized to achieve a detention time of 72 hours. Figure 4-3 shows harvested yields that are related to wetland size and a range of pump capacities. Based on these results, a pump capacity of 150 L/s -250 L/s and a wetland area of 2,000 $m^2$ – 3,000 $m^2$ was selected for further analysis and reliabilities were calculated for these. Figure 4-4 shows a storage size of 1,750 kL achieves a 75% reliability of supply. This will provide a cost effective system while allowing some flexibility for factors such as the catchment at the diversion point and inlet pond volume. Figure 4-5 shows a typical wetland designed for harvesting stormwater.

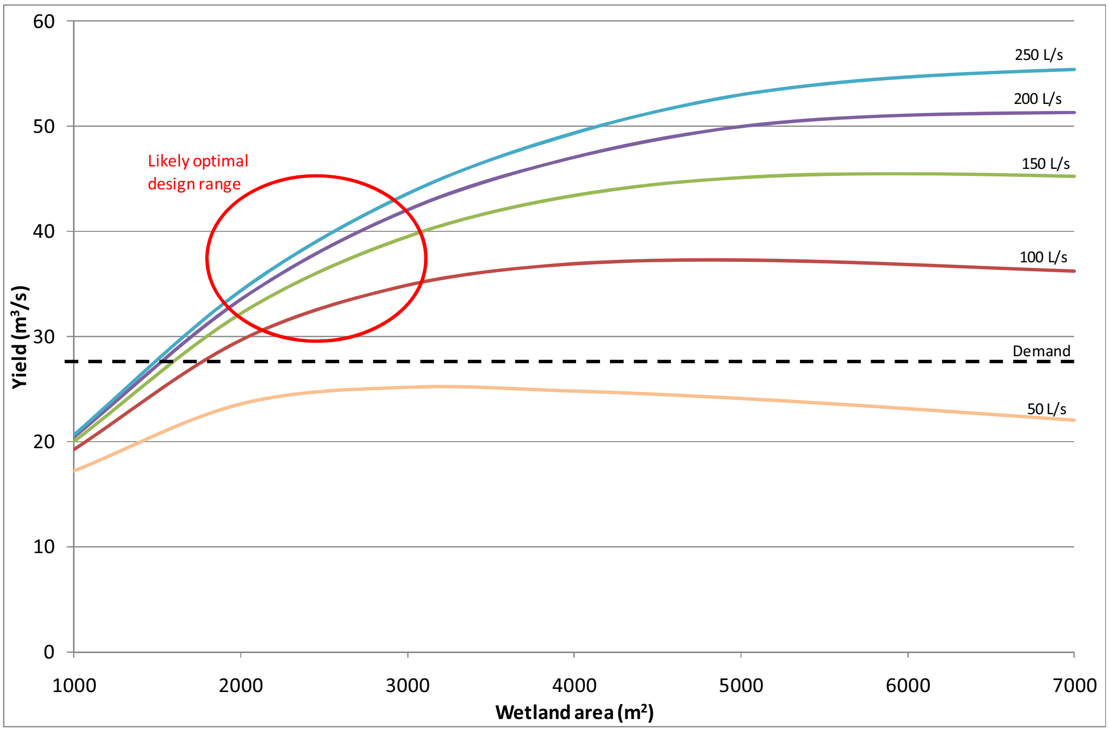


**Figure 4-3 Wetland areas and potential yields**

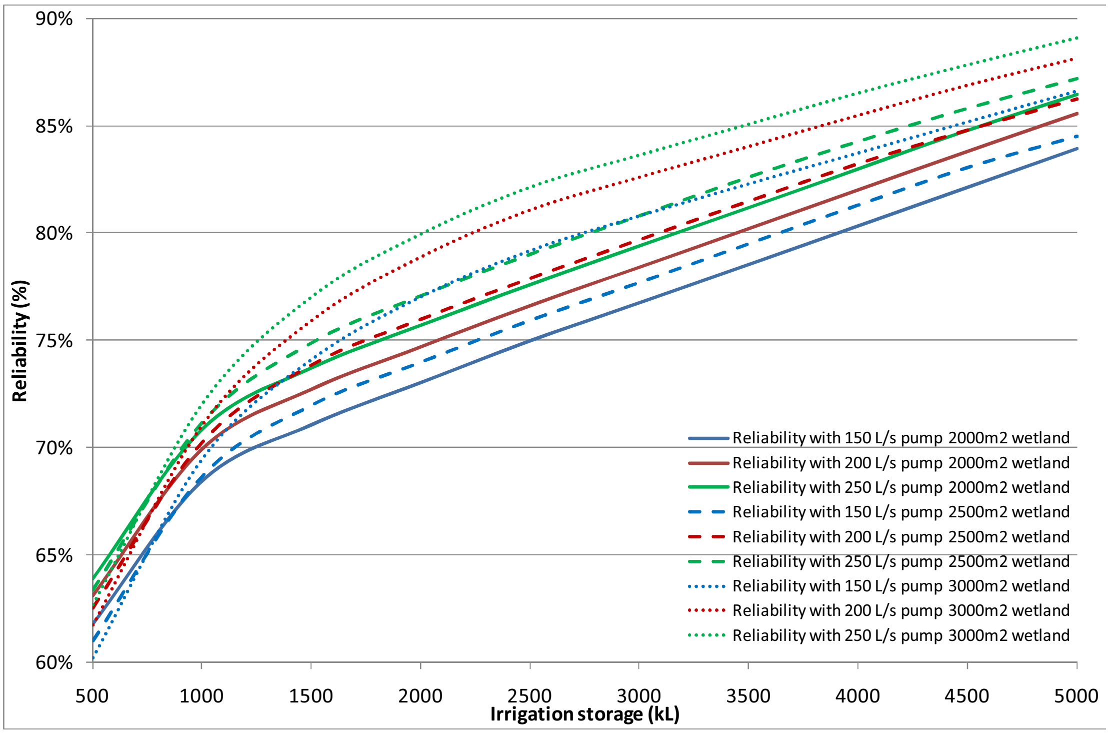


**Figure 4-4 Storage volumes and reliabilities for selected pump sizes and wetland areas**

**Figure 4-5 A typical wetland at Royal Park**

### Bioretention

The bioretention systems assume a filter depth of 0.5 m and extended detention of 0.3 m. A hydraulic conductivity of 100 mm was assumed to allow for some clogging to occur given the high flow volumes that will be passed through the system. A potential yield curve was created and the results are shown in Figure 4-6. The results indicate that a bioretention system 750 $m^2$ to 1000 $m^2$ in area will be required using a pump with a capacity of 150 L/s -250 L/s to provide an efficient design and reduce storage requirements.

A pump capacity of 250 L/s and bioretention system areas of 800 $m^2$, 900 $m^2$ and 1,000 $m^2$ were selected for analysis of reliability. A range of potential storage sizes were modelled to develop reliability curves, shown in Figure 4-7. The results from the reliability analysis indicate that a bioretention system size of 1,000 $m^2$ would be preferred, with a storage of 1,850 kL to achieve a reliability of 75%. Figure 4-8 shows a newly established bioretention system designed for harvesting stormwater.

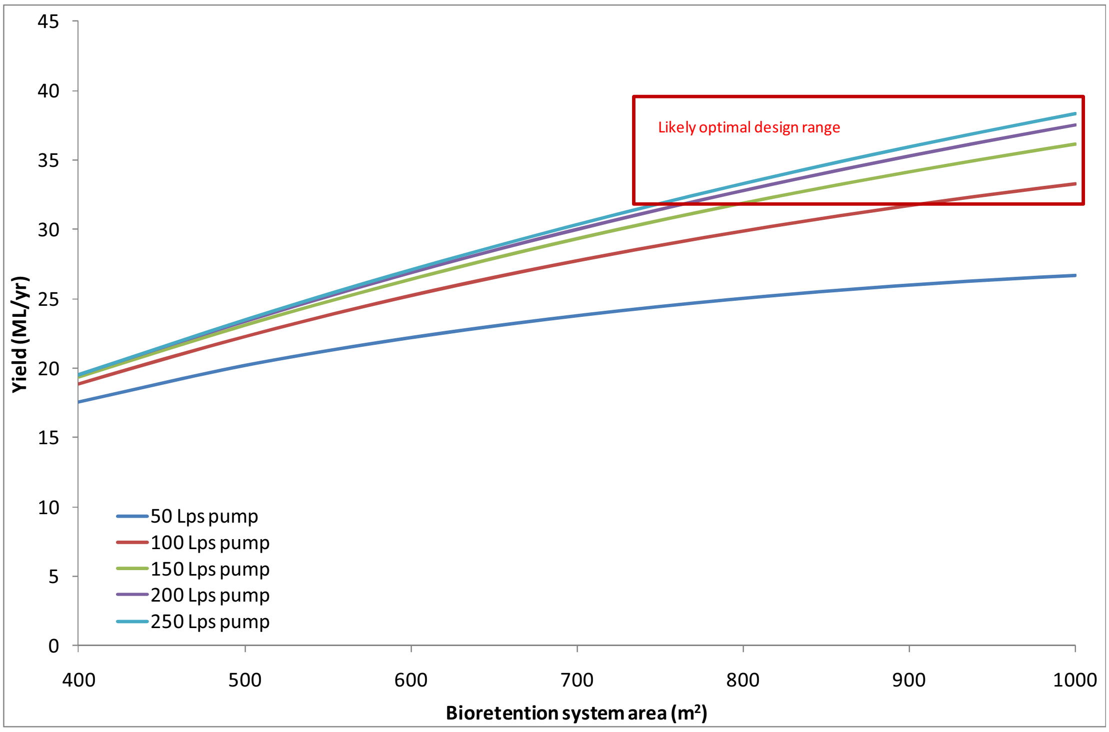


**Figure 4-6 Potential yields for a bioretention treatment**

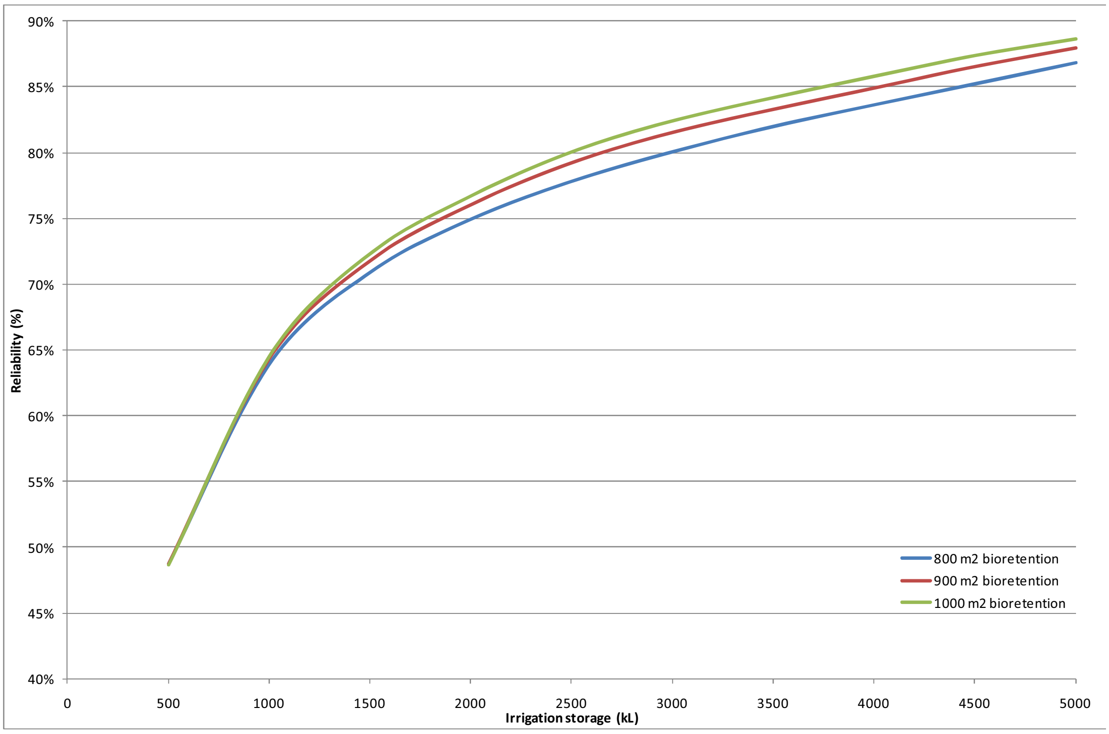


**Figure 4-7 Potential yields for a bioretention treatment**

**Figure 4-8 A recently established bioretention system at Royal Melbourne Golf Course**

**Choosing a preferred option**

*Based on discussions at the workshop the wetland was considered to be clearly the preferred treatment option with potential to reinvigorate the reserve* rather than just meeting water management objectives. A wetland could be designed to become an attractive park feature, a visual highlight that will draw people to the park and help to create a pleasant environment for passive recreational activity. There is potential that a wetland could be combined with other facilities such as BBQ's and a walking path to create opportunities for a wider range of activities at the reserve. As a result, the wetland can contribute substantially towards the broader objectives of the masterplan to enhance passive recreational areas and increase the breadth of users at the park as well as provide stormwater treatment. While the wetland will require a larger land take and greater cost (up to $200,000 more than a bioretention system) the additional cost is considered well worthwhile.

### 4.8.7 Stormwater harvesting locations

The reserve was reviewed during the workshop to identify suitable locations for the stormwater harvesting scheme. The possible suitable locations identified are shown in Figure 4-10.

**Diversion**

It is recommended that the diversion weir is located in the Plummer Street Drain close to the western boundary of the reserve. A pump sump will be located within a pit upstream of the weir. Connections to the relief swale will need to be provided upstream and downstream of the weir. Diverted flows will be pumped to the treatment system.

**Treatment and storage**

Two sites are considered to be suitable for the preferred wetland option. The first is in the south eastern corner of the reserve. There is a large area here that would allow construction of the wetland to be well integrated within a broader surrounding landscape and other facilities. There is more than adequate space for the inlet pond and batter slopes. This site is prominently located on the corner of Graham Street and Williamstown Road and would add to the aesthetic appeal of the park and visual amenity of the area.

This area is used by the dog club and their space needs should be considered and accommodated within the masterplan and wetland design. A significant portion of the space can be retained for their use and other purposes. The open space to the west of the pavilion can also be used by the dog club and other passive recreational users.

The second possible location is the open space area surrounded by Anderson Oval, Williams Oval and the heritage landscaped area around the pavilion. This space would have (just) enough space for the wetland to be located. However, there would be little opportunity for any surrounding infrastructure to allow the wetland to be effectively integrated within the broader reserve landscape and other facilities such as BBQ's.

Underground tanks can be located almost anywhere. Ideally disturbance of existing facilities, sports fields and trees should be avoided. It is recommended that they are located in close proximity to the treatment system either below passive recreational space or within the depot. The depot would be a good site for storage tanks that would provide ready access for maintenance without disturbing reserve users. An additional benefit would be that council tankers could access the tanks for other purposes if surplus water is harvested. There is potential that contaminated soils may be encountered and these will need to be addressed during excavation.


**Figure 4-9 Potential treatment and underground storage locations**

**Recommended location for treatment**

The south east open space is considered to be the preferred location for the treatment system. It is recommended that underground tanks are located within the depot to realise synergies with existing infrastructure and facilitate maintenance with minimal disturbance.

A schematic was prepared to illustrate approximate locations of various components of the stormwater harvesting scheme as shown in Figure 4-10.

A concept drawing of the wetland to illustrate how it may look was also prepared and included in Appendix 2 – Wetland concept.


**Figure 4-10 Schematic of preferred option for Murphy Reserve stormwater harvesting scheme**

### 4.8.8 Catchment stormwater management outcomes

The pollutant loads from the catchment and treatment benefits of the proposed stormwater harvesting scheme were considered. The scheme treats diverted flows from the catchment. It provides treatment of these flows and then loads are further reduced through reuse of this water. When treated flows are in excess of the irrigation storage, treated flows will be redirected to the drain.

In practice, pumping would cease when the wetland is full as there are no significant treatment or reliability benefits to additional pumping. A standard model in MUSIC assumes that pumping will always occur and would over-estimate treatment. This was overcome by running the model once, using the wetland levels to determine a pumping regime where no inflows occur once the wetland is full and then re-running the model using the revised inflow pattern. This approximation will provide a better estimate of the likely treatment performance of the wetland.

The results are summarised in Table 4-6. It can be seen that the wetland treatment performance relative to the loads actually pumped to the system is very good, comfortably achieving best practice treatment and providing confidence that outflows will be adequate for irrigation use. While only part of the total catchment flow is treated by the wetland, significant reductions in catchment pollutant loads being discharged to Port Phillip Bay are achieved and contribute towards the City of Port Phillips stormwater pollutant reduction objectives.

It is estimated that 21 ML/year of stormwater will be harvested for irrigation reuse (3 ML is consumed by evapotranspiration). This represents a substantial contribution towards the City of Port Phillip's targets for 2020 potable water use and alternative water sources, representing more than one quarter of the targets for 2020 (refer to Table 4-7.

**Table 4-6 Stormwater pollutant loads and reductions**

| Parameter | Catchment A | Total load reduction | Wetland treatment performance | % of catchment load | Port Phillip annual target |
|---|---|---|---|---|---|
| Flow (ML/yr) | 103 | 24 | 9% | 23% | - |
| Total suspended solids (kg/yr) | 10,500 | 3624 | 95% | 35% | 10,973-15,255* |
| Total phosphorus (kg/yr) | 24.9 | 7.35 | 77% | 30% | 18-25* |
| Total nitrogen (kg/yr) | 194 | 53 | 49% | 27% | 88-122* |

*Range is for Port Phillip's adopted pollutant load reduction scenarios in the Draft Water Plan

**Table 4-7 Murphy Reserve contribution towards City of Port Phillip Water Management Targets**

| | 2020 | Annual* |
|---|---|---|
| Target for reduction in potable water use | 57 | 5.7 |
| Target for alternative water sources | 78 | 7.8 |
| Murphy Reserve | 21 | 21 |

*The target reduction in council potable water use is 70% of 2000/2001 levels resulting in a target water use of 155 ML/year. Assuming a return to ideal irrigation water use, Council water use will increase from 155 ML/year to 212 ML/year. To sustain the achievement of the 70% target will require further reductions of 57 ML/year by 2020.

#### 4.8.9 Diversion, relief swale, pipes, pumps and other infrastructure

A weir level of 0.6 m AHD was selected as the preferred weir height to minimise the frequency of tidal intrusion reaching the pump sump. This would reduce the frequency to approximately 1% of the total time, occurring on 10% of days for the year.

*Preliminary estimate - Weir at Salmon Street*

Based on available information for the Plummer St drain from City of Port Phillip GIS data, pipe dimensions for the Plummer Street drain were determined to be 1350 mm, increasing to 1575 mm before the drain reaches the western end of the reserve.

Initially, a weir located close to Salmon Street was assumed. The invert of the pipe is at 0.2 m AHD at Salmon Street, therefore a weir height of approximately 0.4 m would be required at this location. This would restrict about 20% of the total area of the pipe. The 1 in 5 year flow for the catchment was estimated to be 3.6 $m^3$/s.

Preliminary calculations assuming a slope of 1 in 1000 and pipe dimensions of 1350 mm increasing to 1575 mm were used to estimate pipe capacity. Estimates of the flow capacity for the drain under pressurised flow (based on hydraulic grade line using GIS data) and for the weir under free or submerged conditions indicates that both would have sufficient capacity for the 1 in 5 year flow without surcharging.

A relief swale can provide additional capacity to compensate for the head loss and reduction in pipe capacity due to the weir. Calculations based on a weir height of 0.4m at Salmon Street indicate that a swale of length 80 m with a base width of 6 m and top width of 10 m would be needed. The swale would have capacity for up to half the design flow and minimise head losses.

*Weir further upstream adjacent to Anderson Oval*

This section is based on additional work undertaken as part of a variation to the original scope to further refine the weir design. Additional information was obtained from a Fisher Stewart report in March 2010, providing details of existing pipe sizes and drain invert levels as well as details of proposed upgrades to rectify variations in the grade and improve capacity. The location of the change in pipe size is close to the intersection with Salmon St rather than further upstream as indicated by the GIS information and this needs to be confirmed. It is not known whether any upgrades have occurred over the past decade although this appears unlikely as pipe sizes correspond to those in the GIS data. Therefore it was assumed that the existing levels provided in the Fisher Stewart report are the best available estimates of pipe invert levels.

The invert level is 0.2m AHD at the corner of Plummer St and Salmon St and 0.7m AHD at the corner of Plummer St and Graham St, see Figure 11. These suggest a pipe slope in the order of 1:1300 for the whole reach, although this varies significantly. The drain reaches an invert level of 0.59m AHD opposite Anderson Oval. *Therefore, while the catchment will be slightly smaller at this location, it would be possible to achieve a diversion with a* ***minimal weir of less than 100mm****. This would minimise any flood mitigation impacts and the costs of the diversion structure and flood mitigation works.*

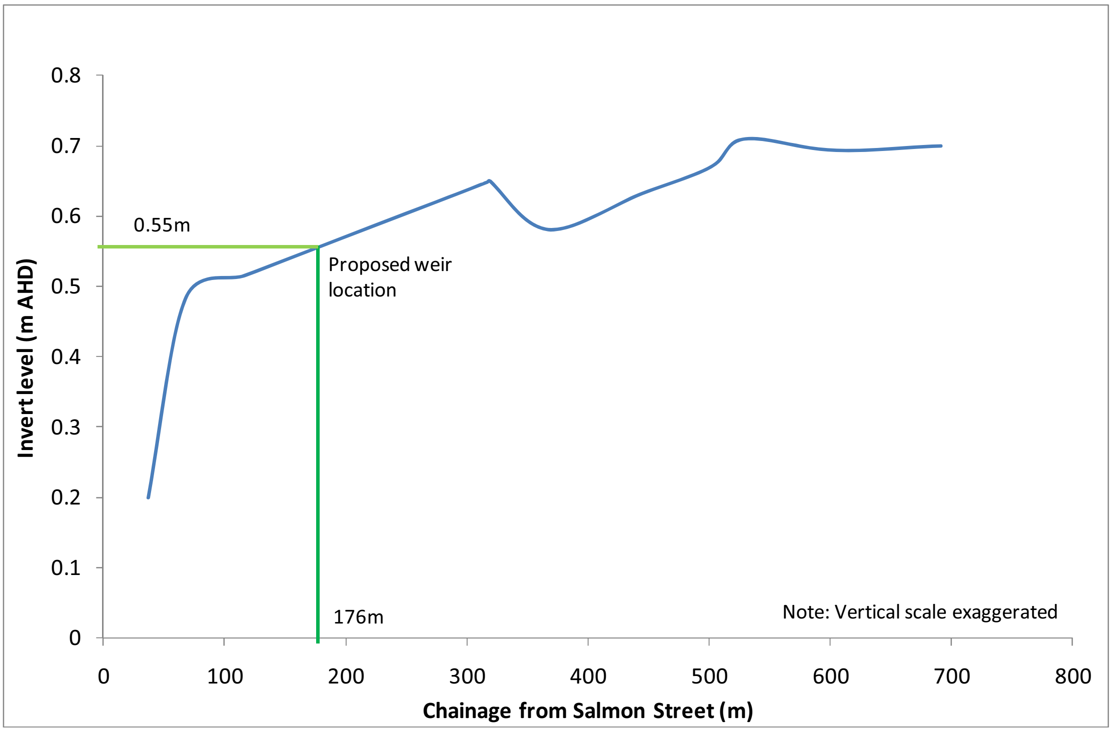


**Figure 11 Plummer Street Drain Invert Levels**

A hydraulic grade line was created for the Plummer Street drain to determine the likely extent of flooding. A 1 in 5 year design event was considered with an estimated flow of up to 3.4 $m^3/s$. A tail water level of 0.6m was assumed. This level is exceeded approximately 1% of the time, therefore the probability of a higher water level occurring at the same time as a 1 in 5 year event is very low. The results indicate that the drain has just enough capacity for the 1 in 5 year event without surcharging given this tail water condition. Based on contour information surface levels adjacent to the reserve range from 3 to 4 metres, increasing to the east.

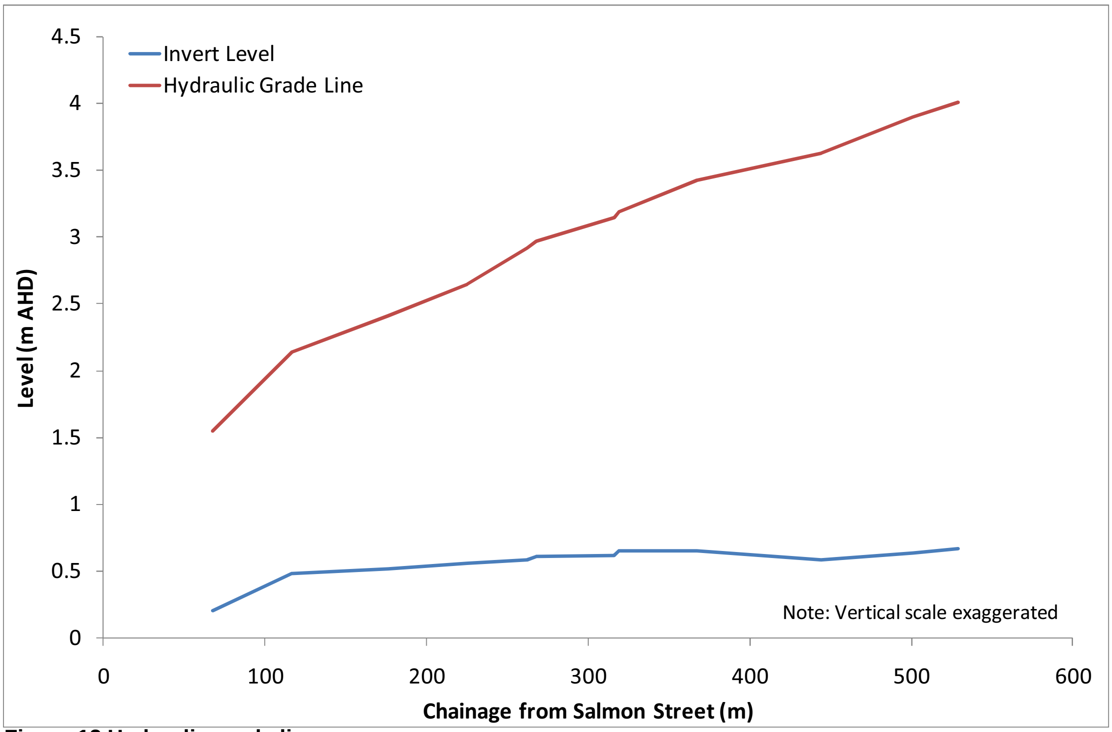


**Figure 12 Hydraulic grade line**

It is proposed that the weir and offtake to the pump sump will be located within one pit. If possible, the weir will be retro-fitted into an existing pit. If this cannot be done, one of the existing pits will be replaced with a new pit containing the weir and offtake. This will minimise any additional head losses due to the addition of a new pit. The weir will be transitioned to minimise any losses.

Hydraulic calculations were undertaken for the weir as a 100mm obstruction within the pipeline. The change in depth between upstream and the weir location that would occur for a 1 in 5 year flow of 3.4 $m^3/s$ at this location was estimated. This was used as a conservative estimate of potential head losses due to the weir. The change in head was estimated at 20mm. This is very small and the actual head losses would be even less than this. For comparison, the anticipated head losses in the existing drain would be 70-100mm. It is considered that there may be minimal need for additional flood mitigation infrastructure and any required swale will be relatively small.

The precise design, location and height of the weir will be determined during the functional design. The treatment and storage will be remodelled to account for the slightly reduced catchment area. While this may increase the size and costs of these systems, it is expected that the reduced costs of weir and flood mitigation works will more than offset these.

*Diversion pumps*

It was determined that diversion pumps with a capacity of 250 L/s would be desirable to provide a good flow capacity and reduce the size of the treatment and storage. These will be located within a pump sump constructed adjacent to the drain.

### 4.8.10 Pre-treatment

Gross pollutants and sediment will tend to accumulate in the pump sump and may adversely impact upon the operation of the pump. It is recommended that action is taken upstream of the system to minimise gross pollutant (litter) and sediment inflows. Given the small size of the catchment, there is only a limited number of upstream inlet pits (approximately 50). It is recommended that these are fitted with screens to capture gross pollutants. Alternatively a large gross pollutant trap could be used although it would be preferable for litter to be captured close to the source.

The catchment area is relatively small and well defined with relatively few large landowners. This makes it ideally suited for an education campaign to educate landowners and employees about the stormwater harvesting scheme and the importance of managing litter, potential sources of sediment and risks of spills effectively. A guideline such as the Hume City Council Industrial Stormwater Code of Practice could be used to guide this process. Many of the surrounding businesses are likely to use the reserve for functions such as BBQ's and effective engagement would encourage them to take ownership of the catchment and scheme and proactively reduce litter loads. This would also provide an excellent opportunity to teach and illustrate the benefits of stormwater treatment and harvesting and show how it can benefit the community through improving the amenity of the reserve.

#### 4.8.11 Management of energy impacts

The most significant energy requirement for the stormwater harvesting scheme is pumping. It is estimated that up to 37,200 kL/year will be pumped to the wetland for treatment with 21,000 kL/year of stored water used for irrigation. Preliminary investigations suggest that two pumps are required to provide 250 L/s, head of 4 m and power requirement of 22 kW may be sufficient. It is likely that a smaller variable flow pump may also be used to reduce the need to switch on and run large pumps when low flows occur and reduce pump wear.

The pump energy requirements were estimated based on the flow volumes, pump energy requirements and the estimated pumping time required. This resulted in an energy use of approximately 1 MWh per year, while the irrigation requirements would result in total use of less than 2 MWh per year. To put this in perspective it is a fraction of the average annual energy requirement for a typical Melbourne household and would be a small part of the reserve energy use.

This energy use could be readily sourced from renewable energy as part of a larger system to supply renewable electricity using solar or wind generation for one of the pavilions. It is recommended that renewable energy sources are considered for the reserve as part of the masterplanning process and that these incorporate the energy use of the stormwater harvesting scheme.

#### 4.8.12 Cost estimate

A preliminary cost estimate was made for the stormwater harvesting scheme based on typical costs for wetlands, storage tanks and estimated costs for structures. These costs need to be refined at the functional design stage.

| **Description** | Cost |
|---|---:|
| Diversion weir structure, pumps and sump | $180,000 |
| Litter traps or gross pollutant trap | $100,000 |
| Reticulation to treatment including pipe connection and directional bore under Plummer Street | $50,000 |
| Relief swale including excavation, planting and pit connections to Plummer St drain | $30,000 |
| Wetland excavation, lining, planting and structures (4,000m$^2$ allowing for inlet pond and batter slopes) | $280,000 |
| Storage (1,750 kL) | $750,000 |
| Connection to irrigation system, temporary storage and UV | $80,000 |
| **Sub-total** | **$1,490,000** |
| Design, management and contingency @ 30% | $432,000 |
| **Total** | **$1,937,000** |

The storage costs comprise half of the total costs. These could potentially be reduced by making use of the existing council tanks at the depot. The potential to retain the existing wastewater recycling scheme and associated benefits should be considered.

# 5.0 Recommendations and conclusions

## 5.1 Preferred options

It was identified that Murphy Reserve has an irrigation demand of 27,900 kL/year. A range of opportunities to meet or contribute towards meeting the demands were considered as illustrated in Figure 5-1. The following recommendations are made regarding the various options:

### Opportunities to act on

- ***It is considered that stormwater harvesting is the most practical option that has the potential to meet a large proportion of the irrigation demand.*** Alternative options can meet only a small part of this demand or would have greater cost, risk and infrastructure requirements.
- Demand management measures and warm season grasses have largely been implemented for the reserve. It is recommended that irrigation systems are maintained and upgraded as required to maintain a minimum overall efficiency of at least 75%.

### Opportunities that are complementary

- The existing wastewater recycling scheme can remain in place in conjunction with stormwater harvesting if practical. Stormwater harvesting or shared use would be a more efficient use of the existing tanks.
- Synthetic turf may offer some water management benefits but the main benefit is its durability allowing increased access to use the surface. The issues and costs need to be considered more broadly by council. Implementation for the soccer training field would increase the reliability of supply for the other ovals from the stormwater harvesting scheme.

### Opportunities that are not recommended

- Sewer mining and aquifer storage and recovery will have higher costs and greater technical difficulty and risks than the stormwater harvesting scheme and as such are unlikely to be justifiable.
- Groundwater resources in the area have low yields, high salinities and sustainable yields are unknown. The sustainable use of groundwater is unlikely to be practical for the reserve.
- Doing nothing will have consequences for the local community through:
    - Reduced access to sporting facilities and viability of sporting clubs
    - Adverse social impacts due to reduced recreational options
    - Missed opportunities to benefit the environment and local community
    - No improvement in reserve facilities


**Figure 5-1 Water management opportunities for Murphy Reserve**

## 5.2 Stormwater harvesting

Stormwater harvesting was identified as the primary preferred option. The proposed stormwater harvesting scheme is illustrated in Figure 4-10 and has the following characteristics:

- Diversion weir in Plummer Street Drain at 0.6 m AHD
- Diversion pump with capacity of 250 L/s
- Relief swale up to 3 m wide
- Rising main to treatment system of approximately 600 m
- Wetland with 500 $m^2$ inlet sedimentation pond and macrophyte zone up to 3,500 $m^2$ with permanent pool volume of 750 $m^3$ and extended detention depth of 0.5 m.
- Irrigation storage tanks with a capacity of 1,750 kL

It is intended that the wetland be integrated within the broader landscape of the reserve and additional facilities such as BBQ areas, shelter, mounds and walking paths. This would greatly enhance the opportunities for passive recreational enjoyment of the reserve, attracting a broader range of people from the community.

The proposed stormwater harvesting scheme, wetlands and additional landscaping and facilities will deliver a range of benefits to the community including:

- A reliable source of water providing an average of 21,000 kL/year for irrigating sports fields
- Increased access to sporting fields and higher participation rates for local sporting clubs
- A greatly enhanced passive recreational space
- A landmark entrance feature to the reserve greatly enhancing its appearance and amenity
- Substantial water savings of 21 ML/year achieving more than 25% of the City of Port Phillip's potable water use reduction and alternative water source targets for 2020
- A significant contribution towards reducing stormwater pollutant loads to the bay equating to around half of the City of Port Phillip's annual target

The cost of the stormwater harvesting scheme including the wetland is estimated at approximately $1.94M including design and management.

## 5.3 Further work

A number of areas were identified for further investigation to progress the preferred option of a stormwater harvesting scheme.

#### Stormwater monitoring

It has been identified that there is a need for monitoring of the drain to determine water quantity and quality, particularly baseflows during summer periods as this may allow further refinements to the design and cost savings. It is recommended that these investigations are commenced as a priority at the earliest opportunity.

#### Functional design of stormwater harvesting scheme

Following initial community consultation, it will be essential to progress the design to a functional design of the stormwater harvesting scheme. This will provide greater detail and certainty to allow council to bring the proposal to the community with confidence and also effectively seek any opportunities for external funding.

The functional design will include:

- Hydraulic investigations, design and modelling for the diversion structure, relief swale, pumps and sump

- Functional design of the wetland including the shape, exact location, levels, vegetation and inlet and outlet structures
- Selection and design of irrigation storage including consideration of above and below ground options, pumping requirements, inlet, pumps and disinfection treatment
- Preparation of catchment community engagement program for at-source management of pollutants

**Integrated wetland landscape plan**

It is important that the wetland is effectively integrated with the surrounding landscape and other facilities in the reserve as part of the masterplan. This requires setting, layout and design of the wetland itself in conjunction with the surrounding features such as mounds, boardwalk access points, walking paths, gardens and BBQ areas. It is recommended that a landscape plan for the wetland and its surrounding area is commissioned in tandem with the functional design of the stormwater harvesting scheme.

# 6.0 References

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# 7.0 Appendix 1 – Weir levels, storage sizes and reliability

### 7.1.1 Preliminary estimates of storage and reliability

Preliminary estimates of storage sizes for different diversion locations and weir levels were undertaken. Tidal levels were taken into account by assuming that flows could only be extracted from the drain when the tidal level was below a specified level. Weir levels at 0.2 m AHD, 0.4 m AHD and 0.6 m AHD were considered. To model these conditions, inflows were set to zero when tidal levels exceeded the specified extraction level. A weir height of 0.2 m above the pipe invert is likely to be low enough to have a minimal impact on the hydraulic capacity of the drains while a larger weir up to a level of 0.6 m AHD would be preferable to reduce the frequency of tidal ingress.

A range of storage sizes were considered to determine the potential volumes and reliabilities that could be obtained and the results of the modelling are shown in Figure 7-1 and Table 7-1. The analysis showed that tidal influences significantly impact upon the reliability of the scheme, reducing the flow volumes available for harvesting. Comparing the maximum potential yields with minimal weir levels (0.2 m for Plummer Street and 0.057 m for Williamstown Road) indicates that approximately 10% of reliability is lost due to being unable to access flows when the tide rise above the weir.

The installation of a tidal barrier to prevent tidal flow up the drain could potentially be used to restrict tidal flows and allow stormwater to be extracted at any time. This would increase the reliability of the scheme or reduce the required storage size. This would require the use of a tidal gate at the outlet or within the drain that completely blocks tidal influx while allowing outflows to occur when flood levels exceed tide levels. The potential influence of groundwater inflows and their salinity would also need to be considered.

**Table 7-1 Storage sizes and reliability with a demand of 27,900 kL/year**

| Source / Tank size | 1,000 kL | 2,000 kL | 3,000 kL | 4,000 kL | 5,000 kL |
|---|---|---|---|---|---|
| **Catchment A weir 0.2m (29%)** | 52% | 63% | 69% | 73% | 76% |
| **Catchment A weir 0.4m (8%)** | 60% | 74% | 80% | 85% | 88% |
| **Catchment A weir 0.6m (1%)** | 63% | 76% | 83% | 86% | 89% |
| **Catchment A all flows** | 62% | 76% | 83% | 87% | 89% |
| **Catchment A, B and C weir 0.057m (50%)** | 49% | 61% | 67% | 70% | 73% |
| **Catchment A, B and C weir 0.2m (29%)** | 61% | 77% | 83% | 87% | 91% |
| **Catchment A, B and C weir 0.4m (8%)** | 66% | 81% | 86% | 90% | 93% |
| **Catchment A, B and C all flows** | 66% | 82% | 89% | 92% | 94% |

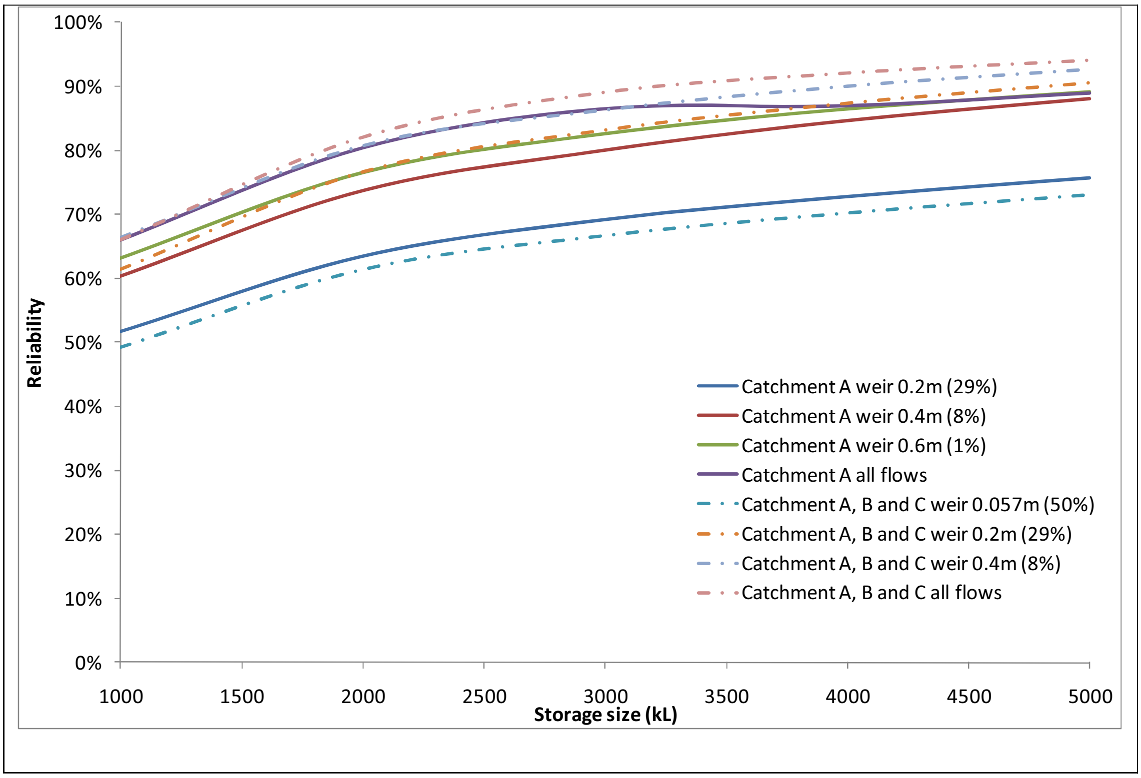


**Figure 7-1 Reliability curves for varying weir levels**

The storage required to achieve a reliability of 75% was calculated for each of the scenarios and summarised in Table 7-2. The most practical weir levels of 0.4 m or 0.6 m AHD at Plummer St or 0.4 m AHD at Williamstown Road require storage in the order of 2,000 kL. It is recommended a 2,000 kL storage is required to provide 75% reliability in supply. *Further analysis of the potential impacts of the diversion weir on flood levels is recommended.*

**Table 7-2 Storage sizes required for a reliability of 75%**

| Source | Weir level (m AHD) | Weir height (m) | Tank Size (kL) |
|---|---|---|---|
| Catchment A | 0.2 | 0 | 4,775 |
| | 0.4 | 0.2 | 2,204 |
| | 0.6 | 0.4 | 1,889 |
| | - | All flows | 1,624 |
| Catchment A, B and C | 0.057 | 0.1 | 5,568 |
| | 0.2 | 0.24 | 1,892 |
| | 0.4 | 0.44 | 1,602 |
| | - | All flows | 1,563 |

# 8.0 Appendix 2 – Wetland concept