Water Sensitive Urban Design

Engineering procedures for stormwater management in Tasmania

Tasmania
Explore the possibilities

nrm SOUTH
NATURAL RESOURCE MANAGEMENT

Derwent Estuary Program

NRM North
Tamar Estuary and Esk Rivers

Acknowledgments

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1.1 Why WSUD?

Recent years have seen an increasing number of initiatives to manage urban water in a more sustainable way. These initiatives are underpinned by the key principles of sustainability: water conservation, integrated water cycle management, waste minimisation and environmental protection. Water Sensitive Urban Design (WSUD) integrates these principles within urban planning and design, and is the best management design practice standard for development across Australia.

Key elements of WSUD include managing urban stormwater as a resource and the protection of receiving waterways and aquatic ecosystems. These aims have been identified as important for effective natural resource management in Tasmania by all levels of government.

This manual sets out a framework to assist in the design of stormwater treatment systems applicable to urban landscapes throughout Tasmania. It has been adapted to be applicable for all of Tasmania from Derwent Estuary Program's WSUD Engineering Procedures: Stormwater for Southern Tasmania (2006) and Melbourne Water's WSUD Engineering Procedures: Stormwater (2004).

This document provides construction, engineering and development assessment advice for stormwater management systems. The source of urban stormwater pollution and the justification for implementing WSUD practices have been widely published and are easily accessible (see Appendix C Resource Guide).

WSUD incorporates water management features into the urban landscape and has multiple environmental and aesthetic benefits such as:

- reducing stormwater flows and pollutant loads – thereby protecting downstream waterways – by collecting and treating stormwater in wetlands, ponds, bioretention swales or grass swales
- conserving potable water by collecting roof runoff and stormwater in rainwater tanks or underground storage for reuse in gardens and toilets
- minimising impervious surfaces by use of porous pavements (e.g. for carparks, roads and driveways) and minimising housing footprints
- providing public open spaces for stormwater treatment (e.g. wetlands), recreation and visual amenity (which also increases land values).

WSUD applies to both urban and rural developments and can either be retrofitted into existing urbanised catchments or incorporated at the design stage of new developments.

This manual will help ensure that WSUD designs represent best practice and a consistent design approach. This manual also provides assessment officers with checklists to assess the adequacy of proposed works submitted for approval. The target audience consists of both design engineers and local and state government approval officers. Design guidance in this manual is intended for professionals with experience in urban hydrology and hydraulics.

While the manual is primarily directed at engineers, it is recognised that all WSUD developments require the involvement of a range of professionals including planners, urban designers, landscape architects and environmental scientists. Sections of this manual contain expert design input from such professionals, based on Derwent Estuary Program's 'WSUD Engineering Procedures: Stormwater for Southern Tasmania (2006)' and Melbourne Water's 'WSUD Engineering Procedures: Stormwater (2004)'.

1.2 The Tasmanian context

The Tasmanian State Policy on Water Quality Management 1997 (SPWQM) sets the water quality management and objectives for the State including stormwater. The purpose of the SPWQM is to achieve the sustainable management of Tasmania's surface water and groundwater resources by protecting or enhancing their qualities while allowing for sustainable development in accordance with the objectives of Tasmania's Resource Management and Planning System (Schedule 1 of the State Policies and Projects Act 1993). The SPWQM has the following stormwater provisions.

<u>Clause 31 – Runoff from land disturbance</u>

- SPWQM states that planning schemes should require stormwater management strategies for development proposals that have the potential to give rise to off-site polluted stormwater runoff.
- The stormwater management strategies should address both the construction phase and the operational phase with maintenance of water quality objectives as a performance objective.

Clause 33 – Urban runoff

- The SPWQM states that erosion and stormwater controls must be specifically addressed at the design phase of proposals in accordance with Clause 31.
- The SPWQM states that state and local governments should also develop and maintain strategies for the reduction of stormwater pollution at source.
- Where stormwater has the potential to prejudice the achievement of water quality objectives, the SPWQM states that councils should prepare and implement a stormwater management plan.

To facilitate the implementation of the stormwater provisions contained in Division 3 of the SPWQM a State Stormwater Strategy was prepared and released by the Tasmanian EPA Division in 2010. The State Stormwater Strategy sets out key principles and standards for stormwater management in Tasmania, and identifies accepted guidance documents.

Managing stormwater in new developments

All new developments that create 500m^2 or more of additional impervious surface, including subdivisions, roads and other large developments, should incorporate best practice stormwater management. The following standards are recommended:

Construction stage

Soil and water management controls should be required and implemented through the Development Application process, including detailed Soil and Water Management Plans where warranted. Best practice guidance on sediment and erosion control measures is provided in the document Soil and Water Management on Building and Construction Sites (2009).

Operational stage

New developments should be designed to minimise impacts on stormwater quality and, where necessary, downstream flooding or flow regimes. Stormwater should be managed and treated at source using best management design practices (eg Water Sensitive Urban Design) to achieve the following stormwater management targets:

- 80 per cent reduction in the annual average load of total suspended solids
- 45 per cent reduction in the annual average load of total phosphorus
- 45 per cent reduction in the annual average load of total nitrogen

Best practice guidance on stormwater treatment options to achieve these targets is provided in this manual – WSUD Engineering Procedures: Stormwater for Tasmania (2012)

Managing stormwater in established urban areas

Management of urban runoff from established catchments should be based on a risk based prioritisation of catchments and stormwater pollution sources. Stormwater management plans should be prepared for high priority catchments, with a focus on 'at source' management. In particular, commercial areas, industrial sites and major roads tend to be significant sources of stormwater pollution. Best practice guidance on the development of stormwater management plans is provided in the Derwent Estuary Program's A Model Stormwater Management Plan for Hobart Regional Councils – Focus on New Town Rivulet Catchment (2005).

Maintaining natural drainage systems

Urban waterways, including rivulets, creeks and natural drainage lines, provide important water quality, ecological and amenity values and should be maintained, enhanced or restored. Piping or lining of natural channels should be seen only as a last resort. It is recommended that buffer zones be established to protect the values of urban waterways, and that any development within these areas be carefully managed. Best practice guidance on managing urban waterways is provided in the document Tasmanian Waterways and Wetlands Works Manual (2003).

In light of the above policies and guidelines, it is clear there is sufficient impetus for the broad adoption of WSUD to protect waterways from the impacts of stormwater pollution.

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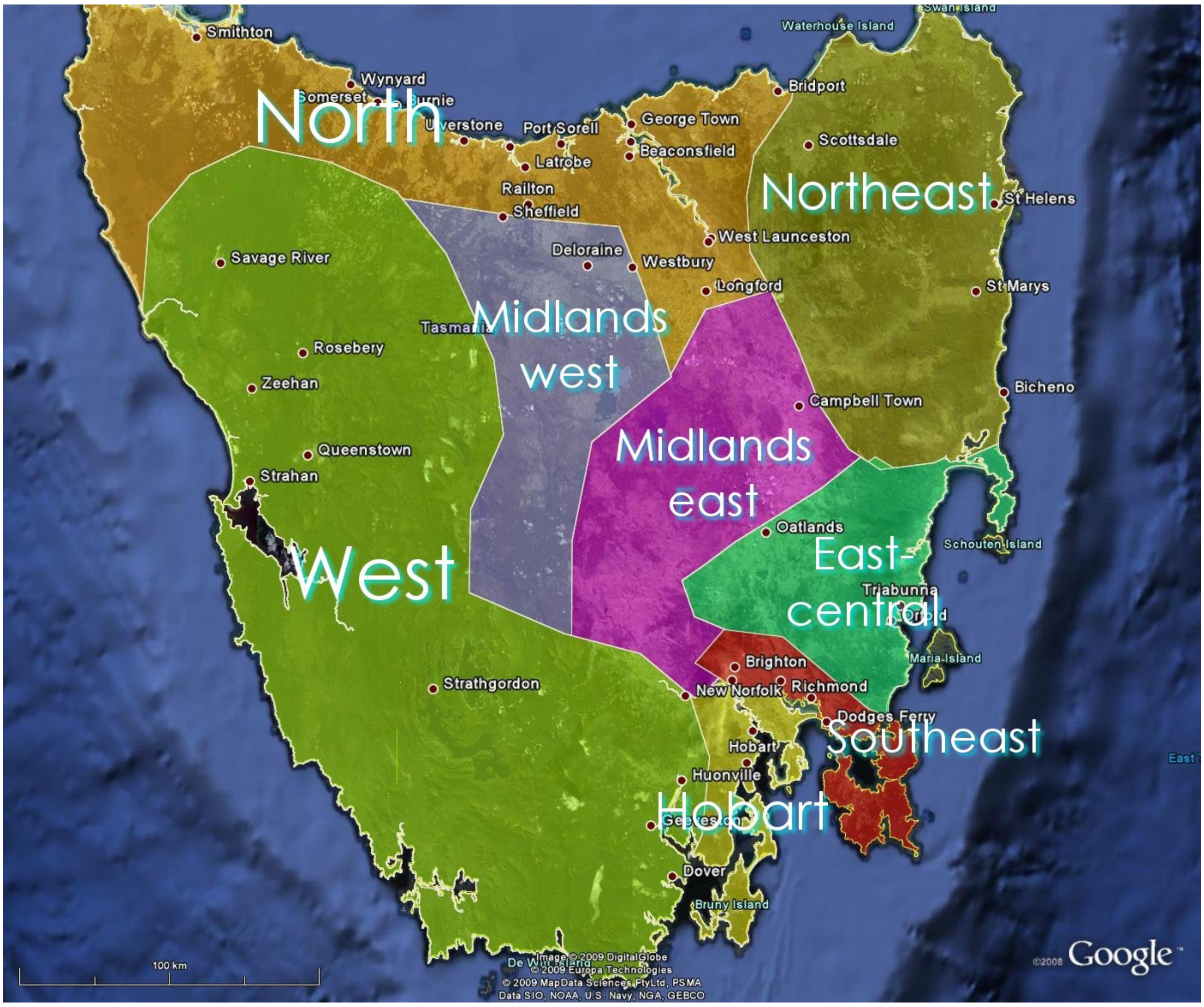

Figure 1.1. Map showing that this manual applies to the whole of Tasmania

1.3 WSUD elements

To effectively manage stormwater from different land uses, a combination of WSUD elements may be required to form what is commonly referred to as a **treatment train** – a series of treatment systems in a catchment that complement one another to achieve a desired stormwater management outcome. The selection and placement of these elements within a catchment should be determined during the concept design stage of a stormwater treatment strategy. An example of how WSUD elements could be incorporated into the urban landscape is shown in Figure 1.2.

Elements discussed in this manual include:

- sediment basins
- bioretention swales
- bioretention basins
- sand filters
- swale/ buffer systems
- constructed wetlands
- ponds
- infiltration measures
- rainwater tanks
- aquifer storage and recovery.

Additionally, Chapter 13 describes other stormwater management options such as proprietary products (including gross pollutant traps - GPTs), porous pavements and other treatment devices.

In combination, the placement of WSUD elements within a catchment or development will reduce stormwater pollution loads, peak flows and can help to conserve water.

The underlying principle of WSUD Engineering Procedures: Stormwater for Tasmania 2012 is the reduction of stormwater pollution from urban developments. The following pollutant reductions from the State Stormwater Strategy (2010) are recommended for the protection of receiving waters in Tasmania:

- total nitrogen 45% reduction
- total phosphorus 45% reduction
- total suspended solids 80% reduction

Stormwater treatment systems need to be sized to achieve this level of treatment. Inflows to any treatment system are assumed to contain concentrations of suspended solids and nutrients at levels typical to urban stormwater in Australia. All sizing guidance in the document is based on modelling using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC). It is suggested that to develop alternate, more specific treatment estimates, site-specific modelling is completed by the designer.

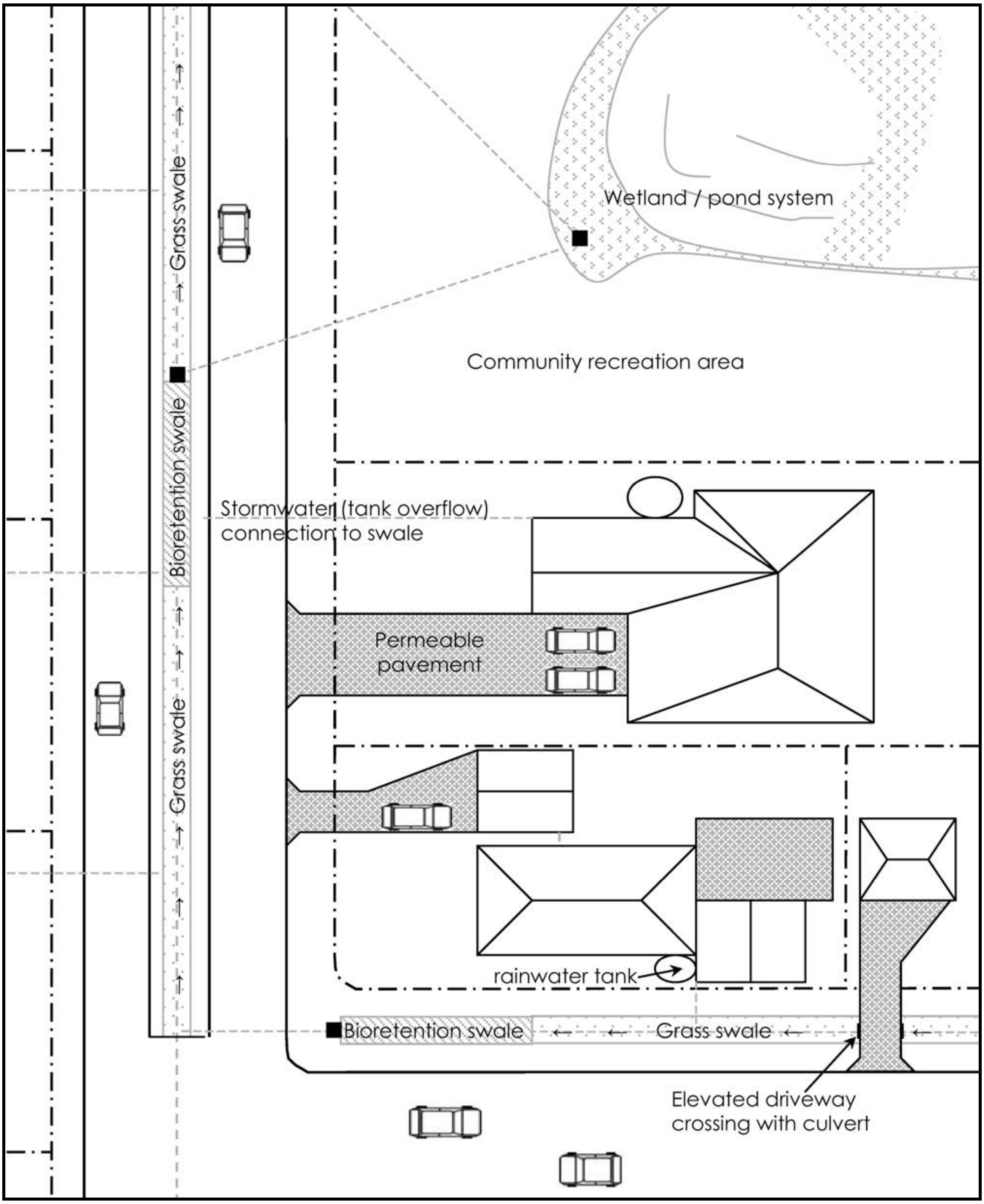

Figure 1.2. Example of WSUD incorporated into residential streetscape

1.3.1 Sediment basins

Sedimentation basins are used to retain coarse sediments (recommended particle size > 0.125mm) and are often the first element in a treatment train. Basins reduce flow velocities and encourage sediments to settle out of the water column. They can be designed to drain during dry periods (filling during runoff events) or to have a permanent pool.

Sedimentation basins are often used as pre-treatments to protect downstream WSUD elements (e.g. wetlands) from becoming loaded or smothered with sediments. They are also used for trapping sediment in runoff from construction sites.

Sediment basins can have various configurations including hard edges and base (e.g. concrete) or a more natural form with edge vegetation creating an attractive urban element. They are, however, typically turbid and maintenance usually requires de-watering and dredging collected sediments. The frequency of maintenance depends on the nature of the catchment and sediment loads.

1.3.2 Bioretention swales

Bioretention swales (or biofiltration trenches) are systems located within the base of a swale. Runoff is filtered through vegetation (e.g. grass, sedges or bushes) and a fine media layer as it percolates downwards. It is then collected via perforated pipes and flows to downstream waterways or to storages for reuse.

Bioretention swales provide efficient stormwater treatment through fine filtration, extended detention and some biological uptake, as well as providing a conveyance function (along the swale). They provide some flow retardation for frequent rainfall events and are particularly efficient at removing nitrogen and other soluble or fine particulate contaminants.

They can form attractive streetscapes and provide landscape features in an urban development. They are commonly located in the median strip of divided roads and car parks.

Bioretention systems are well suited to a wide range of soil conditions including areas affected by soil salinity and saline groundwater, as they are generally designed to minimise or eliminate the likelihood of stormwater infiltration.

Vegetation prevents erosion, continuously breaks up the soil through plant growth to prevent clogging, and provides biofilms on plant roots that pollutants can absorb to. The filtration process generally improves with denser and higher vegetation.

1.3.3 Bioretention basins

Bioretention basins operate with the same treatment processes as bioretention swales except they do not have a conveyance function. High flows are either diverted away from a basin or are discharged into an overflow structure.

They can form attractive streetscapes and provide landscape features in an urban development. Bioretention basins are extremely flexible in scale and shape making them a practical addition to any landscaping feature.

They can be located along streets at regular intervals and treat runoff prior to entry into an underground drainage system or be located at outfalls of a drainage system.

A wide range of vegetation can be used within a bioretention basin assisting integration into a landscape theme of an area. Smaller systems can be integrated into traffic calming measures or parking bays, reducing their requirement for space. They also suit retrofit scenarios.

Traffic, deliveries and washdown wastes need to be kept from bioretenion basins to reduce any potential for damage to the vegetation or the filter media surface.

1.3.4 Sand filters

Sand filters operate in a similar manner to bioretention systems with the exception that they have no vegetation growing on their surface. This is because they are either installed underground (therefore lack of available light limits vegetation growth) or the filter media does not retain sufficient moisture.

They are particularly useful in areas where space is a premium and treatment is best achieved underground (Due to the absence of vegetation, they require regular maintenance to ensure the surface of the sand filter media remains porous and does not become clogged with accumulated sediments).

Prior to entering a sand filter, flows are generally subjected to a pre-treatment to remove litter, debris and coarse sediments (typically a sedimentation chamber). Following pre-treatment, flows are spread over the sand filtration media and water percolates downwards to perforated pipes located at the base of the sand media. The perforated pipes collect treated water for conveyance downstream.

During higher flows, water can pond on the surface of the sand filter increasing the volume of water treated. Very high flows are diverted around sand filters to protect the sand media from scour.

1.3.5 Swale/ buffer systems

Vegetated swales use overland flows and mild slopes (generally 2% to 4%) to slowly convey water downstream. The interaction with vegetation promotes an even flow distribution and reduced velocities thus encouraging coarse sediments to be retained.

Swales can convey stormwater instead of pipes and provide a desirable 'buffer' between receiving waters (e.g. creek, wetland) and impervious areas of a catchment. They can be incorporated in urban designs along streets or parklands to add to the aesthetic character of an area.

The longitudinal slope of a swale is the most important consideration as slopes under 2% can tend to become waterlogged and have stagnant ponding (although the use of underdrains can alleviate this problem). Slopes steeper than 4%, may require the use of check banks to distribute flows evenly across swales as well as slow velocities. Dense vegetation and drop structures can be used to serve the same function as check dams but care needs to be exercised to ensure that velocities are not excessively high.

Vegetation is required to cover the whole width of a swale, be capable of withstanding design flows and be of sufficient density to provide good filtration. For best treatment performance, vegetation height should be above treatment flow water levels.

1.3.6 Constructed wetlands

Constructed wetland systems are shallow extensively vegetated water bodies that use enhanced sedimentation, fine filtration and biological pollutant uptake processes to remove pollutants from stormwater.

Wetlands provide some flow retardation for frequent rainfall events and are particularly efficient at removing nitrogen and other soluble or fine particulate contaminants. They provide habitat for wildlife and a focus for recreation, such as walking paths and resting areas.

Wetlands generally consist of an inlet zone (sediment basin to remove coarse sediments), a macrophyte zone (a shallow heavily vegetated area to remove fine particulates and uptake of soluble pollutants) and a high flow bypass channel (to protect the macrophyte zone). Water levels rise during rainfall events and outlets are configured to slowly release flows, typically over three days, back to dry weather water levels.

Wetlands can be constructed on many scales, from house block scale to large regional systems. In highly urban areas they can have a hard edge form and be part of a streetscape or forecourts of buildings. In regional settings they can be over 10 hectares in size and provide significant habitat for wildlife.

In addition to playing an important role in stormwater treatment, wetlands can also have significant community benefits. They can also improve the aesthetics of a development and be a central feature in a landscape.

1.3.7 Ponds

Ponds can be used for water quality treatment in areas where wetlands are not feasible (e.g. very steep terrain). Ponds (or lakes) promote particle sedimentation, adsorption of nutrients by phytoplankton and ultra violet disinfection. They can be used as storages for reuse schemes and urban landform features for recreation as well as wildlife habitat.

Ponds can provide some flow retardation for frequent rainfall events and are well suited to steep confined valleys where storage volumes can be maximised.

Ponds should be designed to settle fine particles and promote submerged macrophyte growth. Fringing vegetation contributes little to improving water quality but is necessary to reduce bank erosion. Ponds still require pre-treatment such as a sediment basin that needs maintaining more regularly than the main open waterbody.

1.3.8 Infiltration measures

Infiltration measures encourage stormwater to infiltrate into surrounding soils thereby reducing runoff and trapping pollutants. They are highly dependent on local soil characteristics and are best suited to sandy soils with deep groundwater. All infiltration measures require significant pre-treatment of stormwater before infiltration to avoid clogging of the surrounding soils and to protect groundwater quality.

Infiltration measures generally consist of a shallow excavated trench or 'tank' that is designed to detain a certain volume of runoff and subsequently infiltrate to the surrounding soils. Generally these measures are well suited to highly permeable soils, so that water can infiltrate at a sufficient rate. Areas with lower permeability soils may still be suitable, but larger areas for infiltration and detention storage volumes are required. In addition, infiltration measures are required to have sufficient set-back distances from structures to avoid any structural damage, these distances depend on local soil conditions. Careful design is crucial on slopes to prevent nuisance throughflow.

Infiltration measures can also be vegetated and provide some landscape amenity to an area. These systems provide improved pollutant removal through active plant growth improving filtration and ensuring the soil does not become 'clogged' with fine sediments.

1.3.9 Rainwater tanks

Rainwater tanks collect runoff from roof areas for subsequent reuse that reduces the demand on potable mains supplies and reduces stormwater discharges. In addition, they can provide a flood retardation function provided adequate temporary storage is available either through appropriate sizing (e.g. even small tanks can offer significant retention of roof runoff if drawn down frequently) or through temporary detention storage.

There are many forms of rainwater tanks available. They can be incorporated into building designs so they do not impact on the aesthetics of a development. They can also be located underground or some newer designs incorporate tanks into fence or wall elements or as part of a gutter system itself.

To improve the quality of the stored water, tanks can be fitted with 'first flush diverters'. These are simple mechanical devices that divert the first portion of runoff volume (that typically carries debris) away from the tank. After the first flush diversion, water passes directly into the tank.

Collected roof water is suitable for direct use for garden irrigation or toilet flushing with no additional treatment. Tank water can also be used in hot water systems, although some additional treatment may be required to reduce the risk of pathogens depending on the design of the system. This generally involves UV disinfection and ensuring that a hot water service maintains a temperature of at least 60 degrees.

Roof runoff that is reused also prevents stormwater pollutants (generated on roofs) from washing downstream.

1.3.10 Aquifer storage and recovery

Aquifer storage and recovery (ASR) is a means of enhancing water recharge to underground aquifers through either pumping or gravity feed. It can be a low cost alternative to store water compared to surface storages. Excess water produced from urban areas during wet periods (e.g. winter) can be stored underground and subsequently harvested during dry periods to reduce reliance on mains supply.

Harvesting urban runoff and diverting it into underground groundwater systems requires that the quality of the injected water is sufficient to protect the beneficial uses of the receiving ground water. The level of treatment required is dependent on the quality of the groundwater. In most instances the treatment measures described in this manual will provide sufficient treatment prior to injection.

The viability of an ASR scheme is highly dependent on the underlying geology of an area and the presence and nature of aquifers. There are a range of aquifer types that can accommodate an ASR scheme including fracture unconfined rock and confined sand and gravel aquifers. Detailed geological investigations are required to establish the feasibility of any ASR scheme. This manual provides an overview of the main elements of the system and directs readers to more specific guidance documents.

1.4 Document framework

This document provides design and development guidance and is structured in the following manner:

Chapter 2 describes the concept of ‘hydrologic regions’ each of which have allocated a set of ‘adjustment factors’ that allow required sizes of treatment systems for anywhere in Tasmania to be calculated from the sizes provided for the reference site used in this document, i.e. Hobart.

Chapter 3 onwards outlines the design process of each WSUD element while providing siting, management and construction advice along the way.

Comprehensive guidance on designing a treatment train or assessing the requirements for stormwater treatment is not covered thoroughly in this document. However, Appendix C Resource Guide contains references and some commentary on where to find this information. The list is by no means exhaustive but contains a broad variety of relevant literature that will help in this area.

1.5 Acknowledgment

WSUD Engineering Procedures for Stormwater Management Tasmania (2012) is an adaptation of Derwent Estuary Program’s WSUD Engineering Procedures: Stormwater for Southern Tasmania (2006) and Melbourne Water’s WSUD Engineering Procedures: Stormwater (2004).

Chapters one and two in this document are loosely based on Melbourne Water’s WSUD Engineering Procedures: Stormwater. The content of these introductory chapters have been rewritten to better suit Tasmania, its climatic, policy and regulatory environments.

Textual content in Chapters three to thirteen have been modified only minimally in the adaptation. Maps and algorithms within this document have been developed using all available, suitable, Tasmanian rainfall data. The modelling software and methodology used in this process is based on those used in Melbourne Water’s WSUD Engineering Procedures: Stormwater.

1.6 Disclaimer

The purpose of this document is to provide guidance only in the design and implementation of Water Sensitive Urban Design elements and should be used as a part of a holistic and rigorous design process by a qualified designer experienced in the field.

In the adoption of the guidelines within this document, the user acknowledges and agrees that –

1. Whilst the Crown in right of Tasmania (including the Department of Primary Industries, Parks, Water and Environment) has made every effort to ensure the accuracy and reliability of this document, the Crown does not accept any responsibility for the accuracy, completeness, or relevance to the user's purpose, of the information in this document. Those using it for whatever purpose are advised to verify its accuracy and to obtain appropriate professional advice. The Crown, its officers, employees, agents

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2. The worked examples provided in the document and the figures/information included therein are not to be used as actual design data. Designers are to use site-specific data and adhere to the requirements defined by the relevant Local Authorities and recognised industry standard procedures for the design being undertaken.

3. Rainfall data specific to the area of study is to be used where possible. Where such data is not available, IFD curves are to be derived from standard procedures as defined in Australian Rainfall and Runoff (2003).

4. The mean annual rainfall and adjustment factor methodology should be used in all cases where more detailed modelling is not performed, even in close proximity to the reference site. Sizing calculations have been developed using current industry-accepted data and software, however, treatment performance for any given size cannot be guaranteed due to the inherent variability in natural systems.

Chapter 2 Hydrologic Design Regions

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2.1 Introduction

This chapter outlines a methodology for sizing stormwater treatment systems, or WSUD elements, across Tasmania.

The following process has been developed through extensive stormwater modelling using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC) developed by eWater. For a higher level of accuracy, it is suggested that site-specific modelling using MUSIC be performed.

Simulation of the performance of WSUD systems were completed using pluviographic rainfall data for 48 Bureau of Meteorology (BOM) rainfall stations distributed throughout Tasmania. Results of these simulations were used to develop eight Hydrologic Regions (see figure 3.1). The Hydrologic Regions allow the WSUD designer to determine the size of a WSUD element for a given pollutant removal objective, with the only rainfall data required being the mean annual rainfall (MAR) of a site.

2.2 Development of Hydrologic Regions

Constructed wetlands, ponds, bioretention systems and vegetated swales were modelled at 48 locations distributed right across Tasmania. The locations modelled were chosen based on the availability of BOM pluviographic rainfall data. The data used was six-minute time step rainfall data from 1980 onwards. Of the stations used, the duration of data sets ranged from 1-24 years. No pre-1980 data was used due to a shift in rainfall patterns.

Each WSUD element was modelled to determine the size of system required at each location to provide a 45% reduction in total nitrogen (TN). TN was used because it is commonly the limiting parameter in meeting best practice stormwater quality objectives. 45% was chosen as an arbitrary, although realistic, performance objective.

The 48 sites were modelled individually to determine the difference in system size needed to meet the required performance objective compared to one reference site. These differences were used to develop 'adjustment factors' so that sizing information from a reference site could easily be adapted for anywhere in Tasmania.

Specific, more-detailed modelling was performed at one site (see Figure 2.1) to provide a reference site from which the size of WSUD elements could be calculated wherever in Tasmania.

More detail on the methodology used to develop the Hydrological Region Process is provided in Appendix A: Tasmanian Hydrological Regions.

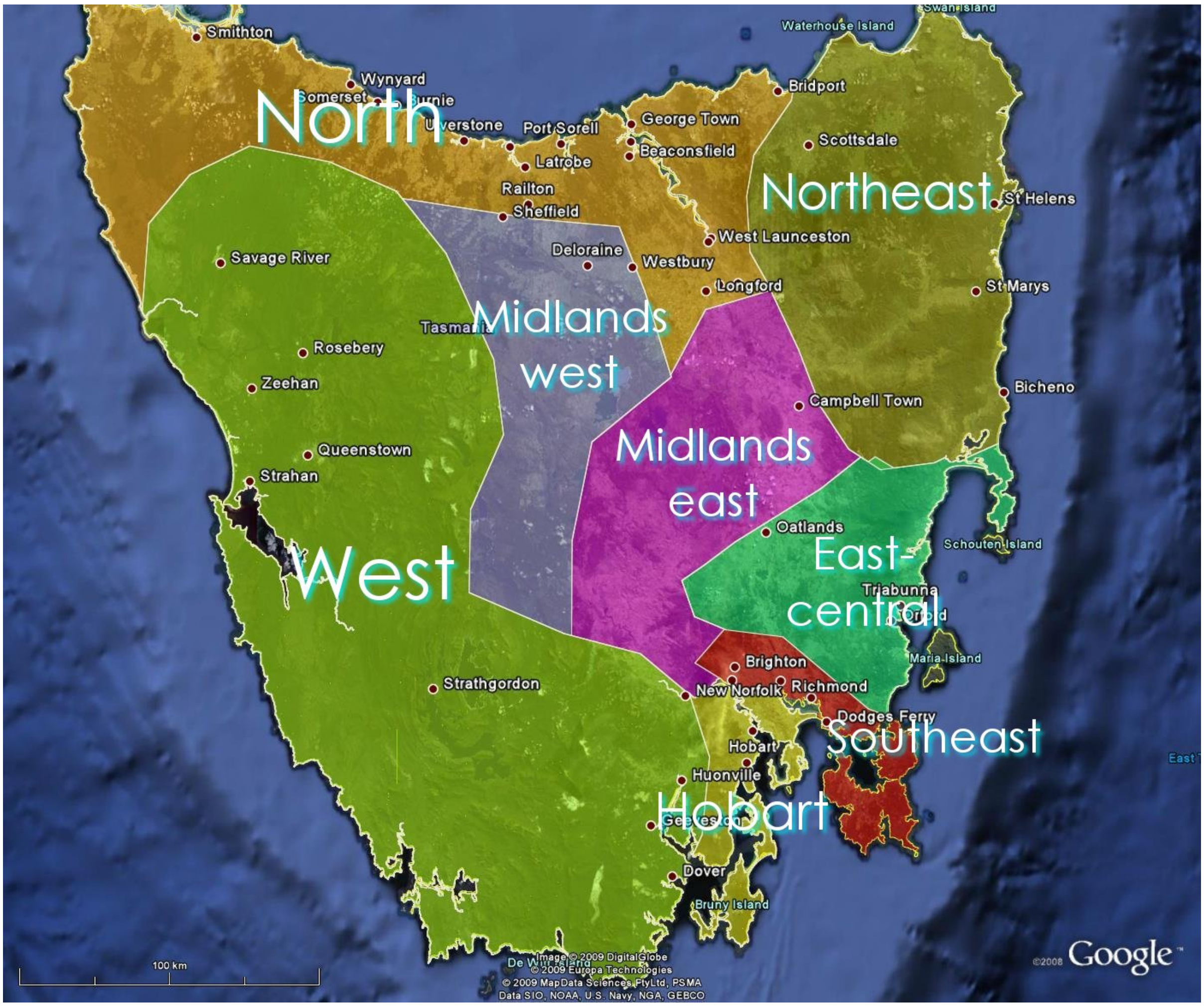

Figure 2.1. Map showing the eight hydrologic regions for Tasmania

2.3 Sizing WSUD elements

Chapters 4, 5, 7, 8 & 9 contain WSUD element sizing graphs for the reference site. Three sizing graphs are provided showing the required size of a treatment system to meet a desired pollutant reduction. The graphs represent sizing information to meet objectives of three pollutants, total suspended solids (TSS), total phosphorus (TP) and total nitrogen (TN) (see figure 3.2).

Figure 2.2 demonstrates how to determine the required size of a wetland, at the reference site, designed to provide a 45% reduction in TN, 45% reduction in TP and an 80% reduction in

TSS. A horizontal line is drawn from the desired % reductions on the y-axis to meet the pollutant curve. In this case, 3 pollutant removal objectives have been set so the size of wetland chosen must meet all the targets. In this example, the TN objective requires the largest wetland, therefore, a wetland equivalent in surface area to 3% of the contributing impervious catchment is needed to provide greater than or equal to the required objective for all pollutants.

To determine the size of a wetland that will meet pollutant removal objectives, the required size at the reference site is first calculated using the above methodology. An adjustment factor is then applied to that size to determine the size wetland required in the area of interest.

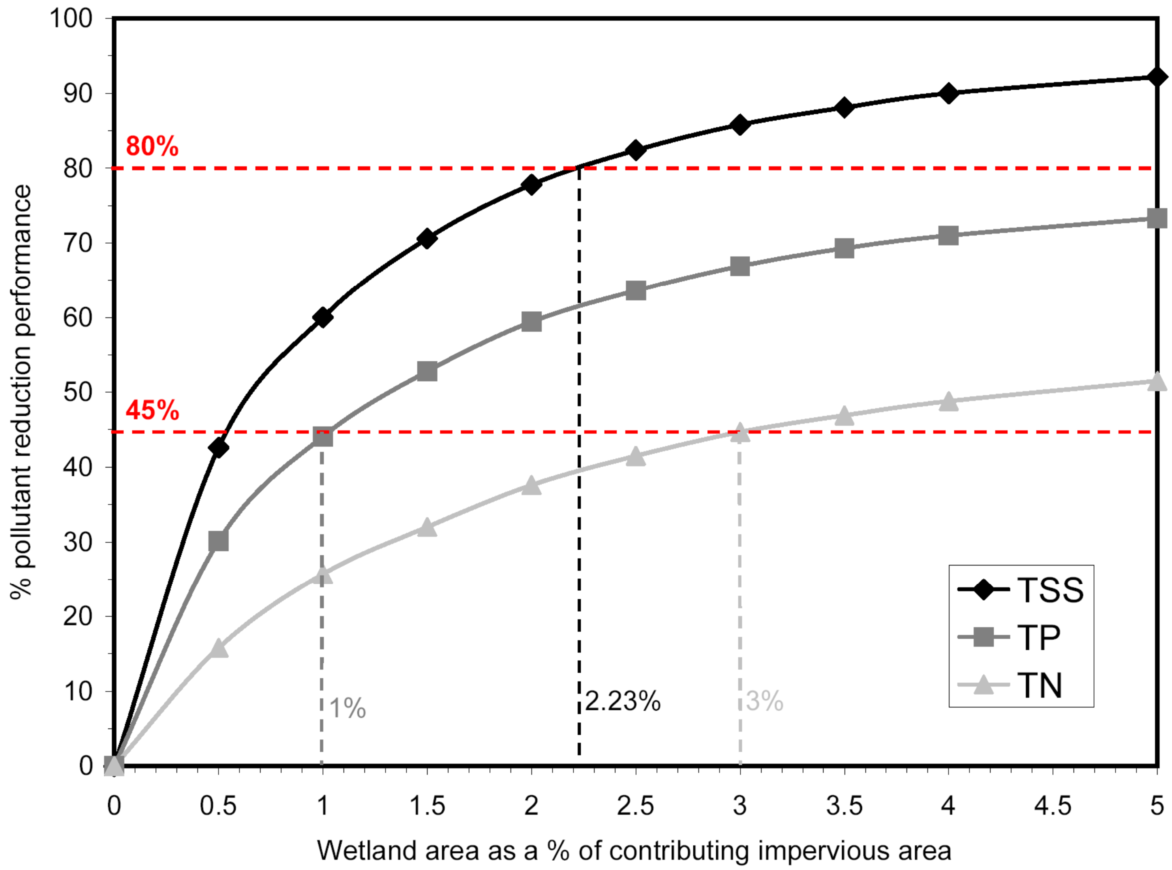

Figure 2.2. Size of wetland required to meet desired pollutant reduction in Hobart

2.4 Calculating adjustment factors

To calculate an adjustment factor (A_f) for anywhere in Tasmania, the only site specific data required is mean annual rainfall (MAR) for the site.

Different adjustment factors are used for each WSUD element (wetlands, bioretention, swales and ponds).

To account for uncertainty in the relationship between mean annual rainfall and the variation in size of treatment systems across the state, a safety factor of 10% is included in the adjustment factor relationships. To avoid this requirement, site-specific modelling should be completed.

Table 2-1. Adjustment factor equations

	Bioretention	Pond	Swale	Wetland
East-central	0.0053(MAR)-1.2818	0.0031(MAR)-0.5156	0.0004(MAR)+0.8559	0.0024(MAR)-0.1239
Hobart	0.0041(MAR)-0.9728	0.0044(MAR)-1.3281	0.0002(MAR)+0.9274	0.0017(MAR)+0.1123
Midlands-east	-0.0038(MAR)+2.835	0.0046(MAR)-1.3754	-0.0071(MAR)+4.5261	-0.0016(MAR)+1.6456
Midlands-west	-0.0072(MAR)+9.4801	0.0027(MAR)+0.0379	-0.0007(MAR)+1.8961	-0.001(MAR)+2.5053
North	0.0016(MAR)+0.3281	0.0033(MAR)-0.7432	0.0005(MAR)+0.794	0.0006(MAR)+0.5906
Northeast	0.0034(MAR)-0.8879	0.003(MAR)-0.4068	0.0005(MAR)+0.9196	0.0012(MAR)+0.5645
Southeast	0.0025(MAR)-0.1253	0.003(MAR)-0.562	0.00003(MAR)+1.0481	0.0012(MAR)+0.3184
West	0.0009(MAR)+0.4632	0.0029(MAR)-0.4404	0.0005(MAR)+0.6041	0.0007(MAR)+0.5005

2.5 Using the adjustment factor methodology

Step 1 – determine treatment objectives

The first step should be to check with the local authority to see if performance objectives for stormwater treatment are required. It is recommended that the stormwater management targets set in State Stormwater Strategy (2010) be used. The Strategy sets an 80% reduction in TSS, 45% reduction in TP and 45% reduction in TN. These targets should provide an adequate level of treatment for the protection of receiving waters throughout Tasmania.

Step 2 – calculate the size of system required at the reference site

Follow the methodology detailed in 2.3 Sizing WSUD elements to calculate the required size of the planned WSUD element in Hobart, the reference site using the sizing graphs provided in the detailed design chapter for that WSUD element.

Step 3 – calculate the appropriate adjustment factor

Determine the Hydrologic Region in which the WSUD element is to be built from figure 2.1. Find out the mean annual rainfall (MAR) at the site (MAR of the site may be obtained from the Bureau of Meteorology if not already known). Apply the adjustment factor equation (from table 3.1) with the MAR of the proposed site to determine the adjustment factor. (NOTE: **MAR** SHOULD BE EXPRESSED IN METRES)

Step 4 – calculate the required size of WSUD element

Multiply the size of treatment system calculated in step 2 by the adjustment factor calculated in step 3 to determine the actual size of system required at the location of interest.

2.6 Worked example

A large ecotourism development in Bicheno plans to install a constructed wetland within its 12 Ha development site. The project proponents would like to achieve the State Stormwater Strategy (2010) stormwater management targets – a 45% reduction in TN, 45% reduction in TP and an 80% reduction in TSS for stormwater leaving the site so that they may assure visitors that the site will not adversely affect local receiving waters. Bicheno has a mean annual rainfall (MAR) of 675.8 mm. The project designers have calculated that the site will have impervious surfaces making up 45% of the total site area.

Calculations for sizing the wetland are as follows:

1. From Figure 2.2, a wetland at the reference site needs a surface area equal to 3% of the contributing impervious area to meet objectives,

   i.e. contributing impervious area = (45/100) x (12 x 10,000) = 54,000 m^2

   reference wetland area = 0.03 x 54,000 = 1,620 m^2.

2. From figure 2.1, it is apparent that Bicheno lies within the NORTHEAST hydrologic region.

3. The *Adjustment Factor* for the NORTHEAST region is computed using the wetland adjustment equation for the NORTHEAST region in Table 2-1:

   Adjustment Factor = 0.0012 (MAR) + 0.5645

   =0.0012 (0.6758) + 0.5645 = 0.5653

4. The required wetland area is 0.5653 x 1,620 = 916m^2.

Thus, a wetland treating a 12 Ha catchment in Bicheno (with 675.8mm annual rainfall) is required to be 916 m^2 to give the same level of treatment as a 1,620 m^2 wetland at the Hobart reference site.

Chapter 3 Sediment Basins

Definition:

A sediment basin is a pond like structure designed to remove coarse sediments from stormwater by reducing flow velocities to the settling velocity of the target sediment size.

Purpose:

- To retain coarse sediments from runoff.
- Are typically the first elements in a treatment train and play an important role by protecting downstream elements from becoming overloaded or smothered with sediments.
- Provide some minor flood attenuation by providing storage.
- Where the target sediment size is below 0.125mm, low levels of hydrocarbons (that have adhered to the particulates) will be removed.

Implementation considerations:

- Sediment basins can require considerable area to achieve desired treatment levels of the target particle size.
- When used for trapping sediment in runoff from construction sites, maintenance frequency can increase dramatically from the high sediment yields.
- Sediment basins can have various configurations including hard edges and base (e.g. concrete) or a more natural form with edge vegetation creating an attractive urban element. They are, however, typically turbid and maintenance usually requires significant disturbance of the system.

Maintenance involves dewatering and dredging collected sediments. This is required approximately every five years.

Sedimentation basins can be installed into hard or soft landscapes

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3.1 Introduction

The reduction of sediment loads is a key process in protecting downstream waterways ensuring the effective operation and long term efficiency of downstream stormwater treatment measures (known as treatment trains). Sedimentation basins are specifically employed to remove coarse to medium-sized sediments (generally 0.125mm and greater) by reducing flow velocities and providing retention of these particulates.

Sedimentation basins can take various forms (generally ponds) and can be classified as any treatment system that primarily functions to promote settling of sediments through processes of temporary detention and reduction of flow velocities

They can be made permanent structures and incorporated into urban designs (parkland or recreational area water features) or temporary structures to control sediment runoff during the development of new areas.

Figure 3.1 shows the layout of a typical permanent sedimentation basin.

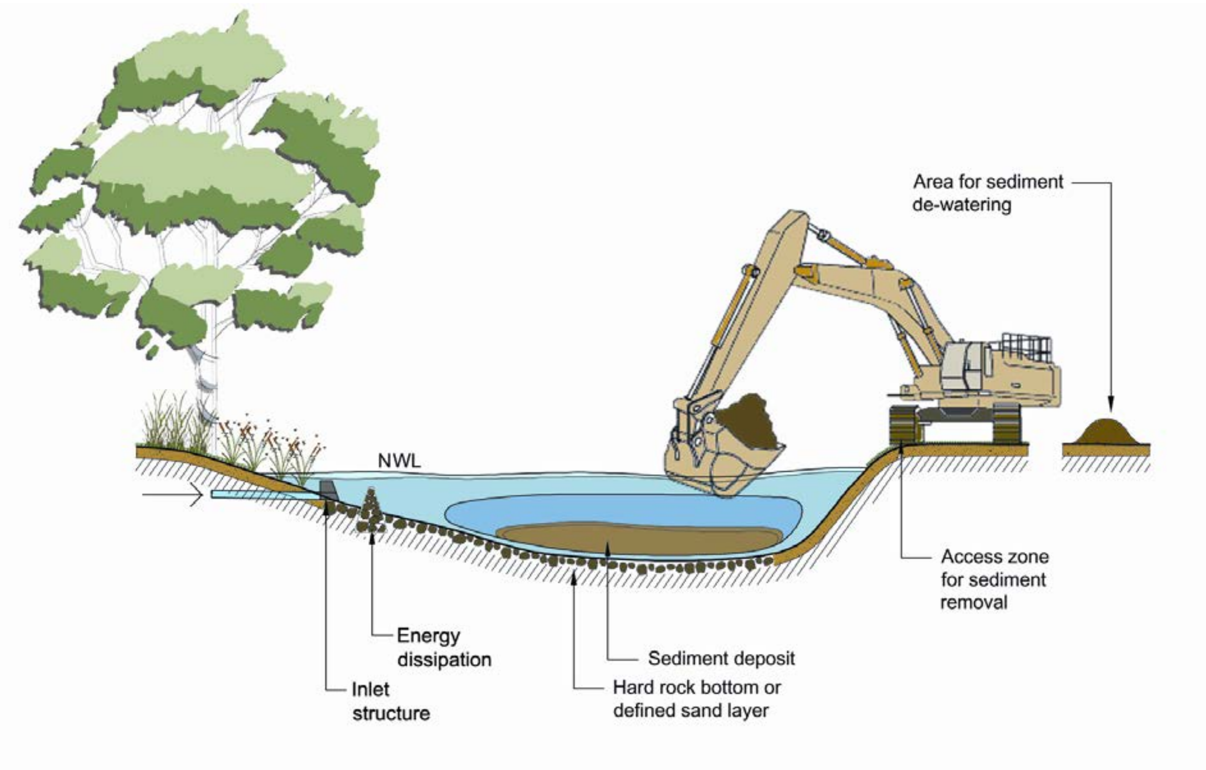


Figure 3.1. Sedimentation basin layout

The required treatment size of a sedimentation basin is calculated primarily on two main factors:

1. The settling velocity of the target sediment size, and
2. The design flow at the required design rainfall intensity.

Analysis of typical urban catchment sediment loads suggest that between 50% to 80% of suspended solids conveyed in urban stormwater are 125 μm or larger. Basins sized to target these size particulates are expected to capture sediment that has low levels of contamination (because of the larger sediment sizes) and is unlikely to require special handling and disposal.

Particular care needs to be undertaken when sizing sedimentation systems because a basin that is under sized may have limited effectiveness (insufficient retention of particulates) and cause smothering of downstream treatment measures reducing their treatment efficiency. Conversely, basins that are oversized run an increased risk of excess accumulation of fine particulates resulting in higher contaminant concentrations that could require specialist handling facilities for maintenance (de-watering and disposal).

A further consideration in sizing a sedimentation basin is the provision of adequate storage for settled sediment to prevent the need for frequent de-silting. A desirable maintenance frequency for permanent facilities is once every five years (normally triggered when sediment accumulates to half the basin depth). Temporary systems will require more frequent maintenance depending on the catchment area and likely sediment loads.

Apart from the issues associated with the appropriate sizing of a sedimentation basin for effective capture and retention of sediment, design considerations are similar to those for ponds and constructed wetlands.

3.2 Verifying size for treatment

Figure 3.2 shows relationships between a required basin area and design discharge for 125 μm sediment capture efficiencies of 70%, 80% and 90% using a typical shape and configuration ($\lambda = 0.5$, see Section 4.3.2).

The influence of a permanent pool reduces flow velocities in the sedimentation basin and thus increases detention times in the basin (and hence removal efficiency). A typical two metre depth permanent pool was used to define the lower limit of the required sedimentation basin thus forming three shaded areas in Figure 3.2.

The performance of typical designs of sedimentation basins can be expected to fall within the shaded curves shown and they can be used to verify the selected size of a proposed sedimentation basin. As the design charts relate the size of a required sedimentation basin to a design flow, they are applicable in all regions and do not require any adjustments for the different hydrologic regions.

The volume of a permanent pool in a sedimentation basin should have sufficient capacity to ensure that desilting of the basin is not more frequent than once every 5 years (unless it is to be used for temporary sediment control when cleaning every 6-months may be appropriate).

A developing catchment can be expected to discharge between 50 m^3 and 200 m^3 of sediment per hectare each year. In a developed catchment, the annual sediment export is generally one to two orders of magnitude lower with an expected mean annual rate of 1.60 m^3/ha. There are different methods used to estimate sediment loads and some authorities have produced charts of sediment loading rates (ACT Government, 1994; NSW Department of

Housing, 1998). Desilting is required when the permanent pool is half full with deposited sediment.

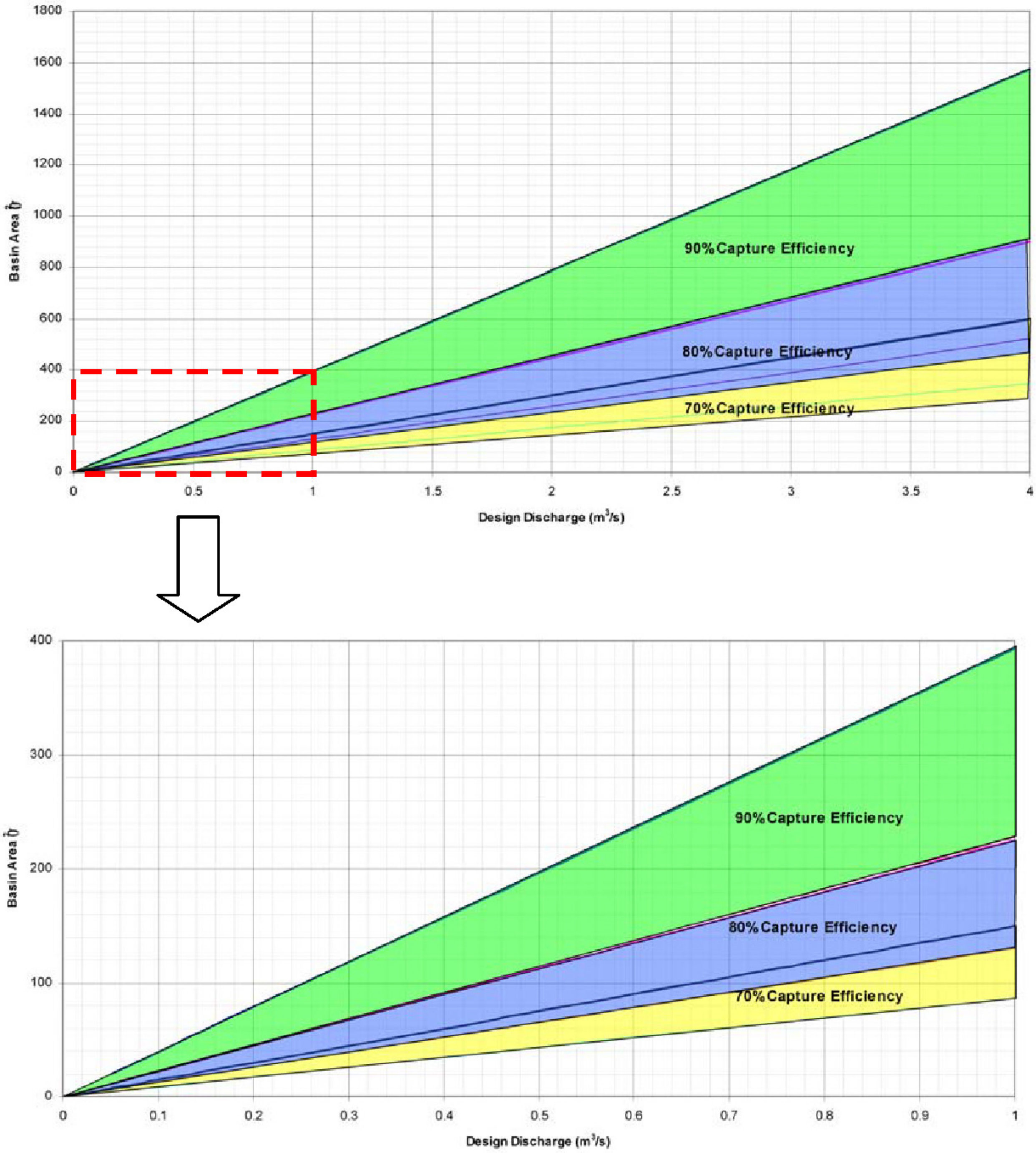


Figure 3.2. Sedimentation Basin Area Vs Design Discharges for varying capture efficiencies of 125 µm sediment

3.3 Design procedure: sedimentation basins

3.3.1 Estimating design flows

3.3.1.1 Design Discharges

Two, possibly three, design flows are required for sedimentation basins:

- Design operation flow - required to enable calculation of the size of the basin to allow settling and retention of the design particulate size (Local councils and regional catchment management authorities may stipulate the design operation flow).
- Minor system design flow - for the design of the inlet and outlet structures to ensure system flows can be accommodated.
- Major flood flows - for the design of the basin overflow structure.

Typical design operation flows are the 1-year ARI (Average Recurrence Interval) or 2-year ARI peak discharge, and for permanent sedimentation basins used as pre-treatments for downstream stormwater treatment measures, this is normally the 1 year ARI peak design.

The design of inflow structures are typically based on higher ARI rainfall events as they need to have capacity to convey the design discharge of the upstream stormwater drainage system. In Tasmania the general design ARI for stormwater drainage systems (other than specific flood-way or conveyance structures) is the 10 year ARI. High risk areas such as commercial, industrial or dense residential may be designed for a 20 year ARI. Local authorities must be consulted prior to design to confirm the desired design ARI.

All flows should be directed through a sedimentation basin so that some level of sedimentation is achieved even during high flow conditions. In situations where the basin forms part or a major drainage system (through flow) the capacity of the inlet and outlet structures must be equal to or greater than the design ARI of the upstream and downstream infrastructure. Local authorities must be consulted prior to design to confirm the design ARI of these components.

3.3.1.2 Minor and major flood estimation

Catchment peak discharge rates can be calculated using a range of hydrologic methods. The Rational Method Design Procedure is suitable for most, however for large catchments (greater than 100Ha), the Kinematic Wave Equation may be employed. These methods are described in detail in the document titled Australian Rainfall and Runoff.

Flood estimation in larger or complex catchments with multiple catchments and flow routing requires specialist knowledge of flood estimation methods and normally involves detailed topographical, hydrologic and hydraulic study of the catchment area. The Rational Method or Kinematic Wave Equation should not be relied upon singly for estimation in these situations.

3.3.2 Size and shape of sedimentation basins

Estimating the required area (A) of a sedimentation basin may be based on the mathematical expression derived by Fair and Geyer (1954), formulated for wastewater sedimentation basin design:

$$R = 1 - \left(1 + \frac{1}{n}\frac{v_s}{Q/A}\right)^{-n}$$

Equation 3.1

where	R	=	fraction of target sediment removed
	v_S	=	settling velocity of target sediment
	Q/A	=	rate of applied flow divided by basin surface area
	n	=	turbulence or short-circuiting parameter

Note that Equation 3.1is strictly applicable for systems with no permanent pool, and will generally over-estimate the required area of a sedimentation basin. This equation is thus often considered to provide an upper limit estimate of the required size for sedimentation basins.

In Equation 3.1the key factor for design is the ‘n’ value which is known as the turbulence parameter. This factor takes into account the configuration of the basin (location of inlet and outlet structures, flow spreaders, flow diversion structure for mixing etc) and the effects of the different configurations on settling efficiency directly relating to the ability of the system to short circuit. In order to calculate ‘n’ a hydrodynamic adjustment value (λ) must be selected that best represents the configuration of the basin and is described in Figure 3.3.

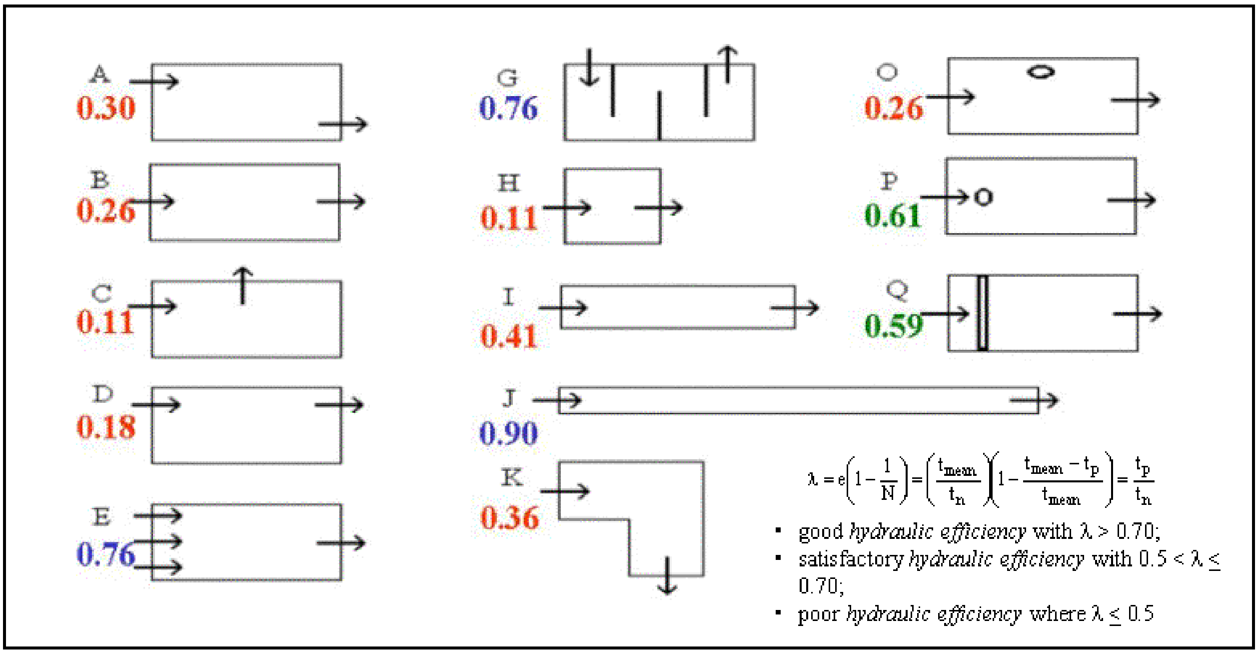


Figure 3.3. Hydraulic Efficiency - λ - A measure of Flow Hydrodynamic Conditions in Constructed Wetlands and Ponds; Range is from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment (Persson et al., 1999)

Figure 3.3 provides some guidance on what is considered to be good basin design with the higher values (of λ) representing basins with good sediment retention properties, where a value of λ greater than 0.5 should be a design objective. If the basin configuration yields a lower value, modification to the basin configuration should be explored to increase the λ value (e.g. inclusion of baffles, islands or flow spreaders).

The shape of the basin has a large impact on the effectiveness of the basin to retain sediments and generally a length to width ratio of at least 3:1 should be aimed for. In addition, the location of the inlet and outlet, flow spreaders and internal baffles impact the hydraulic efficiency of the basin for stormwater treatment. These types of elements are noted in Figure 3.3 as the figure “o” in diagrams O and P (which represent islands in a waterbody) and the double line in diagram Q which represents a structure to distribute flows evenly.

DESIGN ADVICE –

Consideration of maintenance access to a basin is also required when developing the shape of a basin as this can impact the allowable width (if access is from the banks) or the shape if access ramps into a basin are required. An area for sediment de-watering should also be accommodated and it is required to drain back into the basin. This too may impact on the footprint area required for a sedimentation basin system.

Once a design layout has been derived and an appropriate value of λ has been selected, a value for ‘n’ is then calculated using the following relationship:

$$\lambda = 1 - 1/n; \quad n = \frac{1}{1-\lambda}$$

Equation 3.2

DESIGN ADVICE –

Good practice in the design of sedimentation basins will include a permanent pool of water to reduce flow velocities and provide storage of settled sediment. The presence of a permanent pool reduces flow velocities in the sedimentation basin and thus increases detention times.

With the outlet structure being located above the bed of a sedimentation basin, it is also not necessary for sediment particles to settle all the way to the bottom of the basin to be effectively retained. A reasonable depth is considered to be approximately 1 m below the permanent pool level.

If a permanent pool is to be employed, Equation 3.1can be re-derived to account for the effect of the permanent pool storage as follows:

$$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)} \right]^{-n}$$

Equation 3.3

where d_e is the extended detention depth (m) above the permanent pool level

d_p is the depth (m) of the permanent pool

d^* is the depth below the permanent pool level that is sufficient to retain the target sediment (m) – adopt 1.0 or d_p whichever is lower

R is the fraction of target sediment removed

v_S is the settling velocity of target sediment

Q/A is the rate of applied flow divided by basin surface area

n is the turbulence or short-circuiting parameter

The table below lists the typical settling velocities of sediments.

Table 3-1. Settling velocities under ideal conditions

Classification of particle size	Particle diameter (μm)	Settling velocities (mm/s)
Very coarse sand	2000	200
Coarse sand	1000	100
Medium sand	500	53
Fine sand	250	26
Very fine sand	125	11
Coarse silt	62	2.3
Medium silt	31	0.66
Fine silt	16	0.18
Very fine silt	8	0.04
Clay	4	0.011

A further check to confirm the size of a sedimentation basin is the required volume for storage of accumulated sediments and the impact of this volume on required cleaning frequencies.

Loading rates (estimates of loading rates are described in Section 3.2) can then be used to estimate the required storage volume for each clean out and this volume checked against the allowable sediment accumulation volume given the basin configuration estimated using Equation 3.1 or Equation 3.3. The storage volume should be estimated using half the permanent pool volume as this level of accumulation is recommended to be adopted as the trigger for a clean out.

The required volume of sediment storage (S) can be estimated using Equation 3.4:

$$S_t = C_a \times R \times L_o \times F_r$$

Equation 3.4

Where: S_t = volume of storage required (m^3)

C_a = contributing catchment area (ha)

R = capture efficiency (%), estimated from Equation 4.1 or 4.3

L_o = sediment loading rate (m^3/ha/year) **

F_r = desired cleanout frequency (years)

The fraction of sediment removed for the target pollutant (R) is assumed to represent the fraction of the total sediment load removed. In fact, a higher fraction of coarser particles than the target pollutant and a lower fraction for finer particles will be retained than the R-value. R however provides a reasonable estimate of the overall sediment capture efficiency.

** - This figure can be obtained from widely available sediment yield tables. A table should be selected that matches the proposed development type, local soil conditions and topography.

3.3.3 Cross Sections

With the exception of temporary sedimentation basins for construction site management, batter slopes on approaches and immediately under the water line of a basin should be configured with the utmost consideration to public safety.

Generally there are two approaches to construction: either hard or soft edge treatments. These can be applied individually or combined to compliment the landscape of a surrounding area.

Soft edge treatments involve using gentle slopes to the waters edge, extending below the water line for a distance before batter slopes steepen into deeper areas. This is illustrated in Figure 3.4.

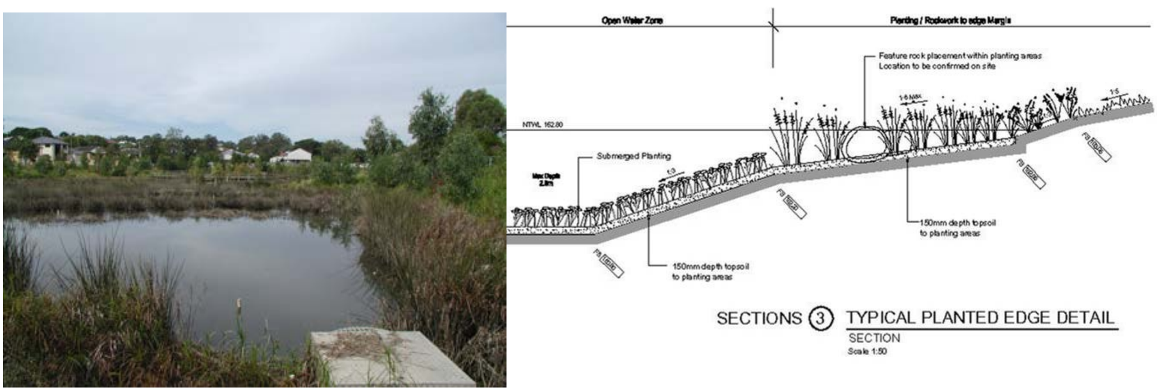


Figure 3.4. Illustration of a soft edge treatment for open waterbodies (GbLA, 2004)

An alternative to the adoption of a flat batter slope beneath the water line is to provide a 3 m "safety bench" that is less than 0.2 m deep below the permanent pool level around the waterbody.

Figure 3.5 shows two options for hard edge details. One has a larger vertical wall and associated handrail for public safety and the other is a low vertical wall. In both hard edge details, it is proposed to line the bottom of the waterbody with rock to prevent vegetation (particularly weed) growth.

The safety requirements for individual basins may vary from site to site, and it is recommended that an independent safety audit be conducted of each design.

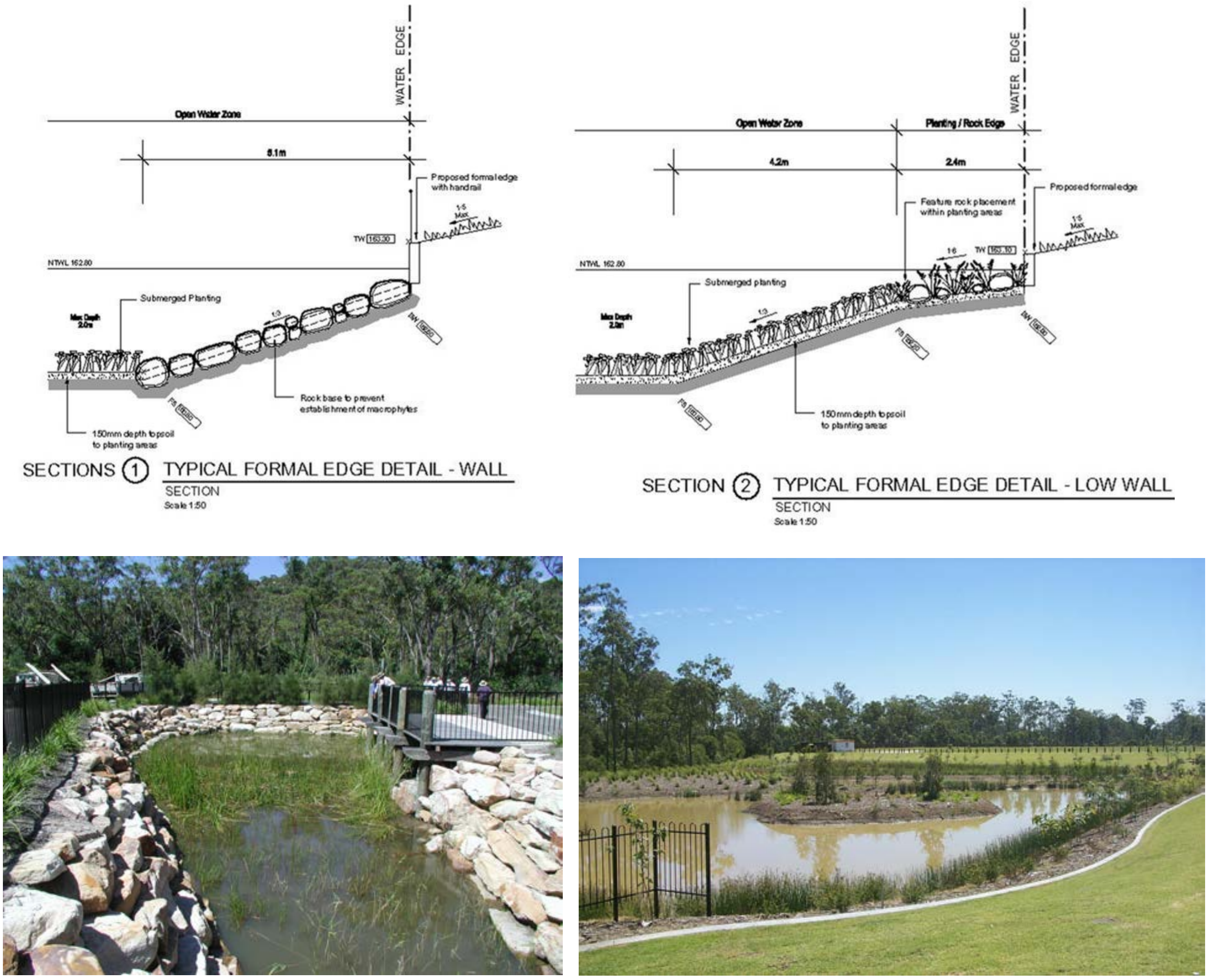


Figure 3.5. Illustration of Hard Edge Treatment for Open Waterbodies (GbLA, 2004)

3.3.4 Hydraulic Structures

Hydraulic structures are required at the inlet and outlet of a sedimentation basin. Their function is essentially one of conveyance of flow with provisions for (i) energy dissipation at the inlet structure(s), (ii) extended detention (if appropriate) at the outlet, and (iii) overflow pathway for above design conditions.

3.3.4.1 Inlet Structure

Sedimentation basins are generally configured so that flows from the upstream reticulated system discharge directly into the basin. The inlet structure must therefore have the capacity to adequately convey these flows. It is necessary to ensure that at the point of discharge, energy is adequately dissipated to minimise localised scouring.

There are many methods and designs of energy dissipating and scour reducing structures and as such they will not be covered in detail in this document. Local authorities may have standard drawings of preferred methods of velocity reduction structures and scour protection systems. Additionally see the resource appendices for further references.

Litter control is also normally required at an inlet structure and it is generally recommended that some form of gross pollutant trap (GPT) be installed as part of an inlet structure. The provision of a GPT will depend on catchment activities as well as any upstream measures in place. There are a number of proprietary products for removing gross pollutants and these are discussed in Chapter 7 in Australian Runoff Quality (Engineers Australia, 2006) and additional references are provided in the resource appendices.

3.3.4.2 Outlet Structure

An outlet structure of a sedimentation basin can be configured in many ways and is dependent on the specified operation of the system (ie. whether designed as a "stand-alone" sedimentation basin for managing construction site runoff or as part of a wetland system).

The outlet structure generally consists of an outlet pit and discharge control structure to control the rate of discharge from the basin under normal operation. During events that exceed the normal design flow, the discharge control structure must have adequate capacity to convey the design operation flow.

Structures for construction sites –

As a sedimentation basin for managing runoff from a construction site, landscape amenity is not an important design outcome. Therefore, floating discharge control structures, as shown in Figure 3.6, are considered the most effective outlets for construction sedimentation basins. They draw flows from the surface, which generally have the lowest suspended sediment concentrations. The discharge control structure consists of one or more slotted pipes mounted with floats to enable them to rise with the progressive filling of the basin as shown in Figure 3.6. Discharge from the basin is maintained at a relatively constant rate independent of the depth of water in the basin.

Permanent applications –

A weir is a more appropriate outlet structure for permanent sedimentation basins, and those that also serve as a landscape element. Where possible, a narrow weir structure (see Figure 3.6) should be adopted to promote a larger extended detention depth range while ensuring adequate capacity to convey the design discharge.

Figure 3.6. Sedimentation Basin Outlet Structures (L) a floating skimmer and (R) a narrow weir

Design considerations –

An outlet pit and associated discharge control structure should be designed with the following considerations in mind:

- Ensure that the crest of the pit is set at or above the permanent pool level of the sedimentation basin
- Ensure that the dimension of the pit provides discharge capacity that is greater than or equal to the discharge capacity of the inlet structure
- Discharge capacity does not exceed that of the downstream infrastructure
- Protection is provided against blockage by flood debris
- Maintenance is simple to undertake and suitable provisions are made for access

Figure 3.7 summarises the design elements of the various components of a sedimentation basin.

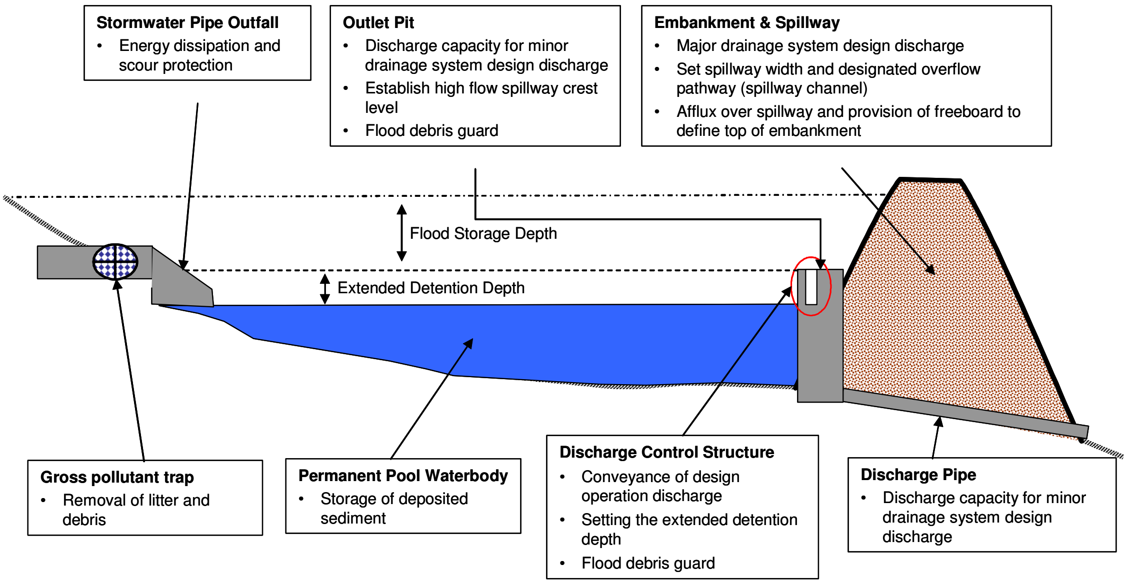


Figure 3.7. Overview of Design Elements of a Sedimentation Basin and Main Design Considerations

Hydraulic Design of the Outlet Structure

An outlet structure is sized with a discharge capacity equal to or greater than the minor drainage system (e.g. 5 year ARI). The two flow conditions that must be considered when calculating the dimensions of an outlet pit are weir and orifice flow (Equations 3.5 and 3.6)

A blockage factor is also employed to account for any blockage of the structure by debris. It is generally considered that a blockage factor of 50% reasonably represents real world conditions.

1. **Weir flow condition** – *when free overall conditions occur over the pit (usually when the extended detention storage of the retarding basin is not fully engaged),* ie.

$$P = \frac{Q_{des}}{B \cdot C_w \cdot H^{1.5}}$$

Equation 3.5

P	=	Perimeter of the outlet pit
B	=	Blockage factor (0.5)
H	=	Depth of water above the crest of the outlet pit
Q_{des}	=	Design discharge (m^3/s)
C_w	=	weir coefficient (1.7)

2. **Orifice flow conditions** – *when the inlet pit is completely submerged (corresponding to conditions associated with larger flood events*), ie.

$$A_o = \frac{Q_{des}}{B \cdot C_d \sqrt{2gH}}$$

Equation 3.6

C_d	=	Orifice Discharge Coefficient (0.6)
B	=	Blockage factor (0.5)
H	=	Depth of water above the centroid of the orifice (m)
A_o	=	Orifice area (m^2)
Q_{des}	=	Design discharge (m^3/s)

It is important that an outlet pit is prevented from blockage by debris. Design consideration needs to include means of preventing blockage of the outlet structure.

Examples of this are shown below.



__Discharge Control Structure__

Three types of discharge control structures can be used, ie.

1. Overflow weir – the length of the weir computed with the weir flow equation (Equation 3.5) but checked to ensure that there is adequate capacity when the structure operates under submerged conditions using the orifice flow equation (Equation 3.6).
2. Riser outlet – a vertical pipe with orifices located along the length of the pipe. The placement of outlet orifices and determining their appropriate diameters is designed iteratively by varying outlet diameters and levels, using the orifice equation (Equation 3.6) applied over discrete depths along the length of a riser up to the maximum detention depth. This can be performed with a spreadsheet as illustrated in the worked example. (see Chapter 8 for more discussion.)
3. Floating slotted pipe – the size and number of slots required to pass the operation design flow can be computed using the orifice flow equation (Equation 3.6).

With riser-type discharge control structures, an outlet orifice is likely to be small and it is important that these are prevented from clogging by debris. Some form of debris guard is recommended as illustrated in the images below.



3.3.5 Overflow Structure

The provision of a high-flow overflow structure is an essential design element. An overflow structure is normally a weir spillway structure. The required width of the spillway can be computed using the weir flow equation (Equation 3.5) with the design discharge being selected according to discussion in Section 3.3.1.1.

Figure 3.8. Overflow Structure of a Sedimentation Basin

3.3.6 Vegetation specification

Vegetation planted along the littoral zone of a sedimentation basin serve the primary function of inhibiting public access to the open waterbody and preventing edge erosion. Terrestrial planting beyond the littoral zone may also be recommended to screen areas and provide an access barrier to uncontrolled areas of the stormwater treatment system.

Appendix 'B' provides a list of suggested plant species suitable for sedimentation basin littoral zones in Tasmania.

3.3.7 Design calculation summary

The table on the following page provides a design calculation summary sheet for the key design elements of a sedimentation basin to aid the design process.

Sedimentation Basin — CALCULATION CHECKSHEET

	CALCULATION TASK	OUTCOME		CHECK
1	**Identify design criteria**			
	Design ARI Flow for inlet hydraulic structures		year	
	Design ARI Flow for outlet hydraulic structures		year	
	Design ARI for overflow hydraulic structures		year	☐
2	**Catchment characteristics**			
	Residential		Ha	
	Commercial		Ha	
	Roads		Ha	
	Fraction impervious			
	Residential			
	Commercial			
	Roads			☐
3	**Estimate design flow rates**			
	Time of concentration			
	estimate from flow path length and velocities		minutes	☐
	Identify rainfall intensities			
	station used for IFD data:			
	Design Rainfall Intensity for inlet structure(s)		mm/hr	☐
	Design runoff coefficient			
	inlet structure(s)			☐
	Peak design flows			
	Inlet structure(s)		m^3/s	
	Outlet structure(s)		m^3/s	
	Overflow structure(s)		m^3/s	☐
4	**Basin Dimension and Layout**			
	Area of sedimentation basin		m^2	
	Aspect Ratio		L:W	
	Hydraulic Efficiency			
	Depth of permanent pool		m	
	Permanent Pool Volume		m^3	
	Cross Section Batter Slope		V:H	☐
5	**Basin Performance**			
	Capture efficiency (of 125 µm sediment)		%	
	Sediment Cleanout Frequency		years	☐
6	**Hydrualic Structures**			
	Inlet Structure			
	Provision of energy dissipation			
	Outlet Structure			
	Pit dimension		L x B	
	or		mm diam	
	Discharge capacity of outlet		m^3/s	
	Provision of debris trap			☐
	Discharge Pipe			
	Discharge Capacity of Discharge Pipe		m^3/s	☐
7	**Spillway**			
	Discharge Capacity of Spillway		m^3/s	
	Afflux		m	
	Freeboard to top of embankment		m	☐

3.4 Checking tools

This section provides a number of checking aids for designers and referral authorities. In addition, advice on construction techniques and lessons learnt from building sediment basins are provided.

Checklists are provided for:

- Design assessments
- Construction (during and post)
- Operation and maintenance inspections
- Asset transfer (following defects period).

3.4.1 Design assessment checklist

The checklist below presents the key design features that should be reviewed when assessing a design of a sediment basin either for temporary or permanent use. These considerations include configuration, safety, maintenance and operational issues that should be addressed during the design phase.

Where an item results in an "N" when reviewing the design, referral should be made back to the design procedure to determine the impact of the omission or error.

In addition to the checklist, a proposed design should have all necessary permits for its installations. The referral agency should ensure that all relevant permits are in place. These can include permits to clear vegetation, to dredge, create a waterbody, divert flows or disturb fish or platypus habitat.

Land ownership and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the asset transfer checklist (see 3.4.4).

Sediment Basin Design Assessment Checklist					
Basin Location:					
Hydraulics	Minor Flood: (m^3/s)		Major Flood: (m^3/s)		
Area	Catchment Area (ha):		Basin Area (ha)		
Treatment				Y	N
Treatment performance verified from curves?					
Basin Configuration				Y	N
Inlet pipe/structure sufficient for maximum design flow (minor and/or major flood event)?					
Scour protection provided at inlet?					
Basin capacity sufficient for maintenance period >=5 years?					
Configuration of basin (aspect, depth and flows) allows settling of particles >125µm?					
Maintenance access allowed for into base of sediment basin?					
Public access to inlet zone prevented through vegetation or other means?					
Gross pollutant protection measures provided on inlet structures?					
Freeboard provided above extended detention depth?					
Batter slopes shallow or safety bench provided in case of accidental entry into basin?					
Hydraulic Structures				Y	N
Outlet perimeter >= design discharge of outlet pipe?					
Outlet configuration suitable for basin type (eg riser for construction sediment, weir for wetland pretreatment etc)?					
Riser diameter sufficient to convey Q_1 flows when operating as a “glory hole” spillway?					
Maintenance drain provided?					
Discharge pipe from has sufficient capacity to convey the maintenance drain flows or Q_1 flows (whichever is higher)?					
Protection against clogging of orifice provided on outlet structure?					

3.4.2 Construction advice

This section provides general advice for the construction of sedimentation basins. It is based on observations from actual construction projects around Australia.

Building phase damage

It is important to have protection from upstream flows during construction of a sediment basin. A mechanism to divert flows around a construction site and to provide protection from litter and debris is required.

High flow contingencies

Contingencies to manage risks associated with flood events during construction are required. All machinery should be stored above acceptable flood levels and the site stabilised as best as possible at the end of each day as well as plans for de-watering following storms made.

Maintenance access

An important component of a sediment basin is accessibility for maintenance. Should excavators be capable of reaching all parts of the basin an access track may not be required to the base of the inlet zone, however an access track around the perimeter of the basin would be required. If sediment collection is by using earthmoving equipment, then a stable ramp will be required into the base of the inlet zone (maximum slope 1:10).

Solid base

To aid maintenance it is recommended to construct the inlet zone either with a hard (ie rock or concrete) bottom or a distinct sand layer. These serve an important role for determining the levels that excavation should extend to during sediment removal (i.e. how deep to dig) for either systems cleaned from the banks or directly accessed. Hard bases are also important if maintenance is by driving into the basin.

De-watering removed sediments

An area should be constructed that allows for de-watering of removed sediments from a sediment basin. This allows the removed sediments to be transported as ‘dry’ material and can greatly reduce disposal costs compared to liquid wastes. This area should be located such that water from the material drains back into the basin. Material should be allowed to drain for a minimum of overnight before disposal.

Inlet checks

It is good practice to check the operation of inlet erosion protection measures following the first few rainfall events. It is important to check for these early in the systems life, to avoid continuing problems. Should problems occur in these events the erosion protection should be enhanced.

3.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Sediment basin

INSPECTED BY:	
DATE:	
TIME:	
WEATHER:	
CONTACT DURING VISIT:	

SITE: ____________________

CONSTRUCTED BY: ____________________

DURING CONSTRUCTION									
Items inspected	Checked		Satisfactory	Unsatisfactory		Checked		Satisfactory	Unsatisfactory
Preliminary works	Y	N			**Structural components**	Y	N		
1. Erosion and sediment control plan adopted					16. Location and levels of outlet as designed				
					17. Safety protection provided				
2. Limit public access					18. Pipe joints and connections as designed				
3. Location same as plans					19. Concrete and reinforcement as designed				
4. Site protection from existing flows					20. Inlets appropriately installed				
Earthworks					21. Inlet energy dissipation installed				
5. Integrity of banks					22. No seepage through banks				
6. Batter slopes as plans					23. Ensure spillway is level				
7. Impermeable (solid) base installed					24. Provision of maintenance drain				
8. Maintenance access (eg. ramp) installed					25. Collar installed on pipes				
9. Compaction process as designed					**Vegetation**				
10. Levels of base, banks and spillway as designed					25. Stablisation immediately following earthworks				
					26. Planting as designed (species and densities)				
					27. Weed removal before stabilisation				
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of banks				
2. Confirm structural element sizes					7. Inlet erosion protection working				
3. Check batter slopes					8. Maintenance access provided				
4. Vegetation as designed					9. Construction generated sediment removed				
5. Draining area for maintenance provided									

COMMENTS ON INSPECTION

ACTIONS REQUIRED

1.
2.
3.
4.
5.
6.

3.4.4 Asset transfer checklist

Asset Handover Checklist		
Asset Location:		
Construction by:		
Defects and Liability Period		
Treatment	**Y**	**N**
System appears to be working as designed visually?		
No obvious signs of under-performance?		
Maintenance	**Y**	**N**
Maintenance plans provided for each asset?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
Asset inspected for defects?		
Asset Information	**Y**	**N**
Design Assessment Checklist provided?		
As constructed plans provided?		
Copies of all required permits (both construction and operational) submitted?		
Proprietary information provided (if applicable)?		
Digital files (eg drawings, survey, models) provided?		
Asset listed on asset register or database?		

3.5 Maintenance requirements

Sediment basins treat runoff by slowing flow velocities and promoting settlement of coarse to medium sized sediments. Maintenance revolves around ensuring inlet erosion protection is operating as designed, monitoring sediment accumulation and ensuring that the outlet is not blocked with debris.

Inspections of the inlet configuration following storm events should be made soon after construction to check for erosion. In addition, regular checks of sediment build up will be required as sediment loads from developing catchments or construction sites vary enormously. ***The basins should be cleaned out if more than half full of accumulated sediment.***

Similar to other types of practices, debris removal is an ongoing maintenance function. Debris, if not removed, can block inlets or outlets, and can be unsightly if located in a visible location. Inspection and removal of debris should be done regularly, but debris should be removed whenever it is observed on the site.

3.5.1 Operation & maintenance inspection form

The form below should be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

Sediment Basin Maintenance Checklist			
Inspection Frequency:	3 monthly	Date of Visit:	
Location:			
Description:			
Site Visit by:			

Inspection Items	**Y**	**N**	**Action Required (details)**
Litter within inlet or open water zones?			
Sediment within inlet zone requires removal (record depth, remove if >50%)?			
Overflow structure integrity satisfactory?			
Evidence of dumping (building waste, oils etc)?			
Terrestrial vegetation condition satisfactory (density, weeds etc)?			
Weeds require removal from within basin?			
Settling or erosion of bunds/batters present?			
Damage/vandalism to structures present?			
Outlet structure free of debris?			
Maintenance drain operational (check)?			
Comments:			

3.6 Sedimentation basin worked example

3.6.1 Worked example introduction

A sedimentation basin and wetland system is proposed to treat runoff from a highway. This worked example focuses on the sediment basin (inlet zone) component of the system.

The site is triangular in shape with a surface area of 500 m^2 as shown in Figure 3.9. Road runoff is conveyed by stormwater pipes (design capacity up to the 100 year ARI event) and there are two highway outfall pipes that discharge to the two top apexes of the site. Each outfall services about 1km of the highway with the total contributing area of 4Ha (90% impervious) to each outfall. The site of the site has a fall of approximately 2m (from 5m to 3m AHD) towards a degraded watercourse.

Site constraints limit the size available for the stormwater treatment system therefore it is important to ensure that the size of the inlet zones (ie. sedimentation basins) are not compromised, to ensure that larger sediments are effectively trapped and prevented from smothering the planted zone (thereby creating future maintenance problems).

The consequence of this action is the reduction in the overall hydrologic effectiveness of the system (i.e. the proportion of mean annual runoff subjected to the full wetland treatment), but not its functional integrity.

All stormwater runoff will be subjected to primary treatment, by sedimentation of coarse to medium size sediment. The inlet zone will operate under by-pass conditions more often owing to a smaller macrophyte zone in this case.

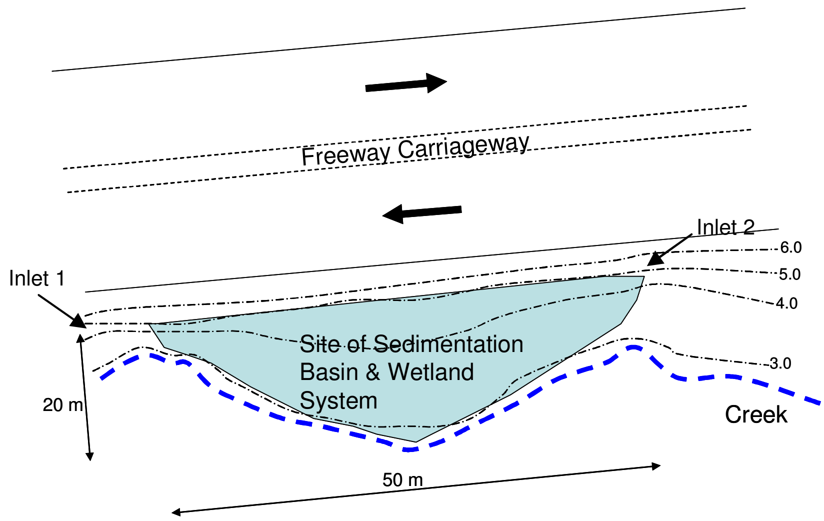


Figure 3.9Layout of Proposed site for Sedimentation Basin

3.6.1.1 Design Objectives

As the sedimentation basins form part of a treatment train (with a small macrophyte wetland), sizing to meet the overall best practice stormwater quality objectives does not apply. Instead, the design requirements of the sedimentation basin system are to:

- Promote sedimentation of particles larger than 125µm (0.125mm) with a 90% capture efficiency for flows up to the 1-year ARI (un-attenuated) peak discharge.
- Provide for connection to the downstream macrophyte wetland zone with discharge capacity corresponding to the 1-year ARI (un-attenuated) peak discharge.
- Provide for by-pass operation when the inundation of the downstream macrophyte wetland zone reaches the design maximum extended detention depth with a discharge capacity corresponding to the 100-year ARI peak discharge.

Analyses to be undertaken during the detailed design phase include the following:

- Sizing the sedimentation basin (depth and area) using sedimentation theory (an extended detention depth of **0.25m** above the permanent pool level has been nominated to match the proposed maximum water level of the downstream macrophyte zone)
- Configure the layout of the basin such that the system *hydraulic efficiency* (see figure 3.3) can be optimised
- Design of the inlet structure to provide for energy dissipation of inflows up to the 100 year ARI peak discharge
- Design of by-pass structure to provide for flow by-pass of downstream wetland for events up to the 100 year ARI event
- Design of the basin outlet structure connecting to the macrophyte wetland zone, including debris trap.

In addition, landscape design will be required and this will include the following:

- Littoral zone vegetation
- Terrestrial vegetation

3.6.2 Estimating Design Flows

The procedures in Australian Rainfall and Runoff (ARR) are used to estimate the design flows. The site has two contributing catchments, each catchment is 4 Ha in area, 1km long (along the freeway) and is drained by culverts.

See Appendix E Design Flows – t_c for a discussion on methodology for calculation of time of concentration.

Step 1 – Calculate the time of concentration.

Velocity within the pipes is assumed to be 1m/s for the purposes of estimating the time of concentration (t_c).

t_c=1000m/1m/s = 1000s = 17 minutes

Rainfall Intensities for the area of study (for the 1, 10 and 100 year average recurrence intervals) are estimated using ARR (1998) with a time of concentration of = 17 minutes and are:

I_1 = 27 mm/hr*

I_{10} = 56 mm/hr*

I_{100} = 95 mm/hr*

* These figures are for the worked example only. The appropriate region and corresponding rainfall intensities must be selected for each individual project.

Step 2 – Calculate design runoff coefficients (using the method outlined in Australian Rainfall and Runoff Book VIII (Engineers Australia, 2003)).

Where – Fraction impervious (f) = 0.9

Rainfall intensity (${}^{10}I_1$) = 26.4mm/hr (from the relevant IFD chart)

Calculate C^1_{10} (pervious runoff coefficient)

$C^1_{10} = 0.1 + 0.0133\ ({}^{10}I_1 - 25) = 0.12$

Calculate C_{10} (10 year ARI runoff coefficient)

$C_{10} = 0.9f + C^1_{10}\ (1-f)$

$C_{10} = 0.82$

Step 3 – Convert C_{10} to values for C_1 and C_{100}

Where – $C_y = F_y \times C_{10}$

Runoff coefficients as per Table 1.6 Book VIII ARR 1998

ARI (years)	Frequency Factor, *Fy*	Runoff Coefficient, *Cy*
1	0.8	0.66
10	1.0	0.82
100	1.2	0.98

Step 4 – Calculate peak design flow (calculated using the Rational Method).

$$Q = \frac{CIA}{360}$$

Where – C is the runoff coefficient (C_1, C_{10} and C_{100})

I is the design rainfall intensity mm/hr (I_1 , I_{10} and I_{100})

A is the catchment area (Ha)

Q_1 = $0.20 m^3/s$

Q_{10} = $0.51 m^3/s$

Q_{100} = $1.0\ m^3/s$

Operation Design Discharge = $0.20\ m^3/s$

Design discharge for connection to microphyte zone = $0.20\ m^3/s$

Spillway Design Discharge = $1.0\ m^3/s$

3.6.3 Size and Shape of Sedimentation Basin

The inlet zone is to be sized to remove at least 90% of 125μm particles for the peak 1-year flow.

Pollutant removal is estimated using the following:

$$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)}\right]^{-n}$$

An aspect ratio of 1 (w) to 4 (L) is adopted based on the available space (Figure 3.9). Using Equation 3.2, the hydraulic efficiency (λ) is estimated to be 0.4. This value is less than desirable, however, site constraints prevent any other configuration. The turbulence factor (n) is computed from Equation 3.2

Hydraulic efficiency (λ) = 0.4

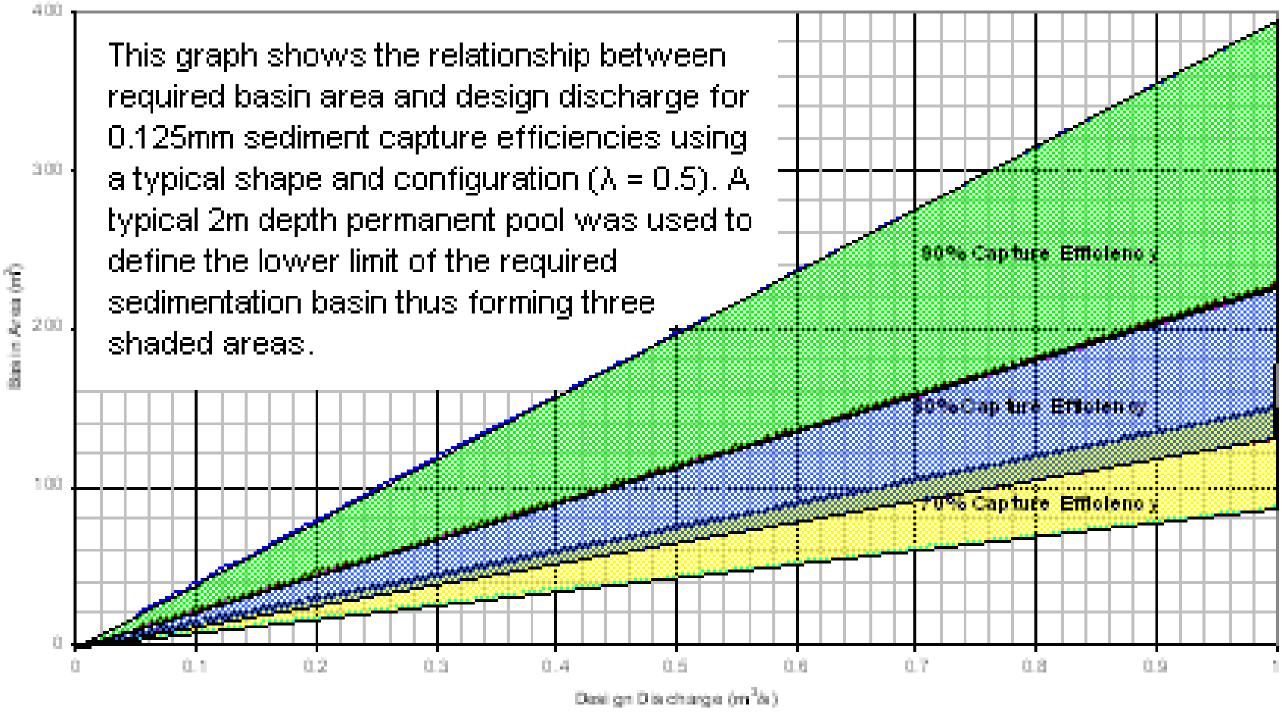
Turbulence factor (n) = 1.67

The proposed extended detention depth of the basin is 0.25 (as outlined in Section 3.6.1) and a notional permanent pool depth of 2 m has been adopted, ie.

$d_p = 2.0$ m | Vs = 0.011m/s for 125μm particles

$d^* = 1.0$m | Q = Design operation flow rate = $0.20\ m^3/s$

$d_e = 0.25$m



From above, the required sedimentation basin area to achieve target sediment (125 μm) capture efficiency of 90% is 50 m². With a W to L ratio of 1:4, the notional dimensions of the basin are 3.5 m x 14 m. This figure correlates with the graph above.

The available sediment storage is 50 x 2 = 100 m³. Cleanout is to be scheduled when the storage is half full, therefore the available sediment storage prior to clean out is 50m³.

The required volume of sediment storage to ensure cleaning is not required more frequently than every five years is estimated using Equation 3.4 (using a sediment discharge rate of 1.6 m³/Ha/yr).

Require storage ($S_{t)}$) $= C_a \times R \times L_o \times F_r$

$= 4 \times 0.9 \times 1.6 \times 5 = 29$ m³

Available storage volume is 50m³, therefore it is OK.

The required clean out frequency is estimated to be (by rearranging Equation 3.4):

Frequency of basin desilting $= \dfrac{0.5 \times 100}{1.6 \times 4 \times 0.9} = 8.6$ years > 5 years ➔ OK

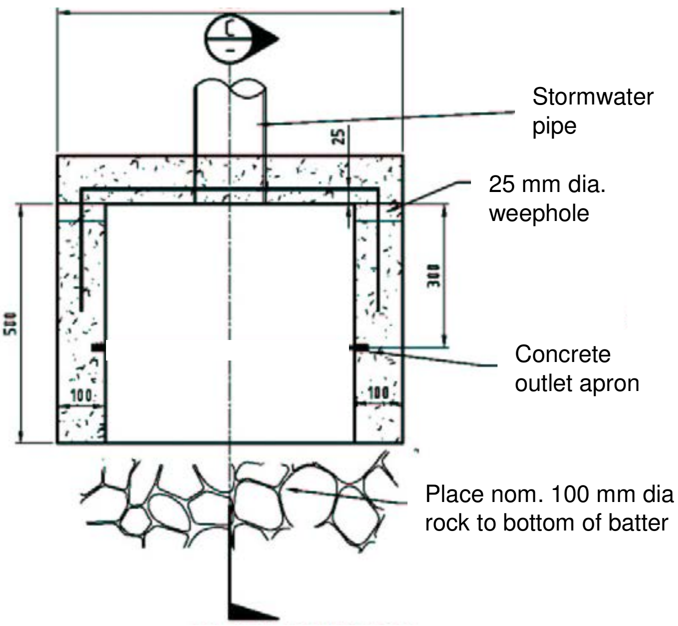
Open Water Area = 50 m²
Width = 3.5 m; Length = 12 m
Depth of Permanent Pool (d_p) = 2.0 m
Depth of Extended Detention (d_e) = 0.25m

3.6.4 Hydraulic Structure design

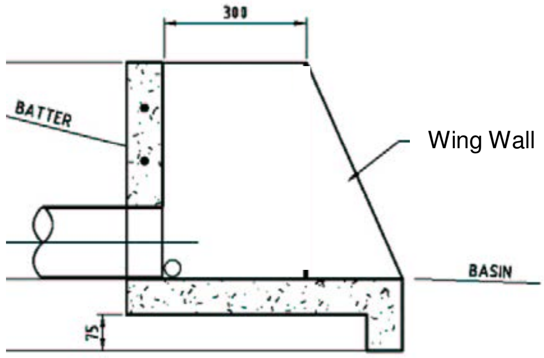
3.6.4.1 Inlet Structure

To prevent scour of deposited sediments from flows in the inlet pipes, it is necessary to limit velocities adjacent to the inlet to below 1m/s. Culvert invert assumed to be RL 3.5m AHD

Rock beaching will be required in this area to ensure that excessive scour does not occur.



Plan Inlet Structure



Section Inlet Structure

Provide energy dissipation and erosion protection in the form of rock beaching at the inlet structure; $Q_{des} = 0.51\ m^3/s$.

3.6.4.2 Outlet Structure

The outlet structure is to consist of an outlet pit with the top of the pit set at the permanent pool level, creating a permanent pool depth of 2 m. The dimension of the pit should ensure adequate discharge capacity to discharge the design flow for the connection to the macrophyte zone (ie. 1 year ARI peak discharge of 0.2 m^3/s).

According to section 3.3.4.2, two possible flow conditions need to be checked, i.e. weir flow conditions (with extended detention of 0.25 m) and orifice flow conditions.

Weir Flow Conditions

From Equation 3.5

, the required perimeter of the outlet pit to pass 0.2 m^3/s with an afflux of 0.25 m can be calculated:-

$$P = \frac{Q_{des}}{B \cdot C_w \cdot H^{1.5}} = \frac{0.2}{0.5 \cdot 1.7 \cdot 0.25^{1.5}} = 1.88 \text{ m}$$

Orifice Flow Conditions

From Equation 3.6, the required area of the outlet pit can be calculated as follows:-

$$A_o = \frac{Q_{des}}{B \cdot C_d \sqrt{2gH}} = \frac{0.2}{0.5 \cdot 0.6\sqrt{2g(0.25)}} = 0.30 \text{ m}^2$$

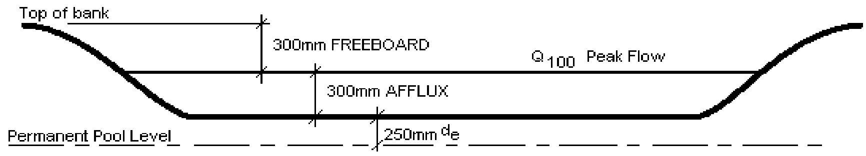
Adopt 600 x 600 mm pit: Area = 0.36 m^2; Perimeter = 2.4 m; Q_{cap} = 0.24 m^3/s ➔ OK

The top of the pit should be fitted with a standard grating to prevent flood debris from blocking the outlet pit.

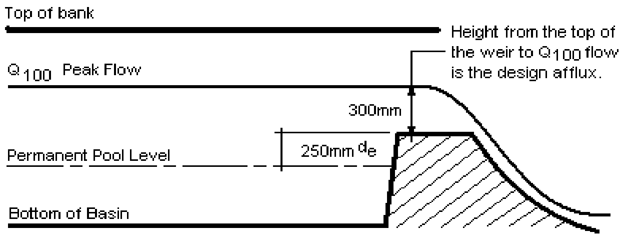
Outlet Pit = 600 x 600 mm diameter with standard grating

3.6.4.3 Overflow Structure

The overflow structure must be capable of conveying the Q_{100} peak flow. A weir with a crest elevation set at 0.25 m (ie. d_e) above the permanent pool level, and an afflux of 300mm has been adopted in order to calculate the required length of the spillway weir.



LONG SECTION OF WEIR



CROSS SECTION OF WEIR

It is common practice to allow for 300 mm of freeboard above the afflux level when setting the top of embankment elevation. This value was adopted as a trade off between the bank height and the width of the weir. A bank height of 600mm (300mm afflux and 300mm freeboard) above the normal water level was deemed acceptable. The length is calculated using the weir flow equation with a weir coefficient of 1.7, i.e.

$$L = \frac{Q_{des}}{C_w \cdot H^{1.5}} = \frac{1.0}{1.7 \cdot 0.3^{1.5}} = 3.6 \text{ m}$$

Bypass weir is located adjacent to inflow culvert to minimise risk of sediment scour.

Spillway length = 3.6 m set at 0.25 m above permanent pool level Top of embankment set at 0.6 m above the permanent pool level.

3.6.4.4 Discharge to Macrophyte Zone

A culvert connection between the sedimentation basin (inlet zone) and macrophyte zone will also need to be designed with the design criterion that the culvert will need to have adequate capacity to pass the 1 year ARI peak discharge when the water level in macrophyte zone is at its permanent pool level. This will also provide the flow control into the wetland.

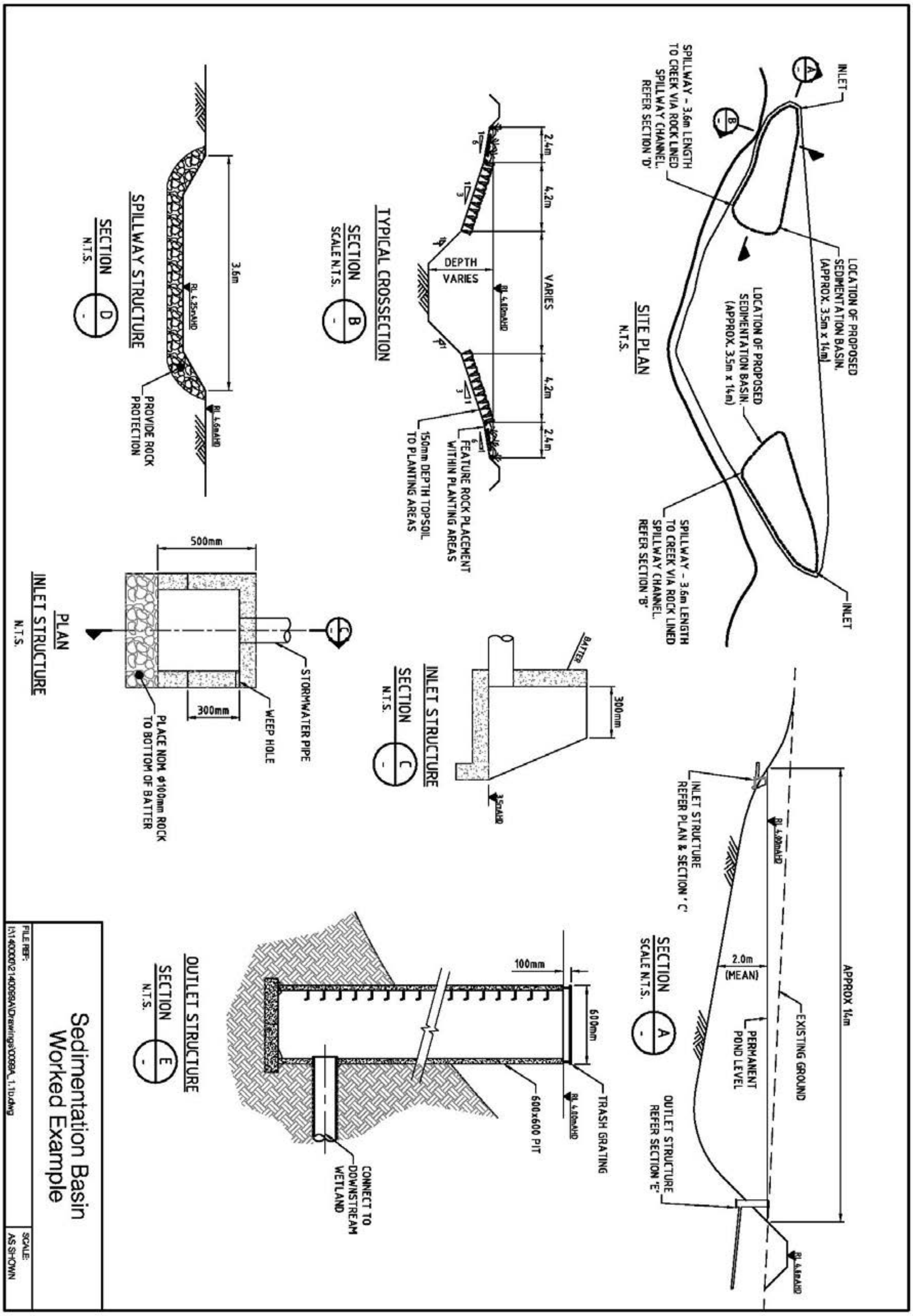
The design calculation and configuration of this connection is described in Chapter 8 on constructed wetland design.

Sedimentation Basin

CALCULATION CHECKSHEET

	CALCULATION TASK		OUTCOME		CHECK
1	**Identify design criteria**				✓
	Design ARI Flow for inlet hydraulic structures		10	year	
	Design ARI Flow for outlet hydraulic structures		1	year	
	Design ARI for overflow hydraulic structures		100	year	
2	**Catchment characteristics**				✓
		Residential	0	Ha	
		Commercial	0	Ha	
		Roads	4	Ha	
	Fraction impervious				✓
		Residential	N/A		
		Commercial	N/A		
		Roads	0.9		
3	**Estimate design flow rates**				
	Time of concentration				
	estimate from flow path length and velocities		17	minutes	✓
	Identify rainfall intensities				
		station used for IFD data:	Hobart		
		Design Rainfall Intensity for inlet structure(s)	27 to 56	mm/hr	✓
	Design runoff coefficient				
		inlet structure(s)	0.66 to 0.98		✓
	Peak design flows				✓
		Inlet structure(s)	0.51	m^3/s	
		Outlet structure(s)	0.20	m^3/s	
		Overflow structure(s)	1.00	m^3/s	
4	**Basin Dimension and Layout**				✓
		Area of sedimentation basin	50	m^2	
		Aspect Ratio	4(L):1(W)	L:W	
		Hydraulic Efficiency	0.4		
		Depth of permanent pool	2	m	
		Permanent Pool Volume	100	m^3	
		Cross Section Batter Slope	1(V):8(H)	V:H	
5	**Basin Performance**				✓
		Capture efficiency (of 125 µm sediment)	90	%	
		Sediment Cleanout Frequency	8.6	years	
6	**Hydraulic Structures**				
	Inlet Structure				✓
		Provision of energy dissipation	Y		
	Outlet Structure				✓
		Pit dimension	600 x 600	L x B	
		or		mm diam	
		Discharge capacity of outlet	0.21	m^3/s	
		Provision of debris trap	Y		
	Discharge Pipe				✓
		Discharge Capacity of Discharge Pipe	0.2	m^3/s	
7	**Spillway**				✓
		Discharge Capacity of Spillway	1	m^3/s	

3.6.5 Construction drawings



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Chapter 4 Bioretention Swales

Definition:

A bioretention swale (or biofiltration trench) is a bioretention system that is located within the base of a swale.

Purpose:

- To provide a conveyance function.
- Removal of fine and coarse sediments.
- Efficient removal of hydrocarbons and other soluble or fine particulate contaminants from biological uptake.
- To provide a low levels of extended detention.
- Provide flow retardation for frequent (low ARI) rainfall events.

Implementation considerations:

- Bioretention swales can form attractive streetscapes and provide landscape features in an urban development.
- Bioretention systems are well suited to a wide range of soil conditions including areas affected by soil salinity and saline groundwater as their operation is generally designed to minimise or eliminate the likelihood of stormwater exfiltration from the filtration trench to surrounding soils.
- Vegetation that grows in the filter media enhances its function by preventing erosion of the filter medium, continuously breaking up the soil through plant growth to prevent clogging of the system and providing biofilms on plant roots that pollutants can adsorb to. The type of vegetation varies depending on landscape requirements and climatic conditions.

Bioretention swales are commonly located in median strips of roads and carparks

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4.1 Introduction

Bioretention swales provide both stormwater treatment and conveyance functions where a bioretention system is installed in the base of a swale that is designed to convey minor floods. The swale component provides pretreatment of stormwater to remove coarse to medium sediments while the bioretention system removes finer particulates and associated contaminants. Figure 4.1 shows the layout of a bioretention swale.

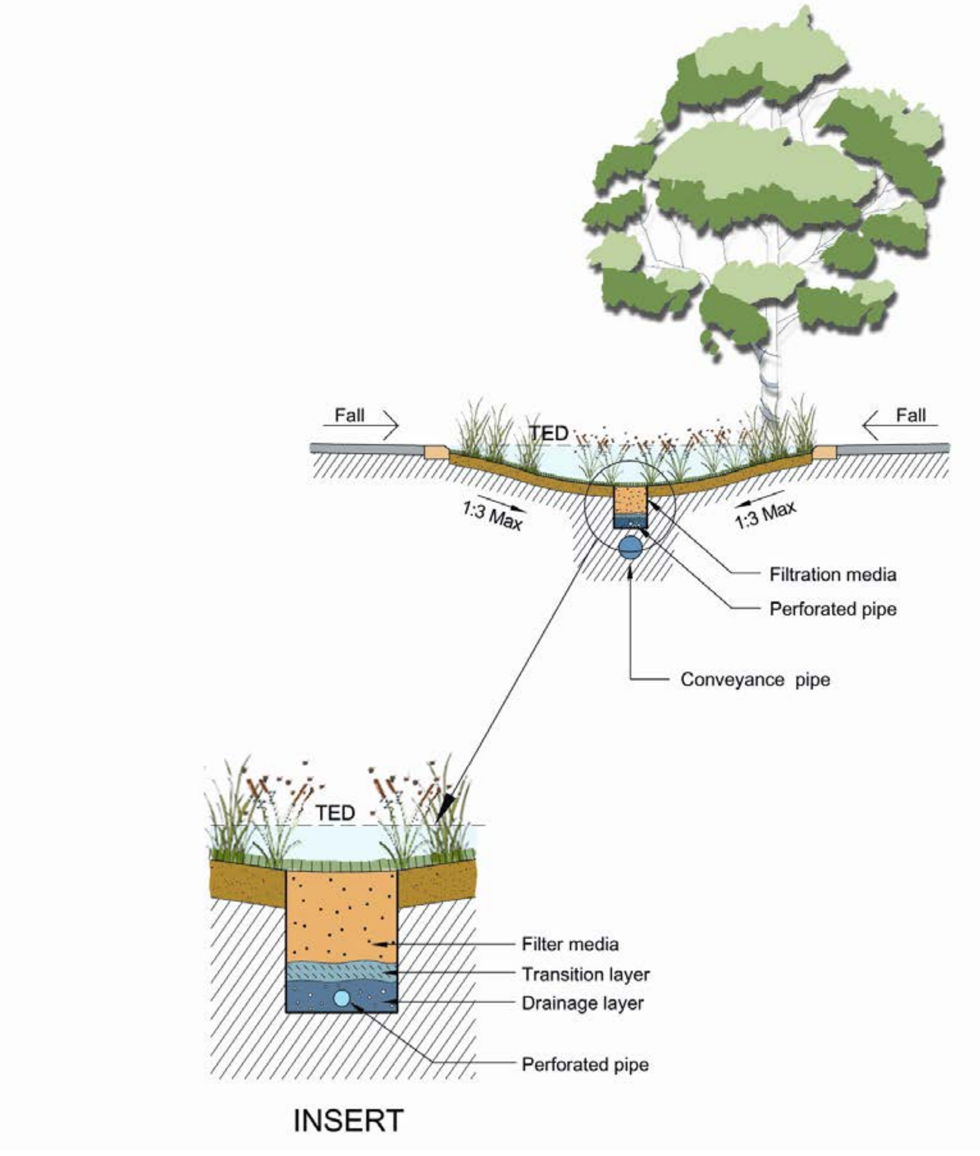


Figure 4.1. Bioretention swale as a centre road median

A bioretention system can be installed in part of a swale, or along the full length of a swale, depending on treatment requirements. Typically, these systems should be installed with slopes of between 1 and 4 %. Depending on the length of the swale and steepness of the terrain, check dams can be used to manage steep slopes and also to provide ponding over a bioretention surface. In this way increased volumes of runoff can be treated through a bioretention system prior to bypass.

Runoff can be directed into bioretention swales either through direct surface runoff (e.g. with flush kerbs) or from an outlet of a pipe system. In either case it is important to keep traffic away from the filter media as compaction can change the filter media functions substantially.

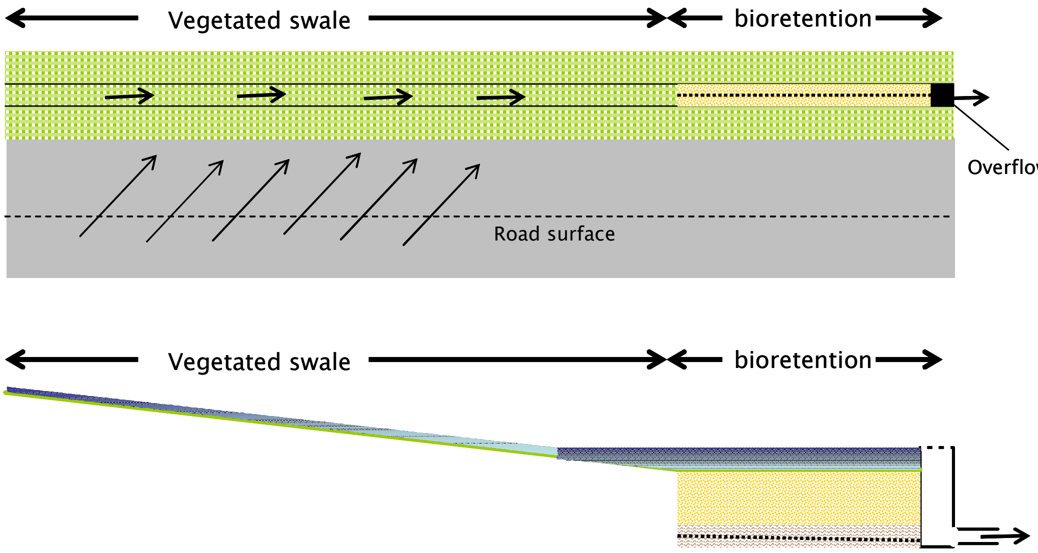


Figure 4.2. Bioretention swale example layout

When designing a system, separate calculations are performed to design the swale and the bioretention system, with iterations to ensure appropriate criteria are met in each section.

In many urban situations, the width available for a swale system will be fixed (as well as the longitudinal slope), therefore the length of the swale to safely convey a minor storm will also be fixed.

A common way to design these systems is as a series of discrete 'cells'. Each cell has an overflow pit that discharges flow to an underground pipe system (Figure 4.2 is an example of a 'cell'). Bioretention systems can then be installed directly upstream of the overflow pits. This also allows an area for ponding over the filtration media and the overflow pit provides a point of connection from the bioretention system's underdrain to the piped stormwater network.

With flood flows being conveyed along the bioretention surface, it is important to ensure that velocities are kept low to avoid scouring of collected pollutants and vegetation.

The treatment system operates by firstly filtering surface flows through surface vegetation and then percolating runoff through prescribed filtration media that provides treatment through fine filtration, extended detention and some biological uptake. They also provide flow retardation for frequent storm events and are particularly efficient at removing nutrients.

The bioretention systems can be designed to either encourage infiltration (where reducing volumes of stormwater runoff is important) or as conveyance systems that do not allow infiltration (where soils are not suitable for infiltration or in close proximity to surrounding structures).

Where bioretention systems are not intended to encourage infiltration they convey collected water to downstream waters (or collection systems for reuse) with any loss in runoff mainly attributed to maintaining soil moisture of the filter media itself (which is also the growing media for the vegetation).

Where bioretention systems perform pretreatment for infiltration, they are designed to facilitate infiltration by removing the collection system at the base of the filtration media allowing contact with surrounding soils.

Figure 4.3 shows a typical section of a swale bioretention combination.

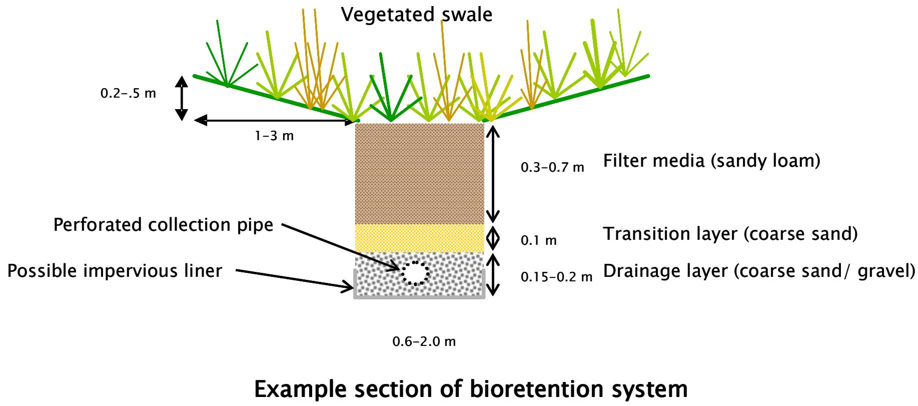


Figure 4.3. Typical section of a bioretention swale

Vegetation that grows in the filter media enhances its function by preventing erosion of the filter medium, continuously breaking up the soil through plant growth to prevent clogging of the system or caking at the soil surface and providing biofilms on plant roots that pollutants can adsorb to. The type of vegetation varies depending on landscaping requirements. Generally, the denser and higher the vegetation the better the filtration process.

Vegetation is critical to maintaining porosity of the filtration layer. Selection of an appropriate filtration media is a key issue that involves balancing sufficient hydraulic conductivity (ie. passing water through the filtration media as quickly as possible), stormwater detention for treatment and providing a suitable growing media to support vegetation growth (ie. retaining sufficient soil moisture and organic content). Typically a sandy loam type material is suitable, however soils can be tailored to a vegetation type.

As shown in Figure 4.3, a bioretention trench could consist of three layers. In addition to the filtration media, a drainage layer is required to convey treated water into the perforated underdrains. This material surrounds the perforated underdrainage pipes and can be either coarse sand (1 mm) or fine gravel (2-5 mm). Should fine gravel be used, it is advisable to install a transition layer of sand or a geotextile fabric (with a mesh size equivalent to sand size) to prevent any filtration media being washed into the perforated pipes.

Keeping traffic off bioretention swales is an important consideration in the design phase of such a system. Traffic can ruin the vegetation, create ruts that cause preferential flow paths that do not offer filtration and compact the filter media. Traffic control can be achieved by selecting vegetation that discourages the movement of traffic or by providing physical barriers to traffic movement. For example, barrier kerbs with breaks in them (to allow distributed water entry, albeit with reduced uniformity of flows compared with flush kerbs) or bollards along flush kerbs can be used to prevent vehicle movement onto swales.

The design process for a bioretention swale involves firstly designing the system for treatment and secondly ensuring the system can convey a minor flood.

Key design issues to be considered are:

Verifying size and configuration for treatment

Determine design capacity and treatment flows

Dimension the swale

Specify details of the filtration media

Above ground components:

- check velocities
- design of inlet zone and overflow pits
- check above design flow operation

Below ground components:

- prescribe soil media layer characteristics (filter, transition and drainage layers)
- underdrain design and capacity check
- check requirement for bioretention lining

Recommended plant species and planting densities

Provision for maintenance.

4.2 Verifying size for treatment

The curves below show pollutant removal performance expected for bioretention systems (either swales or basins) at varying depths of ponding. Average ponding depth should be used in design as average depth is likely to be less than the maximum depth.

These curves are based on the performance of a system in Hobart, the reference site and were derived using the Model for Urban Stormwater improvement Conceptualisation (MUSIC, eWater, 2009). To estimate the size of a bioretention swale anywhere in Tasmania, the required area for a system in Hobart, the reference site should be multiplied by the appropriate adjustment factor listed in Chapter 2. In preference to using the curves, local data should be used to model the specific treatment performance of the system.

The curves were derived assuming the systems receive direct runoff (i.e. no pretreatment) and have the following characteristics:

- Hydraulic conductivity of 36mm/hr

- Filtration media depth of 600 mm
- Filter media particle size (D_{50}) of 0.45 mm

These curves can be used to check the expected performance of the bioretention system for removal of TSS, TP and TN. The x-axis is the area of bioretention expressed as a percentage of the bioretention area of the *impervious* contributing catchment area.

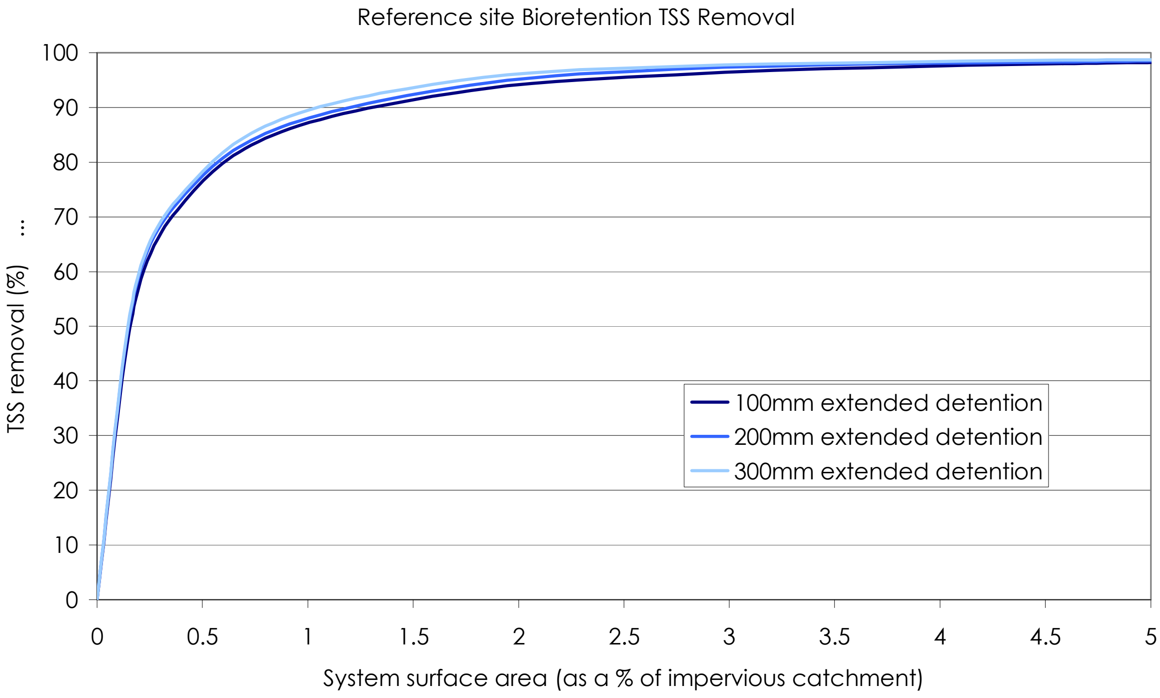


Figure 4.4. TSS removal in bioretention systems with varying extended detention

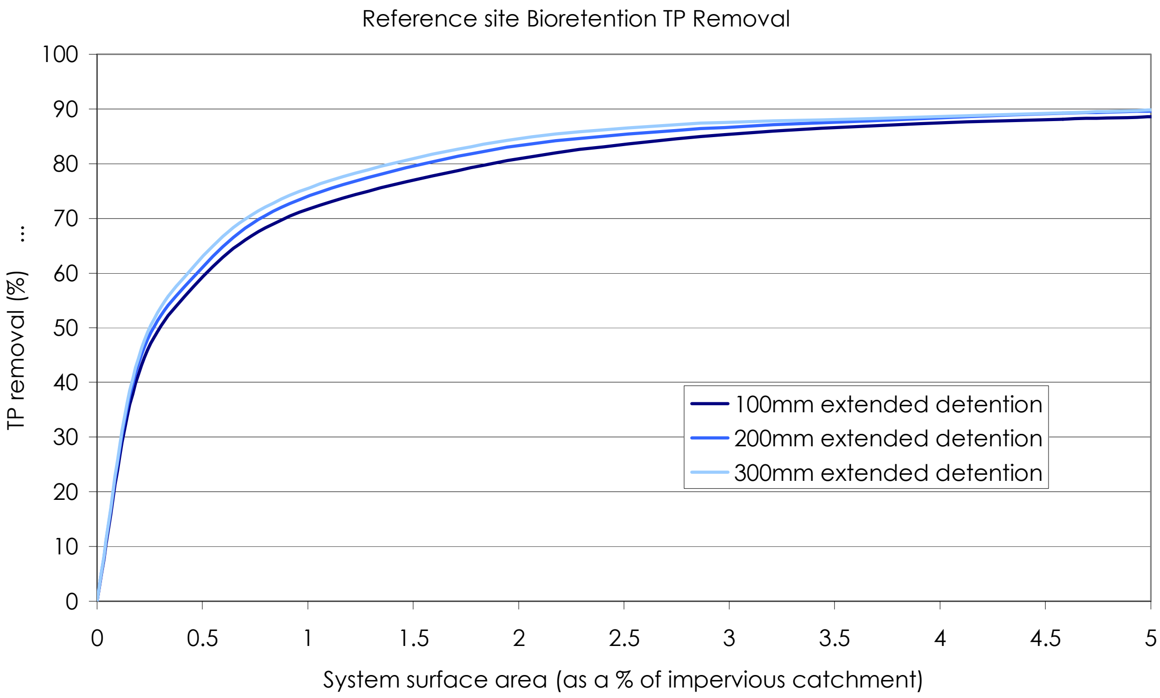


Figure 4.5. TP removal in bioretention systems with varying extended detention

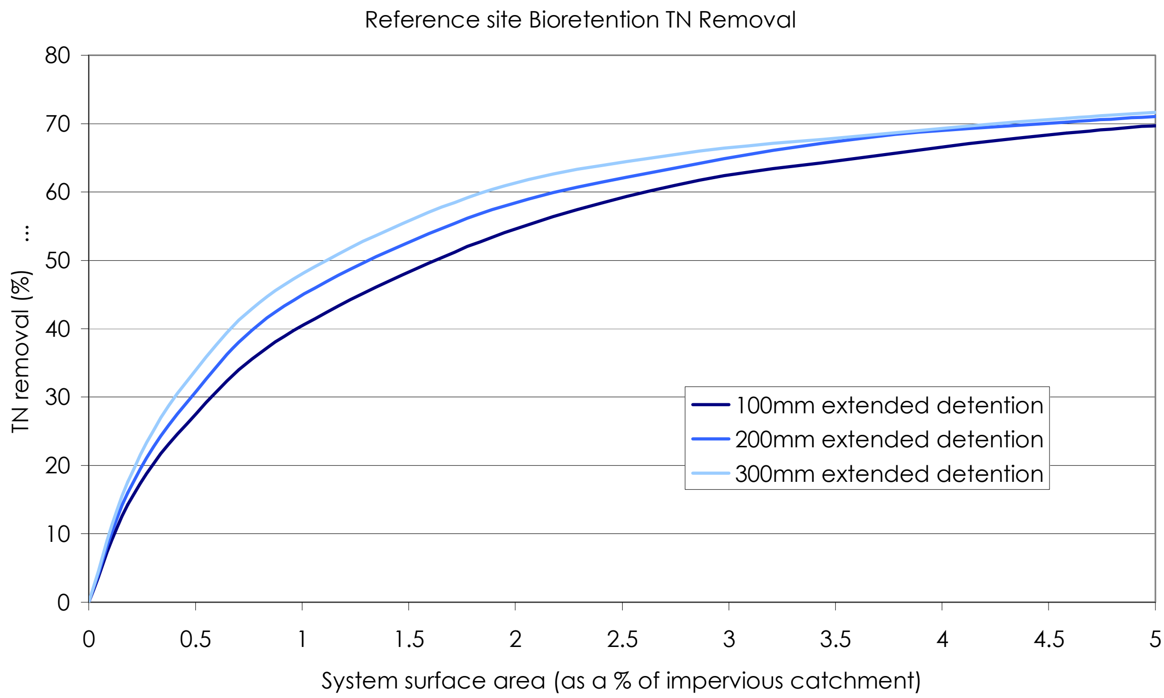


Figure 4.6. TN removal in bioretention systems with varying extended detention

4.3 Design procedure for bioretention swales

The following sections detail the design steps required for bioretention swales.

4.3.1 Estimating design flows

Three design flows are required for bioretention swales:

- Minor flood rates (typically 5-year ARI) to size the overflows to allow minor floods to be safely conveyed and not increase any flooding risk compared to conventional stormwater systems
- Major flood rates (typically 100 year ARI) to check that flow velocities are not too large in the bioretention system, which could potentially scour pollutants or damage vegetation
- Maximum infiltration rate through the filtration media to allow for the underdrainage to be sized, such that the underdrains will allow filter media to drain freely.

4.3.1.1 Minor and major flood estimation

A range of hydrologic methods can be applied to estimate design flows. With typical catchment areas being relatively small, the Rational Method Design Procedure is considered to be a suitable method for estimating design flows.

4.3.1.2 Maximum infiltration rate

The maximum infiltration rate represents the design flow for the underdrainage system (i.e. the slotted pipes at the base of the filter media). The capacity of the underdrains needs to be greater than the maximum infiltration rate to ensure the filter media drains freely and doesn't become a 'choke' in the system.

A maximum infiltration rate (Q_{max}) can be estimated by applying Darcy's equation:

$$Q_{\max} = k \cdot L \cdot W_{base} \cdot \frac{h_{\max} + d}{d}$$

Equation 4.1

where k is the hydraulic conductivity of the soil filter (m/s)

W_{base} is the base width of the ponded cross section above the soil filter (m)

L is the length of the bioretention zone (m)

h_{max} is the depth of pondage above the soil filter (m)

d is the depth of filter media

4.3.2 Swale design

The swale component of a bioretention swale needs to be determined first to set the broad dimensions of the system. Typically the swale will be trapezoidal in shape with side slopes ranging from 1:9 to 1:3 depending on local council regulations and any requirements for driveway crossings. The base of the swale is where a bioretention system can be installed. A minimum base width of 300mm is suggested, however this would more typically be 600-1,000mm.

The swale design involves either:

i. Determining the width of swale required to pass the design flow for the minor drainage system (if the catchment areas are known) or

ii. Determining the maximum length of swale prior to discharge into an overflow pit (i.e. maximum length of each cell) for a given width of swale.

Manning's equation is used to size the swale given the site conditions. Selection of an appropriate Manning's 'n' is a critical consideration (see 4.3.2.2) and this will vary depending on the vegetation type. Consideration of landscape and maintenance elements of vegetation will need to be made before selecting a vegetation type.

4.3.2.1 Slope considerations

Two considerations for slope are required for the swale component of a bioretention swale, namely side slopes and longitudinal slopes.

Selection of an appropriate side slope is dependent on local council regulations and will relate to traffic access and the provision of driveway crossings (if required). The provision of driveway crossings can significantly impact on the required width of the swale/ bioretention system with driveway crossings either being ‘elevated’ or ‘at-grade’. Elevated crossings provide a culvert along the swale to allow flows to continue downstream, whereas at-grade crossings act as small fords and flows pass over the crossings.

The slope of at-grade crossings (and therefore the swale) is governed by the trafficability of the change in slope across the base of the swale. Typically 1:9 side slopes, with a small flat base, will provide sufficient transitions to allow for suitable traffic movement.

Where narrower swales are required, elevated crossings can be used (with side slopes typically of 1 in 5) which will require provision for drainage under the crossings with a culvert or similar.

Crossings can provide good locations for promoting extended detention within the bioretention swale and also for providing overflow points in the bioretention swale that can also be used to achieve ponding over a bioretention system (e.g. Figure 4.2). The distance between crossings will determine how feasible having overflow points at each one is.

Selection of an appropriate crossing type should be made in consultation with urban and landscape designers.

4.3.2.2 Selection of Mannings “n”

Manning’s ‘*n*’ is a critical variable in the Manning’s equation relating to roughness of the channel. It varies with flow depth, channel dimensions and the vegetation type. For constructed swale systems, the values are recommended to be between 0.15 and 0.4 for flow depths shallower than the vegetation height (preferable for treatment) and significantly lower (e.g. 0.03) for flows with greater depth than the vegetation. It is considered reasonable for Manning’s ‘n’ to have a maximum at the vegetation height and then sharply reduce as depths increase. Figure 4.7 shows a plot of varying Manning’s *n* with flow depth for a grass swale. It is reasonable to expect the shape of the Manning’s *n* relation with flow depth to be consistent with other swale configurations, with the vegetation height at the boundary between *Low flows* and *Intermediate flows* (Figure 4.7) on the top axis of the diagram. The bottom axis of the plot has been modified from Barling and Moore (1993) to express flow depth as a percentage of vegetation height.

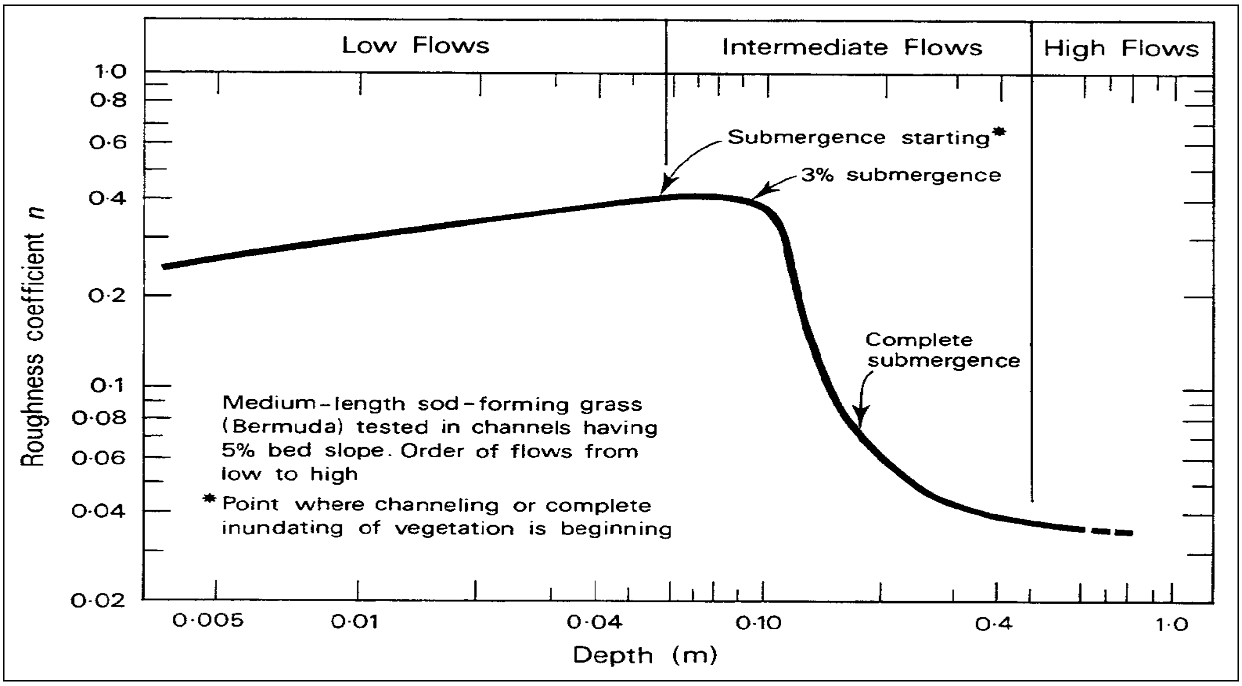


Figure 4.7. Impact of flow depth on hydraulic roughness adapted from Barling and Moore (1993)

Further discussion on selecting an appropriate Manning's 'n' for swales is provided in Appendix F of the MUSIC modelling manual (eWater, 2009).

4.3.3 Inlet details

Stormwater inflow to bioretention swales can be uniformly distributed (e.g. from flush kerbs along a road) or directly from pipe outlets. Combinations of these two entrance pathways can be used.

4.3.3.1 Distributed inflows

An advantage of flows entering a swale system in a distributed manner (ie. entering perpendicular to the direction of the swale) is that inflows are distributed and inflow depths are shallow which maximises contact with vegetation. This provides good pretreatment prior to flows entering the bioretention system. Creating distributed inflows can be achieved either by having flush kerbs or by using kerbs with regular breaks (Figure 4.9).

For distributed inflows, it is important to provide an area for coarse sediments to accumulate off the road surface. The photograph in Figure 4.8 shows sediment accumulating on a street surface where the vegetation is at the same level as the road. To avoid this accumulation, a tapered flush kerb can be used that sets the top of the vegetation between 40-50mm lower than the road surface (Figure 4.8). This requires the top of the ground surface (before turf is placed) to be between 80-100 mm below the road surface. This allows sediments to accumulate off any trafficable surface.

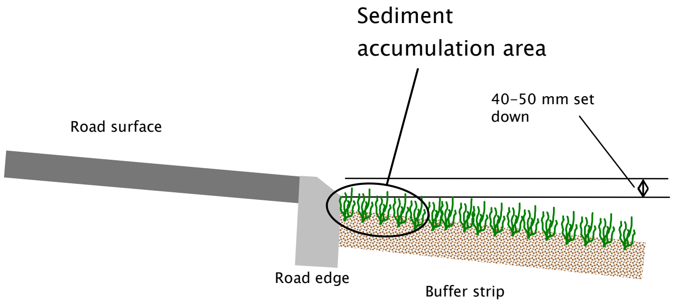


Figure 4.8. Photograph of flush kerb without setdown, edge detail showing setdown

Figure 4.9. Photograph of kerbs with breaks to distribute inflows

4.3.4 Direct entry points

Direct entry of flows can either be through a break in a kerb or from a pipe system. Entrances through kerb breaks may cause some level of water ponding around the entry points. The width of the flow inundation on the road prior to entry will need to be checked and the width of the required opening determined to meet Council requirements. These issues are discussed further in Chapter 5, Bioretention basins.

For piped entrances into bioretention swales, energy dissipation at the pipe outlet point is an important consideration to minimise any erosion potential. This can usually be achieved with a rock mattress and dense vegetation or pipe outlet structures with specific provision for energy dissipation.

The most common constraint on this system is bringing the pipe to the surface of the bioretention swale within the available width. Generally the maximum width of the system will be fixed, as will maximum batter slopes along the swale (1:5 is typical, however 1:3 may be possible for shallow systems with bollards). Further constraints are the cover required for a pipe that crosses underneath a road, as well as the required grade of the pipe. These constraints need to be considered carefully.

In situations where geometry doesn't permit the pipe to reach the surface, a surcharge pit can be used to bring flows to the surface. This is considered preferable to discharging flows below the surface directly into the bioretention filter media because of the potential for blockage and the inability to monitor operation.

Surcharge pits should be designed so that they are as shallow as possible and they should also have pervious bases to avoid long term ponding in the pits and to allow flows from within the pits to drain through the bioretention media and receive treatment. The pits need to be accessible so that any build up of coarse sediment and debris can be monitored and removed if necessary.

These systems are most frequently used when allotment runoff is required to cross a road into a swale on the opposite side. Several allotments can generally be combined prior to crossing the road to minimise the number of road crossings. Figure 4.10 shows an example of a surcharge pit discharging into a bioretention swale.

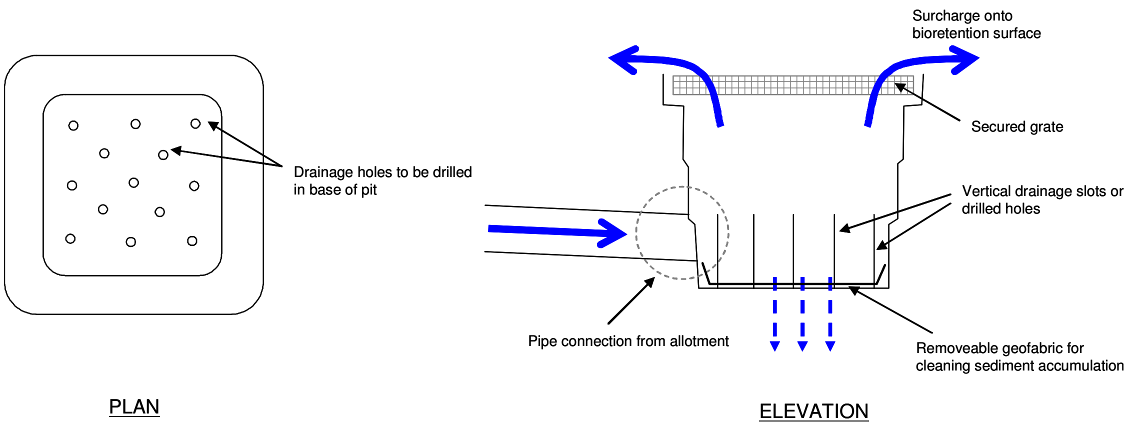


Figure 4.10. Example of surcharge pit for discharging allotment runoff into a bioretention swale

4.3.5 Vegetation scour velocity check

Scour velocities over the vegetation along the swale need to be checked. Manning's equation is used to estimate the mean velocity in the swale. An important consideration is the selection of an appropriate Manning's 'n' that suits the vegetation height. The selection of an appropriate 'n' is discussed in 4.3.2.2.

Manning's equation should be used to estimate flow velocities and ensure the following criteria are met:

- Less than 0.5 m/s for flows up to the design discharge for the minor drainage system (e.g.5-year ARI)
- Less than 1.0 m/s for flows up to the 100-year ARI

4.3.5.1 Velocity check – safety

As swales are generally accessible by the public, it is important to check that the combined depth and velocities product is acceptable from a public risk perspective. To avoid people being swept away by flows along swales, a velocity-depth product check should be performed for design flow rates, as in ARR BkVIII Section 1.10.4.

Velocity (m/s) x depth (m) < 0.4 m^2/s

4.3.6 Size perforated collection pipes

Perforated or slotted collection pipes at the base of bioretention systems collect treated water for conveyance downstream. The collection pipes (there may need to be multiple pipes) should be sized so that the filtration media are freely drained and the collection system does not become a 'choke' in the system.

If gravel is used around the perforated pipes and the filtration media is finer than sand, it is recommended to install an additional 'transition' layer to prevent the fine filtration media being washed into the perforated pipes. Typically this is sand to coarse sand (0.7 – 1.0 mm). Alternatively, a geotextile fabric could be used above the drainage layer to prevent finer material from reaching the perforated pipes, however, caution should be taken to ensure this material is not too fine as if it becomes blocked the whole system will require resetting.

Considerations for the selection of a drainage layer include the slot widths in the perforated pipes as well as construction techniques. In addition, where the bioretention system can only have limited depth (e.g. max depth to perforated pipe <0.5m) it will be preferable to install just one drainage layer with a geotextile fabric providing the function of the transition layer.

Installing parallel pipes is a means to increase the capacity of the perforated pipe system. 100 mm diameter is recommended as the maximum size for the perforated pipes to minimise the thickness of the drainage layer. Either flexible perforated pipe (e.g. AG pipe) or slotted PVC pipes can be used, however care needs to be taken to ensure that the slots in the pipes are not so large that sediment would freely flow into the pipes from the drainage layer. This should also be a consideration when specifying the drainage layer media.

DESIGN NOTE – The use of slotted uPVC over the more traditional choice of flexible agricultural pipe (Agriflex) has numerous advantages:

- Increased structural strength resulting in greater filter media depths without failure.
- Consistent grades to maintain self cleansing velocities are more easily maintained.
- Larger drainage slots allow for faster drainage and less risk of blockage thus increasing service life of the filter bed.
- Higher flow capacities therefore requiring lower numbers of pipes.

The maximum spacing of the perforated pipes should be 1.5m (centre to centre) so that the distance water needs to travel through the drainage layer does not hinder drainage of the filtration media.

To ensure the slotted pipes are of adequate size, several checks are required:

- Ensure the perforations are adequate to pass the maximum infiltration rate
- Ensure the pipe itself has capacity
- Ensure that the material in the drainage layer will not be washed into the perforated pipes (consider a transition layer).

These checks can be performed using the equations outlined in the following sections, or alternatively manufacturers' design charts can be adopted to select appropriately sized pipes.

4.3.6.1 Perforations inflow check

To estimate the capacity of flows through the perforations, orifice flow conditions are assumed and a sharp edged orifice equation can be used. Firstly the number and size of perforations needs to be determined (typically from manufacturer's specifications) and used to estimate the flow rate into the pipes using a head of the filtration media depth plus the ponding depth. Secondly, it is conservative but reasonable to use a blockage factor (e.g. 50% blocked) to account for partial blockage of the perforations by the drainage layer media.

$$Q_{perforation} = B \cdot C \cdot A_{perforation} \sqrt{2gh}$$

Equation 4.2

where	B	is the blockage factor (0.5-0.75)
	C	is the orifice coefficient (~0.6)
	A	is the area of the perforation
	h	is depth of water over the collection pipe

The combined discharge capacity of the perforations in the collection pipe should exceed the design discharge of the sand filter unless the specific intention is to increase detention time in the sand filter by limiting the discharge through the collection pipe.

4.3.6.2 Perforated pipe capacity

One form of the Colebrook-White equation can be applied to estimate the velocity and hence flow rate in the perforated pipe. The capacity of this pipe needs to exceed the maximum infiltration rate.

V = -2(2gDSf) 0.5 x log [(k/3.7D) + (2.51v/D(2gDSf)0.5)]

V = Q / A

Therefore

Q = -2(2gDSf) 0.5 x log [(k/3.7D) + (2.51v/D(2gDSf)0.5)] x A

Equation 4.3

Where D = pipe diameter

A = area of the pipe

S_f = pipe slope

k = wall roughness

ν = viscosity

g = gravity constant

4.3.6.3 Drainage layer hydraulic conductivity

The composition of the drainage layer should be considered when selecting the perforated pipe system, as the slot sizes in the pipes may determine a minimum size of drainage layer particle size. Coarser material (e.g. fine gravel) should be used if the slot sizes are large enough that sand will be washed into the slots.

The material size differential should be an order of magnitude between layers to avoid fine material being washed through the voids of a lower layer. Therefore, if fine gravels are used, then a transition layer is recommended to prevent the filtration media from washing into the perforated pipes. The addition of a transition layer increases the overall depth of the bioretention system and may be an important consideration for some sites (therefore pipes with smaller perforations may be preferable where depth of the system is limited by site constraints).

4.3.6.4 Impervious liner requirement

Should surrounding soils be very sensitive to any exfiltration from the bioretention system (e.g. sodic soils, shallow groundwater or close proximity to significant structures), an impervious liner can be used to contain all water within the bioretention system. The liner could be a flexible membrane or a concrete casing.

The intention of the lining is to eliminate the risk of exfiltration from the bioretention trench. The greatest risk of exfiltration is through the base of a bioretention trench. Gravity and the difference in hydraulic conductivity between the filtration media and the surrounding native soil would act to minimise exfiltration through the walls of the trench. It is recommended that if lining is required, only the base and the sides of the *drainage layer* be lined. Furthermore, it is recommended that the base of the bioretention trench be shaped to promote a more defined flow path of treated water towards the perforated pipe.

The amount of water lost to surrounding soils is highly dependant on local soils and the hydraulic conductivity of the filtration media in the bioretention system. Typically the hydraulic conductivity of filtration media should be selected such that it is 1-2 orders of magnitude greater than the native surrounding soil profile to ensure that the preferred flow path is into the perforated underdrainage system.

4.3.7 High-flow route and overflow design

The design for high flows must safely convey flows up to the design storm for the minor drainage system (e.g. 5-year ARI flows) to the same level of protection that a conventional stormwater system provides. Flows are to be contained within the bioretention swale. Where the capacity of the swale system is exceeded at a certain point along its length, an overflow pit is required. This discharges excess flows into an underground drainage system for conveyance downstream. The frequency of overflow pits is determined in the swale design (4.3.2). This section suggests a method to dimension the overflow pits.

Locations of overflow pits are variable, but it is desirable to locate them at the downstream end of the bioretention system and to have their inverts higher than the filter media to allow ponding and therefore more treatment of flow before bypass occurs.

Typically grated pits are used and the allowable head for discharges is the difference in level between the invert and the nearby road surface. This should be at least 100 mm, but preferably more.

To size a grated overflow pit, two checks should be made to check for either drowned or free flowing conditions. A broad crested weir equation can be used to determine the length of weir required (assuming free flowing conditions) and an orifice equation used to estimate the area between opening required (assumed drowned outlet conditions). The larger of the two pit configurations should be adopted. In addition, a blockage factor is be to used that assumes the orifice is 50% blocked.

3. **Weir flow condition** – *when free overall conditions occur over the pit (usually when the extended detention storage of the retarding basin is not fully engaged),* ie.

$$P = \frac{Q_{des}}{B \cdot C_w \cdot H^{1.5}}$$

Equation 4.4

P	=	Perimeter of the outlet pit
B	=	Blockage factor (0.5)
H	=	Depth of water above the crest of the outlet pit
Q_{des}	=	Design discharge (m^3/s)
C_w	=	weir coefficient (1.7)

4. **Orifice flow conditions** – *when the inlet pit is completely submerged (corresponding to conditions associated with larger flood events*), ie.

$$A_o = \frac{Q_{des}}{B \cdot C_d \sqrt{2gH}}$$

Equation 4.5

C_d	=	Orifice Discharge Coefficient (0.6)
B	=	Blockage factor (0.5)
H	=	Depth of water above the centroid of the orifice (m)
A_o	=	Orifice area (m^2)
Q_{des}	=	Design discharge (m^3/s)

It is important that an outlet pit is prevented from blockage by debris. Design consideration needs to include means of preventing blockage of the outlet structure.

4.3.8 Soil media specification

At least two and possibly three types of soil media are required for the bioretention component of the system.

A filter media layer provides the majority of the treatment function, through fine filtration and also by supporting vegetation that enhances filtration, keeps the filter media porous and provides some uptake of nutrients and other contaminants. It is required to have sufficient depth to support vegetation, this usually is between 300-1,000 mm.

A drainage layer is used to convey treated flows into the perforated underdrainage pipes. Either coarse sand or fine gravel can be used. The layer should surround the perforated pipes and be 150 or 200 mm thick. Should fine gravel be used, a 100 mm transition layer is recommended that will prevent finer filter media being washed into the perforated pipes.

Materials similar to that given in the sections below should provide adequate substrate for vegetation to grow in and sufficient conveyance of stormwater through the bioretention system.

4.3.8.1 Filter media specifications

The material can be of siliceous or calcareous origin. The material will be placed and lightly compacted. Compaction is only required to avoid subsidence and uneven drainage. The material will periodically be completely saturated and completely drained. The bioretention system will operate so that water will infiltrate into the filter media and move vertically down through the profile. Maintaining the prescribed hydraulic conductivity is crucial.

The material shall meet the geotechnical requirements set out below:

Material - Sandy loam or equivalent material (ie similar hydraulic conductivity, 36-180 mm/hr) free of rubbish and deleterious material.

Particle Size- Soils with infiltration rates in the appropriate range typically vary from sandy loams to loamy sands. Soils with the following composition are likely to have an infiltration rate in the appropriate range - clay 5 - 15 %, silt <30 %, sand 50 - 70 %, assuming the following particle size ranges (clay < 0.002 mm, silt 0.002 - 0.05 mm, sand 0.05 - 2.0 mm).

Soils with the majority of particles in this range would be suitable. Variation in large particle size is flexible (ie. an approved material does not have to be screened). Substratum materials should avoid the lower particle size ranges unless hydraulic conductivity tests can demonstrate an adequate hydraulic conductivity (36-180 mm/hr).

Organic Content - between 5% and 10%, measured in accordance with AS1289 4.1.1.

pH - is variable, but preferably neutral, nominal pH 6.0 to pH 7.5 range. Optimum pH for denitrification, which is a target process in this system, is pH 7-8. It is recognised that siliceous materials may have lower pH values.

Any component or soil found to contain high levels of salt, high levels of clay or silt particles (exceeding the particle size limits set above), extremely low levels of organic carbon or any other extremes which may be considered a retardant to plant growth and denitrification should be rejected.

4.3.8.2 *Transition layer specifications*

Transition layer material shall be sand/ coarse sand material. A typical particle size distribution is provided below:

% passing	1.4 mm	100 %
	1.0 mm	80 %
	0.7 mm	44 %
	0.5 mm	8.4 %

This grading is based on a Unimin 16/30 FG sand grading.

The transition layer is recommended to be a minimum of 100mm thick. Table 4.1 presents hydraulic conductivities for a range of media sizes (based on D_{50} sizes) that can be applied in either the transition or drainage layers.

Table 4.1. Hydraulic conductivity for a range of media particle sizes (d_{50}), Engineers Australia (2003)

Soil type	Particle Size (mm)	Saturated Hydraulic Conductivity (mm/hr)	Saturated Hydraulic Conductivity (m/s)
Gravel	2	36000	1×10^{-2}
Coarse Sand	1	3600	1×10^{-3}
Sand	0.7	360	1×10^{-4}
Sandy Loam	0.45	180	5×10^{-5}
Sandy Clay	0.01	36	1×10^{-5}

4.3.8.3 *Drainage layer specifications*

The drainage layer specification can be either coarse sand (similar to the transition layer) or fine gravel, such as a 2mm or 5 mm screenings. Alternative material can also be used (such as recycled glass screenings) provided it is inert and free draining.

This layer should be a minimum of 150mm, and preferably 200mm, thick.

4.3.9 Vegetation specification

Appendix B Plant Lists provides lists of plants that are suitable for bioretention swales. Consultation with landscape architects is recommended when selecting vegetation, to ensure the treatment system compliments the landscape of the area.

4.3.9.1 Design calculation summary

Bioretention Swales — **CALCULATION SUMMARY**

	CALCULATION TASK		OUTCOME	CHECK
1	**Identify design criteria**			
	conveyance flow standard (ARI)		year	
	area of bioretention		m^2	
	maximum ponding depth		mm	
	Filter media type		mm/hr	
2	**Catchment characteristics**			
			m^2	
			m^2	
		slope	%	
	Fraction impervious			
3	**Estimate design flow rates**			
	Time of concentration			
	estimate from flow path length and velocities		minutes	
	Identify rainfall intensities			
		station used for IFD data:		
		major flood – 100 year ARI	mm/hr	
		minor flood – 5 year ARI	mm/hr	
	Peak design flows			
		Q_{minor}	m^3/s	
		Q_{100}	m^3/s	
		Q_{infil}	m^3/s	
3	**Swale design**			
		appropriate Manning's n used?		
4	**Inlet details**			
		adequate erosion and scour protection?		
5	**Velocities over vegetation**			
		Velocity for 5 year flow (<0.5m/s)	m/s	
		Velocity for 100 year flow (<1.0m/s)	m/s	
		Safety: Vel x Depth (<0.4)	m/s	
6	**Slotted collection pipe capacity**			
		pipe diameter	mm	
		number of pipes		
		pipe capacity	m^3/s	
		capacity of perforations	m^3/s	
		soil media infiltration capacity	m^3/s	
8	**Overflow system**			
		system to convey minor floods		
9	**Surrounding soil check**			
		Soil hydraulic conductivity	mm/hr	
		Filter media	mm/hr	
		MORE THAN 10 TIMES HIGHER THAN SOILS?		
10	**Filter media specification**			
		filtration media		
		transition layer		
		drainage layer		
11	**Plant selection**			

4.4 Checking tools

This section provides a number of checking aids for designers and referral authorities. In addition, advice on construction techniques and lessons learnt from building bioretention systems are provided.

Checklists are provided for:

- Design assessments
- Construction (during and post)
- Operation and maintenance inspections
- Asset transfer (following defects period).

4.4.1 Design assessment checklist

The checklist below presents the key design features that should be reviewed when assessing a design of a bioretention basin. These considerations include configuration, safety, maintenance and operational issues that should be addressed during the design phase.

Where an item results in an “N” when reviewing the design, referral should be made back to the design procedure to determine the impact of the omission or error.

In addition to the checklist, a proposed design should have all necessary permits for its installations. The referral agency should ensure that all relevant permits are in place. These can include permits to clear vegetation, to dredge, create a waterbody, divert flows or disturb fish or platypus habitat.

Land ownership and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the asset transfer checklist (see 4.4.4).

Bioretention Swale Design Assessment Checklist					
Bioretention location:					
Hydraulics	Minor Flood: (m^3/s)		Major Flood: (m^3/s)		
Area	Catchment Area (ha):		Bioretention Area (ha)		
Treatment				Y	N
Treatment performance verified from curves?					
Inlet zone/hydraulics				Y	N
Station selected for IFD appropriate for location?					
Longitundinal slope of invert >1% and <4%?					
Mannings 'n' selected appropriate for proposed vegetation type?					
Overall flow conveyance system sufficient for design flood event?					
Maximum flood conveyance width does not impact on traffic amenity?					
Overflow pits provided where flow capacity exceeded?					
Inlet flows appropriately distributed?					
Energy dissipation provided at inlet?					
Velocities within bioretention cells will not cause scour?					
Set down of at least 50mm below kerb invert incorporated?					
Collection System				Y	N
Slotted pipe capacity > infiltration capacity of filter media?					
Transition layer/geofabric barrier provided to prevent clogging of drainage layer?					
Cells				Y	N
Maximum ponding depth and velocity will not impact on public safety (v x d <0.4)?					
Selected filter media hydraulic conductivity > 10x hydraulic conductivity of surrounding soil?					
Maintenance access provided to invert of conveyance channel?					
Protection from gross pollutants provided (for larger systems)?					
Vegetation				Y	N
Plant species selected can tolerate periodic inundation and design velocites?					
Plant species selected integrate with surrounding landscape design?					
Detailed soil specification included in design?					

4.4.2 Construction advice

This section provides general advice for the construction of bioretention basins. It is based on observations from construction projects around Australia.

Building phase damage

Protection of filtration media and vegetation is important during building phase, uncontrolled building site runoff is likely to cause excessive sedimentation, introduce weeds and litter and require replanting following the building phase. Can use a staged implementation – i.e. during building use geofabric and some soil and instant turf (laid perpendicular to flow path) to provide erosion control and sediment trapping. Following building, remove and revegetate possibly reusing turf at subsequent stages. Also divert flows around swales during building (divert to sediment controls).

Traffic and deliveries

Ensure traffic and deliveries do not access bioretention swales during construction. Traffic can compact the filter media and cause preferential flow paths, deliveries (such as sand or gravel) can block filtration media. Washdown wastes (e.g. concrete) can also cause blockage of filtration media and damage vegetation. Bioretention areas should be fenced off during building phase and controls implemented to avoid washdown wastes.

Management of traffic during the building phase is particularly important and poses significant risks to the health f the vegetation and functionality of the bioretention system. Measures such as those proposed above (e.g. staged implementation of final landscape) should be considered.

Inlet erosion checks

It is good practice to check the operation of inlet erosion protection measures following the first few rainfall events. It is important to check for these early in the systems life, to avoid continuing problems. Should problems occur in these events the erosion protection should be enhanced.

Sediment build-up on roads

Where flush kerbs are to be used, a set-down from the pavement surface to the vegetation should be adopted. This allows a location for sediments to accumulate that is off the pavement surface. Generally a set down from kerb of 50mm to the top of vegetation (if turf) is adequate. Therefore, total set down to the base soil is approximately 100 mm (with 50mm turf on top of base soil).

Tolerances

It is importance to stress the importance of tolerances in the construction of bioretention swales (eg base, longitudinal and batters) – having flat surfaces is particularly important for a well distributed flow paths and even ponding over the surfaces. Generally plus or minus 50mm is acceptable.

Erosion control

Immediately following earthworks it is good practice to revegetate all exposed surfaces with sterile grasses (e.g. hydroseed). These will stabilise soils, prevent weed invasion yet not prevent future planting from establishing.

Timing for planting

Timing of vegetation is dependent on a suitable time of year (and potential irrigation requirements) as well as timing in relation to the phases of development. For example temporary planting during construction for sediment control (e.g. with turf) then removal and planting with long term vegetation. Alternatively temporary (e.g. turf or sterile grass) can used until a suitable season for long term vegetation.

Planting strategy

A planting strategy for a development will depend on the timing of building phases was well as marketing pressure. For example, it may be desirable to plant out several entrance bioretention systems to demonstrate long term landscape values, and use the remainder of bioretention systems as building phase sediment control facilities (to be planted out following building). Other important considerations include the time of year and whether irrigation will be required during establishment.

Perforated pipes

Perforated pipes can be either a PVC pipe with slots cut into the length of it or a flexible ribbed pipe with smaller holes distributed across its surface (an AG pipe). Both can be suitable. PVC pipes have the advantage of being stiffer with less surface roughness therefore greater flow capacity, however the slots are generally larger than for flexible pipes and this may cause problems with filter or drainage layer particle ingress into the pipe. Stiff PVC pipes however can be cleaned out easily using simple plumbing equipment. Flexible perforated pipes have the disadvantage of roughness (therefore flow capacity) however have smaller holes and are flexible which can make installation easier. Blockages within the flexible pipes can be harder to dislodge with standard plumbing tools.

Clean filter media

Ensure drainage media is washed prior to placement to remove fines.

4.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Bioretention swales

INSPECTED BY:
DATE:
TIME:
WEATHER:
CONTACT DURING VISIT:

SITE: ____________________

CONSTRUCTED BY: ____________________

DURING CONSTRUCTION									
Items inspected	Checked		Satisfactory	Unsatisfactory		Checked		Satisfactory	Unsatisfactory
Preliminary works	Y	N			**Structural components**	Y	N		
1. Erosion and sediment control plan adopted					16. Location and levels of pits as designed				
					17. Safety protection provided				
2. Traffic control measures					18. Location of check dams as designed				
3. Location same as plans					19. Swale crossings located and built as designed				
4. Site protection from existing flows									
Earthworks					20. Pipe joints and connections as designed				
5. Level bed of swale					21. Concrete and reinforcement as designed				
6. Batter slopes as plans					22. Inlets appropriately installed				
7. Dimensions of bioretention area as plans					23. Inlet erosion protection installed				
8. Confirm surrounding soil type with design					24. Set down to correct level for flush kerbs				
9. Provision of liner					**Vegetation**				
10. Perforated pipe installed as designed					25. Stablisation immediately following earthworks				
11. Drainage layer media as designed									
12. Transition layer media as designed					26. Planting as designed (species and densities)				
13. Filter media specifications checked									
14. Compaction process as designed					27. Weed removal before stabilisation				
15. Appropriate topsoil on swale									
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of soil				
2. Traffic control in place					7. Inlet erosion protection working				
3. Confirm structural element sizes					8. Maintenance access provided				
4. Check batter slopes					9. Construction generated sediment removed				
5. Vegetation as designed									

COMMENTS ON INSPECTION

ACTIONS REQUIRED

1.
2.
3.
4.
5.
6.

4.4.4 Asset transfer checklist

Asset Handover Checklist		
Asset Location:		
Construction by:		
Defects and Liability Period		
Treatment	Y	N
System appears to be working as designed visually?		
No obvious signs of under-performance?		
Maintenance	Y	N
Maintenance plans provided for each asset?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
Asset inspected for defects?		
Asset Information	Y	N
Design Assessment Checklist provided?		
As constructed plans provided?		
Copies of all required permits (both construction and operational) submitted?		
Proprietary information provided (if applicable)?		
Digital files (eg drawings, survey, models) provided?		
Asset listed on asset register or database?		

4.5 Maintenance requirements

Vegetation plays a key role in maintaining the porosity of the soil media of the bioretention system and a strong healthy growth of vegetation is critical to its performance. The potential for rilling and erosion down the swale component of the system needs to be carefully monitored during establishment stages of the system.

The most intensive period of maintenance is during the plant establishment period (first two years) when weed removal and replanting may be required. It is also the time when large loads of sediments could impact on plant growth particularly in developing catchments with poor building controls.

Other components of the system that will require careful consideration are the inlet points (if the system does not have distributed inflows). These inlets can be prone to scour and build up of litter and surcharge pits in particular will require routine inspections. Occasional litter removal and potential replanting may be required.

Maintenance is primarily concerned with:

- Maintenance of flow to and through the system
- Maintaining vegetation
- Preventing undesired vegetation from taking over the desirable vegetation
- Removal of accumulated sediments
- Litter and debris removal

Vegetation maintenance will include:

- Removal of noxious plants or weeds
- Re-establishment of plants that die

Sediment accumulation at the inlet points needs to be monitored. Depending on the catchment activities (e.g. building phase) the deposition of sediment can tend to smother plants and reduce the available ponding volume. Should excessive sediment build up occur, it will impact on plant health and require removal before it impacts on plants, leading to a reduction in their capacity to maintain the infiltration rate of the filter media.

Similar to other types of stormwater practices, debris removal is an ongoing maintenance function. Debris, if not removed, can block inlets or outlets, and can be unsightly if located in a visible location. Inspection and removal of debris should be done regularly, but debris should be removed whenever it is observed on the site.

Inspections are also recommended following large storm events to check for scour.

4.5.1 Operation & maintenance inspection form

The form below should be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

Bioretention Swale Maintenance Checklist			
Inspection Frequency: 3 monthly		Date of Visit:	
Location:			
Description:			
Site Visit by:			
Inspection Items	**Y**	**N**	**Action Required (details)**
Sediment accumulation at inflow points?			
Litter within swale?			
Erosion at inlet or other key structures (eg crossovers)?			
Traffic damage present?			
Evidence of dumping (eg building waste)?			
Vegetation condition satisfactory (density, weeds etc)?			
Replanting required?			
Mowing required?			
Clogging of drainage points (sediment or debris)?			
Evidence of ponding?			
Set down from kerb still present?			
Damage/vandalism to structures present?			
Surface clogging visible?			
Drainage system inspected?			
Resetting of system required?			
Comments:			

4.6 Bioretention swale worked example

4.6.1 Worked example introduction

This worked example describes the detailed design of a grass swale and bioretention system located in a median separating an arterial road and a local road within the residential estate. The layout of the catchment and bioretention swale is shown in Figure 4.11. A photograph of a similar bioretention swale in a median strip is shown in Figure 4.12 (although the case study is all turf).

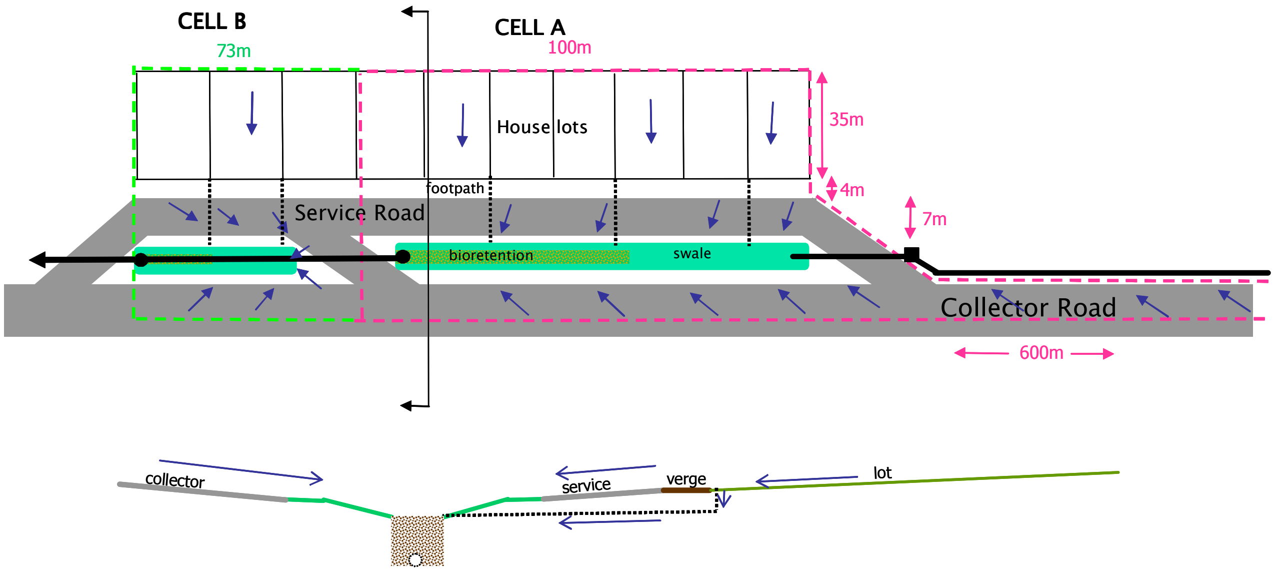


Figure 4.11. Catchment area layout and section for worked example

Figure 4.12. Photograph of bioretention swale

The site is comprised of the arterial road and a service road separated by a median of some 6m width. The median area offers the opportunity for a local treatment measure. The area available is relatively large in relation to the catchment, however is elongated in shape. The catchment area for the swale and bioretention area includes the road reserve and the adjoining allotment (of approximately 35m depth and with a fraction impervious of 0.6).

Three crossings of the median are required and the raised access crossings can be designed as the separation mounds between the swale and bioretention treatment system, thus resulting in a two-cell system.

Each bioretention swale cell will treat its individual catchment area. Runoff from the Arterial Road is conveyed by the conventional kerb and gutter system into a stormwater pipe and discharged into the surface of the swale at the upstream end of each cell. Runoff from the local street can enter the swale as distributed inflow (sheet flow) along the length of the swale.

As runoff flows over the surface of the swale, it receives some pretreatment and coarse to medium sized particles are trapped by vegetation on the swale surface. During runoff events, flow is temporarily impounded in the bioretention zone at the downstream end of each cell. Filtered runoff is collected via a perforated pipe in the base of the bioretention zone. Flows in excess of the capacity of the filtration medium pass through the swale as surface flow and overflow into the piped drainage system at the downstream end of each bioretention cell.

Simulation using MUSIC found that the required area of bioretention system to meet current best practice of 80% reduction in TSS and 45% reduction in TP and TN from values typically generated from urban catchments is approximately 110 m^2 and 42 m^2 for Cell A and B respectively. The filtration medium used is sandy loam with a notional saturated hydraulic conductivity of 180 mm/hr. The required area of the filtration zone is distributed to the two cells according to their catchment area.

4.6.1.1 Design Objectives

- Treatment to meet current best practice objectives of 80%, 45% and 45% reductions of TSS, TP and TN respectively
- Sub-soil drainage pipe to be designed to ensure that the capacity of the pipe exceeds the saturated infiltration capacity of the filtration media (both inlet and flow capacity)
- Design flows within up to 10-year ARI range are to be safely conveyed into a piped drainage system without any inundation of the adjacent road.
- The hydraulics for the swale need to be checked to confirm flow capacity for the 10 year ARI peak flow.
- Acceptable safety and scouring behaviour for 100 year ARI peak flow.

4.6.1.2 Constraints and Concept Design Criteria

- Depth of the bioretention filter layer shall be a maximum of 600mm
- Maximum ponding depth allowable is 200mm
- Width of median available for siting the system is 6m
- The filtration media available is a sandy loam with a saturated hydraulic conductivity of 180mm/hour.

4.6.1.3 Site Characteristics

- Land use Urban, low density residential
- Overland flow slopes Cell A and B=1.3%
- Soil Clay
- Catchment areas:

	Allotments	Collector road	Local road	Footpath	Swale
Cell A	100m x 35m	600m x 7m	100m x 7m	100m x 4m	103m x 7.5m
Cell B	73m x 35m	73m x 7m	73m x 7m	73m x 4m	44m x 7.5m

- Fraction impervious 0.60 (lots); 0.90 (roads); 0.50 (footpaths); 0.0 (Swale)

4.6.2 Confirm size for treatment

Interpretation of 4.2Verifying size for treatment with the input parameters below is used to estimate the reduction performance of the bioretention system to ensure the design will achieve target pollutant reductions.

- Hobart location
- 200mm extended detention
- treatment area to impervious area ratio:

Cell A 110m^2/ 6710 m^2 = 1.61%

Cell B 42m^2/ 2599 m^2 = 1.6%

From the graphs, the expected pollutant reductions are 85%, 70% and 45% for TSS, TP and TN respectively and exceed the design requirements of 80%, 45% and 45%.

DESIGN NOTE – The values derived from 4.2Verifying size for treatment will only be valid if the design criteria for the proposed installation are similar to those used to create the Figures. Site specific modelling using programs such as MUSIC may yield a more accurate result.

4.6.3 Estimating design flows

With a small catchment the Rational Method is considered an appropriate approach to estimate the 10 and 100 year ARI peak flow rates. The steps in these calculations follow below.

4.6.3.1 Major and minor design flows

The procedures in Australian Rainfall and Runoff (ARR) are used to estimate the design flows.

See Appendix E Design Flows – tc for a discussion on methodology for calculation of time of concentration.

Step 1 – Calculate the time of concentration.

Cell A and Cell B are effectively separate elements for the purpose of sizing the swales for flow capacity and inlets to the piped drainage system for a 10 year ARI peak flow event. Therefore, the t_c are estimated separately for each cell.

- Cell A– the t_c calculations include consideration of runoff from the allotments as well as from gutter flow along the collector road. Comparision of these travel times concluded the flow along the collector road was the longest and was adopted for t_c.
- Cell B – the t_c calculations include overland flow across the lots and road and swale/bioretention flow time.

Following procedures in ARR, the following t_c values are estimated:

t_c Cell A : 10 mins

t_c Cell B: 8 mins

Rainfall Intensities for the area of study (for the 10 and 100 year average recurrence intervals) are estimated using ARR (1998) with a time of concentration of 10 minutes and 8 minutes respectively are:

	t_c	100yr	10yr
Cell A	10 min	135*	77*
Cell B	8 min	149*	85*

* These figures are for the worked example only. The appropriate region and corresponding rainfall intensities must be selected for each individual project.

Step 2 – Calculate design runoff coefficients (using the method outlined in Australian Rainfall and Runoff Book VIII (Engineers Australia, 2003)).

Apply method outlined in Section1.5.5 (iii) ARR 2001 Bk VIII

$$C_{10} = 0.9f + C^1{}_{10}\,(1-f)$$

Fraction impervious

Roads – adopt f = 0.90

Footpaths - adopt f = 0.50

Swales - adopt f = 0.00

Lots - adopt f = 0.60

- For Cell A (area weighted) f = 0.70
- For Cell B (area weighted) f = 0.61

Calculate C_{10} (10 year ARI runoff coefficient)

$C_{10} = 0.9f + C^{1}_{10}\ (1-f)$

- C_{10} for Cell A = 0.67
- C_{10} for Cell B = 0.61

Step 3 – Convert C_{10} to values for C_1 and C_{100}

Where – $C_y = F_y \times C_{10}$

Runoff coefficients as per Table 1.6 Book VIII ARR 1998

ARI (years)	Frequency Factor, *Fy*	Runoff Coefficient, *Cy (cell A)*	Runoff Coefficient, *Cy (cell B)*
10	1.0	0.67	0.61
100	1.2	0.80	0.73

Step 4 – Calculate peak design flow (calculated using the Rational Method).

$$Q = \frac{CIA}{360}$$

Where – C is the runoff coefficient (C_{10} and C_{100})

I is the design rainfall intensity mm/hr (I_{10} and I_{100})

A is the catchment area (Ha)

	Q_{10}	Q_{100}
Cell A	0.14	0.29
Cell B	0.06	0.11

4.6.3.2 Maximum infiltration rate

The maximum infiltration rate reaching the perforated pipe at the base of the soil media is estimated by using the hydraulic conductivity of the media and the head above the pipes and applying Darcy's equation.

Saturated permeability = 180 mm/hr

Flow capacity of the infiltration media:

$$Q_{\max} = k \cdot L \cdot W_{base} \cdot \frac{h_{\max} + d}{d}$$

$$Q_{\max} = 5E10^{-5} \cdot L \cdot W_{base} \left(\frac{0.2 + 0.6}{0.6} \right)$$

where k is the hydraulic conductivity of the soil filter (m/s)

W_{base} is the base width of the ponded cross section above the soil filter (m)

L is the length of the bioretention zone (m)

h_{max} is the depth of pondage above the soil filter (m)

d is the depth of filter media

Maximum infiltration rate Cell A = 0.004 m^3/s

Maximum infiltration rate Cell B = 0.001 m^3/s

4.6.4 Swale design

The swales need to be sized such that they can convey 10 year ARI flows into the underground pipe network without water encroaching on the road. Manning's equation is used with the following parameters. Note the depth of the swale (and hence the side slopes) were determined by the requirement of discharging allotment runoff onto the surface of the bioretention system. Given the cover requirements of the allotment drainage pipes as they flow under the service road (550 mm minimum cover), it set the base of the bioretention systems at 0.76m below road surface.

- Base width of 1m with 1:3 side slopes, max depth of 0.76m
- Grass vegetation (assume *n* = 0.045 for 10 year ARI with flows above grass height)
- 1.3% slope

The approach taken is to size the swale to accommodate flows in Cell A and then adopt the same dimension for Cell B for Aesthetic reasons (Cell B has lower flow rates).

The maximum capacity of the swale is estimated adopting a 150mm freeboard (i.e. maximum depth is 0.61m).

Q_{cap} = 2.1 m^3/s > 0.14 m^3/s

Therefore, there is adequate capacity given the relatively large dimensions of the swale to accommodate allotment runoff connection.

4.6.5 Inlet details

There are two mechanisms for flows to enter the system, firstly underground pipes (either from the upstream collector road into Cell 1 or from allotment runoff) and secondly direct runoff from road and footpaths.

Flush kerbs with a 50 mm set down are intended to be used to allow for sediment accumulation from the road surfaces.

Grouted rock is to be used for scour protection for the pipe outlets into the system. The intention of these is to reduce localised flow velocities to avoid erosion.

4.6.6 Vegetation scour velocity check

Assume Q_{10} and Q_{100} will be conveyed through the swale/bioretention system. Check for scouring of the vegetation by checking that velocities are below 0.5m/s during Q_{10} and 1.0 m/s for Q_{100}.

Using Manning's equation to solve for depth for Q_{10} and Q_{100} gives the following results:

Q_{10} = 0.14 m^3/s, depth = 0.15m (with n = 0.3), velocity = 0.09m/s < 0.5m/s – therefore, OK

Q_{100} = 0.29 m^3/s, depth = 0.32m (with n = 0.05), velocity = 0.49m/s < 1.0m/s – therefore, OK

Hence, the swale and bioretention system can satisfactorily convey the peak 10 and 100-year ARI flood, with minimal risk of vegetation scour.

4.6.6.1 Safety velocity check

Check velocity – depth product in Cell A during peak 100-year ARI flow for pedestrian safety criteria.

V = 0.49m/s (calculated previously)

D = 0.32m

v.d = 0.49 x 0.32 = 0.16<0.4m^2/s (ARR 2001 Bk VIII Section1.10.4)

Therefore, velocities and depths are OK.

4.6.7 Sizing of perforated collection pipes

4.6.7.1 Perforations inflow check

Estimate the inlet capacity of sub-surface drainage system (perforated pipe) to ensure it is not a choke in the system. To build in conservatism, it is assumed that 50% of the holes are blocked. A standard perforated pipe was selected that is widely available. To estimate the flow rate an orifice equation is applied using the following parameters:

Head = 0.85 m [0.6 m (filter depth) + 0.2 m (max. pond level) + 0.05 (half of pipe diameter)]

The following are the characteristics of the selected slotted pipe

- Clear openings = 2100 mm^2/m
- Slot width = 1.5mm
- Slot length = 7.5mm
- No. rows = 6
- Diameter of pipe = 100mm

For a pipe length of 1.0 m, the total number of slots = 2100/(1.5 x 7.5) = 187.

Discharge capacity of each slot can be calculated using the orifice flow equation

$$Q_{perforation} = C \cdot A_{perforation}\sqrt{2gh} = 2.75 \times 10^{-5}\ m^3/s$$

where h is the head above the slotted pipe, calculated to be 0.85 m.

C is the orifice coefficient (~0.6)

The inflow capacity of the slotted pipe is thus $2.75 \times 10^{-5} \times 187 = 5.1 \times 10^{-3}$ $m^3/s/m$-length

Adopt a blockage factor of 0.5 gives the inlet capacity of each slotted pipe to be 2.57×10^{-3} $m^3/s/m$-length.

Inlet capacity/m x total length =

Cell A = 0.0025 x 61 = 0.15 m^3/s > 0.003 (max infiltration rate), hence 1 pipe has sufficient perforation capacity to pass flows into the perforated pipe.

Cell B = 0.0025 x 22 = 0.05 m^3/s > 0.001 (max infiltration rate), hence 1 pipe is sufficient.

4.6.7.2 Perforated pipe capacity

The **Colebrook-White equation** is applied to estimate the flow rate in the perforated pipe. A slope of 0.5% is assumed and a 100mm perforated pipe (as above) was used. Should the capacity not be sufficient, either a second pipe could be used or a steeper slope. The capacity of this pipe needs to exceed the maximum infiltration rate.

Estimate applying the Colebrook-White Equation

$$\mathbf{Q = -2(2gDS_f)^{0.5} \times \log [(k/3.7D) + (2.51\nu/D(2gDS_f)^{0.5})] \times A}$$

Adopt $D = 0.10m$

$S_f = 0.005m/m$

$g = 9.81 m^2/s$

$k = 0.007m$

$v = 1.007 \times 10^{-6}$

Q_{cap} = 0.004 m^3/s (for one pipe) > 0.003 m^3/s (Cell 1) 0.001 m^3/s (Cell 2), and hence 1 pipe is sufficient to convey maximum infiltration rate for both Cell A and B.

Adopt 1 x ϕ 100 mm perforated pipe for the underdrainage system in both Cell A and Cell B.

4.6.7.3 Drainage layer hydraulic conductivity

Typically flexible perforated pipes are installed using fine gravel media to surround them. In this case study, 5mm gravel is specified for the drainage layer. This media is much coarser than the filtration media (sandy loam) therefore to reduce the risk of washing the filtration layer into the perforated pipe, a transition layer is to be used. This is to be 100 mm of coarse sand.

4.6.7.4 Impervious liner requirement

In this catchment the surrounding soils are clay to silty clays with a saturate hydraulic conductivity of approximately 3.6 mm/hr. The sandy loam media that is proposed as the filter media has a hydraulic conductivity of 50-200 mm/hr. Therefore the conductivity of the filter media is > 10 times the conductivity of the surrounding soils and an impervious liner is not required.

4.6.8 Overflow design

The overflow pits are required to convey 10 year ARI flows safely from above the bioretention systems and into an underground pipe network. Grated pits are to be used at the downstream end of each bioretention system.

The size of the pits are calculated using a broad crested weir equation with the height above the maximum ponding depth and below the road surface, less freeboard (i.e. 0.76 – (.2 + .15)= 0.41m).

First check using a broad crested weir equation

$$P = \frac{Q_{des}}{B \cdot C_w \cdot H^{1.5}}$$

P = Perimeter of the outlet pit

B = Blockage factor (0.5)

H = 0.41Depth of water above the crest of the outlet pit

Q_{des} = Design discharge (m^3/s)

C_w = weir coefficient (1.7)

Gives P = .62m of weir length required (equivalent to 155 x 155mm pit)

Now check for drowned conditions:

$$A_o = \frac{Q_{des}}{B \cdot C_d \sqrt{2gH}}$$

C_d = Orifice Discharge Coefficient (0.6)

B = Blockage factor (0.5)

H = Depth of water above the centroid of the orifice (m)

A_o = Orifice area (m^2)

Q_{des} = Design discharge (m^3/s)

gives A = 0.16 m^2 (equivalent to 400 x 400 pit)

Hence, drowned outlet flow conditions dominate, adopt pit sizes of 450 x 450 mm for both Cell A and Cell B as this is minimum pit size to accommodate underground pipe connections.

4.6.9 Soil media specification

Three layer of soil media are to be used. A sandy loam filtration media (600mm) to support the vegetation, a coarse transition layer (100mm) and a fine gravel drainage layer (200mm). specifications for these are below.

4.6.9.1 Filter media specifications

The filter media is to be a sandy loam with the following criteria:

The material shall meet the geotechnical requirements set out below:

- hydraulic conductivity between 50-200 mm/hr
- particle sizes of between: clay 5 – 15 %, silt <30 %, sand 50 – 70 %
- between 5% and 10% organic content, measured in accordance with AS1289 4.1.1.
- pH neutral

4.6.9.2 Transition layer specifications

Transition layer material shall be coarse sand material such as Unimin 16/30 FG sand grading or equivalent. A typical particle size distribution is provided below:

% passing	1.4 mm	100 %
	1.0 mm	80 %
	0.7 mm	44 %
	0.5 mm	8.4 %

4.6.9.3 Drainage layer specifications

The drainage layer is to be 5 mm screenings.

4.6.9.4 Vegetation specification

To compliment the landscape design of the area a grass species is to be used. For this application a turf with maximum height of 100 mm has been assumed. The actual species will be selected by the landscape designer.

4.6.9.5 Calculation summary

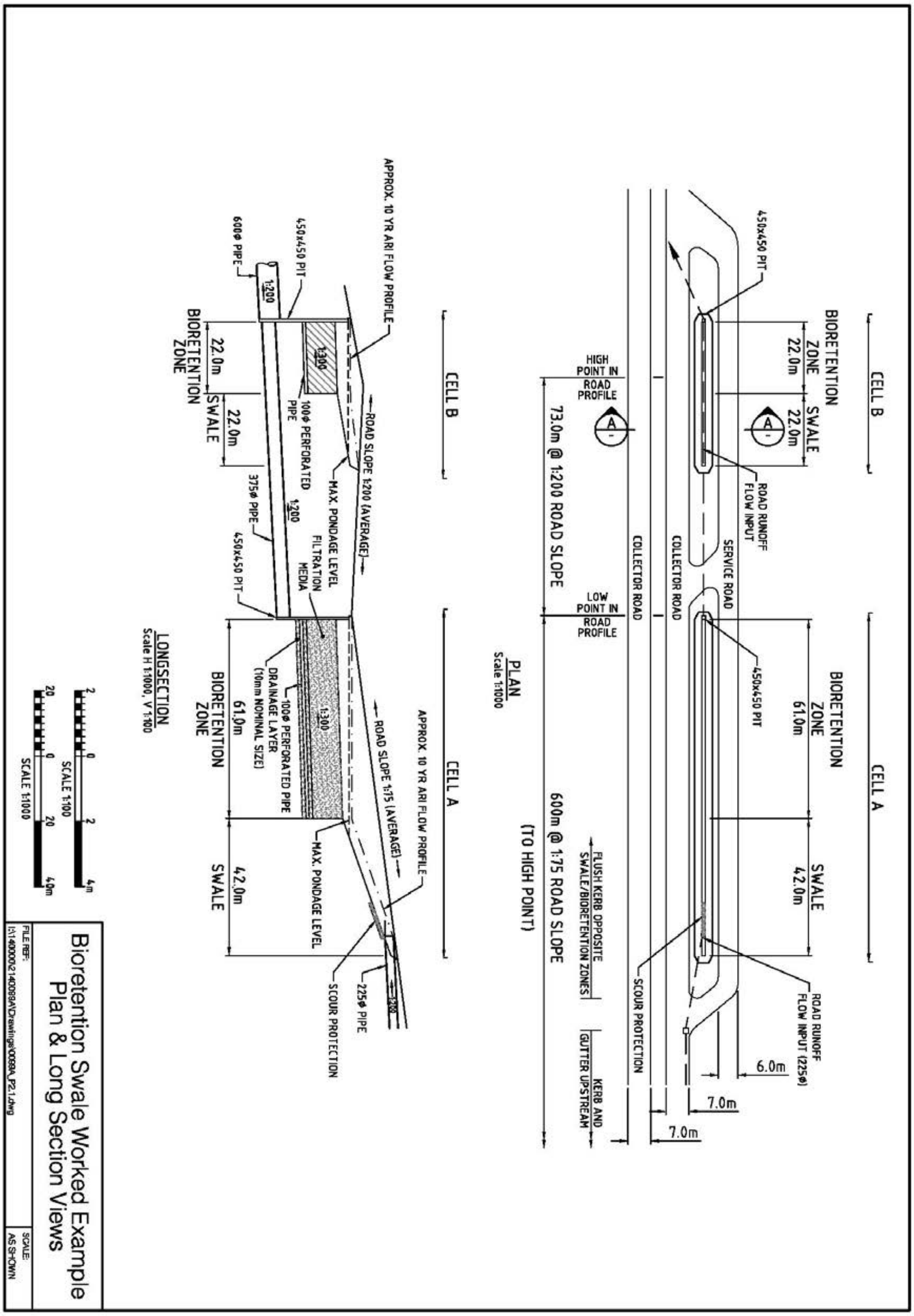
The sheet below shows the results of the design calculations.

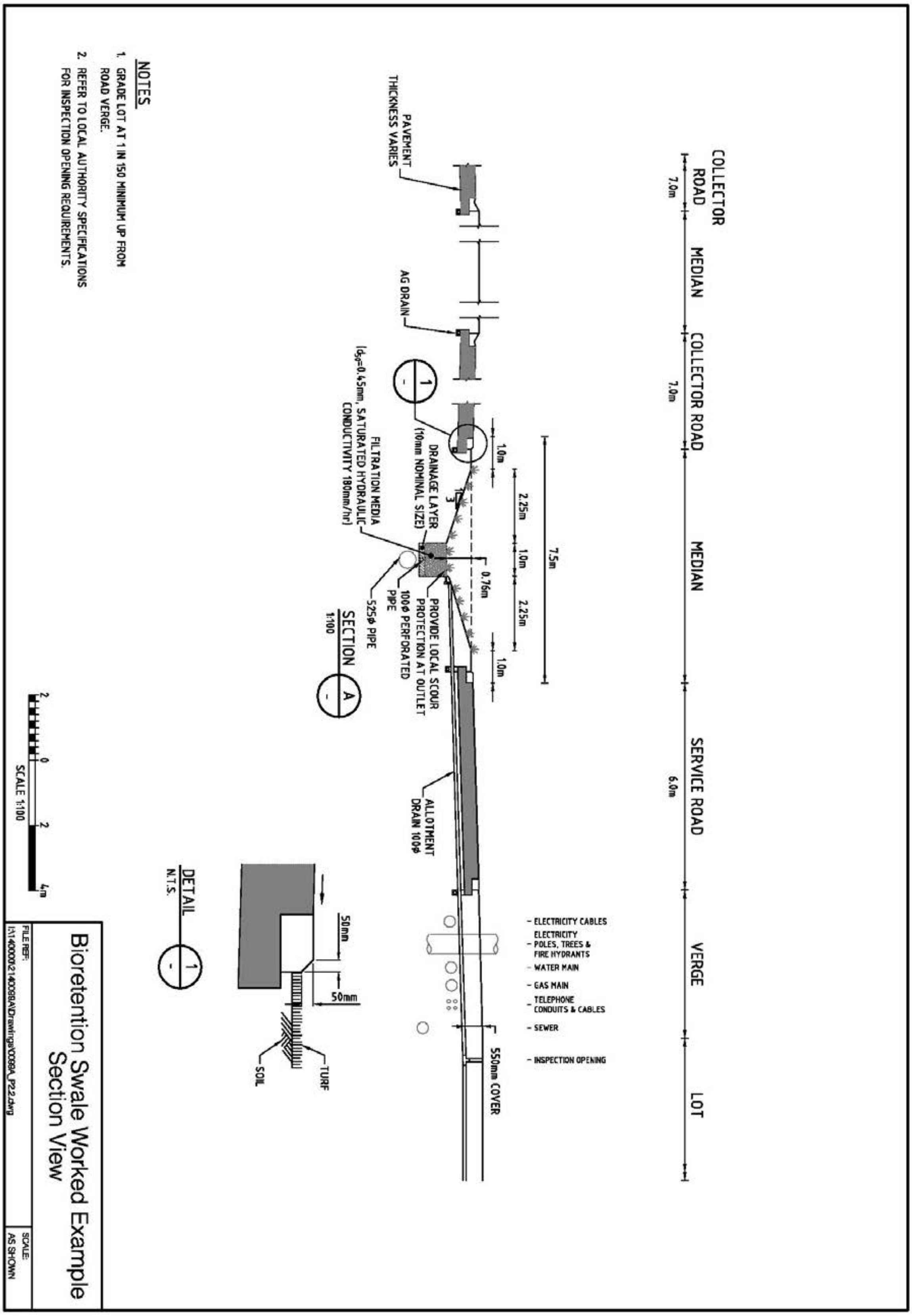
Bioretention Swales

CALCULATION SUMMARY

	CALCULATION TASK		OUTCOME		CHECK
1	**Identify design criteria**				
	conveyance flow standard (ARI)		10	year	
	area of bioretention		110 and 42	m^2	
	maximum ponding depth		200	mm	
	Filter media type		180	mm/hr	✓
2	**Catchment characteristics**				
		Cell A	9600	m^2	
		Cell B	4200	m^2	
		slope	1.3	%	
	Fraction impervious				
		Cell A	70		
		Cell B	0.61		✓
3	**Estimate design flow rates**				
	Time of concentration				
	estimate from flow path length and velocities		Cell A – 10	minutes	✓
			Cell B – 8		
	Identify rainfall intensities				
		station used for IFD data:			
		major flood – 100 year ARI		mm/hr	
		minor flood – 10 year ARI		mm/hr	
	Peak design flows		Cell A, Cell B		
		Q_{minor}	0.14, 0.06	m^3/s	
		Q_{100}	0.29, 0.11	m^3/s	
		Q_{infil}	0.004, 0.001	m^3/s	✓
3	**Swale design**				
		appropriate Manning's n used?	yes		✓
4	**Inlet details**				
		adequate erosion and scour protection?	rock pitching		✓
5	**Velocities over vegetation**				
		Velocity for 10 year flow (<0.5m/s)	0.09	m/s	
		Velocity for 100 year flow (<1.0m/s)	0.49	m/s	
		Safety: Vel x Depth (<0.4)	0.16	m/s	✓
6	**Slotted collection pipe capacity**				
		pipe diameter	100	mm	
		number of pipes	1		
		pipe capacity	0.004	m^3/s	
		capacity of perforations	0.15	m^3/s	
		soil media infiltration capacity	0.003	m^3/s	✓
8	**Overflow system**				
		system to convey minor floods	grated pits 450 x 450		✓
9	**Surrounding soil check**				
		Soil hydraulic conductivity	3.6	mm/hr	
		Filter media	180	mm/hr	
		MORE THAN 10 TIMES HIGHER THAN SOILS?	YES		✓
10	**Filter media specification**				
		filtration media	sandy loam		
		transition layer	sand		
		drainage layer	gravel		✓
11	**Plant selection**		*turf*		

4.6.10 Construction drawings




NOTES
1. GRADE LOT AT 1 IN 150 MINIMUM UP FROM ROAD VERGE.
2. REFER TO LOCAL AUTHORITY SPECIFICATIONS FOR INSPECTION OPENING REQUIREMENTS.
COLLECTOR ROAD
7.0m
MEDIAN
COLLECTOR ROAD
7.0m
MEDIAN
SERVICE ROAD
6.0m
VERGE
LOT
PAVEMENT THICKNESS VARIES
AG DRAIN
7.5m
1.0m
2.25m
1.0m
2.25m
1.0m
0.76m
3
1
DRAINAGE LAYER (10mm NOMINAL SIZE)
FILTRATION MEDIA (d_{50}=0.45mm, SATURATED HYDRAULIC CONDUCTIVITY 180mm/hr)
PROVIDE LOCAL SCOUR PROTECTION AT OUTLET
100ϕ PERFORATED PIPE
525ϕ PIPE
SECTION A
1:100
ALLOTMENT DRAIN 100ϕ
- ELECTRICITY CABLES
- ELECTRICITY POLES, TREES & FIRE HYDRANTS
- WATER MAIN
- GAS MAIN
- TELEPHONE CONDUITS & CABLES
- SEWER
- INSPECTION OPENING
550mm COVER
50mm
50mm
TURF
SOIL
DETAIL 1
N.T.S.
2 0 2 4m
SCALE 1:100
Bioretention Swale Worked Example Section View
FILE REF: I:\140000\2140098A\Drawings\0098A_P2.2.dwg
SCALE: AS SHOWN

4.7 References

Barling, R. D., & Moore, I. D., 1993, *The role of buffer strips in the management of waterway pollution*. In Woodfull, J., Finlayson, P. and McMahon, T.A. (Ed), The role of buffer strips in the management of waterway pollution from diffuse urban and rural sources, The Centre for Environmental Applied Hydrology, University of Melbourne, Report 01/93.

Engineers Australia, 2006, *Australian Runoff Quality Guidelines: A Guide to Water Sensitive Urban Design*, Editor in Chief, Wong, T.H.F., National Committee for Water Engineering.

eWater, 2009, Model for Urban Stormwater Improvement Conceptualisation (MUSIC) User Manual, Version 4.0, September 2009.

Institution of Engineers Australia, 1997, *Australian Rainfall and Runoff – A guide to flood estimation*, Editor in Chief – Pilgram, D.H.

Chapter 5 Bioretention Basins

Definition:

A bioretention basin is a bioretention system that provides efficient treatment of stormwater through fine filtration, extended detention and some biological uptake.

Purpose:

- Removal of fine and coarse sediments.
- Efficient removal of hydrocarbons and other soluble or fine particulate contaminants from biological uptake.
- To provide a low levels of extended detention.
- Provide flow retardation for frequent (low ARI) rainfall events.

Implementation considerations:

- Bioretention basins operate with the same treatment processes as bioretention swales except do not have a conveyance function. High flows are either diverted away from a basin or are discharged into an overflow structure.
- Bioretention basins have an advantage of being applicable at a range of scales and shapes and can therefore have flexibility for locations within a development. They can be located along streets at regular intervals and treat runoff prior to entry into an underground drainage system, or be located at outfalls of a drainage system to provide treatment for much larger areas (e.g. in the base of retarding basins).

A wide range of vegetation can be used within a bioretention basin, allowing them to be well integrated into a landscape theme of an area. Smaller systems can be integrated with traffic calming measures or parking bays, reducing their requirement for space. They are equally applicable to redevelopment as well as greenfield sites.

Bioretention basins are applicable at a range of scales and can be integrated with an urban landscape

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5.2 Verifying size for treatment .... 5—5

5.3 Design procedure: bioretention basins .... 5—7

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5.6 Bioretention basin worked example .... 5—25

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5.1 Introduction

Bioretention basins use ponding to maximise the volume of runoff treated through the filtration media. Their operation is the same as bioretention swales, but typically they convey above design flows through overflow pits or bypass paths, and are not required to convey flood flows over the filtration surface. This has the advantage of not subjecting the filter surface to high velocities that can dislodge collected pollutants or scour vegetation.

These devices can be installed at various scales, for example, in planter boxes, in retarding basins or in streetscapes integrated with traffic calming measures. In larger applications, it is considered good practice to have pretreatment measures upstream of the basin to reduce the maintenance frequency of the bioretention basin. For small systems this is not required.

Figure 5.1 shows an example of a basin integrated into a local streetscape and a car park.

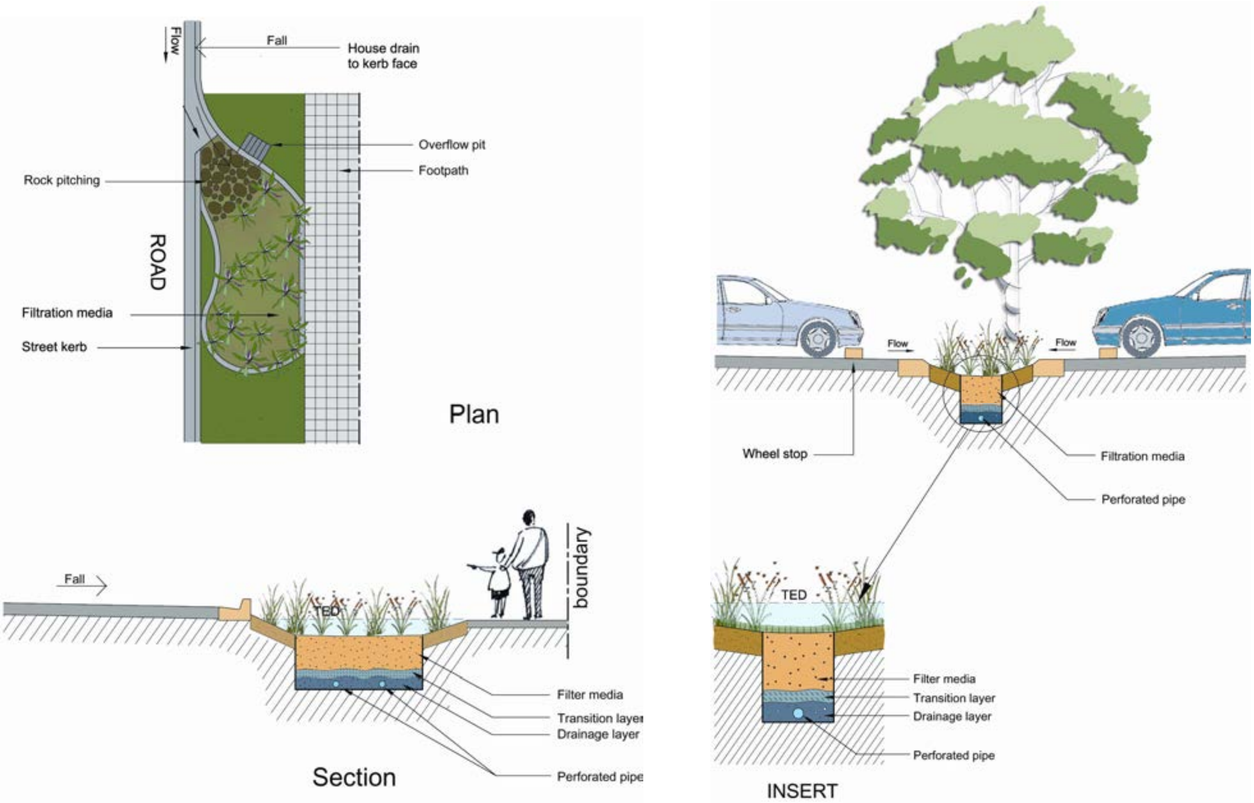


Figure 5.1. Bioretention basin integrated into a local streetscape (L) and a car park (R)

They can be designed to either encourage infiltration (where reducing volumes of stormwater runoff is important) or as conveyance systems that do not allow infiltration (where soils are not suitable for infiltration or in close proximity to surrounding structures).

Where bioretention systems perform a pretreatment for infiltration, they are designed to facilitate infiltration by removing the collection system at the base of the filtration media allowing contact with surrounding soils.

Vegetation that grows in the filter media enhances its function by preventing erosion of the filter medium, continuously breaking up the soil through plant growth to prevent clogging of the system and providing biofilms on plant roots that pollutants can absorb to. The type of vegetation varies depending on landscaping requirements. Generally the denser and higher the vegetation the better the filtration process. Vegetation is critical to maintaining porosity of the filtration layer.

Selection of an appropriate filtration media is a key issue that involves a trade-off between providing sufficient hydraulic conductivity (ie. passing water through the filtration media as quickly as possible) and providing sufficient water retention to support vegetation growth (i.e. retaining sufficient moisture by having low hydraulic conductivities). Typically a sandy loam type material is suitable, however the soils can be tailored to a vegetation type.

A drainage layer is required. This material surrounds the perforated underdrainage pipes and can be either coarse sand (1 mm) or fine gravel (2-5 mm). Should fine gravel be used, it is advisable to install a transition layer of sand or a geotextile fabric to prevent any filtration media being washed into the perforated pipes.

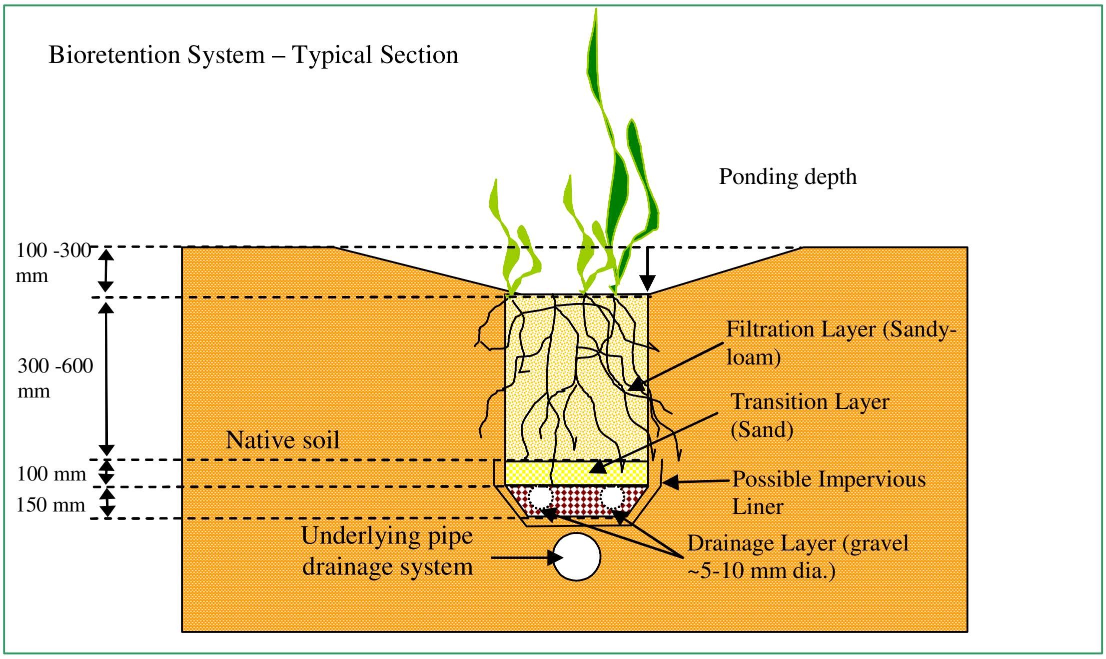


Figure 5.2. Typical section of an bioretention basin

The design process for a bioretention basin is slightly different to bioretention swales, as they do not need to be capable of conveying large floods (e.g. 5-year flows) over their surface and an alternative route for flood flows is required.

Key design issues to be considered are:

- Verifying size and configuration for treatment
- Determine design capacity and treatment flows
- Specify details of the filtration media
- Above ground design:

    - check velocities
    - design of inlet zone and overflow pits
    - check above design flow operation
- Below ground design:
    - prescribe soil media layer characteristics (filter, transition and drainage layers)
    - underdrain design and capacity check
    - check requirement for bioretention lining
- Recommended plant species and planting densities
- Provision for maintenance

5.2 Verifying size for treatment

The curves below show the pollutant removal performance expected for bioretention basins with varying depths of ponding. The curves are based on the performance of the system at the reference site and were derived using the Model for Urban Stormwater improvement Conceptualisation (MUSIC). To estimate an equivalent performance at other locations in Tasmania, the hydrologic design region relationships should be used, refer to Chapter 2. In preference to using the curves, local data should be used to model the specific treatment performance of the system.

The curves were derived assuming the systems receive direct runoff (i.e. no pretreatment) and have the following characteristics:

- Hydraulic conductivity of 36mm/hr
- Filtration media depth of 600 mm
- Particle size of 0.45 mm

These curves can be used to check the expected performance of the bioretention system for removal of TSS, TP and TN.

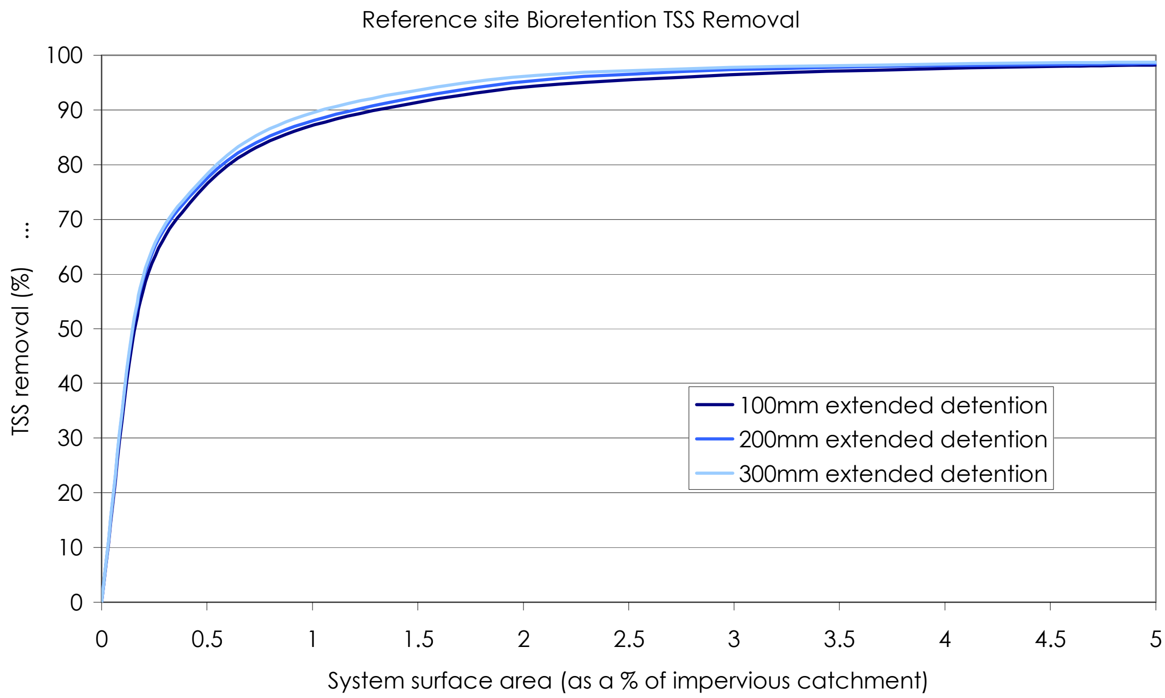


Figure 5.3. TSS removal in bioretention systems with varying extended detention

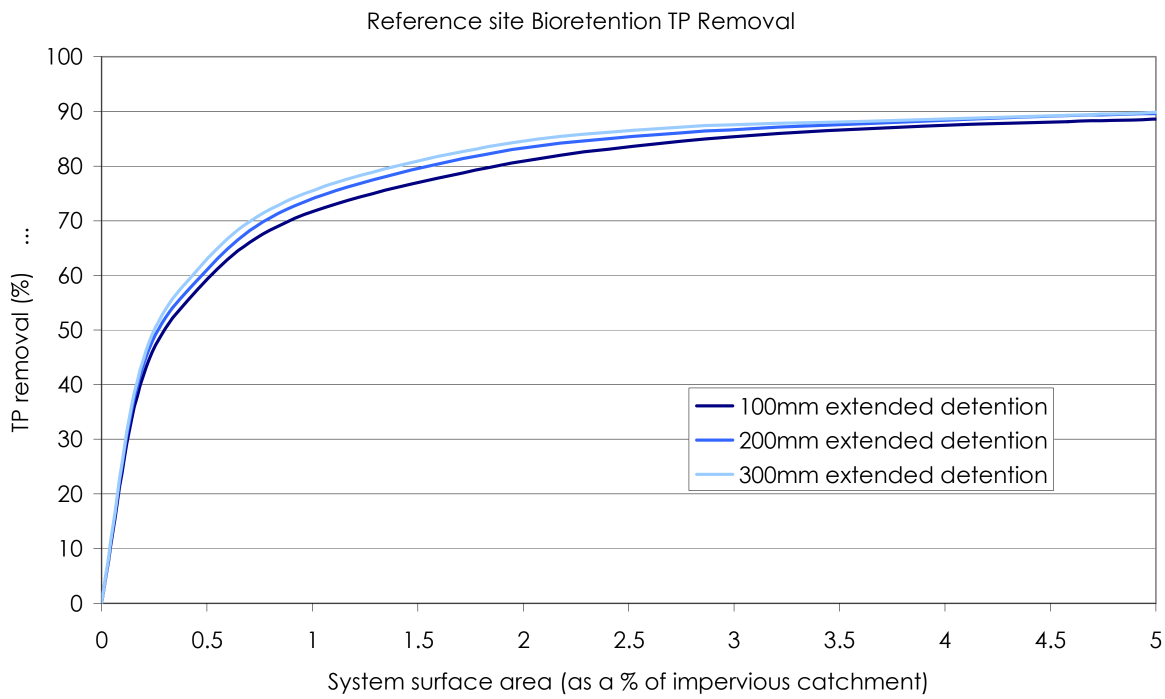


Figure 5.4. TP removal in bioretention systems with varying extended detention

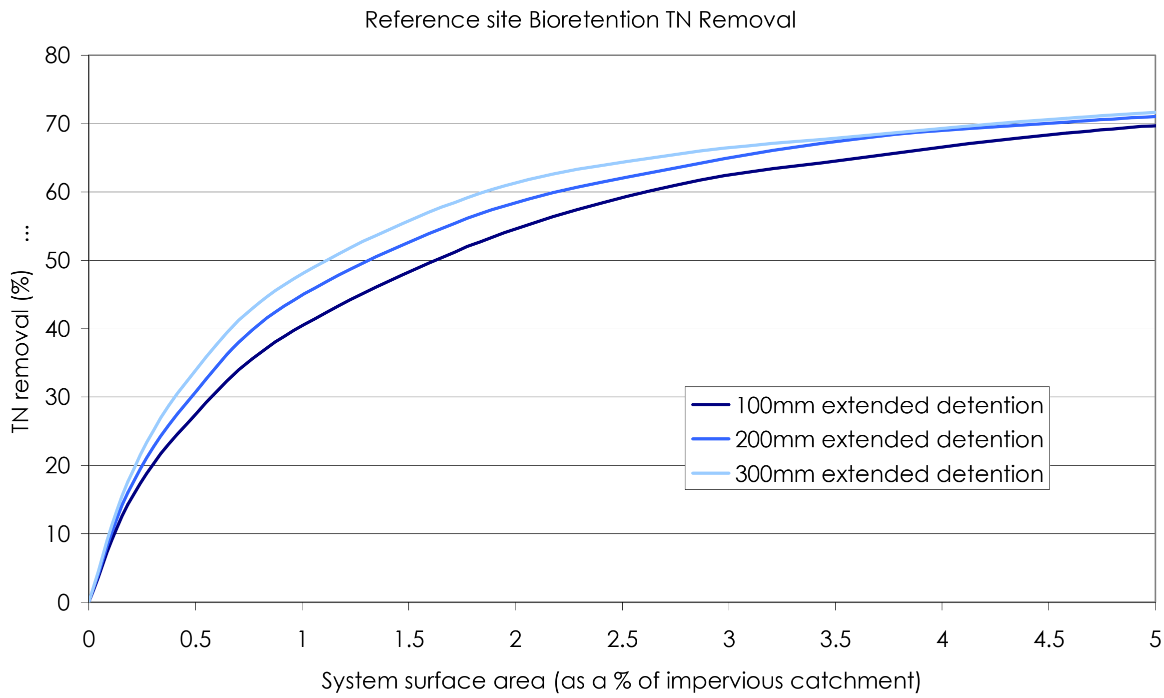


Figure 5.5. TN removal in bioretention systems with varying extended detention

5.3 Design procedure: bioretention basins

The following sections detail the design steps required for bioretention basins.

5.3.1 Estimating design flows

Three design flows are required for bioretention basins:

- minor flood rates (typically 5-year ARI) to size the overflows to allow minor floods to be safely conveyed and not increase any flooding risk compared to conventional stormwater systems
- major flood rates (typically 100 year ARI) to check that flow velocities are not too large in the bioretention system, which could potentially scour pollutants or damage vegetation
- maximum infiltration rate through the filtration media to allow for the underdrainage to be sized, such that the underdrains will allow the filter media to freely drain.

5.3.1.1 Minor and major flood estimation

A range of hydrologic methods can be applied to estimate design flows. With typical catchment areas being relatively small, the Rational Method Design Procedure is considered to be a suitable method for estimating design flows. More detailed flow analysis is required for larger catchment-scale systems.

5.3.1.2 Maximum infiltration rate

The maximum infiltration rate represents the design flow for the underdrainage system (i.e. the slotted pipes at the base of the filter media). The capacity of the underdrains needs to be greater than the maximum infiltration rate to ensure the filter media drains freely and doesn't become a 'choke' in the system.

A maximum infiltration rate (Q_{max}) can be estimated by applying Darcy's equation:

$$Q_{\max} = k \cdot L \cdot W_{base} \cdot \frac{h_{\max} + d}{d}$$

Equation 5.1

where k is the hydraulic conductivity of the soil filter (m/s)

W is the average width of the ponded cross section above the sand filter (m)

L is the length of the bioretention zone (m)

h_{max} is the depth of pondage above the sand filter (m)

5.3.2 Inlet details

Two checks of inlet details are required for bioretention basins, checking the width of flow in the gutter at the inlet (so traffic is not affected) and checking velocities to ensure scour doesn't occur at the entry for both minor and major storm events.

5.3.2.1 Flow widths at entry

The width of flow at the entry during a minor storm event (typically 5 year ARI) needs to be checked. This can be done by applying Manning's equation and ensuring that flows do not exceed local council regulations (e.g. maintaining at least one trafficable lane during a 5-year ARI storm).

5.3.2.2 Kerb opening width at entry

To determine the width of the inlet slot in the kerb into the bioretention basin, Manning's equation can be used with the kerb, gutter and road profile to estimate flow depths at the entry point. Once the flow depths for the minor storm (e.g. 5-year ARI) is estimated, this can be used to calculate the required width of opening in the kerb by applying a broad crested weir equation. This ensures free draining flows into the bioretention basin. The opening width is estimated by applying the flow depth in the gutter (as H) and solving for L (opening width).

$$Q = C.L.H^{3/2} \quad \text{with } C=1.7$$

Equation 5.2

5.3.2.3 Inlet scour protection

It is considered good practice to provide erosion protection for flows as they enter a bioretention basin. Typically velocities will increase as flows drop from the kerb invert into the top of the bioretention soil media. Rock beaching is a simple method for managing these velocities.

5.3.3 Vegetation scour velocity check

Scour velocities over the vegetation are checked through the bioretention basin by assuming the system flows at a depth equal to the ponding depth across the full with of the system. Then by dividing the design flow rate by the cross sectional area, an estimate of flow velocity can be made. It is a conservative approach to assume that all flows pass through the bioretention basin (particularly for a 100 year ARI) however this will ensure the integrity of the vegetation.

Velocities should be kept below:

- 0.5 m/s for 5-year ARI discharges
- 1.0 m/s for 100-year ARI discharges

5.3.4 Size slotted collection pipes

Perforated or slotted collection pipes at the base of bioretention systems collect treated water for conveyance downstream. The collection pipes (there may need to be multiple pipes) should be sized so that the filtration media are freely drained and the collection system does not become a 'choke' in the system.

Treated water that has passed through the filtration media is directed into slotted pipes via a 'drainage layer' (typically fine gravel or coarse sand, 1-5 mm diameter). To convey water from the filtration media and into the perforated pipe, flows must pass through the drainage layer. The purpose of the drainage layer is to efficiently convey treated flows into the perforated pipes while preventing any of the filtration media from being washed downstream.

If gravel is used around the perforated pipes an additional 'transition' layer is recommended to prevent the fine filtration media being washed into the perforated pipes. Typically this is sand to coarse sand (0.7 - 1.0 mm). Alternatively, a geotextile fabric could be used above the drainage layer to prevent finer material from reaching the perforated pipes, however, caution should be taken to ensure this material is not too fine as if it becomes blocked the whole system will require resetting.

Considerations for the selection of a drainage layer include the slot widths in the perforated pipes as well as construction techniques. In addition, where the bioretention system can only have limited depth (e.g. max depth to perforated pipe <0.5m) it will be preferable to install just one drainage layer with a geotextile fabric providing the function of the transition layer.

Installing parallel pipes is a means to increase the capacity of the perforated pipe system. 100 mm diameter is recommended as the maximum size for the perforated pipes to minimise the thickness of the drainage layer. Either flexible perforated pipe (e.g. AG pipe) or slotted PVC pipes can be used, however care needs to be taken to ensure that the slots in the pipes are not so large that sediment would freely flow into the pipes from the drainage layer. This should also be a consideration when specifying the drainage layer media.

<u>DESIGN NOTE</u> – The use of slotted uPVC over the more traditional choice of flexible agricultural pipe (Agriflex) has numerous advantages:

- Increased structural strength resulting in greater filter media depths without failure.
- Consistent grades to maintain self cleansing velocities are more easily maintained.
- Larger drainage slots allow for faster drainage and less risk of blockage thus increasing service life of the filter bed.
- Higher flow capacities therefore requiring lower numbers of pipes.

The maximum spacing of the perforated pipes should be 1.5m (centre to centre) so that the distance water needs to travel through the drainage layer does not hinder drainage of the filtration media.

To ensure the slotted pipes are of adequate size, several checks are required:

- Ensure the perforations are adequate to pass the maximum infiltration rate
- Ensure the pipe itself has capacity
- Ensure that the material in the drainage layer will not be washed into the perforated pipes (consider a transition layer).

These checks can be performed using the equations outlined in the following sections, or alternatively manufacturers' design charts can be adopted to select appropriately sized pipes. Product information may be available from suppliers (for example Vinidex, www.vinidex.com.au; or Iplex, www.iplex.com.au/).

5.3.4.1 Perforations inflow check

To estimate the capacity of flows through the perforations, orifice flow conditions are assumed and a sharp edged orifice equation can be used. Firstly the number and size of perforations needs to be determined (typically from manufacturer's specifications) and used to estimate the flow rate into the pipes using a head of the filtration media depth plus the ponding depth. Secondly, it is conservative but reasonable to use a blockage factor (e.g. 50% blocked) to account for partial blockage of the perforations by the drainage layer media.

$$Q_{perforation} = B \cdot C \cdot A_{perforation} \sqrt{2gh}$$

<u>Equation 5.3</u>

where	B	is the blockage factor (0.5–0.75)
	C	is the orifice coefficient (~0.6)
	A	is the area of the perforation
	h	is depth of water over the collection pipe

The combined discharge capacity of the perforations in the collection pipe should exceed the design discharge of the sand filter unless the specific intention is to increase detention time in the sand filter by limiting the discharge through the collection pipe.

5.3.4.2 Perforated pipe capacity

One form of the Colebrook-White equation can be applied to estimate the velocity and hence flow rate in the perforated pipe. The capacity of this pipe needs to exceed the maximum infiltration rate.

V = -2(2gDSf) 0.5 x log [(k/3.7D) + (2.51v/D(2gDSf)0.5)]

V = Q / A

Therefore

Q = -2(2gDSf) 0.5 x log [(k/3.7D) + (2.51v/D(2gDSf)0.5)] x A

Equation 5.4

Where D = pipe diameter

A = area of the pipe

S_f = pipe slope

k = wall roughness

ν = viscosity

g = gravity constant

5.3.4.3 Drainage layer hydraulic conductivity

The composition of the drainage layer should be considered when selecting the perforated pipe system, as the slot sizes in the pipes may determine a minimum size of drainage layer particle size. Coarser material (e.g. fine gravel) should be used if the slot sizes are large enough that sand will be washed into the slots.

The material size differential should be an order of magnitude between layers to avoid fine material being washed through the voids of a lower layer. Therefore, if fine gravels are used, then a transition layer is recommended to prevent the filtration media from washing into the perforated pipes. The addition of a transition layer increases the overall depth of the bioretention system and may be an important consideration for some sites (therefore pipes with smaller perforations may be preferable where depth of the system is limited by site constraints).

5.3.4.4 Impervious liner requirement

When infiltration is not to be encouraged, stormwater is treated via filtration through a specified soil media with the filtrate collected via a sub-surface drainage system to be either discharge as treated surface flow or collected for reuse. The amount of water lost to surrounding soils is highly dependent on local soils and the hydraulic conductivity of the filtration media in the bioretention system. Typically the hydraulic conductivity of filtration media (sandy loam) is 1-2 orders of magnitude greater than the native surrounding soil profile therefore the preferred flow path is into the perforated underdrainage system.

Where bioretention basins are installed near to significant structures care should be taken to minimise any leakage from the bioretention system. Soil tests of the surrounding soils should be made and the expected hydraulic conductivity estimated (this can be measured with practices described in Chapter 11 Australian Runoff Quality [Engineers Australia, 2006]).

During a detailed design it is considered good practice to provide an impervious liner where the saturated hydraulic conductivity of the surrounding soils is under one order of magnitude less than the filtration media. This is only expected to be required in sandy loam to sandy soils and where infiltration is expected to create problems.

In many roadside applications, a drainage trench runs parallel with the road and will collect any seepage from a bioretention system.

If surrounding soils are very sensitive to exfiltration from the bioretention basin (e.g. sodic soils, shallow groundwater or close proximity to significant structures), an impervious liner can be used to contain all water within the bioretention system. The liner could be a flexible membrane or a concrete casing.

The intention of the lining is to eliminate the risk of exfiltration from a bioretention system. It is considered that the lining of the whole bioretention system in some terrain can be problematic. Fully lined bioretention systems could create sub-surface barriers to shallow groundwater movements. In areas of shallow groundwater any interruption to groundwater movements could increase groundwater levels.

It is considered the greatest risk of exfiltration was through the floor of the bioretention trench. Gravity and the difference in hydraulic conductivity between the filtration media and the surrounding native soil would act to minimise exfiltration through the walls of the trench. To minimise the likelihood of exfiltration from the floor of the bioretention it was concluded the floor of the bioretention should be lined and shaped to ensure the most efficient drainage of the floor of the bioretention basin.

5.3.5 High-flow route and by-pass design

The intention of the high flow design is to convey safely the minor floods (e.g. 5-year ARI flows) to the same level of protection that a conventional stormwater system provides. Bioretention basins are typically served with either grated overflow pits or conventional side entry pits (located downstream of an inlet) to transfer flows into an underground pipe network (the same pipe network that collects treated flows).

The location of the overflow pit is variable but it is desirable to ensure that flows do not pass through extended length of vegetation. Grated pits can be located near the inlet to minimize the flow path length for above design flows. A level of conservatism is built into the design of grated overflow pits by placing their inverts at least 100 mm below the invert of the street gutter (and therefore the maximum ponding depth). This allows the overflow to convey a minor flood prior to any afflux effects in the street gutter. The overflow pit should be sized to pass a five year ARI storm with the available head below the gutter invert (i.e. 100 mm).

Overflow pits can also be located external to bioretention basins, potentially in the kerb and gutter immediately downstream of the inlet to the basin. In this way the overflow pit can operate in the same was as a conventional side entry pit, with flows entering the pit only when the bioretention system is at maximum ponding depth.

To size a grated overflow pit, two checks should be made to check for either drowned or free flowing conditions. A broad crested weir equation can be used to determine the length of weir required (assuming free flowing conditions) and an orifice equation used to estimate the area between opening required (assumed drowned outlet conditions). The larger of the two pit configurations should be adopted. In addition, a blockage factor is be to used that assumes the orifice is 50% blocked.

5. **Weir flow condition** – *when free overall conditions occur over the pit (usually when the extended detention storage of the retarding basin is not fully engaged),* ie.

$$P = \frac{Q_{des}}{B \cdot C_w \cdot H^{1.5}}$$

Equation 5.5

P = Perimeter of the outlet pit

B = Blockage factor (0.5)

H = Depth of water above the crest of the outlet pit

Q_{des} = Design discharge (m^3/s)

C_w = weir coefficient (1.7)

6. **Orifice flow conditions** – *when the inlet pit is completely submerged (corresponding to conditions associated with larger flood events*), ie.

$$A_o = \frac{Q_{des}}{B \cdot C_d \sqrt{2gH}}$$

Equation 5.6

C_d = Orifice Discharge Coefficient (0.6)
B = Blockage factor (0.5)
H = Depth of water above the centroid of the orifice (m)
A_o = Orifice area (m^2)
Q_{des} = Design discharge (m^3/s)

It is important that an outlet pit is prevented from blockage by debris. Design consideration needs to include means of preventing blockage of the outlet structure.

5.3.6 Soil media specification

At least two and possibly three types of soil media are required for bioretention basins.

A filter media layer provides the majority of the treatment function and supports vegetation. It is required to have sufficient depth to support the vegetation, usually between 300-1000 mm.

A drainage layer is used to convey treated flows into the perforated underdrainage pipes. Either coarse sand or fine gravel can be used. The layer should surround the perforated pipes and be 150 or 200 mm thick. Should fine gravel be used, a 100 mm transition layer is recommended that will prevent finer filter media being washed into the perforated pipes.

Materials similar to that given in the sections below should provide adequate substrate for vegetation to grow in and sufficient conveyance of stormwater through the bioretention system.

5.3.6.1 Filter media specifications

The material can be of siliceous or calcareous origin. The material will be placed and lightly compacted. Compaction is only required to avoid subsistence and uneven drainage. The material will periodically be completely saturated and completely drained. The bioretention system will operate so that water will infiltrate into the sediment and move vertically down through the profile. Maintaining the prescribed hydraulic conductivity is crucial.

The material shall meet the geotechnical requirements set out below:

Material - Sandy loam or equivalent material (ie similar hydraulic conductivity, 50-200 mm/hr) free of rubbish and deleterious material.

Particle Size- Soils with infiltration rates in the appropriate range typically vary from sandy loams to loamy sands. Soils with the following composition are likely to have an infiltration rate in the appropriate range - clay 5 - 15 %, silt <30 %, sand 50 - 70 %, assuming the following particle size ranges (clay < 0.002 mm, silt 0.002 - 0.05 mm, sand 0.05 - 2.0 mm).

Soils with majority of particles in this range would be suitable. Variation in large particle size is flexible (ie. an approved material does not have to be screened). Substratum materials should avoid the lower particle size ranges unless tests can demonstrate an adequate hydraulic conductivity ($1\text{-}5\times10^{-5}$ m/s).

Organic Content - between 5% and 10%, measured in accordance with AS1289 4.1.1.

pH - is variable, but preferably neutral, nominal pH 6.0 to pH 7.5 range. Optimum pH for denitrification, which is a target process in this system, is pH 7-8. It is recognised that siliceous materials may have lower pH values.

Any component or soil found to contain high levels of salt, clay or silt particles (exceeding the particle size limits set above), extremely low levels of organic carbon or any other extremes which may be considered retardant to plant growth and denitrification should be rejected.

5.3.6.2 Transition layer specifications

Transition layer material shall be sand/ coarse sand material. A typical particle size distribution is provided below:

% passing	1.4 mm	100 %
	1.0 mm	80 %
	0.7 mm	44 %
	0.5 mm	8.4 %

This grading is based on a Unimin 16/30 FG sand grading.

The transition layer is recommended to be a minimum of 100mm thick. Table 5.1 presents hydraulic conductivities for a range of media sizes (based on D_{50} sizes) that can be applied in either the transition or drainage layers.

Table 5-1. Hydraulic conductivity for a range of media particle sizes (d_{50}).

Soil type	Particle Size (mm)	Saturated Hydraulic Conductivity (mm/hr)	Saturated Hydraulic Conductivity (m/s)
Gravel	2	36000	1×10^{-2}
Coarse Sand	1	3600	1×10^{-3}
Sand	0.7	360	1×10^{-4}
Sandy Loam	0.45	180	5×10^{-5}
Sandy Clay	0.01	36	1×10^{-5}

5.3.6.3 Drainage layer specifications

The drainage layer specification can be either coarse sand (similar to the transition layer) or fine gravel, such as a 2mm or 5 mm screenings.

This layer should be a minimum of 150mm and preferably 200mm thick.

5.3.7 Vegetation specification

Appendix B Plant Lists provides lists of plants that are suitable for bioretention basins. Consultation with landscape architects is recommended when selecting vegetation, to ensure the treatment system compliments the landscape of the area.

5.3.8 Design calculation summary

Bioretention basins **CALCULATION SUMMARY**

	CALCULATION TASK		OUTCOME	CHECK
1	**Identify design criteria**			☐
	conveyance flow standard (ARI)		year	
	area of bioretention		m^2	
	maximum ponding depth		mm	
	Filter media type		mm/hr	
2	**Catchment characteristics**			☐
			m^2	
			m^2	
		slope	%	
	Fraction impervious			☐
3	**Estimate design flow rates**			
	Time of concentration			
	estimate from flow path length and velocities		minutes	☐
	Identify rainfall intensities			
		station used for IFD data:		
		100 year ARI	mm/hr	
		5 year ARI	mm/hr	
	Peak design flows			
		Q_5	m^3/s	
		Q_{100}	m^3/s	
		Q_{infil}	m^3/s	☐
4	**Slotted collection pipe capacity**			
		pipe diameter	mm	
		number of pipes		
		pipe capacity	m^3/s	
		capacity of perforations	m^3/s	
		soil media infiltration capacity	m^3/s	
		CHECK PIPE CAPACITY > SOIL CAPACITY		☐
5	**Check flow widths in upstream gutter**			
		Q_5 flow width	m	
		CHECK ADEQUATE LANES TRAFFICABLE		☐
6	**Kerb opening width**			
		width of brak in kerb for inflows	m	☐
7	**Velocities over vegetation**			
		Velocity for 5 year flow (<0.5m/s)	m/s	
		Velocity for 100 year flow (<1.0m/s)	m/s	☐
8	**Overflow system**			
		system to convey minor floods		☐
9	**Surrounding soil check**			
		Soil hydraulic conductivity	mm/hr	
		Filter media	mm/hr	
		MORE THAN 10 TIMES HIGHER THAN SOILS?		☐
10	**Filter media specification**			
		filtration media		
		transition layer		
		drainage layer		☐
11	**Plant selection**			☐

5.4 Checking tools

This section provides a number of checking aids for designers and referral authorities. In addition, advice on construction techniques and lessons learnt from building bioretention systems are provided.

Checklists are provided for:

- Design assessments
- Construction (during and post)
- Operation and maintenance inspections
- Asset transfer (following defects period).

5.4.1 Design assessment checklist

The checklist below presents the key design features that should be reviewed when assessing a design of a bioretention basin. These considerations include configuration, safety, maintenance and operational issues that should be addressed during the design phase.

Where an item results in an "N" when reviewing the design, referral should be made back to the design procedure to determine the impact of the omission or error.

In addition to the checklist, a proposed design should have all necessary permits for its installations. The referral agency should ensure that all relevant permits are in place. These can include permits to clear vegetation, to dredge, create a waterbody, divert flows or disturb fish or platypus habitat.

Land ownership and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the asset transfer checklist (see 5.4.4).

Bioretention Basin Design Assessment Checklist

Bioretention location:					
Hydraulics	Minor Flood: (m^3/s)		Major Flood: (m^3/s)		
Area	Catchment Area (ha):		Bioretention Area (ha)		

Treatment	Y	N
Treatment performance verified from curves?		

Inlet zone/hydraulics	Y	N
Station selected for IFD appropriate for location?		
Overall flow conveyance system sufficient for design flood event?		
Maximum upstream flood conveyance width does not impact on traffic amenity?		
Velocities at inlet and within bioretention system will not cause scour?		
Bypass sufficient for conveyance of design flood event?		
Bypass has set down of at least 100mm below kerb invert?		

Collection System	Y	N
Slotted pipe capacity > infiltration capacity of filter media?		
Maximum spacing of collection pipes <1.5m?		
Transition layer/geofabric barrier provided to prevent clogging of drainage layer?		

Basin	Y	N
Maximum ponding depth will not impact on public safety?		
Selected filter media hydraulic conductivity > 10x hydraulic conductivity of surrounding soil?		
Maintenance access provided to base of bioretention (where reach to any part of a basin >6m)?		
Protection from gross pollutants provided (for larger systems)?		

Vegetation	Y	N
Plant species selected can tolerate periodic inundation?		
Plant species selected integrate with surrounding landscape design?		
Detailed soil specification included in design?		

5.4.2 Construction advice

This section provides general advice for the construction of bioretention basins. It is based on observations from construction projects around Australia.

Building phase damage

Protection of filtration media and vegetation is important during building phase, uncontrolled building site runoff is likely to cause excessive sedimentation, introduce weeds and litter and require replanting following the building phase. Staged implementation may be used - i.e. during building use geofabric, soil (e.g. 50mm) and instant turf (laid perpendicular to flow path) to provide erosion control and sediment trapping. Following building, remove and revegetate possibly reusing turf at subsequent stages.

Traffic and deliveries

Ensure traffic and deliveries do not access bioretention bains during construction. Traffic can compact the filter media and cause preferential flow paths, deliveries can block filtration media. Washdown wastes (e.g. concrete) can cause blockage of filtration media. Bioretention areas should be fenced off during building phase and controls implemented to avoid washdown wastes.

Inlet erosion checks

It is good practice to check the operation of inlet erosion protection measures following the first few rainfall events. It is important to check for these early in the systems life, to avoid continuing problems. Should problems occur in these events the erosion protection should be enhanced.

Sediment build-up on roads

Where flush kerbs are to be used, a set-down from the pavement surface to the vegetation should be adopted. This allows a location for sediments to accumulate that is off the pavement surface. Generally a set down from kerb of 50mm to the top of vegetation (if turf) is adequate. Therefore, total set down to the base soil is approximately 100 mm (with 50mm turf on top of base soil).

Timing for planting

Timing of vegetation is dependent on a suitable time of year (and potential irrigation requirements) as well as timing in relation to the phases of development. For example temporary planting during construction for sediment control (e.g. with turf) then remove and plant out with long term vegetation.

Planting strategy

A planting strategy for a development will depend on the timing of phases was well as marketing pressure. For example, it may be desirable to plant out several entrance bioretention systems to demonstrate long term landscape values, and use the remainder of bioretention systems as building phase sediment control facilities (to be planted out following building).

Perforated pipes

Perforated pipes can be either PVC pipe with slots cut into the length of it or a flexible ribbed pipe with smaller holes distributed across its surface (an AG pipe). Both can be suitable. PVC pipes have the advantage of being stiffer with less surface roughness therefore greater flow capacity, however the slots are generally larger than for flexible pipes and this may cause problems with filter or drainage layer particle ingress into the pipe. Stiff PVC pipes however can be cleaned out easily using simple plumbing equipment. Flexible perforated pipes have the disadvantage of roughness (therefore reduced flow capacity) however have smaller holes and are flexible which can make installation easier. Blockages within the flexible pipes can be harder to dislodge with standard plumbing tools.

Inspection openings

It is good design practice to have inspection openings at the end of the perforated pipes. The pipes should be brought to the surface and have a sealed capping. This allows inspection of sediment buildup and water level fluctuations when required and easy access for maintenance. The vertical component of the pipe should not be perforated otherwise short circuiting can occur.

Clean filter media

Ensure drainage media is washed prior to placement to remove fines.

5.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Bioretention basins

INSPECTED BY:
DATE:
TIME:
WEATHER:
CONTACT DURING VISIT:

SITE: ____________________

CONSTRUCTED BY: ____________________

DURING CONSTRUCTION									
Items inspected	Checked		Satisfactory	Unsatisfactory		Checked		Satisfactory	Unsatisfactory
Preliminary works	Y	N			**Structural components**	Y	N		
1. Erosion and sediment control plan adopted					15. Location and levels of pits as designed				
					16. Safety protection provided				
2. Traffic control measures					17. Pipe joints and connections as designed				
3. Location same as plans					18. Concrete and reinforcement as designed				
4. Site protection from existing flows					19. Inlets appropriately installed				
Earthworks					20. Inlet erosion protection installed				
5. Bed of basin correct shape					21. Set down to correct level for flush kerbs				
6. Batter slopes as plans					**Vegetation**				
7. Dimensions of bioretention area as plans					22. Stablisation immediately following earthworks				
8. Confirm surrounding soil type with design									
9. Provision of liner					23. Planting as designed (species and densities)				
10. Perforated pipe installed as designed									
11. Drainage layer media as designed					24. Weed removal before stabilisation				
12. Transition layer media as designed									
13. Filter media specifications checked									
14. Compaction process as designed									
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of soil				
2. Traffic control in place					7. Inlet erosion protection working				
3. Confirm structural element sizes					8. Maintenance access provided				
4. Check batter slopes					9. Construction generated sediment removed				
5. Vegetation as designed									

COMMENTS ON INSPECTION

ACTIONS REQUIRED

1.
2.
3.
4.
5.
6.

5.4.4 Asset transfer checklist

Asset Handover Checklist		
Asset Location:		
Construction by:		
Defects and Liability Period		
Treatment	**Y**	**N**
System appears to be working as designed visually?		
No obvious signs of under-performance?		
Maintenance	**Y**	**N**
Maintenance plans provided for each asset?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
Asset inspected for defects?		
Asset Information	**Y**	**N**
Design Assessment Checklist provided?		
As constructed plans provided?		
Copies of all required permits (both construction and operational) submitted?		
Proprietary information provided (if applicable)?		
Digital files (eg drawings, survey, models) provided?		
Asset listed on asset register or database?		

5.5 Maintenance requirements

Bioretention basins treat runoff by filtering it through vegetation and then passing the runoff vertically through a filtration media which filters the runoff. Besides vegetative filtration, treatment relies upon infiltration of runoff into an underdrain. Vegetation plays a key role in maintaining the porosity of the surface of the filter media and a strong healthy growth of vegetation is critical to its performance.

The most intensive period of maintenance is during the plant establishment period (first two years) when weed removal and replanting may be required. It is also the time when large loads of sediments could impact on plant growth particularly in developing catchments with poor building controls.

Maintenance is primarily concerned with:

- Maintenance of flow to and through the bioretention basin
- Maintaining vegetation
- Preventing undesired overgrowth vegetation from taking over the bioretention basin
- Removal of accumulated sediments
- Litter and debris removal

Vegetation maintenance will include:

- Fertilising plants
- Removal of noxious plants or weeds
- Re-establishment of plants that die

Sediment accumulation at the inlets needs to be monitored. Depending on the catchment activities (e.g. building phase) the deposition of sediment can smother plants and reduce the ponding depth available. Excessive sediment build up will impact on plant health and require removal before it reduces the infiltration rate of the filter media.

Similar to other types of practices, debris removal is an ongoing maintenance function. Debris, if not removed, can block inlets or outlets, and can be unsightly if located in a visible location. Inspection and removal of debris should be done regularly, but debris should be removed whenever it is observed on the site.

5.5.1 Operation & maintenance inspection form

The form below should be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

Bioretention Basin Maintenance Checklist			
Inspection Frequency: 3 monthly	Date of Visit:		
Location:			
Description:			
Site Visit by:			
Inspection Items	**Y**	**N**	**Action Required (details)**
Sediment accumulation at inflow points?			
Litter within basin?			
Erosion at inlet or other key structures (eg crossovers)?			
Traffic damage present?			
Evidence of dumping (eg building waste)?			
Vegetation condition satisfactory (density, weeds etc)?			
Replanting required?			
Mowing required?			
Clogging of drainage points (sediment or debris)?			
Evidence of ponding?			
Damage/vandalism to structures present?			
Surface clogging visible?			
Drainage system inspected?			
Resetting of system required?			
Comments:			

5.6 Bioretention basin worked example

5.6.1 Worked example introduction

A series of bioretention basins, designed as street traffic parking “out-stands” is to be retrofitted into a local street to treat road runoff. The local street is in inner Hobart. A proposed layout of the bioretention system is shown in Figure 5.6 and an image of a similar system to that proposed is shown in Figure 5.7.

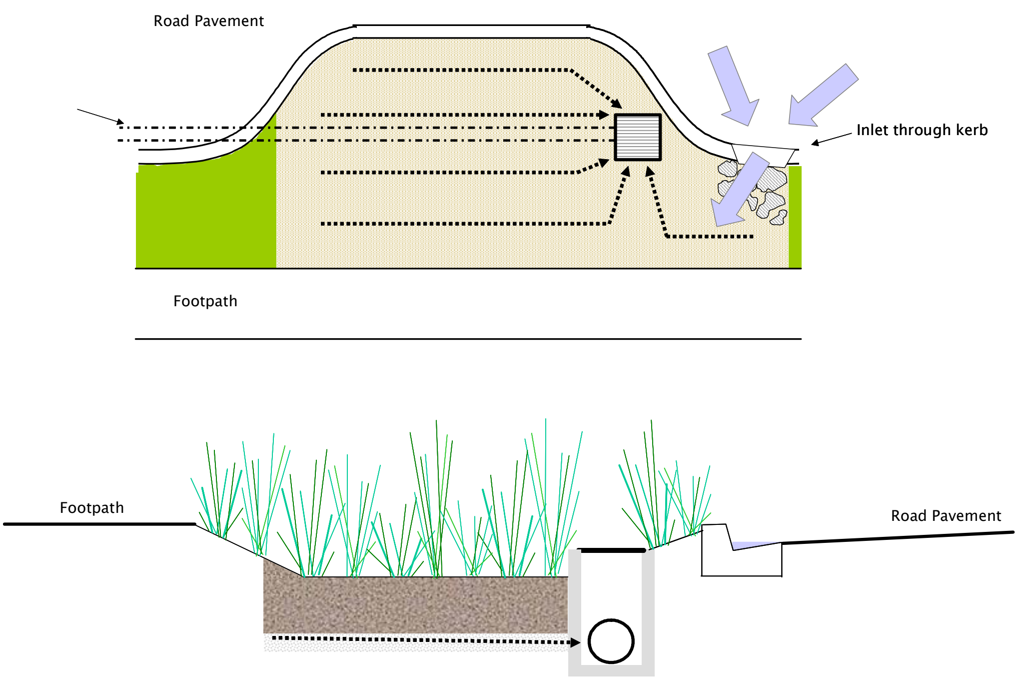


Figure 5.6General Layout and Cross Section of Proposed Bioretention System

Figure 5.7. Retrofitted Bioretention System in a street

The contributing catchment areas to each of the individual bioretention basins consist of 300 m² of road and footpath pavement and 600 m² of adjoining properties. Runoff from adjoining properties (approx. 60% impervious) is discharged into the road gutter and, together with road runoff, is conveyed along a conventional roadside gutter to the bioretention cell.

The aim of the design is to facilitate effective treatment of stormwater runoff while maintaining a 5-year ARI level of flood protection for the local street. The actual size of the cell may however, be increased to suit other streetscape objectives. The maximum width (measured perpendicular to the alignment of the road) of the bioretention basin is to be two metres. Analyses to detail the operation of the bioretention basin are shown below and demonstrate the design procedures. The analyses include:

- Road and gutter details to convey water into the basin
- Detailing inlet conditions to provide for erosion protection
- Configuring and designing a system for above-design operation that will provide the required 5-year ARI flood protection for the local street
- Sizing of below ground drainage system
- Specification of the soil filtration medium
- Landscape layout and details of vegetation

5.6.1.1 Design Objectives

Treatment to meet current best practice objectives of 80%, 45% and 45% reductions of TSS, TP and TN respectively whilst maintaining a 5 year ARI level of flood protection for the local street.

5.6.1.2 Constraints and Concept Design Criteria

Analyses during a concept design determined the following criteria:

- A bioretention basin area of 10 m^2 (minimum) is required to achieve the water quality objectives.
- The maximum width of the bioretention basin is to be 2m.
- The extended detention depth is 200 mm.
- Filter media shall be a sandy loam.

5.6.1.3 Site Characteristics

• Land Use	Urban, paved carpark and footpaths, lots.	
• Overland flow slope	1% typical.	
• Soil	Assumed to be clay	
• Catchment Area	Carpark	= 300 m^2
	Lots	= 600 m^2
• Fraction Impervious	Carpark	= 0.90

Lots = 0.60

5.6.2 Confirm size for treatment

Interpretation of Figures 5.3 to 5.5 with the input parameters below is used to estimate the reduction performance of the bioretention basin for the three pollutants.

- Hobart location
- 200mm extended detention
- treatment area to impervious area ration of:

  $10m^2 / [(0.9x300) + (0.6x600)]m^2 = 1.60\%$

From the graphs, the expected pollutant reductions are 85%, 69% and 45% for TSS, TP and TN respectively and exceed the design requirements of 80%, 45% and 45%.

DESIGN NOTE – The values derived from 5.2 Verifying size for treatment will only be valid if the design criteria for the proposed installation are similar to those used to create the Figures. Site specific modelling using programs such as MUSIC may yield a more accurate result.

5.6.3 Estimating design flows

5.6.3.1 Major and minor design flows

With a small catchment the Rational Method is considered an appropriate approach to estimate the 5 and 100 year ARI peak flow rates. The steps in these calculations follow below.

See Appendix E Design Flows – tc for a discussion on methodology for calculation of time of concentration.

Step 1 – Calculate the time of concentration.

DESIGN NOTE – See Sand Filters chapter section 6.6.3 for more information on Tc.

$$Tc = \frac{91 \times 0.015}{0.063^{0.1} \times 10^{0.2}}$$

$$= 1.365 / 1.202$$

$$= 1.135 \text{ minutes}$$

- Gutter flow: adopt flow path length of 50 m to bioretention.

  Assume gutter velocity = 1m/s

  Flow time = 50/1 = 50sec

Adopt t_c = 1.135 + 0.8 = 1.95 min Assume minimum of 6 minutes

Design rainfall intensities

- Using a time of concentration of 6 minutes, the design rainfall intensities from the IFD chart relevant to the catchment location are –

	5yr	100yr
Intensity (mm/hr)	72+	150+

* These figures are for the worked example only. The appropriate region and corresponding rainfall intensities must be selected for each individual project.

Step 2 – Calculate design run-off coefficients (using the method outlined in Australian Rainfall and Runoff Book VIII (Engineers Australia, 2003)).

Where – Fraction impervious (f) = 0.9

Rainfall intensity ($^{10}I_{1}$) = 28.6mm/hr (from the relevant IFD chart)

Calculate C^{1}_{10} (pervious run-off coefficient)

$$C^{1}_{10} = 0.1 + 0.0133\ (^{10}I_1 - 25) = 0.15$$

Calculate C_{10} (10 year ARI run-off coefficient)

$$f = 0.06 \times 0.6 + 0.03 \times \frac{0.90}{(0.06+.03)}$$

$$= 0.70$$

$$C_{10} = 0.9f + C^{1}_{10}\ (1-f)$$

$$C_{10} = 0.67$$

Step 3 – Convert C_{10} to values for C_5 and C_{100}

Where – $C_y = F_y \times C_{10}$

From Table 1.6 in Australian Rainfall and Runoff – Book VII;

$$C_5 = 0.95 \times C_{10} = 0.64$$

$$C_{100} = 1.2 \times C_{10} = 0.81$$

Step 4 – Calculate peak design flow (calculated using the Rational Method).

$$Q = \frac{CIA}{360}$$

Where – C is the runoff coefficient (C_5 and C_{100})

I is the design rainfall intensity mm/hr (I_5 and I_{100})

A is the catchment area (Ha)

Q_5 = 0.012 m^3/s (16 L/s)

Q_{100} = 0.030 m^3/s (93 L/s)

5.6.3.2 Maximum infiltration rate

The maximum infiltration rate (Q_{max}) through the sand filter is computed using Darcy's equation, i.e.

$$Q_{max} = k \cdot A \cdot \frac{h_{max} + d}{d} \quad = 0.0067 \text{ m}^3\text{/s}$$

where k is the hydraulic conductivity of sand = 5 x10^{-5} m/s (Engineers Australia, 2003, Ch. 9)

A is the surface area of the sand filter = 10 m^2

h_{max} is the depth of pondage above the sand filter = 0.2 m

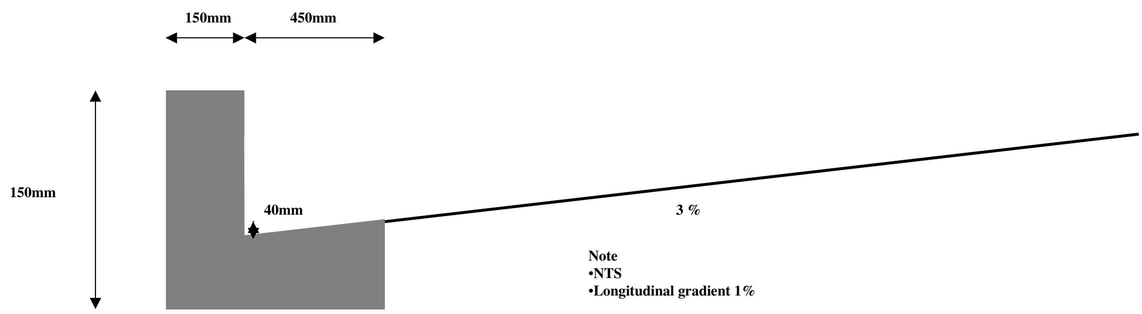
d is the depth of the sand filter = 0.6 m

5.6.3.3 Inlet details

Flow width at entry

A check of the flow capacity of the system and the width of the flow across the road needs to be performed to ensure the road is protected to council standards for a minor (5 year ARI) flood. In this case council has a criterion of having less than 2 metre wide flow in the gutter, which facilitates one trafficable lane during a minor flood.

Adopt following kerb, gutter and road profile, with a longitudinal gradient of 1% along the gutter the following flow and depth estimates can be made using the Manning's equation.



- Check flow capacity and width of flow
- Assume uniform flow conditions, estimate by applying Manning's equation

$Q_{5\ Year}$ = 0.012 m^3/s	Depth of Flow	= 55 mm
	Width of Flow	= 900 mm (within gutter)
	Velocity	= 0.6 m/s (within gutter)

The estimated peak flow width during the $Q_{5\ Year}$ storm event is appropriate for the development (<2.0 m during minor storm flow).

Q_{100} = 0.030 m^3/s	Depth of Flow	= 70 mm
	Width of Flow	= 1.45 m (within gutter)
	Velocity	= 0.8 m/s (within gutter)

Kerb opening at entry

The flow depth in the gutter estimated above is used to determine the required width of opening in the kerb to allow for flows to freely flow into the bioretention system.

Q_5 = 0.012 m^3/s

Assume broad crested weir flow conditions through the slot

$Q_{minor} = B.C.L.H^{3/2}$

with $B = 1.0$,

$C = 1.7$ and

H (Flow depth Q_5) = 0.055 and solving for L

$\therefore$ $L = Q_5/(CH^{3/2}) = (0.012)/(1.7 \times 0.055^{3/2}) = 0.55$ m

Therefore adopt a 0.6m wide opening in the kerb at the inlet.

Inlet scour protection

Rock beaching is to be provided at the inlet to manage flow velocities from the kerb and into the bioretention system. This detail is shown on the diagrams.

5.6.4 Vegetation scour velocity check

Assume Q_5 and Q_{100} will be conveyed through the bioretention system. Check for scouring of the vegetation by checking that velocities are below 0.5m/s during Q_5 and 1.0 m/s for Q_{100}.

Width of bioretention = 2 m

Extended detention depth = 0.2 m

Area = 2 x 0.2 = 0.4 m^2

Q_5 average velocity = 0.012/0.4 = 0.03 m/s < 0.5 m/s – therefore OK

Q_{100} average velocity = 0.03/0.4 = 0.08 m/s < 1.0m/s – therefore OK

Hence, bioretention system can satisfactorily convey the peak 5 and 100-year ARI flood, minimising the potential for scour.

5.6.5 Size perforated collection pipes

5.6.5.1 Perforations inflow check

Estimate the inlet capacity of sub-surface drainage system (perforated pipe) to ensure it is not a choke in the system. To build in conservatism, it is assumed that 50% of the holes are blocked. A standard perforated pipe was selected that is widely available. To estimate the flow rate an orifice equation is applied using the following parameters:

Head = 0.85 m [0.6 m (filter depth) + 0.2 m (max. pond level) + 0.05 (half of pipe diameter)]

The following are the characteristics of the selected slotted pipe

- Clear openings = 2100 mm^2/m
- Slot width = 1.5mm
- Slot length = 7.5mm
- No. rows = 6
- Diameter of pipe = 100mm

For a pipe length of 1.0 m, the total number of slots = 2100/(1.5 x 7.5) = 187.

Discharge capacity of each slot can be calculated using the orifice flow equation, i.e.

$$Q_{perforation} = C \cdot A_{perforation}\sqrt{2gh} \quad = 2.67 \times 10^{-5}\ m^3/s$$

where h is the head above the slotted pipe, calculated to be 0.80 m.

C is the orifice coefficient (~0.6)

The inflow capacity of the slotted pipe is thus $2.67 \times 10^{-5} \times 187 = 5 \times 10^{-3}$ $m^3/s/m$-length

Adopt a blockage factor of 0.5 gives the inlet capacity of each slotted pipe to be 2.5×10^{-3} $m^3/s/m$-length.

Inlet capacity/m x total length = 0.0025 x 5 = 0.0125 m^3/s > 0.0067 (max infiltration rate), hence OK.

5.6.5.2 Perforated pipe capacity

The Colebrook-White equation is applied to estimate the flow rate in the perforated pipe. A slope of 0.5% is assumed and a 100mm perforated pipe (as above) was used. Should the capacity not be sufficient, either a second pipe could be used or a steeper slope. The capacity of this pipe needs to exceed the maximum infiltration rate.

Estimate applying the Colebrook-White Equation

$$Q = -2(2gDS_f)^{0.5} \times \log\left[(k/3.7D) + (2.51\nu/D(2gDS_f)^{0.5})\right] \times A$$

Where D = pipe diameter

A = area of the pipe

S_f = pipe slope

k = wall roughness

ν = viscosity

g = gravity constant

Total discharge capacity = 0.019 m^3/s > maximum infiltration rate of 0.0067 m^3/s ➔ OK

Adopt 1 x ϕ 100 mm perforated pipe for the underdrainage system.

5.6.5.3 Drainage layer hydraulic conductivity

Typically flexible perforated pipes are installed using fine gravel media to surround them. In this case study 5mm gravel is specified for the drainage layer. This media is much coarser than the filtration media (sandy loam) therefore to reduce the risk of washing the filtration later into the perforated pipe a transition layer is to be used. This is to be 100 mm of coarse sand.

5.6.5.4 Impervious liner requirement

In this catchment the surrounding soils are clay to silty clays with a saturate hydraulic conductivity of approximately 3.6 mm/hr. The sandy loam media that is proposed as the filter media has a hydraulic conductivity of 50-200 mm/hr. Therefore the conductivity of the filter media is > 10times the conductivity of the surrounding soils and an impervious liner is not considered to be required.

5.6.6 High flow route and by-pass design

The overflow pit is required to convey 5 year ARI flows safely from above the bioretention system into an underground pipe network. Grated pits are to be used at the upstream end of the bioretention system.

The size of the pits are calculated using a broad crested weir equation with the height above the maximum ponding depth and below the road surface, (i.e. 100mm).

First check using a broad crested weir equation

$$P = \frac{Q_{des}}{B \cdot C_w \cdot H^{1.5}}$$

P = Perimeter of the outlet pit

B = Blockage factor (0.5)

H = 0.1m Depth of water above the crest of the outlet pit

Q_{des} = Design discharge (m^3/s)

C_w = weir coefficient (1.7)

Gives P = .44m of weir length required (equivalent to 115 x 115mm pit)

Now check for drowned conditions:

$$A_o = \frac{Q_{des}}{B \cdot C_d \sqrt{2gH}}$$

C_d = Orifice Discharge Coefficient (0.6)

B = Blockage factor (0.5)

H = Depth of water above the centroid of the orifice (0.1m)

A_o = Orifice area (m^2)

Q_{des} = Design discharge (m^3/s)

gives A = 0.16 m^2 (equivalent to 170 x 170 pit)

Hence, drowned outlet flow conditions dominate, adopt pit sizes of 600 x 600 mm for this systems as this is minimum pit size to accommodate underground pipe connections.

5.6.7 Soil media specification

Three layer of soil media are to be used. A sandy loam filtration media (600mm) to support the vegetation, a coarse transition layer (100mm) and a fine gravel drainage layer (200mm). Specifications for these are below.

5.6.7.1 Filter media specifications

The filter media is to be a sandy loam with the following criteria:

The material shall meet the geotechnical requirements set out below:

- hydraulic conductivity between 50–200 mm/hr
- particle sizes of between: clay 5 – 15 %, silt <30 %, sand 50 – 70 %
- between 5% and 10% organic content, measured in accordance with AS1289 4.1.1.
- pH neutral

5.6.7.2 Transition layer specifications

Transition layer material shall be coarse sand material such as Unimin 16/30 FG sand grading or equivalent. A typical particle size distribution is provided below:

% passing	1.4 mm	100 %
	1.0 mm	80 %
	0.7 mm	44 %
	0.5 mm	8.4 %

5.6.7.3 Drainage layer specifications

The drainage layer is to be 5 mm screenings.

5.6.8 Vegetation specification

With such a small system it is considered to have a single species of plants within the bioretention system. For this application a Tall Sedge (*Carrex appressa*) is proposed with a planting density of 8 plants /m^2. More information on maintenance and establishment is provided in Appendix B.

5.6.9 Calculation summary

The sheet below shows the results of the design calculations.

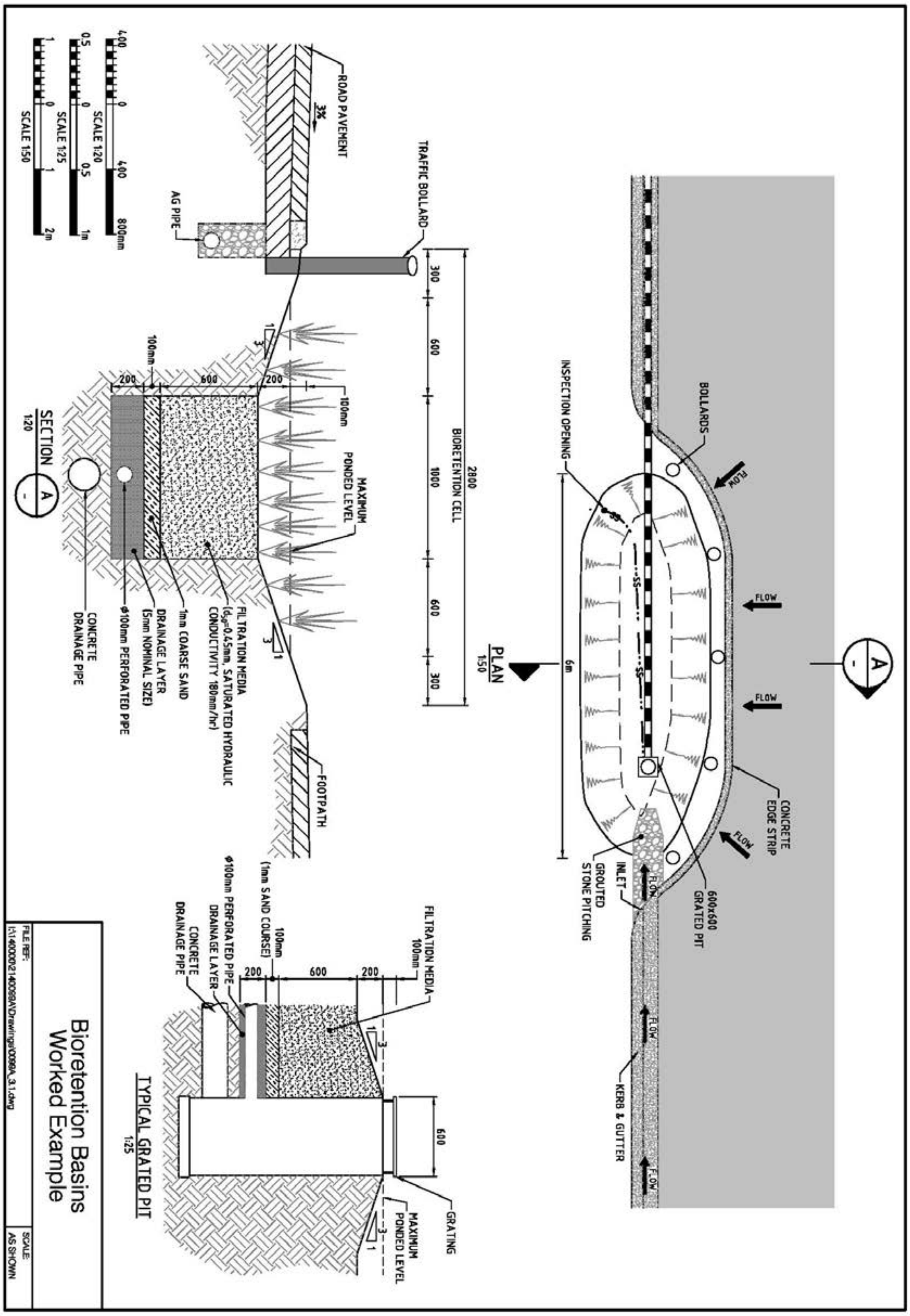
Bioretention basins

CALCULATION SUMMARY

	CALCULATION TASK		OUTCOME		CHECK
1	**Identify design criteria**				✓
	conveyance flow standard (ARI)		5	year	
	area of bioretention		10	m^2	
	maximum ponding depth		200	mm	
	Filter media type		180	mm/hr	
2	**Catchment characteristics**				✓
		car park area	300	m^2	
		allotment area	600	m^2	
		slope	1	%	
	Fraction impervious				✓
		car park	0.9		
		allotments	0.6		
3	**Estimate design flow rates**				
	Time of concentration				
	estimate from flow path length and velocities		6	minutes	✓
	Identify rainfall intensities				
		station used for IFD data:	Hobart		
		100 year ARI	150	mm/hr	
		5 year ARI	72	mm/hr	
	Peak design flows				
		Q_5	0.012	m^3/s	
		Q_{100}	0.030	m^3/s	
		Q_{infil}	0.0003	m^3/s	✓
4	**Slotted collection pipe capacity**				
		pipe diameter	100	mm	
		number of pipes	1		
		pipe capacity	0.019	m^3/s	
		capacity of perforations	0.0125	m^3/s	
		soil media infiltration capacity	0.0067	m^3/s	
		CHECK PIPE CAPACITY > SOIL CAPACITY	YES		✓
5	**Check flow widths in upstream gutter**				
		Q_5 flow width	0.9	m	
		CHECK ADEQUATE LANES TRAFFICABLE	YES		✓
6	**Kerb opening width**				
		width of brak in kerb for inflows	0.6	m	✓
7	**Velocities over vegetation**				
		Velocity for 5 year flow (<0.5m/s)	0.03	m/s	
		Velocity for 100 year flow (<1.0m/s)	0.08	m/s	✓
8	**Overflow system**				
		system to convey minor floods	grated pit 600 x 600		✓
9	**Surrounding soil check**				
		Soil hydraulic conductivity	0.36	mm/hr	
		Filter media	180	mm/hr	
		MORE THAN 10 TIMES HIGHER THAN SOILS?	YES (no liner)		✓
10	**Filter media specification**				
		filtration media	sandy loam		
		transition layer	coarse sand		
		drainage layer	fine gravel		✓

5.6.9.1 Construction drawings

The diagram below shows the construction drawing for the worked example.



5.7 References

Engineers Australia, 2006, *Australian Runoff Quality Australian Runoff Quality: A guide to Water Sensitive Urban Design, Editor-in-Chief, Wong, T.H.F.*

eWater, 2009, Model for Urban Stormwater Improvement Conceptualisation (MUSIC) User Manual, Version 4.0, September.

Institution of Engineers Australia, 1997, *Australian Rainfall and Runoff – A guide to flood estimation*, Editor in Chief – Pilgram, D.H.

Chapter 6 Sand Filters

Definition:

A sand filter is a filter bed used to remove pollutants.

Purpose:

- To capture gross pollutants
- To retain coarse sediments
- Fine filtration of flows

Implementation considerations:

- They are particularly useful in areas where space is a premium and treatment is best achieved underground
- Due to the absence of vegetation, they require regular maintenance to ensure the surface of the sand filter media remains porous and does not become clogged with accumulated sediments.
- Prior to entering a sand filter, flows are generally subjected to a pretreatment to remove litter, debris and coarse sediments (typically a sedimentation chamber).
- Sand filters operate in a similar manner to bioretention systems with the exception that they have no vegetation growing on their surface. This is because they are either installed underground (therefore light limits vegetation growth) or the filter media does not retain sufficient moisture.

Sand filters can be installed above or below ground

6.1 Introduction 6-3

6.2 Verifying size for treatment .... 6—5

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6.5 Maintenance requirements .... 6—20

6.6 Sand filter worked example .... 6—22

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6.1 Introduction

Sand filters operate in a similar manner as bioretention systems with the exception that they do not support any vegetation owing to the filtration media being too free-draining (and therefore dries out too frequently to support vegetation). Their use in stormwater management is suited to confined spaces and where vegetation cannot be sustained (e.g. underground). They are particularly useful treatment devices in heavily urbanized and built up areas.

Key design considerations include the provision of detention storage to yield a high hydrologic effectiveness (i.e. allowing for extended detention above the filter media), discharge control by proper sizing of the perforated underdrain and overflow pathway for above design operation.

A sand filter system typically consists of three chambers as illustrated below.

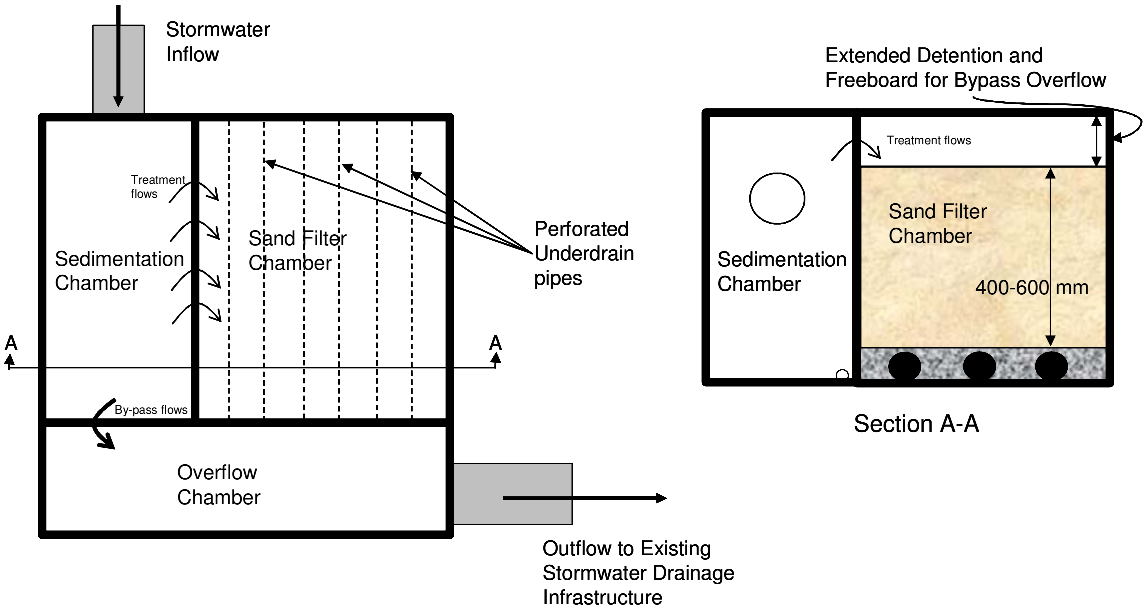


Figure 6.1. Typical layout of a sand filter

Functionality

Water enters a sedimentation chamber either via a conventional side entry pit or through an underground pipe network, where gross pollutants and coarse to medium-sized sediment is retained. This chamber can be designed to either have permanent water storage between events or to drain between storm events via weep holes.

Stormwater overflows from the sedimentation chamber into a sand filter chamber via a weir.

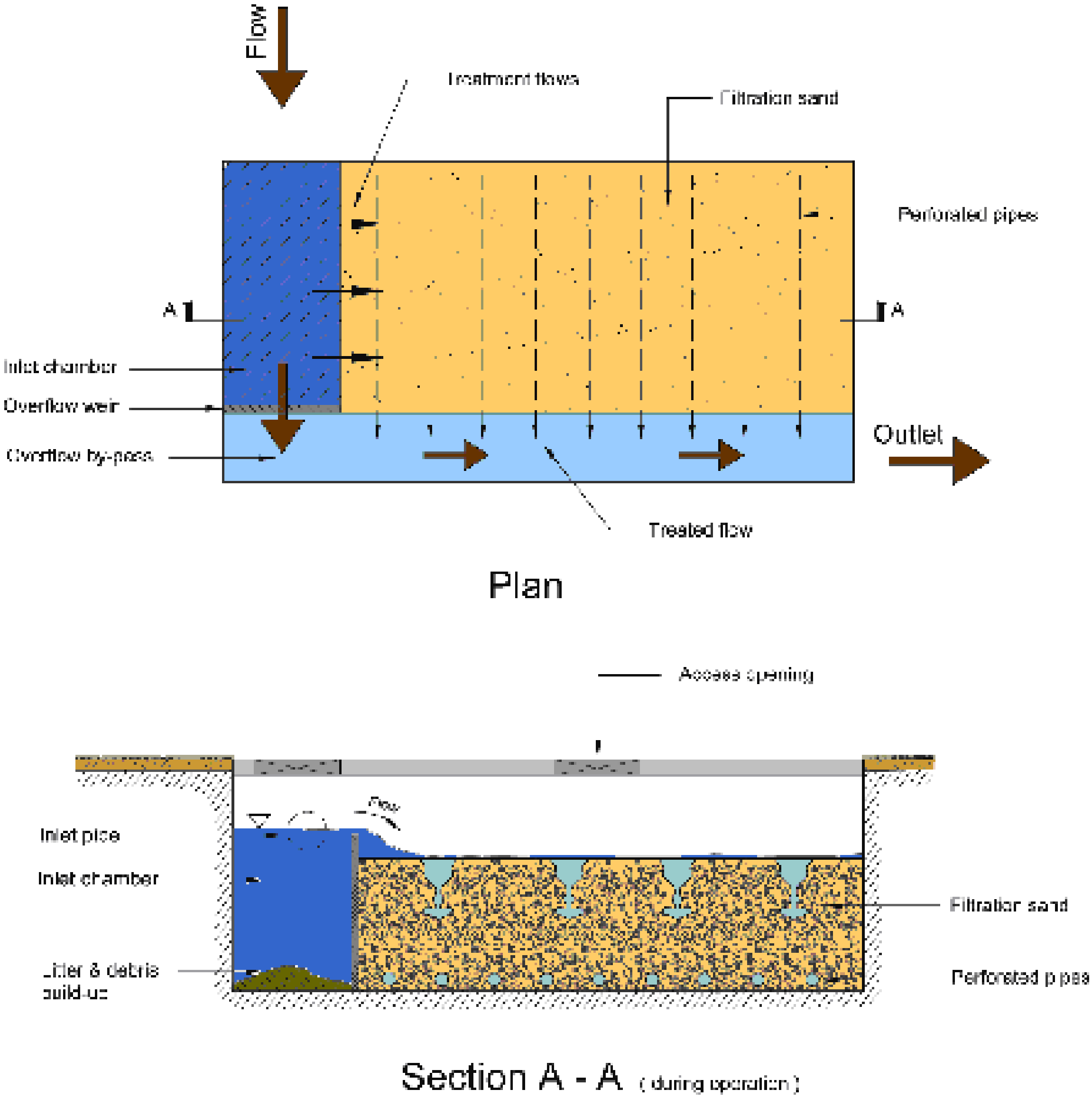
Water percolates through the sand filtration media (typically 400-600 mm depth) and perforated under-drain pipes collect filtered water in a similar manner as in bioretention systems.

There are advantages and disadvantages with either approach –

	Advantages	*Disadvantages*
PERMANENT WATER STORAGE	• Reduces the likelihood of re-suspension of sediments at the start of the following rainfall event as inflows do not fall and scour collected sediments • Minimised potential for mosquito breeding because of the likelihood of sufficient surface oil on incoming flows to prevent larval growth.	• System requires the removal of wet material from the sedimentation chamber during maintenance. • The high organic loads and stagnant water can lead to anaerobic conditions that can also lead to release of soluble pollutants (such as phosphorous). Release of these bio-available pollutants can cause water quality problems downstream (such as excessive algal growth).
FREE DRAINING	• Allowing the sedimentation chamber to drain during inter-event periods (by installation of weep holes) reduces the likelihood of pollutant transformation during the inter-event period.	• The challenge with this type of system is to design weep holes such that they can continue to drain as material (litter, organic material and sediment) accumulates and the holes do not block.

Figure 6.2 shows a sand filter in Auckland and Figure 6.3 shows an illustration of how a sand filter may be configured and operates during storm events.

Figure 6.2. Underground sand filter for a car park in Auckland, New Zealand



<u>**Figure 6.3. Illustration of a sand filter during operation**</u>

Key functions of a sand filter include the following:

- capture of gross pollutants
- sedimentation of particles larger than 125 μm within a sedimentation chamber for flows up to a 1 year ARI (unattenuated) peak discharge
- filtration of stormwater following sedimentation pre-treatment through a sand filtration layer.

6.2 Verifying size for treatment

The graphs below show expected performance of sand filters for retention of TSS, TP and TN respectively. These curves were derived using MUSIC (eWater, 2009) an assumed sand filter depth of 600 mm. Note, Melbourne hydrological data was used in developing these curves for the sizing of sand filters.

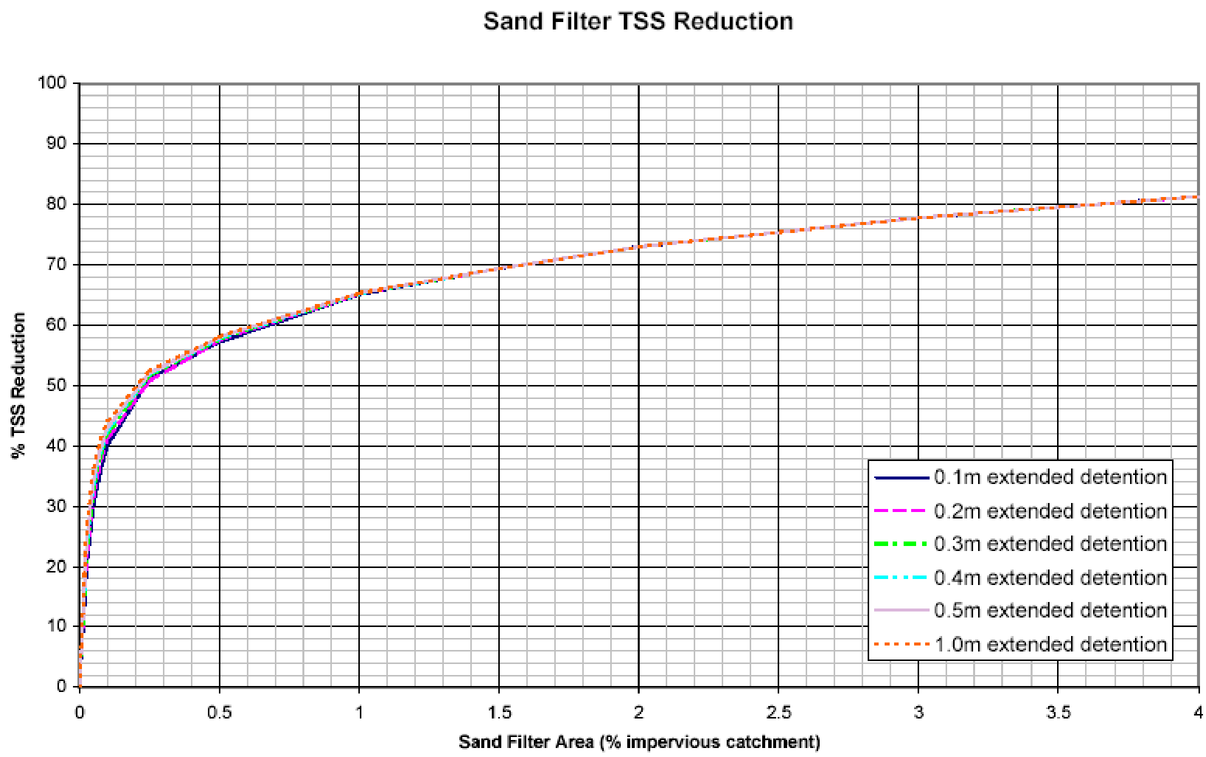


Figure 6.4. Sand Filter TSS removal performance

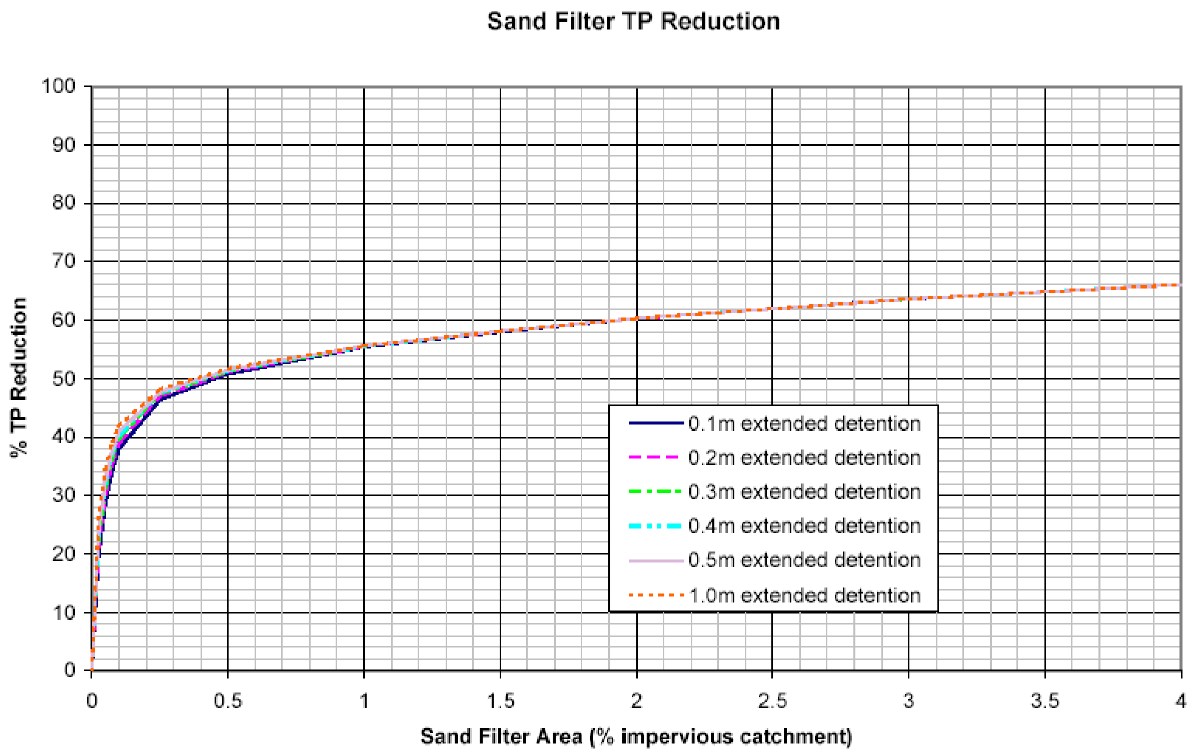


Figure 6.5. Sand Filter TP removal performance

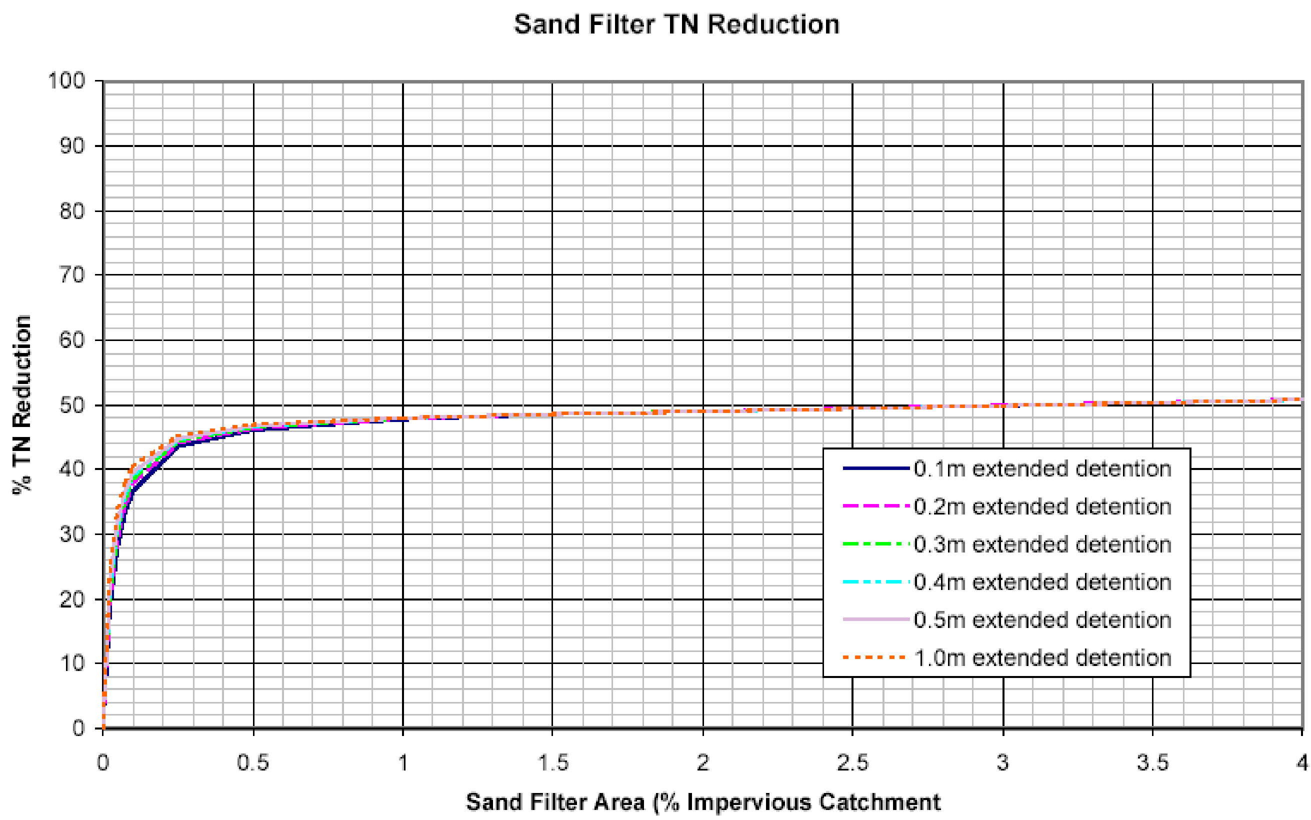


Figure 6.6. Sand Filter TN removal performance

6.3 Design procedure: sand filters

The following sections detail the design steps required for sand filters.

6.3.1 Estimating design flows

Three design flows are required for sand filters:

- Sedimentation chamber design flow – this would normally correspond to the 1 year ARI peak discharge as standard practice for sedimentation basins
- Sand filter design flow – this is the product of the maximum infiltration rate and the surface area of the sand filter, and is used to determine the minimum discharge capacity of the under-drains to allow the filter media to freely drain
- Overflow chamber design flow – this would normally correspond to the minor drainage system (typically 5 or 10-year ARI) to size the weir connecting the sand filter to the overflow chamber. This allows minor floods to be safely conveyed and not increase any flooding risk compared to conventional stormwater systems.

6.3.1.1 Minor and major flood estimation

A range of hydrologic methods can be applied to estimate design flows. With typical catchment areas being relatively small, the Rational Method Design Procedure is considered to be a suitable method for estimating design flows.

6.3.1.2 Maximum infiltration rate

The maximum infiltration rate represents the design flow for the under-drainage system (i.e. the slotted pipes at the base of the filter media). The capacity of the under-drains needs to be greater than the maximum infiltration rate to ensure the filter media drains freely and the pipe doesn't become a 'choke' in the system.

A maximum infiltration rate (Q_{max}) can be estimated by applying Darcy's equation:

$$Q_{\max} = k \cdot A \cdot \frac{h_{\max} + d}{d}$$

Equation 6.1

where k is the hydraulic conductivity of the soil filter (m/s)

A is the surface area of the sand filter (m^2)

h_{max} is the depth of pondage above the sand filter (m)

d is the depth of the filter media (m)

6.3.2 Hydraulic Structure details

6.3.2.1 Sedimentation Chamber

Inlet into the sand filter is via the sedimentation chamber. The dimension of this chamber should be sized to retain sediment larger than 125 μm for the design flow and to have adequate capacity to retain settled sediment such that the cleanout frequency is once a year or longer. A target sediment capture efficient of 70% is recommended. This is lower than would be recommended for sedimentation basins that do not form part of a sand filter (see Chapter 3 for Sediment Basin design).

The lower capture efficiencies can be supported partly due to the required maintenance regime of the filter media and particle size range in the filter being of a similar order of magnitude as the target sediment size of 125 μm.

An inspection of the filter media should be carried out every 6 months and in particular after significant rainfall events to ensure the sediment and litter loads can be controlled by the sedimentation chamber.

Inspections of the sedimentation chamber should be performed at similar intervals however sediment clean out may only be required once every year. This will vary from site to site and records of inspections should be kept from each inspection (See Section 7.5.1).

It is necessary to check that deposited sediments of the target sediment size or larger are not resuspended during the passage of the design peak discharge for the overflow chamber. A maximum flow velocity of 0.2 m/s is recommended.

The reader is referred to Chapter 3 for guidance on the sizing the sedimentation chamber.

6.3.2.2 Sand Filter Chamber

The filter media in the sand filter chamber consist of two layers, being a drainage layer consisting of gravel size material to encase the perforated under-drains and the sand filtration layer. The surface of the sand filter should be set at the crest height of the weir connecting the sedimentation chamber to the sand filter chamber. This will minimise any scouring of the sand surface as water is conveyed into the sand filter chamber.

6.3.2.2.1 Filter media specifications

A range of particle size ranges can be used for sand filters depending on the likely size of generated sediments. Material with particle size distributions described below has been reported as effective for stormwater treatment (ARC, 2003):

% passing	9.5 mm	100 %
	6.3 mm	95-100 %
	3.17 mm	80-100 %
	1.5 mm	50-85 %
	0.8 mm	25-60 %
	0.5 mm	10-30 %
	0.25 mm	2-10 %

This grading is based on TP10 (ARC, 2003).

Alternatively finer material can be used (such as that below), however, it requires more attention to maintenance to ensure the material maintains its hydraulic conductivity and does not become blocked. Inspections should be carried out every 3-months during the initial year of operation as well as after major storms to check for surface clogging.

% passing	1.4 mm	100 %
	1.0 mm	80 %
	0.7 mm	44 %
	0.5 mm	8.4 %

This grading is based on a Unimin 16/30 FG sand grading.

6.3.2.2.2 Drainage layer specifications

The drainage layer specification can be either coarse sand or fine gravel, such as a 5 mm or 10 mm screenings. Specification of the drainage layer should take into consideration the perforated pipe system, in particular the slot sizes. This layer should be a minimum of 150 mm and preferably 200 mm thick.

DESIGN NOTE – The use of slotted uPVC over the more traditional choice of flexible agricultural pipe (Agriflex) has numerous advantages:

- Increased structural strength resulting in greater filter media depths without failure.
- Consistent grades to maintain self cleansing velocities are more easily maintained.
- Larger drainage slots allow for faster drainage and less risk of blockage thus increasing service life of the filter bed.
- Higher flow capacities therefore requiring lower numbers of pipes.

6.3.2.3 Overflow Chamber

The overflow chamber conveys excess flow to downstream drainage infrastructure and the overflow weir should be sized to ensure that it has sufficient capacity to convey the design discharge from the upstream drainage system. The overflow weir should be located in the sedimentation chamber.

When water levels in the sedimentation and sand filter chambers exceed the extended detention depth, water overflows into this chamber and is conveyed into the downstream drainage system.

DESIGN NOTE – Water levels in the overflow chamber should ideally be lower than the crest of the overflow weir. Some level of weir submergence is not expected to severely reduce the discharge capacity of the overflow weir. Water levels must remain below ground when operating at the design discharge of the upstream drainage system.

A broad crested weir equation can be used to determine the length of the overflow weir. i.e.

$$Q_{weir} = C_w \cdot L \cdot H^{1.5}$$

Equation 6.2

where C_w is the weir coefficient (~1.7)

L is the length of the weir (m)

H is the afflux (m)

6.3.3 Size slotted collection pipes

Either flexible perforated pipes (e.g. AG pipe) or slotted uPVC pipes can be used, however care needs to be taken to ensure the slots in the pipes are not so large that sediment can migrate into the pipes from the drainage layer. They should be sized so that the filtration media is freely drained and the collection system does not become a ‘choke’ in the system.

DESIGN NOTE – There are circumstances where it may be desirable to restrict the discharge capacity of the collection system in order to promote longer detention periods within the sand media. One such circumstance is when depth constraints may require a shallower filtration depth and a larger surface area, leading to a higher than desired maximum infiltration rate.

The water that has passed through the filtration media, is directed into the collection pipes via a ‘drainage layer’ (typically fine gravel or coarse sand, 2-10 mm diameter), whose purpose is to efficiently convey treated flows into the collection pipes while preventing any of the filtration media from being washed downstream.

DESIGN NOTE – It is considered reasonable for the maximum spacing of the slotted or perforated collection pipes to be 1.5 m (centre to centre) so that the distance water needs to travel through the drainage layer does not hinder drainage of the filtration media. Installing parallel pipes is a means to increase the capacity of the collection pipe system. A pipe diameter of 100 mm is considered to be a maximum size for the collection pipes.

To ensure the slotted or perforated pipes are of adequate size several checks are required:

- Ensure the perforations (slots) are adequate to pass the maximum infiltration rate (or the maximum required outflow)
- Ensure the pipe itself has adequate capacity
- Ensure the drainage layer has sufficient hydraulic conductivity and will not be washed into the perforated pipes.

6.3.3.1 Perforations inflow check

To estimate the capacity of flows through the perforations, orifice flow conditions are assumed and a sharp edged orifice equation can be used.

Firstly the number and size of perforations needs to be determined (typically from manufacturer’s specifications) and used to estimate the flow rate into the pipes using a design pressure head of the filtration media depth, plus the ponding depth. Secondly, it is conservative but reasonable to use a blockage factor (e.g. 50% blocked) to account for partial blockage of the perforations by the drainage layer media.

$$Q_{perforation} = B \cdot C \cdot A_{perforation} \sqrt{2gh}$$

Equation 6.3

where B is the blockage factor (0.5-0.75)

C is the orifice coefficient (~0.6)

A is the area of the perforation

h is depth of water over the collection pipe

The combined discharge capacity of the perforations in the collection pipe should exceed the design discharge of the sand filter unless the specific intention is to increase detention time in the sand filter by limiting the discharge through the collection pipe.

Prevention of clogging of the perforations is essential and a drainage layer consisting of gravel encasing the slotted pipe is recommended. It is good practice to adopt a blockage factor to account for the likelihood of some of the slots being blocked.

6.3.3.2 Perforated pipe capacity

One form of the Colebrook-White equation can be applied to estimate the velocity and hence flow rate in the perforated pipe. The capacity of this pipe needs to exceed the maximum infiltration rate.

V = -2(2gDSf) 0.5 x log [(k/3.7D) + (2.51v/D(2gDSf)0.5)]

V = Q / A

Therefore

Q = -2(2gDSf) 0.5 x log [(k/3.7D) + (2.51v/D(2gDSf)0.5)] x A

Equation 6.4

Where D = pipe diameter

A = area of the pipe

S_f = pipe slope

k = wall roughness

ν = viscosity

g = gravity constant

6.3.4 Design principles to facilitate maintenance

There are several key decisions during the design process that have significant impact on the ability to perform maintenance of a sand filter. As sand filters do not support vegetation, maintenance is paramount to performance, especially in maintaining the porosity of the surface of the sand filtration media.

Easy access is the most important maintenance consideration during design. This includes both access to the site (e.g. traffic management options) as well as access to the sedimentation and sand filter chambers (as well as less frequent access to the overflow chamber). Regular inspections are also required, particularly following construction and should be conducted following the first several significant rainfall events. This reinforces the requirement for easy access to the site.

Access into the sand filter chamber is particularly important because of the requirement to remove the fine sediments from the surface layer of the sand filter (top 25-50mm) from the entire surface area when accumulated fine sediment forms a 'crust'. This may require multiple entry points to the chamber depending in the scale of the filter. If maintenance crews can not access part of the sand filter chamber it will quickly become blocked and perform no water quality improvement function.

If the sedimentation chamber is required to be drained for maintenance purposes (regardless of whether it is designed to drain between storm events. A drainage valve needs to be designed into systems that have no weep holes that can drain this chamber. Having freely drained material significantly reduces the removal and disposal costs from the sedimentation chamber.

The perforated collection pipes at the base of the sand filter are also important maintenance considerations. Provision should be made for flushing (and downstream capture of flushed material) of any sediment build up that occurs in the pipes. This can be achieved with solid pipe returns to the surface for inspection openings (at the upstream end of the pipes) and a temporary filter sock or equivalent placed over the outlet pipe in the overflow chamber to capture flushed sediment.

6.3.5 Design calculation summary

Below is a design calculation summary sheet for the key design elements of sand filters to aid the design process.

Sand Filters — CALCULATION CHECKSHEET

	CALCULATION TASK	OUTCOME		CHECK
1	**Identify design criteria**			
	conveyance flow standard (ARI)		year	
	treatment flow rate (ARI)		year	
	pretreatment objective		µm	
	sand filter area		m^2	
	sand filter depth		m	
	maximum ponding depth		mm	☐
2	**Catchment characteristics**			
	area		m^2	
	slope		%	
	Fraction impervious			☐
3	**Estimate design flow rates**			
	Time of concentration			
	estimate from flow path length and velocities		minutes	☐
	Identify rainfall intensities			
	station used for IFD data			
	100 year ARI		mm/hr	
	1 year ARI		mm/hr	☐
	Design runoff coefficient			
	C_{10}			
	C_{100}			☐
	Peak design flows			
	Q_1		m^3/s	
	Q_{100}		m^3/s	☐
3	**Sedimentation chamber**			
	required surface area		m^2	
	length:width ratio			
	length x width		m	
	depth		m	
	Inlet weir length		m	
	particle sizes		mm	
	CHECK SCOUR VELOCITY (depends on particle size)		<.51 m/s	
	overflow weir capacity		m^3/s	
	CHECK OVERFLOW CAPACITY			☐
4	**Slotted collection pipe capacity**			
	pipe diameter		mm	
	number of pipes			
	pipe capacity		m^3/s	
	capacity of perforations		m^3/s	
	soil media infiltration capacity		m^3/s	
	CHECK PIPE CAPACITY > SOIL CAPACITY			☐
5	**Sand Filter properties**			
	Particle size	% Passing		
			%	
			%	
			%	
			%	
			%	
			%	
			%	☐

6.4 Checking tools

This section provides a number of checking aids for designers and referral authorities. In addition, advice on construction techniques and lessons learnt from building sand filters are provided.

Checklists are provided for:

- Design assessments
- Construction (during and post)
- Operation and maintenance inspections
- Asset transfer (following defects period).

6.4.1 Design assessment checklist

The checklist below presents the key design features that should be reviewed when assessing a design of a sand filter. These considerations include configuration, safety, maintenance and operational issues that should be addressed during the design phase.

Where an item results in an "N" when reviewing the design, referral should be made back to the design procedure to determine the impact of the omission or error.

In addition to the checklist, a proposed design should have all necessary permits for its installations. The referral agency should ensure that all relevant permits are in place. These can include permits to clear vegetation, to dredge, create a waterbody, divert flows or disturb fish or platypus habitat.

Land ownership and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the asset transfer checklist (see Section 6.4.4).

Sand Filter Design Assessment Checklist					
Sand Filter location:					
Hydraulics	Minor Flood: (m^3/s)		Major Flood: (m^3/s)		
Area	Catchment Area (ha):		Sand Filter Area (m^2)		
Treatment				Y	N
Treatment performance verified from curves?					
Inlet zone/hydraulics				Y	N
Station selected for IFD appropriate for location?					
Sediment chamber dimensions sufficient to retain 125um particles?					
Drainage facilities for sediment chamber provided?					
Overall flow conveyance system sufficient for design flood event?					
Velocities at inlet and within sand filter will not cause scour?					
Bypass sufficient for conveyance of design flood event?					
Collection System				Y	N
Slotted pipe capacity > infiltration capacity of filter media (where appropriate) ?					
Maximum spacing of collection pipes <1.5m?					
Drainage layer >150mm?					
Transition layer provided to prevent clogging of drainage layer?					
Filter Basin				Y	N
Maximum ponding depth will not impact on public safety?					
Selected filter media hydraulic conductivity > 10x hydraulic conductivity of surrounding soil?					
Maintenance access provided to base of filter media (where reach to any part of a basin >6m)?					
Protection from gross pollutants provided (for larger systems)?					
Sand media specification included in design?					

6.4.2 Construction advice

This section provides general advice for the construction of sand filters. It is based on observations from construction projects around Australia.

Building phase damage

Protection of filtration media is very important during building phase, uncontrolled building site runoff is likely to cause excessive sedimentation, introduce debris and litter and could cause clogging of the sand media. Upstream measures should be employed to control the quality of building site runoff. If a sand filter is not protected during building phase it is likely to require replacement of the sand filter media. An additional system of installing a geotextile fabric over the surface of the sand filter during the building phase can also protect the sand filter media below. Accumulated sediment and the geotextile fabric can then be removed following most of the upstream building activity has finished.

Traffic

Ensure traffic and deliveries do not access sand filters during construction. Traffic can compact the filter media and cause preferential flow paths, deliveries can block filtration media. Wash down wastes (e.g. concrete) can cause blockage of filtration media. Sand filters should be fenced off during building phase and controls implemented to avoid wash down wastes.

Sediment basin drainage

When a sediment chamber is designed to drain between storms (so that pollutants are stored in a drained state) weeps holes can be used that are protected from blockage. Blockage can be avoided by constructing a protective sleeve (to protect the holes from debris blockage, e.g. 5mm screen) around small holes at the base of the bypass weir. It can also be achieved with a vertical slotted PVC pipe, with protection from impact and an inspection opening at the surface to check for sediment accumulation. The weep holes should be sized so that they only pass small flows (e.g. 10-15mm diameter).

Perforated pipes

Perforated pipes can be either a uPVC pipe with slots cut into the length of it or a flexible ribbed pipe with smaller holes distributed across its surface (an AG pipe). Both can be suitable. uPVC pipes have the advantage of being stiffer with less surface roughness therefore greater flow capacity, however the slots are generally larger than for flexible pipes and this may cause problems with filter or drainage layer particle ingress into the pipe. Stiff PVC pipes however can be cleaned out easily using simple plumbing equipment. Flexible perforated pipes have the disadvantage of roughness (therefore flow capacity) however have smaller holes and are flexible which can make installation easier. Blockages within the flexible pipes can be harder to dislodge with standard plumbing tools.

Inspection openings in perforated pipes

It is good design practice to have inspection openings at the end of the perforated pipes. The pipes should be brought to the surface (with solid pipes) and have a sealed capping. This allows inspection of sediment buildup when required and easy access for maintenance, such as flushing out accumulated sediments. Sediment controls downstream should be used when flushing out sediments from the pipes to prevent sediments reaching downstream waterways.

Clean filter media

It is essential to ensure drainage media is washed prior to placement to remove fines and prevent premature clogging of the system.

6.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Sand filters

INSPECTED BY:
DATE:
TIME:
WEATHER:
CONTACT DURING VISIT:

SITE: ______________________

CONSTRUCTED BY: ______________________

DURING CONSTRUCTION									
Items inspected	Checked		Satisfactory	Unsatisfactory		Checked		Satisfactory	Unsatisfactory
Preliminary works	Y	N			**Structural components**	Y	N		
1. Erosion and sediment control plan adopted					14. Location and levels of pits as designed				
					15. Safety protection provided				
2. Traffic control measures					16. Pipe joints and connections as designed				
3. Location same as plans					17. Concrete and reinforcement as designed				
4. Site protection from existing flows					18. Inlets appropriately installed				
Earthworks					19. Pipe joints and connections as designed				
5. Level bed					20. Concrete and reinforcement as designed				
6. Side slopes are stable					21. Inlets appropriately installed				
7. Provision of liner					**Filtration system**				
8. Perforated pipe installed as designed					22. Provision of liner				
9. Drainage layer media as designed					23. Adequate maintenance access				
10. Sand media specifications checked					24. Inlet and outlet as designed				
Sedimentation chamber									
11. Adequate maintenance access									
12. Invert level correct									
13. Ability to freely drain (weep holes)									
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of sand				
2. Traffic control in place					7. No surface clogging				
3. Confirm structural element sizes					8. Maintenance access provided				
4. Sand filter media as specified					9. Construction generated sediment and debris removed				
5. Sedimentation chamber freely drains									

COMMENTS ON INSPECTION

ACTIONS REQUIRED

1.
2.
3.
4.
5.
6.

6.4.4 Asset transfer checklist

Asset Handover Checklist		
Asset Location:		
Construction by:		
Defects and Liability Period		
Treatment	**Y**	**N**
System appears to be working as designed visually?		
No obvious signs of under-performance?		
Maintenance	**Y**	**N**
Maintenance plans provided for each asset?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
Asset inspected for defects?		
Asset Information	**Y**	**N**
Design Assessment Checklist provided?		
As constructed plans provided?		
Copies of all required permits (both construction and operational) submitted?		
Proprietary information provided (if applicable)?		
Digital files (eg drawings, survey, models) provided?		
Asset listed on asset register or database?		

6.5 Maintenance requirements

Maintenance of sand filters is primarily concerned with:

- Regular inspections (3-6 monthly) to inspect sedimentation chamber and the sand media surface
- Maintenance of flows to and through the sand filter
- Removal of accumulated sediments and litter and debris removal from the sedimentation chamber
- Checking to ensure the weep holes and overflow weirs are not blocked with debris.

Maintaining the flow through a sand filter involves regular inspection and removal of the top layer of accumulated sediment. Inspections should be conducted after the first few significant rainfall events following installation and then at least every six months following. The inspections will help to determine the long term cleaning frequency for the sedimentation chamber and the surface of the sand media.

Removing fine sediment from the surface of the sand media can typically be performed with a flat bottomed shovel or vacuum machinery. Tilling below this surface layer can also maintain infiltration rates. Access is required to the complete surface area of the sand filter and this needs to be considered during design.

Sediment accumulation at in the sedimentation chamber also needs to be monitored. Depending on the catchment activities (e.g. building phase) the deposition of sediment can overwhelm the sedimentation chamber and reduce flow capacities.

Similar to other types of practices, debris removal is an ongoing maintenance function. Debris, if not removed, can block inlets or outlets, and can be unsightly if located in a visible location. Inspection and removal of debris should be done regularly, but debris should be removed whenever it is observed on the site.

6.5.1 Operation & maintenance inspection form

The form below should be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

Sand Filter Maintenance Checklist			
Inspection Frequency: 6 monthly	**Date of Visit:**		
Location:			
Description:			
Site Visit by:			
Inspection Items	**Y**	**N**	**Action Required (details)**
Litter within filter?			
Scour present within sediment chamber or filter?			
Traffic damage present?			
Evidence of dumping (eg building waste)?			
Clogging of drainage weep holes or outlet?			
Evidence of ponding?			
Damage/vandalism to structures present?			
Surface clogging visible?			
Drainage system inspected?			
Removal of fine sediment required?			
Comments:			

6.6 Sand filter worked example

6.6.1 Worked example introduction

A sand filter system is proposed to treat stormwater runoff from a courtyard/plaza area in Hobart. The site is nested amongst a number of tall buildings and is to be fully paved as a multi-purpose court yard. Stormwater runoff from the surrounding building is to be directed to bioretention planter boxes while runoff from this 2000 m^2 courtyard will be directed into an underground sand filter. Provision for overflow into the underground drainage infrastructure ensures that the site is not subjected to flood ponding for storm events up to the 100 year average recurrence interval. The existing stormwater drainage infrastructure has the capacity to accommodate the 100 year ARI peak discharge from this relatively small catchment.

Key functions of a sand filter include the following:

- Promote the capture of gross pollutants
- Promote sedimentation of particles larger than 125 µm within the inlet zone for flows up to a 1 year ARI (unattenuated) peak discharge.
- Promote filtration following sedimentation pre-treatment through a sand layer
- Provide for by-pass operation by configuring and designing the by-pass chamber.

The concept design suggests that the required area of the sand filter chamber is 80 m^2 and the depth of the sand filter is 600 mm. Outflows from the sand filter are conveyed into a stormwater pipe for discharge into existing stormwater infrastructure (legal point of discharge) via a third chamber, an overflow chamber. Flows in excess of a 200 mm extended detention depth would overflow and discharge directly into the underground stormwater pipe and by-pass the sand filter.

6.6.1.1 Design Objectives

Design objectives include the following:

- Sand filter to consist of 3 chambers, a sedimentation (and gross pollutant trapping) chamber, a sand filter chamber and an overflow chamber.
- The sedimentation chamber shall be designed to capture particles larger than 125µm for flows up to the peak 1yr ARI design flow with a capture efficiency of 80%. The outlet from the chamber will need to be configured to direct flows up to the 1yr ARI into the sand filter, flows in excess of 1yr ARI will bypass to the overflow chamber.
- The sand filter shall be designed to filter the peak 1yr ARI flow. Perforated sub-soil drainage pipes are to be provided at the base of the sand filter and will need to be sized to ensure the flow can enter the pipes, (check inlet capacity) and to ensure they have adequate flow capacity.
- The overflow chamber shall be designed to capture and convey flows in excess of the 1yr ARI peak flow and up to the 100 year ARI peak discharge.

- Sedimentation chamber shall retain sediment and gross pollutants in a dry state and to have sufficient storage capacity to limit sediment cleanout frequency to once a year.
- Inlet / outlet pipes to be sized to convey the 100yr ARI peak discharge.

6.6.1.2 Site Characteristics

The site characteristics are summarised as follows:-

- Catchment area 2,000m^2 (80 m x 25 m)
- Land use/surface type Paved courtyard
- Overland flow slope 1.0%
- Soil type clay
- Fraction impervious 0.90

6.6.2 Verifying size for treatment

The nominated area of the sand filter is 80 m^2.

According to charts in Section 6.2, a sand filter area of 3.5% of the impervious area will be necessary to reduce TSS load by 80%. Smaller areas are required to attain best practice objectives for TP and TN.

With a fraction impervious of 0.9, the impervious area of the courtyard is 1800 m2 and the required sand filter area is 63 m^2 → OK

> **DESIGN NOTE – The values derived from 6.2 Verifying size for treatment will only be valid if the design criteria for the proposed installation are similar to those used to create the Figures. Site specific modelling using programs such as MUSIC (eWater, 2009) may yield a more accurate result.**

6.6.3 Estimating Design Flows

The calculation of the design flow will be undertaken using the Rational Method.

Length of the longest flow path is assumed to consist of overland flowpath (1/2 width of the courtyard = 12.5m) and gutter flow (1/2 perimeter length of the courtyard = 52.5m). Fall across the courtyard is assumed to be 1%.

The travel time of the overland flow path can be estimated using either the Bransby Williams formula for time of concentration or by the overland kinematic wave equation as presented in Australian Rainfall and Runoff (2003).

Each method has advantages and disadvantages. The Kinematic Wave equation is the most accurate method of calculating tc and is generally suited to most catchments. As the equation requires the designer to solve for t and $I^{0.4}$ simultaneously, an iterative approach must be undertaken (or use a previously prepared relationship table for $I^{0.4}$ for the study area). The Bransby Williams formula is well suited to situations where no actual relationships for t_c have been calculated based on observed data, and it does not require an iterative process to reach a solution making it attractive to designers new to these theories.

Kinematic wave equation	**Bransby Williams formula for t_c**
$$t = \frac{6.94(L \cdot n^*)^{0.6}}{I^{0.4} \cdot S^{0.3}}$$	$$t_c = \frac{91\ L}{A^{0.1}\ S_e^{0.2}}$$
Where: **t** is the overland travel time (minutes) **L** is the overland flow path length (m) **N*** is the surface roughness (concrete or asphalt ~ 0.013) **I** is the design rainfall intensity (mm/hr) **S** is the slope	Where: **t_c** is the time of concentration (minutes) **L** is the main stream length measured to the catchment divide (km) **A** is the catchment area (Ha) **S_e** is the grade of the main stream (m/km)

Equation 6.5 and Equation 6.6

NOTE – For this example, the Bransby Williams formula will be used.

Step 1 – Calculate the time of concentration.

From equation 7.4b –

$$Tc = \frac{91 \times 0.065}{0.2^{0.1} \times 10^{0.2}}$$

$$= 5.915 / 1.348$$

$$= 4.38 \text{ minutes (assume 5minutes)}$$

Using a time of concentration of 5 minutes, the design rainfall intensities from the IFD chart relevant to the catchment location are –

I_1 = 44 mm/hr *

I_{100} = 170 mm/hr*

* These figures are for the worked example only. The appropriate region and corresponding rainfall intensities must be selected for each individual project.

Step 2 – Calculate design run-off coefficients (using the method outlined in Australian Rainfall and Runoff Book VIII (Engineers Australia, 2003)).

Where – Fraction impervious (f) = 0.9

Rainfall intensity ($^{10}I_1$) = 28.6mm/hr (from the relevant IFD chart)

Calculate C^1_{10} (pervious run-off coefficient)

$C^1_{10} = 0.1 + 0.0133\,(^{10}I_1 - 25) = 0.15$

Calculate C_{10} (10 year ARI run-off coefficient)

$C_{10} = 0.9f + C^1_{10}\,(1-f)$

$C_{10} = 0.82$

Step 3 – Convert C_{10} to values for C_1 and C_{100}

Where – $C_y = F_y \times C_{10}$

From Table 1.6 in Australian Rainfall and Runoff – Book VII;

$C_1 = 0.8 \times C_{10} = 0.66$

$C_{100} = 1.2 \times C_{10} = 0.98$

Step 4 – Calculate peak design flow (calculated using the Rational Method).

$$Q = \frac{CIA}{360}$$

Where –	C	is the runoff coefficient (C_1 and C_{100})
	I	is the design rainfall intensity mm/hr (I_1 and I_{100})
	A	is the catchment area (Ha)

Q_1 = 0.016 m^3/s (16 L/s)

Q_{100} = 0.093 m^3/s (93 L/s)

Maximum infiltration rate

The maximum infiltration rate (Q_{max}) through the sand filter is computed using Equation 6.1, i.e.

$$Q_{max} = k \cdot A \cdot \frac{h_{max} + d}{d} \quad = 0.084\ m^3/s$$

where k is the hydraulic conductivity of sand = 1 x10^{-3} m/s (Engineers Australia, 2003, Ch. 9)

A is the surface area of the sand filter = 63 m^2

h_{max} is the depth of pondage above the sand filter = 0.2 m

d is the depth of the sand filter = 0.6 m

Design Flows	Q_1 = 0.016 m^3/s; Q_{100} = 0.093 m^3/s;
	Maximum Infiltration Rate = 0.084 m^3/s

6.6.4 Hydraulic Structures

6.6.4.1 Sizing of Sedimentation Basin

The sedimentation chamber is to be sized to remove the 125μm particles for the peak 1-year flow.

Pollutant removal is estimated using Equation 3.3 (see Chapter 3):

$$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)} \right]^{-n}$$

A notional aspect ratio of 1 (w) to 2 (L) is adopted. From Equation 3.3, the hydraulic efficiency (λ) is 0.3. The turbulence factor (n) is computed from Equation 3.2 to be 1.4.

Hydraulic efficiency (λ) = 0.3
Turbulence factor (n) = 1.4

The proposed extended detention depth of the basin is 0.2m (as outlined in Section 6.6.1) and a notional permanent pool depth of 0.6 m (equal to the depth of the sand filter) has been adopted, i.e.

d_p = 0.6 m

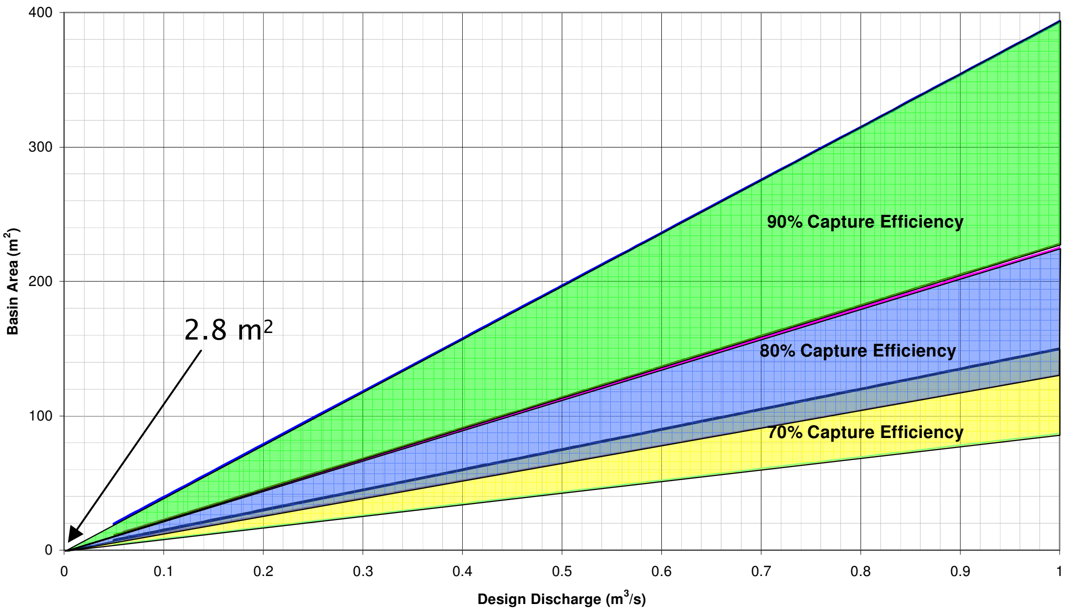
d^* = 0.6 m

d_e = 0.20m

Vs = 0.011m/s for 125μm particles

Q = Design flow rate = 0.016 m^3/s

The required sedimentation basin area to achieve target sediment (125 μm) capture efficiency of 70% is 2.8 m^2. With a W to L ratio of 1:2, the notional dimensions of the basin are 1.4 m x 2.0 m. This size is validated against the curves presented in Figure 3.2 (see Chapter 3).



The available sediment storage is 2.8 x 0.6 = 1.68 m^3. Cleanout is to be scheduled when the storage is half full. Using a sediment discharge rate of 1.6 m^3/Ha/yr, the clean out frequency is estimated to be:

$$\text{Frequency of basin desilting} = \frac{0.5 \times 1.68}{0.7 \times 1.6 \times 0.2} = 3.75 \text{ years} > 1 \text{ year} \rightarrow \text{OK}$$

During the 100 year ARI storm, peak discharge through the sedimentation chamber will be 0.093 m^3/s with flow depth of 0.8 m. It is necessary to check that flow velocity does not re-suspend deposited sediment of 125 μm or larger (≤ 0.2 m/s).

The mean velocity in the chamber is calculated as follows:

$$V_{100} = 0.093/(1.4 \times 0.8) = 0.083 \text{ m/s} \rightarrow \text{OK}$$

The length of the sedimentation chamber is 2.0 m. Provide slots of total length of 1.2 m connecting it to the sand filter chamber. The connection discharge capacity should be greater than the 1 year ARI peak flow (0.016m^3/s) and can be calculated using the weir equation as follows:

$$Q_{connection} = C_w L H^{1.5}$$

where C_w is the weir coefficient (assume = 1.4 for a broad crested weir)

H is the afflux = 0.2 m (extended detention in sedimentation chamber)

L is the length of the weir

The discharge capacity calculated from the above equation is 0.15 m^3/s >> 1 year ARI discharge of 0.016m^3/s.

Sedimentation Chamber = 2.8 m^2

Width = 1.4 m; Length = 2.0 m

Total weir length of connection to sand filter chamber = 1.2 m

Depth of chamber from weir connection to sand filter = 0.6 m
Depth of Extended Detention (d_e) = 0.2m

6.6.4.2 Sand Filter Chamber

6.6.4.2.1 Dimensions

With the length of sedimentation chamber being 2.0 m, the dimension of the sand filter chamber is determined to be 2.0 m x 32.0 m, giving an area of 64 m^2.

Sand filter chamber dimension: 2.0 m x 32.0 m

6.6.4.2.2 Media specifications

Sand filter layer to consist of sand/coarse sand material with a typical particle size distribution is provided below:

% passing	1.4 mm	100 %
	1.0 mm	80 %
	0.7 mm	44 %
	0.5 mm	8.4 %

This grading is based on a Unimin 16/30 FG sand grading.

The drainage layer is to consist of fine gravel, of 5 mm screenings.

No impervious liner is necessary as in situ soil is clay.

Filter layer is to be 600mm deep and consist of sand with 80% greater than 1 mm diameter Drainage layer to be 200mm deep and consist of 5 mm gravel

6.6.4.3 Overflow Chamber

The width of the sedimentation chamber has been selected to be 1.4 m. A weir set at 0.8 m from the base of the sedimentation chamber (or 0.2 m above the surface of the sand filter) of 1.4 m length needs to convey flows up to the 100 year ARI peak discharging into the overflow chamber.

Calculate the afflux resulting from conveying the 100 year ARI peak discharge through a 1.4 m length weir, i.e.

$$H = \left(\frac{Q_{weir}}{C_w \cdot L}\right)^{0.667} = 0.17 \text{ m, say } 0.2 \text{ m}$$

where Q_{weir} is the design discharge = 0.093 m^3/s

C_w is the weir coefficient (~1.7)

L is the length of the weir (m)

H is the afflux (m)

With an afflux of 0.2, the discharge capacity of the overflow weir is 0.21 m^3/s > 100 year ARI peak flow of 0.093 m^3/s.

Crest of overflow weir = 0.2 m above surface of sand filter
Length of overflow weir = 1.4m, 100 year ARI Afflux = 0.2 m
Roof of facility to be at least 0.4 m above sand filter surface

6.6.5 Size slotted collection pipes

6.6.5.1 Perforations inflow check

The following are the characteristics of the selected slotted pipe

- Clear openings = 2100 mm^2/m
- Slot width = 1.5mm
- Slot length = 7.5mm
- No. rows = 6
- Diameter of pipe = 100mm

For a pipe length of 1.0 m, the total number of slots = 2100/(1.5 x 7.5) = 187.

Discharge capacity of each slot can be calculated using the orifice flow equation (Equation 6.3), i.e.

$$Q_{perforation} = C \cdot A_{perforation}\sqrt{2gh} = 2.67 \times 10^{-5} \text{ m}^3/\text{s}$$

where h is the head above the slotted pipe, calculated to be 0.80 m.

C is the orifice coefficient (~0.6)

The inflow capacity of the slotted pipe is thus 2.67 x 10^{-5} x 187 = 5 x 10^{-3} $m^3/s/m$-length

Adopt a blockage factor of 0.5 gives the inlet capacity of each slotted pipe to be 2.5 x 10^{-3} $m^3/s/m$-length.

Maximum infiltration rate is 0.083 m^3/s. The minimum length of slotted pipe required is

$L_{slotted\ pipe}$ = 0.083/2.5x10^{-3} = 33.2 m = 17 lengths of 2.0 m at 1.5 m spacing.

17 slotted pipes (2.0 m length) at 1.5 m spacing required

6.6.5.2 Slotted pipe capacity

The diameter of the slotted pipe is 100 mm. The discharge capacity of the collection pipe is calculated using an Colebrook-White equation (Equation 6.4), i.e.

$$Q = -2(2gDS_f)^{0.5} \times \log [(k/3.7D) + (2.51\nu/D(2gDS_f)^{0.5})] \times A$$

Where D = pipe diameter

A = area of the pipe

S_f = pipe slope

k = wall roughness

ν = viscosity

g = gravity constant

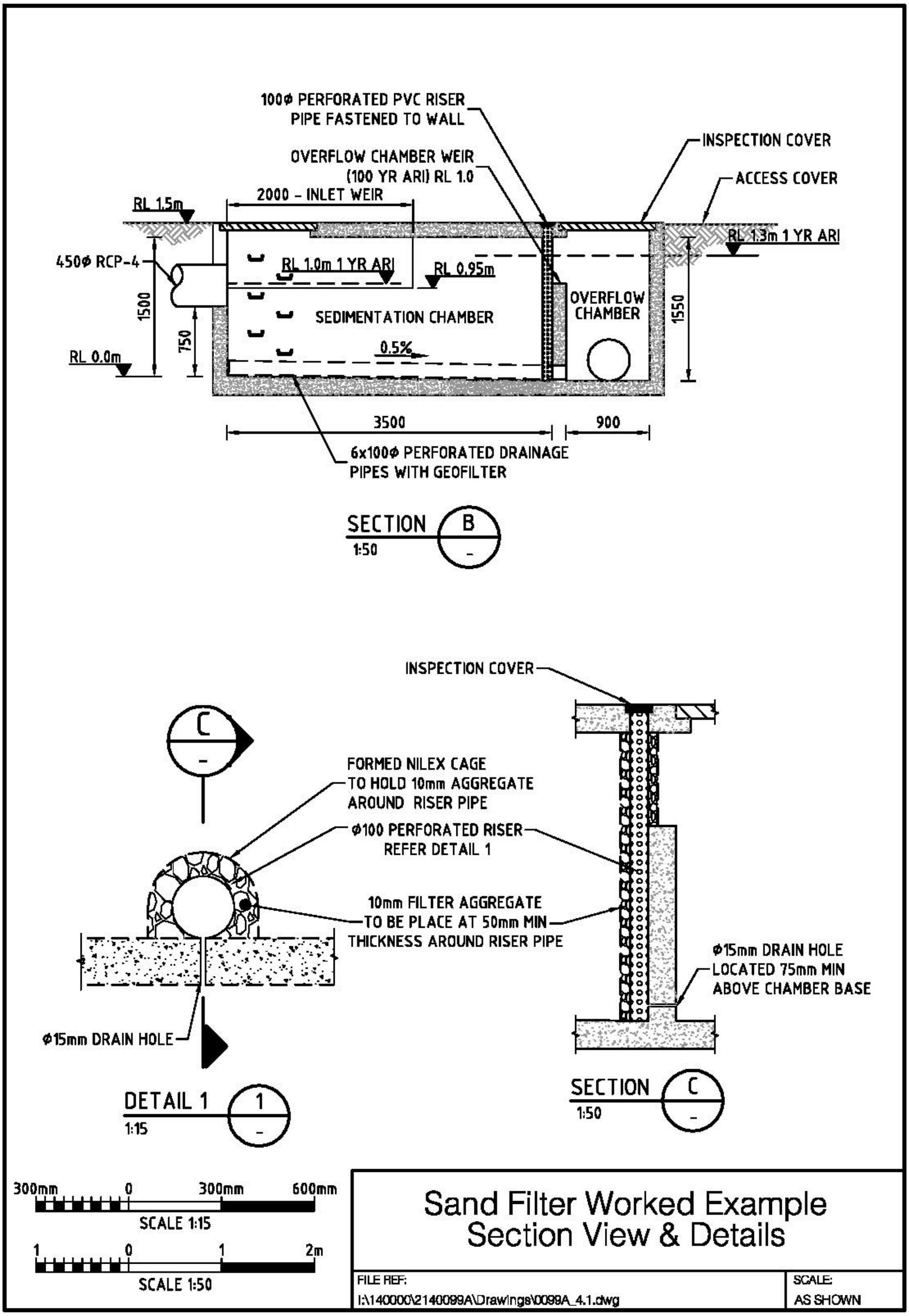
Total discharge capacity (17 pipes) = 0.323 m^3/s > maximum infiltration rate of 0.083 m^3/s ➔ OK

> Combined slotted pipe discharge capacity = 0.323 m^3/s and exceeds the maximum infiltration rate.

6.6.6 Design Calculation Summary

Sand Filters — **CALCULATION CHECKS**

	CALCULATION TASK		OUTCOME		CHECK
1	**Identify design criteria**				
	conveyance flow standard (ARI)		100	year	
	treatment flow rate (ARI)		1	year	
	pretreatment objective		125	µm	
	sand filter area		6	m^2	
	sand filter depth		0.6	m	
	maximum ponding depth		200	mm	✓
2	**Catchment characteristics**				
		area	2000	m^2	
		slope	1	%	
		Fraction impervious	0.9		✓
3	**Estimate design flow rates**				
	Time of concentration				
	estimate from flow path length and velocities		5	minutes	✓
	Identify rainfall intensities				
		station used for IFD data	Hobart		
		100 year ARI	170	mm/hr	
		1 year ARI	44	mm/hr	✓
	Peak design flows				
		Q_1	0.016	m^3/s	
		Q_{100}	0.09	m^3/s	✓
4	**Sedimentation chamber**				
		required surface area	2	m^2	
		length:width ratio	1:2		
		length x width	2 x 1.4	m	
		depth	0.6	m	
		Inlet weir length	1.2	m	
		particle sizes	1.0	mm	
		CHECK SCOUR VELOCITY (depends on particle size)	0.08	<0.2 m/s	
		overflow weir capacity	0.54	m^3/s	
		CHECK OVERFLOW CAPACITY	YES		✓
5	**Slotted collection pipe capacity**				
		pipe diameter	150	mm	
		number of pipes	17		
		combined pipe capacity	0.323	m^3/s	
		capacity of perforations	0.05	m^3/s	
		soil media infiltration capacity	0.083	m^3/s	
		CHECK PIPE CAPACITY > SOIL CAPACITY	YES		✓
6	**Sand Filter properties**				
	Percent Passing	1.40	100	%	
	Unimin 16/30 FG	1.18	96	%	
		1.00	80	%	
		0.85	63	%	
		0.71	44	%	
		0.60	24	%	
		0.50	8	%	
		0.425	1	%	✓


100ϕ PERFORATED PVC RISER PIPE FASTENED TO WALL
OVERFLOW CHAMBER WEIR (100 YR ARI) RL 1.0
INSPECTION COVER
ACCESS COVER
2000 - INLET WEIR
RL 1.5m
RL 1.3m 1 YR ARI
450ϕ RCP-4
RL 1.0m 1 YR ARI
RL 0.95m
OVERFLOW CHAMBER
SEDIMENTATION CHAMBER
1500
750
1550
0.5%
RL 0.0m
3500
900
6x100ϕ PERFORATED DRAINAGE PIPES WITH GEOFILTER
SECTION B
1:50
INSPECTION COVER
C
FORMED NILEX CAGE TO HOLD 10mm AGGREGATE AROUND RISER PIPE
ϕ100 PERFORATED RISER REFER DETAIL 1
10mm FILTER AGGREGATE TO BE PLACE AT 50mm MIN THICKNESS AROUND RISER PIPE
ϕ15mm DRAIN HOLE LOCATED 75mm MIN ABOVE CHAMBER BASE
ϕ15mm DRAIN HOLE
DETAIL 1
1:15
SECTION C
1:50
300mm 0 300mm 600mm
SCALE 1:15
1 0 1 2m
SCALE 1:50
Sand Filter Worked Example
Section View & Details
FILE REF:
I:\140000\2140099A\Drawings\0099A_4.1.dwg
SCALE:
AS SHOWN

6.7 References

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Chapter 7 Swale/ buffer Systems

Definition:

A vegetated swale is a vegetation-lined channel used to convey stormwater in lieu of pipes.

Purpose:

- Vegetated swales provide a desirable 'buffer' between receiving waters (e.g. creek, wetland) and impervious areas of a catchment
- The interaction with vegetation promotes an even distribution and slowing of flows thus encouraging coarse sediments to be retained.

Implementation considerations:

- Swales can be incorporated in urban designs along streets or parklands and add to the Aesthetic character of an area.
- Operates best with longitudinal slopes of 2% to 4%. Milder sloped swales may become waterlogged and have stagnant ponding, the use of underdrains can alleviate this problem. For slopes steeper than 4%, check banks can help to distribute flows as well as slow velocities. Dense vegetation and drop structures can be used to serve the same function as check dams but care needs to be exercised to ensure that velocities are not excessively high.
- Swales can use a variety of vegetation types. Vegetation is required to cover the whole width of a swale, be capable of withstanding design flows and be of sufficient density to provide good filtration. For best treatment performance, vegetation height should be above treatment flow water levels.
- If runoff enters directly into a swale, perpendicular to the main flow direction, the edge of the swale acts as a buffer and provides pre-treatment for the water entering the swale.

Vegetation is selected by required appearance & treatment performance

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7.2 Verifying size for treatment .... 7—6

7.3 Design procedure: Swales .... 7—9

7.4 Checking tools .... 7—17

7.5 Maintenance requirements .... 7—22

7.6 Swale worked example .... 7—24

7.7 References .... 7—35

7.1 Introduction

Vegetated swales are used to convey stormwater in lieu of pipes and remove coarse and medium sediment and are commonly combined with buffer strips. Swales also provide a disconnection of impervious areas from hydraulically efficient pipe drainage systems resulting in slower travel times thus reducing the impact of increased catchment imperviousness on peak flow rates.

Figure 7.1 illustrates vegetated swales with different options for driveway crossings, including at-grade crossings (with mild side slopes) and elevated crossings.

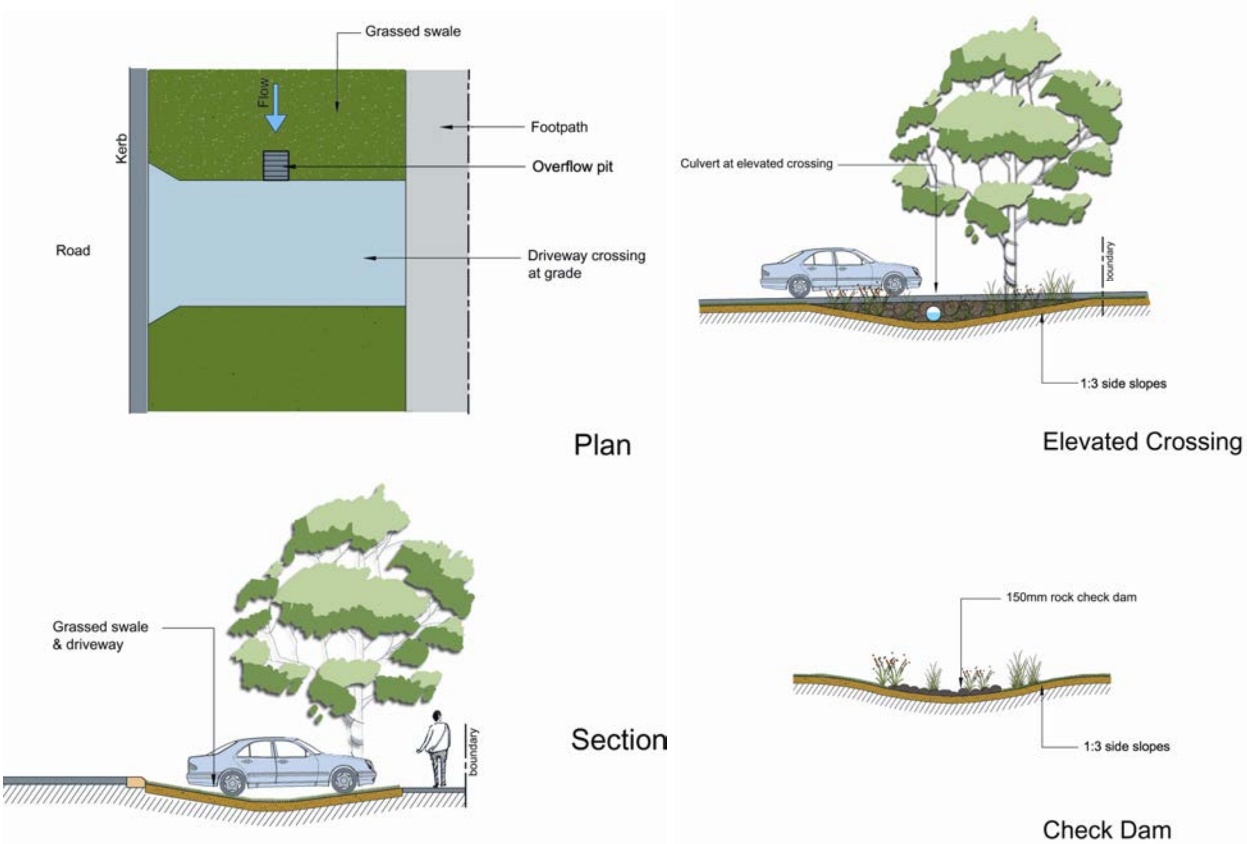


Figure 7.1. Swales with at-grade driveway crossing (L), elevated crossing (top R) and a check dam for flow spreading

The interaction between flow and vegetation along swales facilitates pollutant settlement and retention. Swale vegetation acts to spread flows and reduce velocities, which in turn aids filtration and sediment deposition. Swales alone can rarely provide sufficient treatment to meet objectives for all pollutants, but can provide an important pre-treatment function for other WSUD measures. They are particularly good at coarse sediment removal and can be incorporated in street designs to enhance the Aesthetics of an area.

Buffer strips (or buffers) are areas of vegetation through which runoff passes while travelling to a discharge point. They reduce sediment loads by passing a shallow, well-distributed flow through vegetation. Vegetation reduces velocities and coarse sediments are retained. Buffers can be used as edges to swales, particularly where flows are distributed along the banks of the swale.

To convey flood flows along swales, in excess of a treatment design flow (typically the peak 3-month ARI flow), pits draining to underground pipes can be used. Overflows from the swale enter the pit when a designated depth is reached. This is particularly useful in areas with narrow verges, where a swale can only accommodate flows associated with the minor drainage system (e.g. 5 year ARI) for a certain length.

The longitudinal slope of a swale is the most important consideration in their design. They generally operate best with between 1% and 4% slopes. Slopes milder than this can tend to become waterlogged and have stagnant ponding. However, shallow underdrains or a thin sand layer can alleviate this problem by providing a drainage path for small depressions along a swale. For slopes steeper than 4%, check dams or banks (small porous rock walls) along swales can help to distribute flows and reduce velocities.

Swales can be designed with a variety of vegetation types including turf, sedges and tussock grasses. Vegetation is required to cover the whole width of the swale, be capable of withstanding design flows and be of sufficient density to provide good filtration. For best performance, the vegetation height should be above the treatment flow water level.

Figure 7.2. Swale systems: heavily vegetated, use of check dams, grass swale with elevated crossings

Grassed swales are commonly used and can appear as a typical road verge, however the short vegetation offers sediment retention to only shallow flows. In addition, the grass is required to be mown and well maintained in order for the swale to operate effectively. Denser vegetated swales can offer improved sediment retention by slowing flows more and providing filtration for deeper flows. Conversely, vegetated swales have higher hydraulic roughness and therefore require a larger area to convey flows compared to grass swales. These swales can become features of a landscape and, once established, require minimal maintenance and be hardy enough to withstand large flows.

Another key consideration when designing swales is road or driveway crossings. Crossings can provide an opportunity for check dams (to distribute flows) or to provide temporary ponding above a bioretention system (refer to Chapter 5). A limitation with 'elevated' crossings can be their expense compared to at-grade crossings (particularly in dense urban developments), safety concerns with traffic movement adjacent to the inlet and outlet and the potential for blockage of with small culverts.

Crossings can also be constructed at grade and act like a ford during high flows, however, this reduces maximum swale batter slopes to approximately 1 in 9 (with a flat base) to allow for traffic movement. These systems can be cheaper to construct than elevated crossings but require more space. They are well suited to low density developments.

Figure 7.3. Elevated and at-grade driveway crossings across swales

Swales can also be constructed as centre medians in divided roads and in this case would also enhance the Aesthetics of the street. This also avoids issues associated with crossings.

It is extremely important to keep traffic and deliveries off swales and means to ensure this is a key concern during swale design. Traffic can ruin the vegetation and provide ruts that cause preferential flow paths that do not offer filtration. Traffic control can be achieved by selecting swale vegetation that discourages the movement of traffic or by providing physical barriers to traffic movement. For example, barrier kerbs with breaks in them (to allow distributed water entry, albeit with reduced uniformity of flows compared with flush kerbs) or bollards along flush kerbs can be used to prevent vehicle movement onto swales.

With flood flows being conveyed along a swale surface, it is important to ensure velocities are kept low to avoid scouring of collected pollutants and vegetation.

Swales can be installed at various scales, for example in local streets or on large highways.

The design process for swales involves firstly designing the system for conveyance and secondly ensuring the system has features that maximise it’s treatment performance and long-term viability.

Key design issues to be considered are:

Verifying treatment performance and relation to other measures in a treatment train

Determine design flows

Dimension the swale with site constraints

Above ground design

- check velocities
- check slopes
- design of inlet zone and overflow pits
- check above design flow operation
- Allowances to preclude traffic on swales
- Recommend plant species and planting densities
- Provision for maintenance.

7.2 Verifying size for treatment

The curves below show the pollutant removal performance expected for swales with varying slopes (1%, 3% and 5 %) and vegetation height (0.05 to 0.5m). It is important to recognise that swales in isolation provide limited treatment for fine pollutants, but can perform pretreatment for other measures.

The curves are based on the performance of the system at the reference site and were derived using the Model for Urban Stormwater improvement Conceptualisation (MUSIC) (eWater, 2009). To estimate an equivalent performance at other locations in Tasmania, the hydrologic design region relationships should be used to convert the treatment area into an equivalent treatment area, refer to Chapter 2. In preference to using the curves, local data should be used to model the specific treatment performance of the system.

The curves were derived assuming the systems receive direct runoff (i.e. no pretreatment) and have the following characteristics:

- a base width of 2m
- a top width of 6m
- 1 in 6 side slopes
- no infiltration through the base of the swale.

These curves can be used to check the expected performance of swales for removal of TSS, TP and TN with similar cross sections to the dimensions assumed above. If dimensions of a swale vary significantly from the values above, more detailed modelling of performance should be conducted. The *swale size* is represented as the *top width of the swale times its length* divided by the contributing *impervious catchment*.

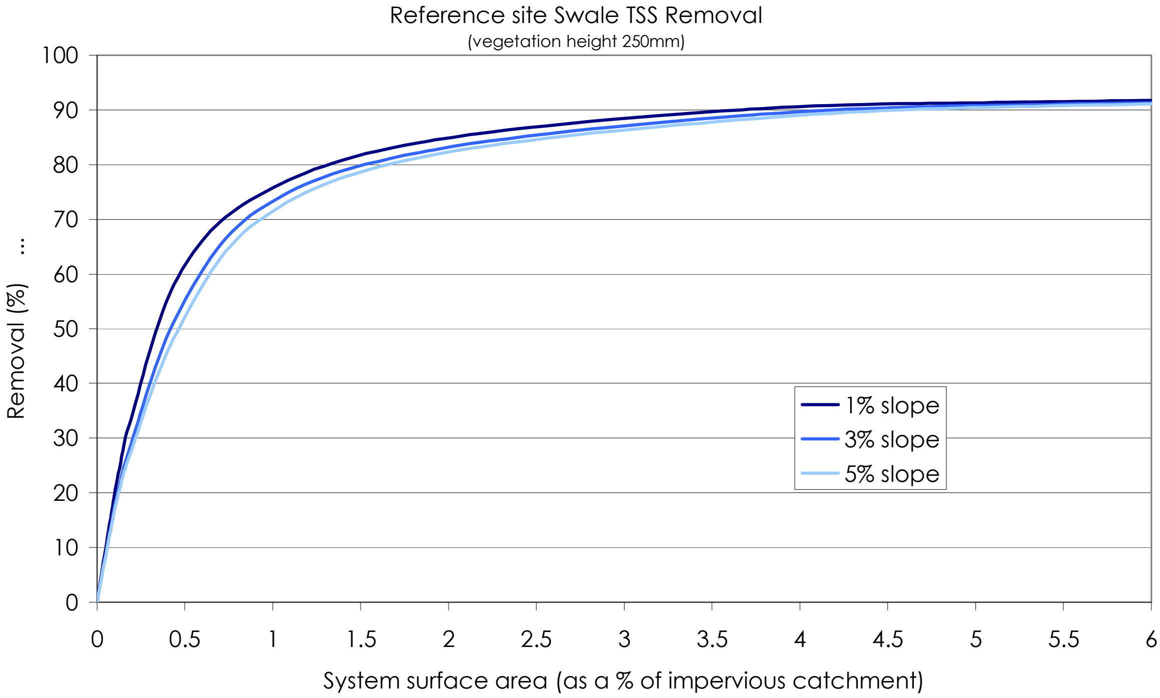


Figure 7.4. TSS removal in swale systems with varying slope

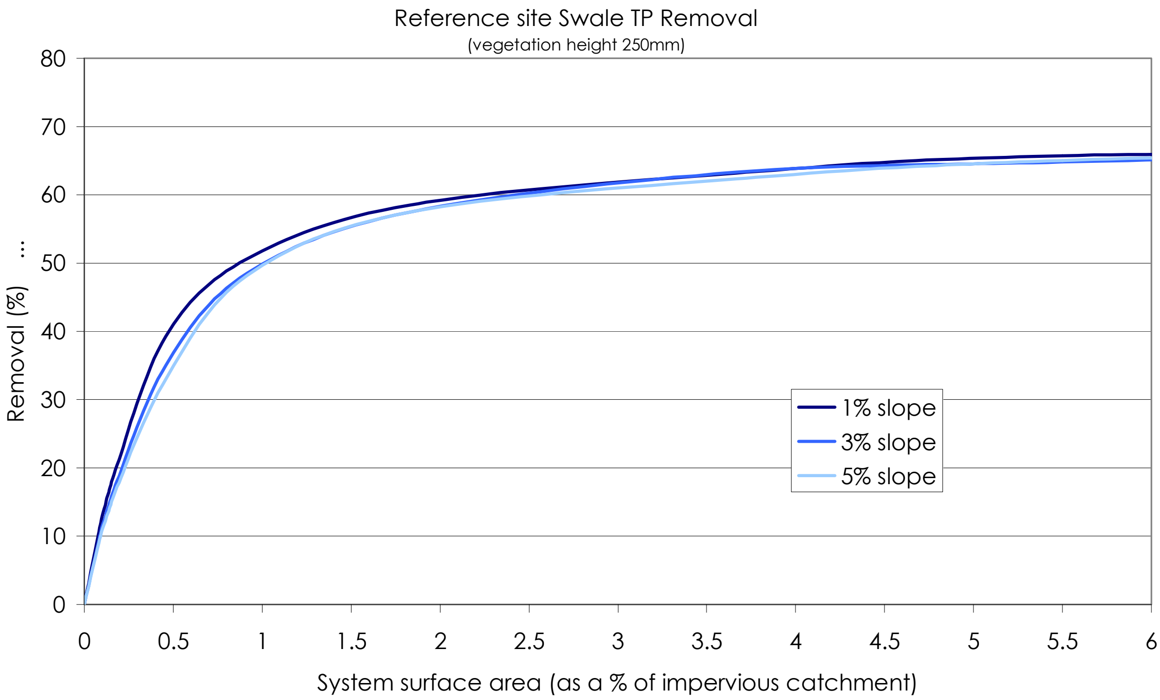


Figure 7.5. TP removal in swale systems with varying slope

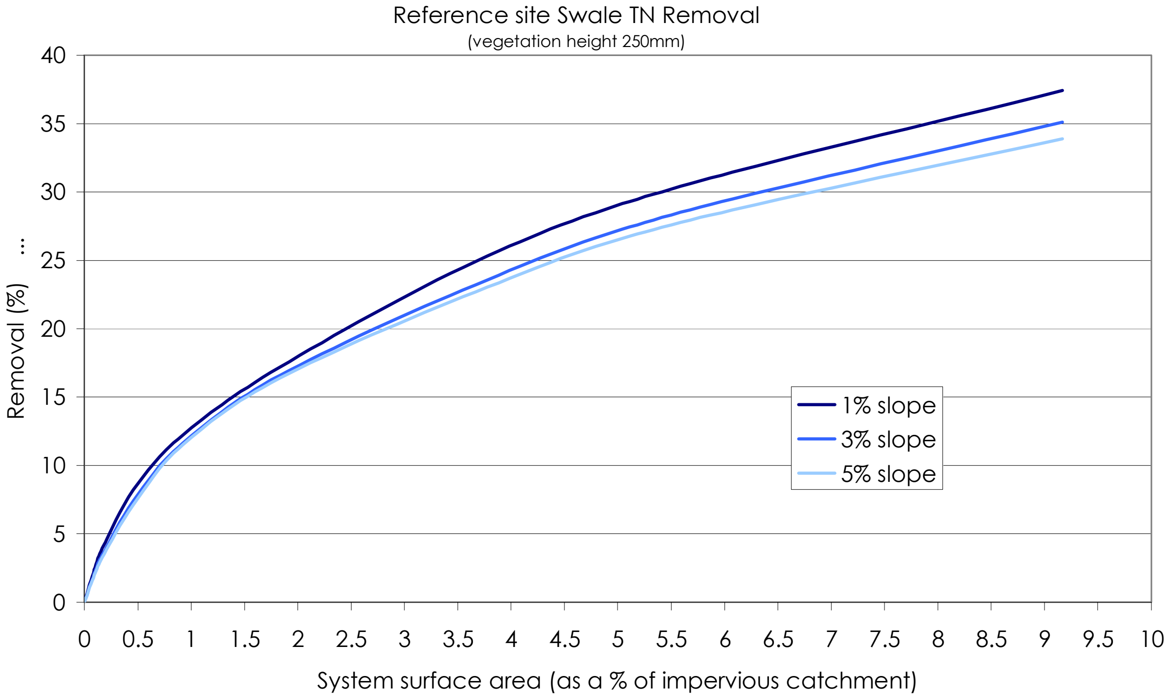


Figure 7.6. TN removal in swale systems with varying slope

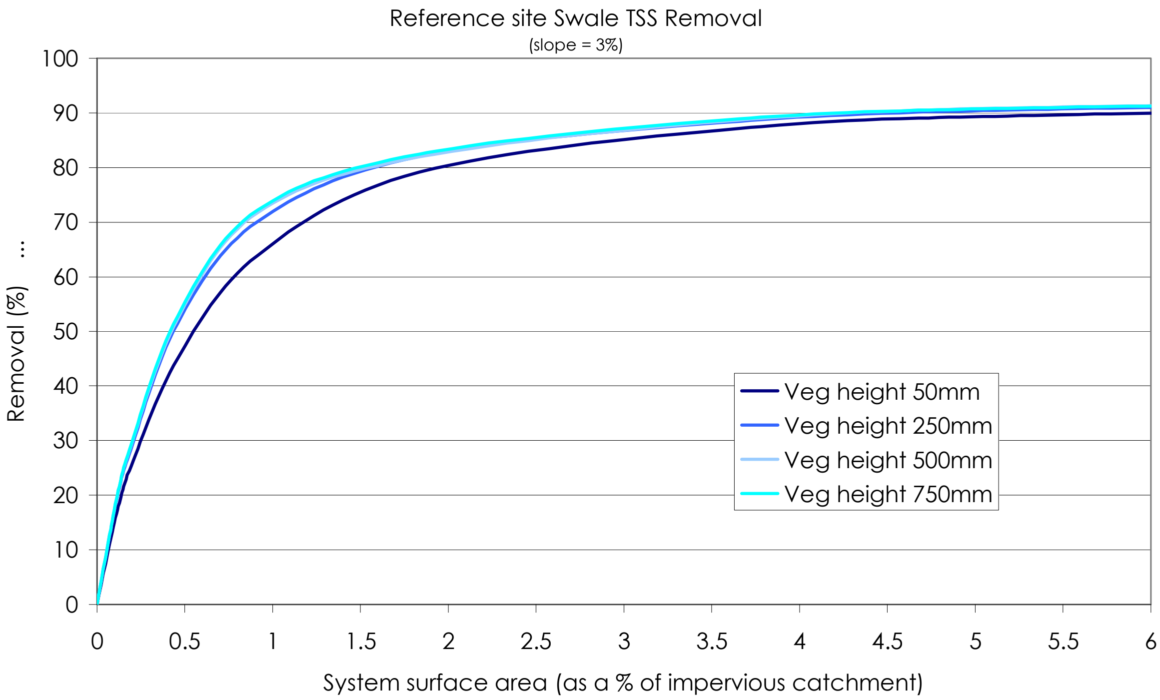


Figure 7.7. TSS removal in swale systems with varying vegetation height

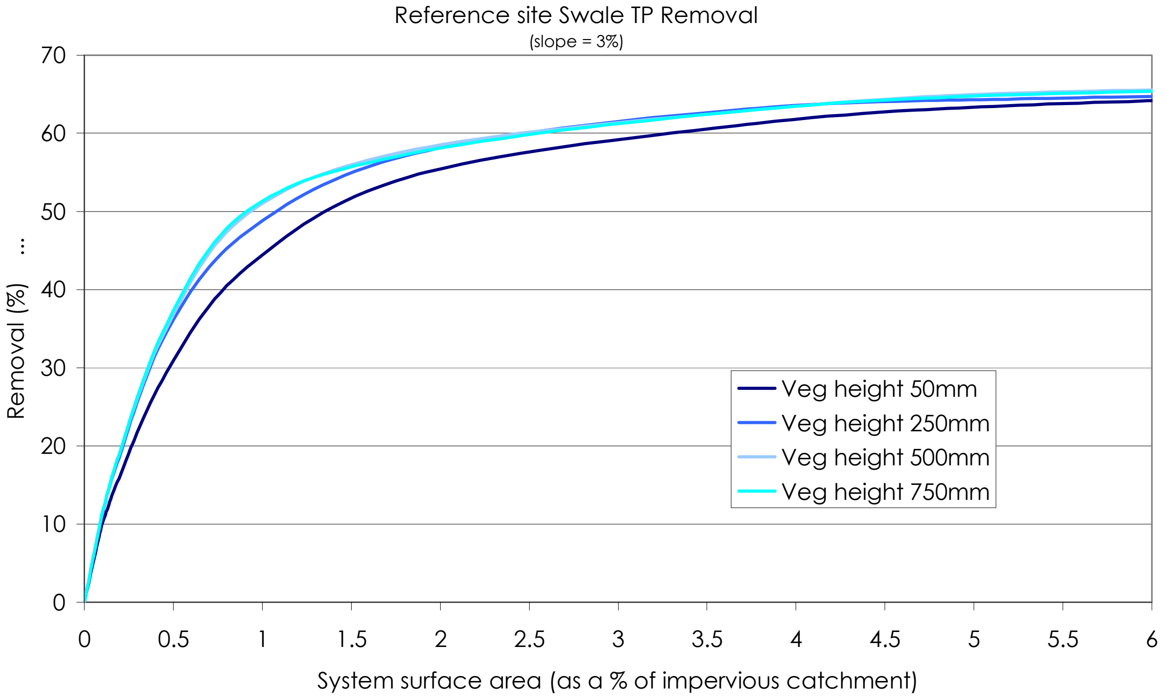


Figure 7.8. TP removal in swale systems with varying vegetation height

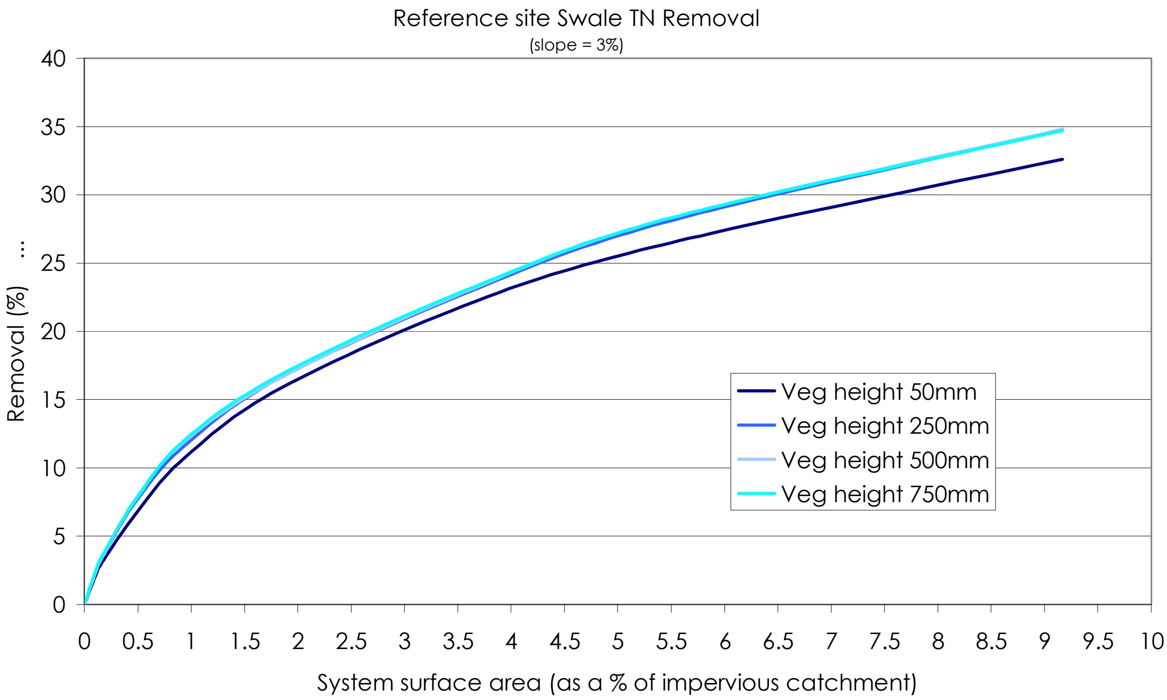


Figure 7.9. TN removal in swale systems with varying vegetation height

7.3 Design procedure: Swales

The following sections detail the design steps required for swale systems.

7.3.1 Estimating design flows

Two design flows are required for swale systems:

- Minor flood rates (typically 5-year ARI) to size overflows and allow conveyance for minor floods and not increase flood risk compared to conventional stormwater systems
- Major flood rates (typically 100 year ARI) to check that flow velocities are not too large in the swale that could scour pollutants or damage vegetation

7.3.1.1 Minor and major flood estimation

A range of hydrologic methods can be applied to estimate design flows. With typical catchment areas being relatively small, the Rational Method Design Procedure is considered to be a suitable method for estimating design flows.

7.3.2 Dimensioning a swale

Constraints relating to a swale alignment and size need to be identified before a swale size can be checked against its flow capacity requirements. Iterations between these factors and an urban concept design may be necessary. Many of these factors should be considered during concept design, nevertheless, should also be checked during detail design. Factors to consider are:

- Allowable width given urban layout
- How flows are delivered into a swale (e.g. cover requirements for pipes or kerb details)
- Longitudinal slope
- Maximum side slopes and base width
- Provision of crossings (elevated or at grade)

Depending on which of the above factors are fixed, other variables can be altered to derive an acceptable configuration.

Once design flows are established, a swale is sized to convey a particular flood frequency or the maximum length of swale is determined for a particular flood frequency. The calculation steps are identical in either approach. The following sections outline some considerations in relation to dimensioning a swale.

7.3.2.1 Side slopes and maximum width of a swale

A maximum width of swale is usually determined from an urban layout, particularly in redevelopment scenarios. This maximum width needs to be identified early in the design process as it informs the remainder of the swale design.

Alternatively, calculations can be made to estimate a required swale width to accommodate a particular flow (e.g. conveyance as the minor drainage system) to inform an urban design.

Other considerations that may influence a swale width are how water is delivered to it and the maximum batter slopes (which can be affected by crossing types).

Selection of an appropriate side slope is heavily dependent on local council regulations and will relate to traffic access and the provision of crossings (if required). The provision of driveway crossings can significantly impact on the required width of the swale. The slope of at-grade crossings (and therefore the swale) are governed by the trafficability of the change in slope across the base of the swale. Typically 1:9 side slopes with a small flat base will provide sufficient transitions to allow for suitable traffic movement for at-grade crossings.

Where narrower swales are required, elevated crossings can be used (with side slopes typically of between 1 in 3 and 1 in 6) and these will require provision for drainage under the crossings with a culvert or similar.

Crossings can provide good locations for overflow points in a swale. However, the distance between crossings will determine how feasible having overflow points at each one is.

Selection of appropriate crossing type should be made in consultation with urban and landscape designers.

7.3.2.2 Maximum length of a swale

In many urban situations, the length of a swale is determined by the maximum allowable width and side slopes (therefore depth). A swale of a set dimension (and vegetation type) will be capable of conveying flows up to a specific rate after which flows will overtop the banks. This point is considered the maximum length of a swale. Overflow pits can be used in these situations where flows surcharge into underground pits and underground pipe networks for conveyance. A swale thus can be adjacent to a long length of road, however, will not convey flows from an entire upstream catchment.

Manning's equation is used to size the swale given the site conditions. This calculation is sensitive to the selection of Manning's *n* and this should vary according to flow depth (as it decreases significantly once flow depths exceed vegetation height). Consideration of the landscape and maintenance of the vegetation will need to be made before selecting a vegetation type.

7.3.3 Swale capacity - selection of Mannings "n"

To calculate the flow capacity of a swale, Manning's equation can be used. This allows the flow rate and levels to be determined for variations in dimensions, vegetation type and slopes.

Manning's $Q = (AR^{2/3}S_o^{1/2})/n$

Equation 7.1

Where A = cross section area

R = hydraulic radius

S = channel slope

n = roughness factor

Manning's '*n*' is a critical variable in the Manning's equation relating to roughness of the channel. It varies with flow depth, channel dimensions and the vegetation type. For constructed swale systems, the values are recommended to be between 0.15 and 0.4 for flow depths shallower than the vegetation height (preferable for treatment) and can be significantly lower (e.g. 0.03) for flows with greater depth than the vegetation (however, it can vary greatly with channel slope and cross section configuration). Further discussion on selecting an appropriate Manning's '*n*' for a swale is provided in Appendix F of the MUSIC modelling manual (eWater, 2009).

It is considered reasonable for Manning's 'n' to have a maximum at the vegetation height and then sharply reduce as depths increase. Figure 8.10 shows a plot of varying Manning's *n* with flow depth for a grass swale. It is reasonable to expect the shape of the Manning's *n* relation with flow depth to be consistent with other swale configurations, with the vegetation height at the boundary between *Low flows* and *Intermediate flows* (Figure 8.10) on the top axis of the diagram. The bottom axis of the plot has been modified from Barling and Moore (1993) to express flow depth as a percentage of vegetation height.

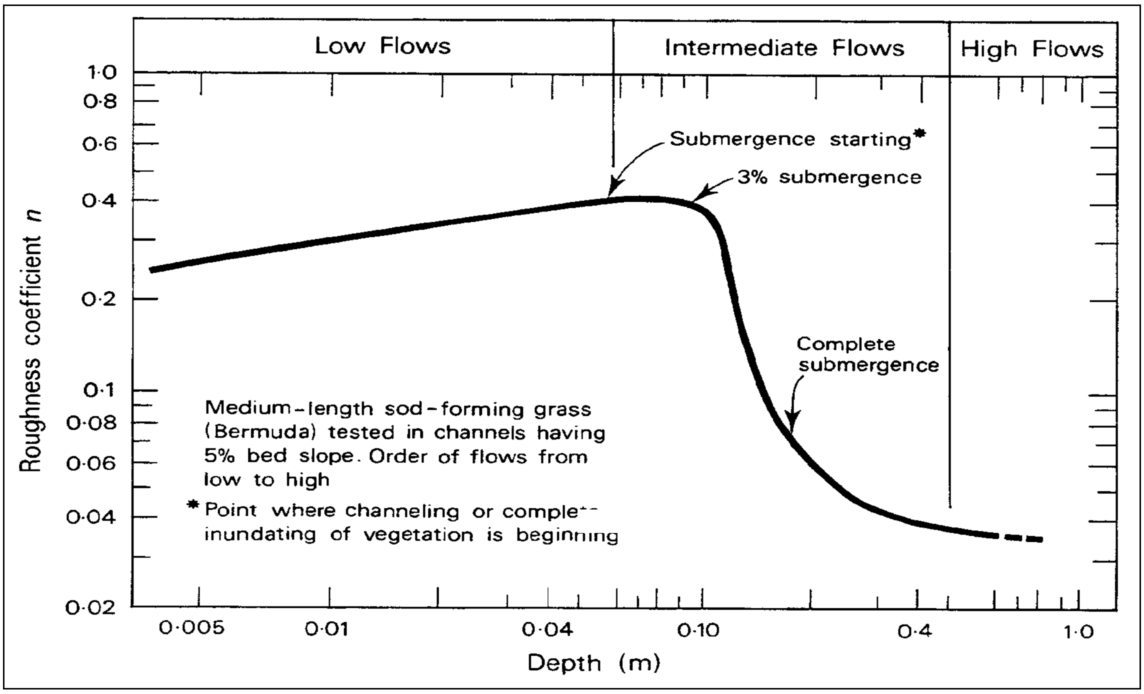


Figure 7.10. Impact of flow depth on hydraulic roughness adapted from Barling and Moore (1993)

7.3.4 Inlet details

Inlets for swale systems can be from distributed runoff (e.g. from flush kerbs along a road) or from point outlets such as pipes. Combinations of these two entrance pathways can also be used.

7.3.4.1 Distributed flows (buffers)

An advantage of flows entering a swale system in a distributed manner (i.e. entering perpendicular to the direction of the swale) is that flow depths are shallow which maximises contact with vegetation. This area is often called a buffer. The requirement of the area is to ensure there is dense vegetation growth, flow depths are kept shallow (below the vegetation height) and erosion is avoided. This provides good pretreatment prior to flows being conveyed down a swale. Creating distributed flows can be achieved either by having a flush kerb or by using kerbs with regular breaks in them to allow for even flows across the buffer surface.

For distributed flows, it is important to provide an area for coarse sediments to accumulate, that is off the road surface. The photograph in Figure 7.11 shows sediment accumulating on a street surface where the vegetation is the same level as the road. To avoid this accumulation, a tapered flush kerb can be used that sets the top of the vegetation between 40-50mm lower than the road surface (Figure 7.11), which requires the top of the ground surface (before turf is placed) to be between 80-100 mm below the road surface. This allows sediments to accumulate off any trafficable surface.

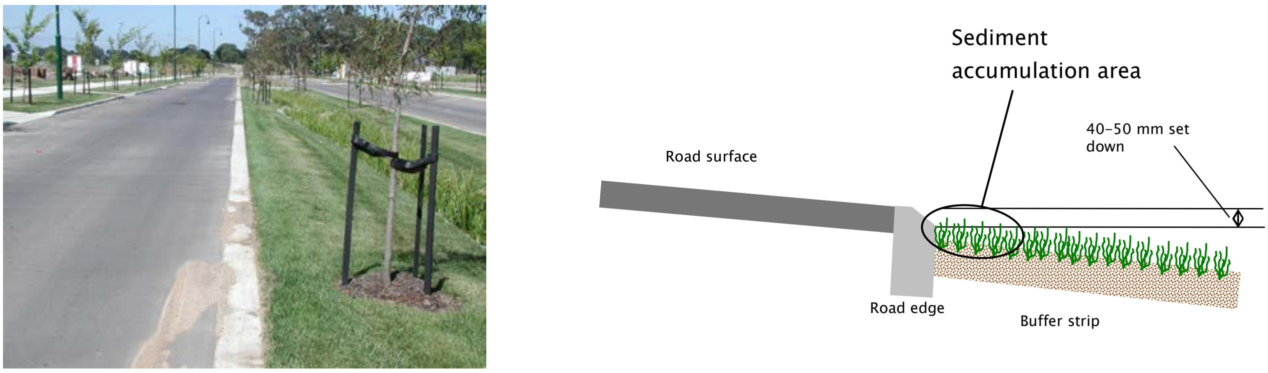


Figure 7.11. Photograph of flush kerb without setdown, edge detail showing setdown

Figure 7.12. Photograph of different arrangements of kerbs with breaks to distribute inflows

7.3.4.2 Direct entry points

Direct entry of flows can either be from overland flow or from a pipe system. For all point entrances into swales, energy dissipation at the inlet point is an important consideration to minimise any erosion potential. This can usually be achieved with rock beaching and dense vegetation.

The most common constraint on pipe systems is bringing the pipe to the surface of a swale within the available width. Generally the maximum width of the system will be fixed and so will maximum batter slopes along the swale (5:1 is typical, however 3:1 may be possible for shallow systems with bollards). Further constraints are the cover required for a pipe that crosses underneath a road, as well as the required grade of the pipe. These constraints need to be considered carefully.

In situations where geometry doesn't permit the pipe to reach the surface, a 'surcharge' pit can be used to bring flows to the surface. Surcharge pits should be designed so that they are as shallow as possible and have pervious bases to avoid long term ponding in the pits (this may require underdrains to ensure it drains, depending on local soil conditions). The pits need to be accessible so that any build up of coarse sediment and debris can be monitored and removed if necessary.

These systems are most frequently used when allotment runoff is required to cross a road into a swale on the opposite side. Several allotments can generally be combined prior to crossing the road to minimise the number of road crossings. Figure 7.13 shows an example of a surcharge pit discharging into a swale.

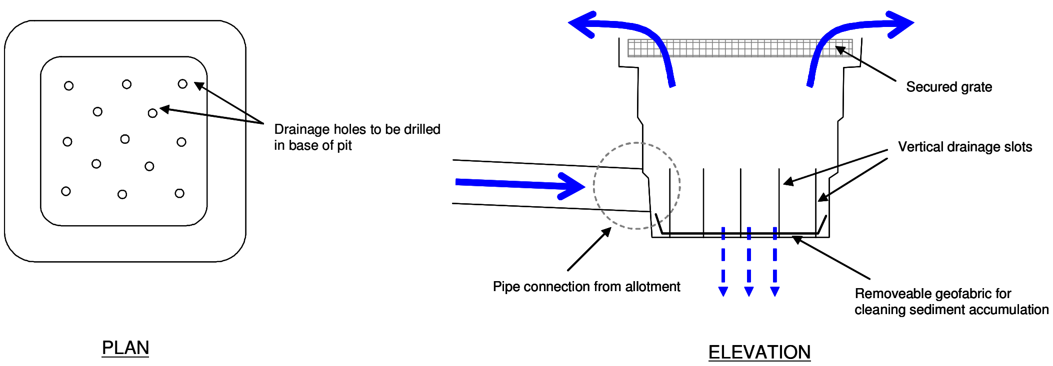


Figure 7.13. Example of surcharge pit for discharging allotment runoff into a swale

7.3.5 Vegetation scour velocity check

Scour velocities over the vegetation along the swale are checked by applying Manning's equation. An important consideration is the selection of an appropriate Manning's 'n' that suits the vegetation height. The selection of an appropriate 'n' is discussed more in the Section 4.3.

Manning's equation should be used to estimate flow velocities and ensure the following criteria are met:

- Less than 0.5 m/s for minor storm (e.g.5-year ARI) discharges
- Less than 1.0 m/s for major storm (e.g.100-year ARI) discharges

7.3.5.1 Velocity check – safety

As swales are generally accessibly by the public it is important to check that flow depths and velocities are acceptable from a public risk perspective. To avoid people being swept away by flows along swales a velocity-depth product check should be performed for design flow rates, as in ARR BkVIII Section 1.10.4.

$$\text{Velocity (m/s) x depth (m)} < 0.4\ \text{m}^2/\text{s}$$

7.3.5.2 Check dams

For steep swales (>4%), check dams can be used to help distribute flows across a swale to avoid preferential flow paths and maximise contact with vegetation. Check dams are typically low level (e.g. 100mm) rock weirs or driveway crossings that are constructed across the base of a swale. A rule of thumb for locating check dams is for the crest of a downstream check dam should be at 4% grade from 100 mm below the toe of an upstream check dam (see Figure 8.14).

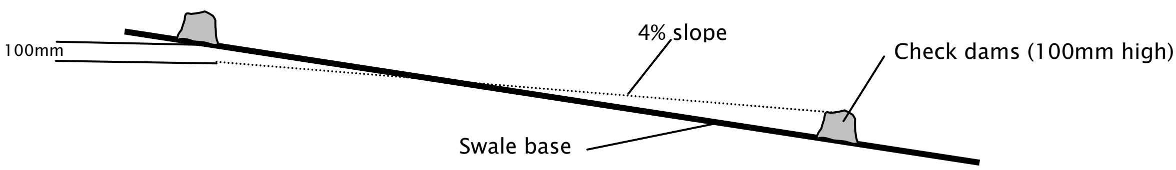


Figure 7.14. Location of check dams in swales

7.3.6 High-flow route and overflow design

The design for high flows must safely convey flows associated with a minor drainage system (e.g. 5-year ARI flows) to the same level of protection that a conventional stormwater system provides. Flows are to be contained within the swale. Where the capacity of the swale system is exceeded at a certain point along it's length, an overflow pit is required. This will discharge excess flows into an underground drainage network for conveyance downstream. The frequency of overflow pits is determined from the capacity of the swale. This section suggests a method to dimension the overflow pits.

The locations of overflow pits is variable, but it is desirable to locate them just upstream of crossings to reduce flows across the crossing.

Typically grated pits are used and the allowable head for discharges is the difference in level of the invert and the nearby road surface. This should be at least 100 mm, but preferably more.

To size a grated overflow pit, two checks should be made to check for either drowned or free flowing conditions. A broad crested weir equation can be used to determine the length of weir required (assuming free flowing conditions) and an orifice equation used to estimate the area between opening required (assumed drowned outlet conditions). The larger of the two pit configurations should be adopted. In addition, a blockage factor is be to used that assumes the orifice is 50% blocked.

Weir flow condition – *when free overall conditions occur over the pit (usually when the extended detention storage of the retarding basin is not fully engaged),* ie.

$$P = \frac{Q_{des}}{B \cdot C_w \cdot H^{1.5}}$$

Equation 7.2

P = Perimeter of the outlet pit

B = Blockage factor (0.5)

H = Depth of water above the crest of the outlet pit

Q_{des} = Design discharge (m^3/s)

C_w = weir coefficient (1.7)

Orifice flow conditions – *when the inlet pit is completely submerged (corresponding to conditions associated with larger flood events*), ie.

$$A_o = \frac{Q_{des}}{B \cdot C_d \sqrt{2gH}}$$

Equation 7.3

C_d = Orifice Discharge Coefficient (0.6)
B = Blockage factor (0.5)
H = Depth of water above the centroid of the orifice (m)
A_o = Orifice area (m^2)
Q_{des} = Design discharge (m^3/s)

It is important that an outlet pit is prevented from blockage by debris. Design consideration needs to include means of preventing blockage of the outlet structure.

7.3.7 Vegetation specification

Table B.1 in Appendix B provides lists of plants that are suitable for swales. Consultation with landscape architects is recommended when selecting vegetation, to ensure the treatment system compliments the landscape of the area.

7.3.8 Design calculation summary

Swales **CALCULATION SUMMARY**

	CALCULATION TASK		OUTCOME		CHECK
1	**Identify design criteria**				
	conveyance flow standard (ARI)			year	
	vegetation height			mm	☐
2	**Catchment characteristics**				
				m^2	
				m^2	
		slope		%	
	Fraction impervious				
		f_{imp}			☐
3	**Estimate design flow rates**				
	Time of concentration				
	estimate from flow path length and velocities			minutes	☐
	Identify rainfall intensities				
		station used for IFD data:			
		major flood – 100 year ARI		mm/hr	
		minor flood – 5 year ARI		mm/hr	
	Peak design flows				
		Q_{minor}		m^3/s	
		Q_{100}		m^3/s	☐
4	**Swale design**				
		Manning's n below vegetation height			
		Manning's n at capacity			☐
5	**Inlet details**				
		adequate erosion and scour protection?			
		flush kerb setdown?		mm	☐
6	**Velocities over vegetation**				
		Velocity for 5 year flow (<0.5m/s)		m/s	
		Velocity for 100 year flow (<1.0m/s)		m/s	
		Safety: Vel x Depth (<0.4)		m^2/s	☐
7	**Overflow system**				
		spacing of overflow pits			
		pit type			☐
8	**Plant selection**				☐

7.4 Checking tools

This section provides a number of checking aids for designers and referral authorities. In addition, advice on construction techniques and lessons learnt from building swale systems are provided.

Checklists are provided for:

- Design assessments
- Construction (during and post)
- Operation and maintenance inspections
- Asset transfer (following defects period).

7.4.1 Design assessment checklist

The checklist below presents the key design features that should be reviewed when assessing a design of a swale. These considerations include configuration, safety, maintenance and operational issues that should be addressed during the design phase.

Where an item results in an "N" when reviewing the design, referral should be made back to the design procedure to determine the impact of the omission or error.

In addition to the checklist, a proposed design should have all necessary permits for its installations. The referral agency should ensure that all relevant permits are in place.

Land ownership and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the asset transfer checklist (see 7.4.4).

Swale Design Assessment Checklist

Swale location:					
Hydraulics	Minor Flood: (m^3/s)		Major Flood: (m^3/s)		
Area	Catchment Area (ha):				
Treatment				**Y**	**N**
Treatment performance verified from curves?					
Inlet zone/hydraulics				**Y**	**N**
Station selected for IFD appropriate for location?					
Longitudinal slope of invert >1% and <4%?					
Mannings 'n' selected appropriate for proposed vegetation type?					
Overall flow conveyance system sufficient for design flood event?					
Maximum flood conveyance width does not impact on traffic amenity?					
Overflow pits provided where flow capacity exceeded?					
Inlet flows appropriately distributed?					
Energy dissipation provided at inlet?					
Velocities within swale cells will not cause scour?					
Set down of at least 50mm below kerb invert incorporated?					
Cells				**Y**	**N**
Maximum ponding depth and velocity will not impact on public safety (v x d <0.4)?					
Maintenance access provided to invert of conveyance channel?					
Protection from gross pollutants provided (for larger systems)?					
Vegetation				**Y**	**N**
Plant species selected can tolerate periodic inundation and design velocites?					
Plant species selected integrate with surrounding landscape design?					

7.4.2 Construction advice

This section provides general advice for the construction of swales. It is based on observations from construction projects around Australia.

Building phase damage

Protection of soil and vegetation is important during building phase, uncontrolled building site runoff is likely to cause excessive sedimentation, introduce weeds and litter and require replanting following the building phase. Can use a staged implementation – i.e. during building use geofabric, soil (e.g. 50mm) and instant turf (laid perpendicular to flow path) to provide erosion control and sediment trapping. Following building, remove and revegetate possibly reusing turf at subsequent stages.

Traffic and deliveries

Ensure traffic and deliveries do not access swales during construction. Traffic can compact the filter media and cause preferential flow paths, deliveries can smother vegetation. Washdown wastes (e.g. concrete) can disturb vegetation and cause uneven slopes along a swale. Swales should be fenced off during building phase and controls implemented to avoid washdown wastes.

Inlet erosion checks

It is good practice to check the operation of inlet erosion protection measures following the first few rainfall events. It is important to check for these early in the systems life, to avoid continuing problems. Should problems occur in these events the erosion protection should be enhanced.

Sediment build-up on roads

Where flush kerbs are to be used, a set-down from the pavement surface to the vegetation should be adopted. This allows a location for sediments to accumulate that is off the pavement surface. Generally a set down from kerb of 50mm to the top of vegetation (if turf) is adequate. Therefore, total set down to the base soil is approximately 100 mm (with 50mm turf on top of base soil).

Timing for planting

Timing of vegetation is dependent on a suitable time of year (and potential irrigation requirements) as well as timing in relation to the phases of development. For example temporary planting during construction for sediment control (e.g. with turf) then remove and plant out with long term vegetation.

7.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Swales

INSPECTED BY:
DATE:
TIME:
WEATHER:
CONTACT DURING VISIT:

SITE: ____________________

CONSTRUCTED BY: ____________________

DURING CONSTRUCTION									
Items inspected	Checked		Satisfactory	Unsatisfactory		Checked		Satisfactory	Unsatisfactory
Preliminary works	Y	N			**Structural components**	Y	N		
1. Erosion and sediment control plan adopted					11. Location and levels of pits as designed				
					12. Safety protection provided				
2. Traffic control measures					13. Location of check dams as designed				
3. Location same as plans					14. Swale crossings located and built as designed				
4. Site protection from existing flows									
Earthworks					15. Pipe joints and connections as designed				
5. Level bed of swale					16. Concrete and reinforcement as designed				
6. Batter slopes as plans					17. Inlets appropriately installed				
7. Longitudinal slope in design range					18. Inlet erosion protection installed				
8. Provision of shallow drainage for mild slopes					19. Set down to correct level for flush kerbs				
					Vegetation				
9. Compaction process as designed					20. Stablisation immediately following earthworks				
10. Appropriate topsoil on swale									
					21. Planting as designed (species and densities)				
					22. Weed removal before stabilisation				
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of soil				
2. Traffic control in place					7. Inlet erosion protection working				
3. Confirm structural element sizes					8. Maintenance access provided				
4. Check batter slopes					9. Construction sediment removed				
5. Vegetation as designed					10. Evidence of local surface ponding				

COMMENTS ON INSPECTION

ACTIONS REQUIRED

1.
2.
3.
4.
5.
6.

7.4.4 Asset transfer checklist

Asset Handover Checklist		
Asset Location:		
Construction by:		
Defects and Liability Period		
Treatment	**Y**	**N**
System appears to be working as designed visually?		
No obvious signs of under-performance?		
Maintenance	**Y**	**N**
Maintenance plans provided for each asset?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
Asset inspected for defects?		
Asset Information	**Y**	**N**
Design Assessment Checklist provided?		
As constructed plans provided?		
Copies of all required permits (both construction and operational) submitted?		
Proprietary information provided (if applicable)?		
Digital files (eg drawings, survey, models) provided?		
Asset listed on asset register or database?		

7.5 Maintenance requirements

Swale systems treat runoff by filtering it through vegetation and then passing the runoff downstream. Treatment relies upon contact with vegetation and therefore maintaining vegetation growth is the main maintenance objective. In addition, they have a flood conveyance role that needs to be maintained to ensure adequate flood protection for local properties.

The potential for rilling and erosion down a swale needs to be carefully monitored, particularly during establishment stages of the system.

The most intensive period of maintenance is during the plant establishment period (first two years) when weed removal and replanting may be required. It is also the time when large loads of sediments could impact on plant growth, particularly in developing catchments with poor building controls.

Other components of the system that will require careful consideration are the inlet points (if the system does not have distributed inflows). The inlets can be prone to scour and build up of litter and surcharge pits in particular will require routine inspections. Occasional litter removal and potential replanting may be required.

Overflow pits also require routine inspections to ensure structural integrity and that they are free of blockages with debris.

Maintenance is primarily concerned with:

- Maintenance of flow to and through the system
- Maintaining vegetation
- Preventing undesired vegetation from taking over the desirable vegetation
- Removal of accumulated sediments
- Litter and debris removal

Vegetation maintenance will include:

- Removal of noxious plants or weeds
- Re-establishment of plants that die

Sediment accumulation at the inlet points needs to be monitored. Depending on the catchment activities (e.g. building phase), the deposition of sediment can tend to smother plants and reduce the ponding volume available. Should excessive sediment build up, it will impact on plant health and require removal before it reduces the infiltration rate of the filter media.

Similar to other types of practices, debris removal is an ongoing maintenance function. Debris, if not removed, can block inlets or outlets, and can be unsightly if located in a visible location. Inspection and removal of debris should be done regularly, but debris should be removed whenever it is observed on a site.

Inspections are also recommended following large storm events to check for scour.

7.5.1 Operation & maintenance inspection form

The form below should be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

Swale and Buffer Maintenance Checklist			
Inspection Frequency: 3 monthly	Date of Visit:		
Location:			
Description			
Site Visit by:			
Inspection Items	**Y**	**N**	**Action Required (details)**
Sediment accumulation at inflow points?			
Litter within swale?			
Erosion at inlet or other key structures (eg crossovers)?			
Traffic damage present?			
Evidence of dumping (eg building waste)?			
Vegetation condition satisfactory (density, weeds etc)?			
Replanting required?			
Mowing required?			
Sediment accumulation at outlets?			
Clogging of drainage points (sediment or debris)?			
Evidence of ponding?			
Set down from kerb still present?			
Comments:			

7.6 Swale worked example

7.6.1 Worked example introduction

As part of a development in Hobart, runoff from allotments and a street surface is to be collected and conveyed in a vegetated swale system to downstream treatments, the intention being for a turf swale system. An additional exercise in this worked example is to investigate the consequences on flow capacity of using a vegetated (e.g. sedges) swale (vegetation height equal to 300mm).

A concept design for the development suggested this system as part of a treatment train. The street will have a one-way crossfall (to the high side) with flush kerbs, to allow for distributed flows into the swale system across a buffer zone.

The swale is to convey minor flood events, including all flows up to a five-year ARI storm. However, the width of the swale is fixed (at 4.5m) and there will be a maximum catchment area the swale can accommodate, above which an underground pipe will be required to preserve the conveyance properties of the downstream swale. Access to the allotments will be via an at-grade crossover with a maximum slope of 1 in 9 (11%).

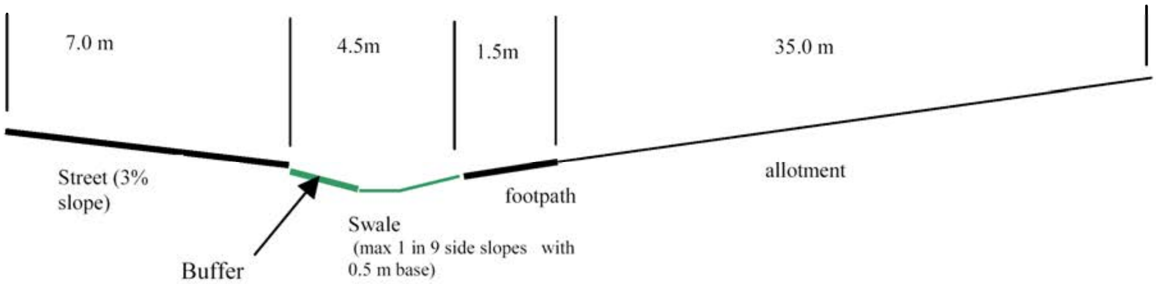


Figure 7.15. Cross section of proposed buffer/swale system

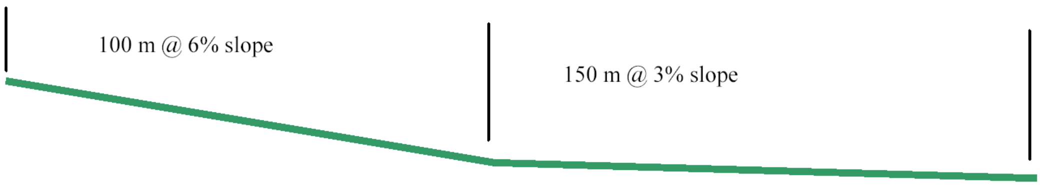


Figure 7.16. Long section of proposed buffer/swale system

The contributing catchment area includes 35 m deep (and 10m wide) allotments on one side, a 7m wide road pavement surface and a 1.5 m footpath and 4.5 m swale and services easement (Figure 7.15). The area is 250 m long with the top 100m having a 6% slope and the bottom 150m having a 3% slope (Figure 8.16).

Allotment runoff is to be discharged under a footpath via a conventional stormwater pipe directly into the swale system with appropriate erosion control.

Figure 7.17. Similar buffer swale system for conveying runoff

Design criteria for the buffer/ swale system are to:

- Promote sedimentation of coarse particles through the buffer by providing for an even flow distribution and areas for sediment accumulation (i.e. set down at kerb edge);

- Provide traffic management measures that will preclude traffic damage (or parking) within the buffer or swale (e.g. bollards or parking bays);
- Provide check dams to control velocities and spread flows (potentially using crossings);
- Provision of driveway access to lots given side slope limits; and
- Provision to convey 5-year ARI flows within the swale and underground pipe system.

This worked example focuses on the design of the buffer strip and vegetated swale conveyance properties. Analyses to be undertaken during the detailed design phase include the following:

- Design the swale system to accommodate driveway crossovers and check dams where required
- Select vegetation such that the hydraulic capacity of the swale is sufficient
- Determine maximum length of swale to convey 5 year flows before an underground pipe is required
- Check velocities are maintained to acceptable levels
- Overflow structure from swale to underground pipe (if required).

Additional design elements will be required, including:

- Configure the street kerb details such that sheet flow is achieved through the buffer strip
- Configure house lot drainage so that erosion control is provided
- Buffer strip vegetation
- Swale vegetation (integral with hydraulic design of the system).

7.6.1.1 Design Objectives

- Swale shall convey at least all flows up to the peak 5-year ARI storm event.
- Sedimentation of coarse particles will be promoted within the buffer by providing an even flow distribution.
- Prevent traffic damage to the buffer swale system.
- Flow velocities to be controlled to prevent erosion.
- Allowance for suitable driveway gradients (max 1:9) to be provided at crossovers into properties.

7.6.1.2 Site Characteristics

Catchment area	8,750m^2	(lots)
	2,125 m^2	(roads and concrete footpath)
	1,125 m^2	(swale and services easement)
	12,000 m^2	
Landuse/surface type	Residential lots, roads/concrete footpaths, swale and service easement.	
Overland flow slope:	Total main flowpath length = 250m	
	Upper section = 100m@ 6% slope	
	Lower section = 150m@ 3% slope	
Soil type:	Clay	
Fraction impervious:	lots f = 0.65	
	roads/footpath f = 1.00	
	swale/service easement f = 0.10	

7.6.1.3 Confirm size for treatment

Interpretation of 7.2 Verifying size for treatment with the input parameters below is used to estimate the reduction performance of the swale system to ensure the design will achieve target pollutant reductions.

- Reference site location
- Average slope of 5% along swale
- Vegetation height of 50 mm

To interpret the graphs the area of swale base to the impervious catchment needs to be estimated.

Area of swale base / impervious catchment area

0.5x250/ [(0.65x8750)+(1.0x2125)+(0.1x1125)] = 1.6%

To apply the performance curves the area = 1.6%

From the figures using an equivalent area in the reference site, it is estimated that pollutant reductions are 90%, 63% and 28% for TSS, TP and TN respectively. For real-world design, the adjustment factor/hydrologic region methodology should be applied to calculate the actual size of system required at the development site.

DESIGN NOTE – The values derived from 7.2 Verifying size for treatment will only be valid if the design criteria for the proposed installation are similar to those used to create the Figures. Site specific modelling using programs such as MUSIC (eWater, 2009) may yield a more accurate result.

7.6.2 Estimating design flows

With a small catchment, the Rational Method is considered an appropriate approach to estimate the 5 and 100 year ARI peak flow rates. The steps in these calculations follow below.

See Appendix E Design Flows – t_c for a discussion on methodology for calculation of time of concentration.

7.6.2.1 Major and minor design flows

The procedures in Australian Rainfall and Runoff (ARR) are used to estimate the design flows.

Step 1 – Calculate the time of concentration.

The time of concentration is estimated assuming overland flow across the allotments and along the swale. From procedures in AR&R, t_c is estimated to be 10 minutes.

Rainfall Intensities for the area of study (for the 5 and 100 year average recurrence intervals) are estimated using ARR (1998) with a time of concentration of 10 minutes are:

t_c	100yr	5yr
10 min	140*	67*

* These figures are for the worked example only. The appropriate region and corresponding rainfall intensities must be selected for each individual project.

Step 2 – Calculate design runoff coefficients (using the method outlined in Australian Rainfall and Runoff Book VIII (Engineers Australia, 2003)).

Apply method outlined in Section1.5.5 (iii) ARR 2001 Bk VIII

$C_{10} = 0.9f + C^1{}_{10}\ (1-f)$

Fraction impervious

$f \quad = (8750 \times 0.65 + 2125 \times 1 + 1125 \times 0.1)/12000$

$\quad = 0.66$

Apply the rational formula method outlined in Section 1.5.5 (iii) AR&R 2001 Bk VIII:

${}^{10}I_1 = 30.1$ mm/hr (Hobart)

$C^1{}_{10} = 0.1 + 0.0133\ ({}^{10}I_1 - 25)$

$C^1{}_{10} = 0.17$

Calculate C_{10} (10 year ARI runoff coefficient)

$C_{10} = 0.9f + C^1{}_{10}\ (1-f)$

$C_{10} = 0.65$

Step 3 – Convert C_{10} to values for C_5 and C_{100}

Where – $C_y = F_y \times C_{10}$

Runoff coefficients as per Table 1.6 Book VIII ARR 1998

	C_5	C_{100}
Cell A	0.65	0.78

Step 4 – Calculate peak design flow (calculated using the Rational Method).

$$Q = \frac{CIA}{360}$$

Where – C is the runoff coefficient (C_5 and C_{100})

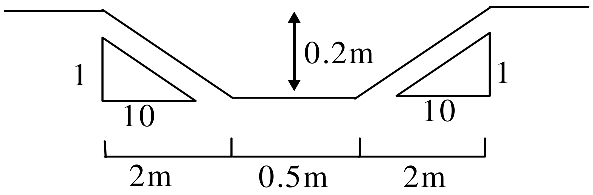
I is the design rainfall intensity mm/hr (I_5 and I_{100})

A is the catchment area (Ha)

Q_5	Q_{100}
0.14	0.36

7.6.3 Swale dimensions

To facilitate at-grade driveway crossings the following cross section is proposed:



7.6.4 Swale flow capacity

The capacity of the swale is firstly estimated at the most downstream point. It is considered to be the critical point in the swale as it has the largest catchment and has the mildest slope (it is assumed that the dimension of the swale will be the same for both the steep and mild sloped areas for Aesthetic reasons. Flow velocities will also need to be checked at the downstream end of the steep section of swale.

The worked example firstly considers the swale capacity using a grass surface with a vegetation height of 50 mm. An extension of the worked example is to investigate the consequence of using 300 mm high vegetation (e.g. sedges) instead of grass.

7.6.4.1 *Selection of manning* n

A range of Manning's *n* values are selected for different flow depths appropriate for grass. It is firstly assumed that the flow height for a 5 year ARI storm will be above the vegetation and therefore Manning's *n* is quite low. A figure of 0.04 is adopted. (The flow depth will need to be checked to ensure it is above the vegetation)

- Adopt slope 3% (minimum longitudinal slope)
- Manning's *n* = 0.04 (at 0.2m depth)
- Side slopes 1(v):10(h)

Manning's $Q=(AR^{2/3}S_o^{1/2})/n$

$Q_{cap} = 0.50m^3/s >> Q_5\ (0.14m^3/s)$

The nominated swale has sufficient capacity to convey the required peak Q_5 flow without any requirement for an additional piped drainage system. The capacity of the swale (Q_{cap} = $0.50m^3/s$) is also sufficient to convey the entire peak Q_{100} flow of $0.36m^3/s$ without impacting on the adjacent road and footpath.

To investigate flow rates at lower depths, Manning's *n* is varied according to the flow depth relating to the vegetation height. This can be performed simply in a spreadsheet application. The values adopted here are:

Table 7-1. Manning's *n* and flow capacity variation with flow depth - turf

Flow Depth	Mannings n	Flow rate
(m)		(m3/s)
0.05	0.30	0.003
0.1	0.30	0.01
0.15	0.10	0.10
0.2	0.04	0.50

From the table of Manning's equation output, it can be seen that the 5 year ARI flow depth is above the vegetation height and therefore the Manning's *n* assumption would seem reasonable.

7.6.4.2 Option 2 – assume higher vegetation

For the purposes of this worked example, the capacity of the swale is also estimated when using 300mm high vegetation (e.g. sedges). The higher vegetation will increase the roughness of the swale (as flow depths will be below the vegetation height) and therefore a higher Manning's *n* should be adopted.

The table on the following page presents the adopted Manning's *n* values and the corresponding flow capacity of the swale for different flow depths.

Table 7-2. Manning's *n* and flow capacity variation with flow depth - sedges

Flow Depth	Mannings n	Flow rate
(m)		(m3/s)
0.05	0.35	0.003
0.1	0.32	0.01
0.15	0.30	0.03
0.2	0.30	0.07

It can be seen above that the swale with current dimensions is not capable of conveying a 5-year discharge. Either the swale depth would need to be increased or overflow pits provided to convey a 5-year ARI flow.

This worked example continues using grass for the remainder.

7.6.5 Inlet details

There are two ways for flows to reach the swale, either directly from the road surface or from allotments via an underground 100mm pipe.

Direct runoff from the road enters the swale via a buffer (the grass edge of the swale). The pavement surface is set 50 mm higher than the start of the swale and has a taper that will allow sediments to accumulate in the first section of the buffer, off the pavement surface. Traffic control is achieved by using traffic bollards.

Flows from allotments will discharge into the base of the swale and localised erosion protection is provided with grouted rock at the outlet point of the pipe.

These are detailed in the construction drawings.

7.6.6 Velocity checks

Two velocity checks are performed to ensure vegetation is protected from erosion at high flow rates. 5-year and 100-year ARI flow velocities are checked and need to be kept below 0.5m/s and 1.0 m/s respectively.

Velocities are estimated using Manning's equation:

Firstly, velocities are checked at the most downstream location (ie. slope = 3%)

$d_{5\text{-year}} = 0.16$ m

$V_{5\text{-year}} = 0.44$ m/s < 0.5 m/s therefore OK

$D_{100\text{-year}} = 0.19$ m

$V_{100\text{-year}} = 0.70$ m/s < 1.0 m/s therefore OK

Secondly, velocities are checked at the bottom of the steeper section (ie. slope = 6% with reduced catchment area)

$d_{5\text{-year}} = 0.13$ m ($Q_5 = 0.06 m^3/s$)

$V_{5\text{-year}} = 0.29$ m/s < 0.5 m/s therefore OK

$D_{100\text{-year}} = 0.15$ m ($Q_{100} = 0.15 m^3/s$)

$V_{100\text{-year}} = 0.47$ m/s < 1.0 m/s therefore OK

7.6.6.1 Safety check

Check at both critical points (bottom of steep section and bottom of entire swale) that velocity depth product is less than 0.4 during a 100 year ARI flow.

At bottom of steep section:

V= 0.47 m/s, d= 0.15m; therefore V.d = 0.07 m^2/s <0.4 therefore OK.

At bottom of swale:

V= 0.70 m/s, d= 0.19m; therefore V.d = 0.13 $m^2.s$ <0.4 therefore OK.

7.6.6.2 Check dams

Given the steep slope of the upper part of the swale (6%), check dams are required to help to distribute flows across the base of the swale in the upper section. These are to be placed every 10 m along the steep part of the swale, be approximately 100 mm high and be constructed of stone. The check dams are to cross the base of the swale and merge into the batters.

7.6.7 Overflow structures

As the swale can carry a five year ARI discharge, overflow structures are not required for this worked example. See Chapter 4 for an example including the design of an overflow pit.

7.6.8 Vegetation specification

To compliment the landscape design of the area, a turf species is to be used. For this application a turf with a height of 50 mm has been assumed. The actual species will be selected by the landscape designer.

7.6.9 Calculation summary

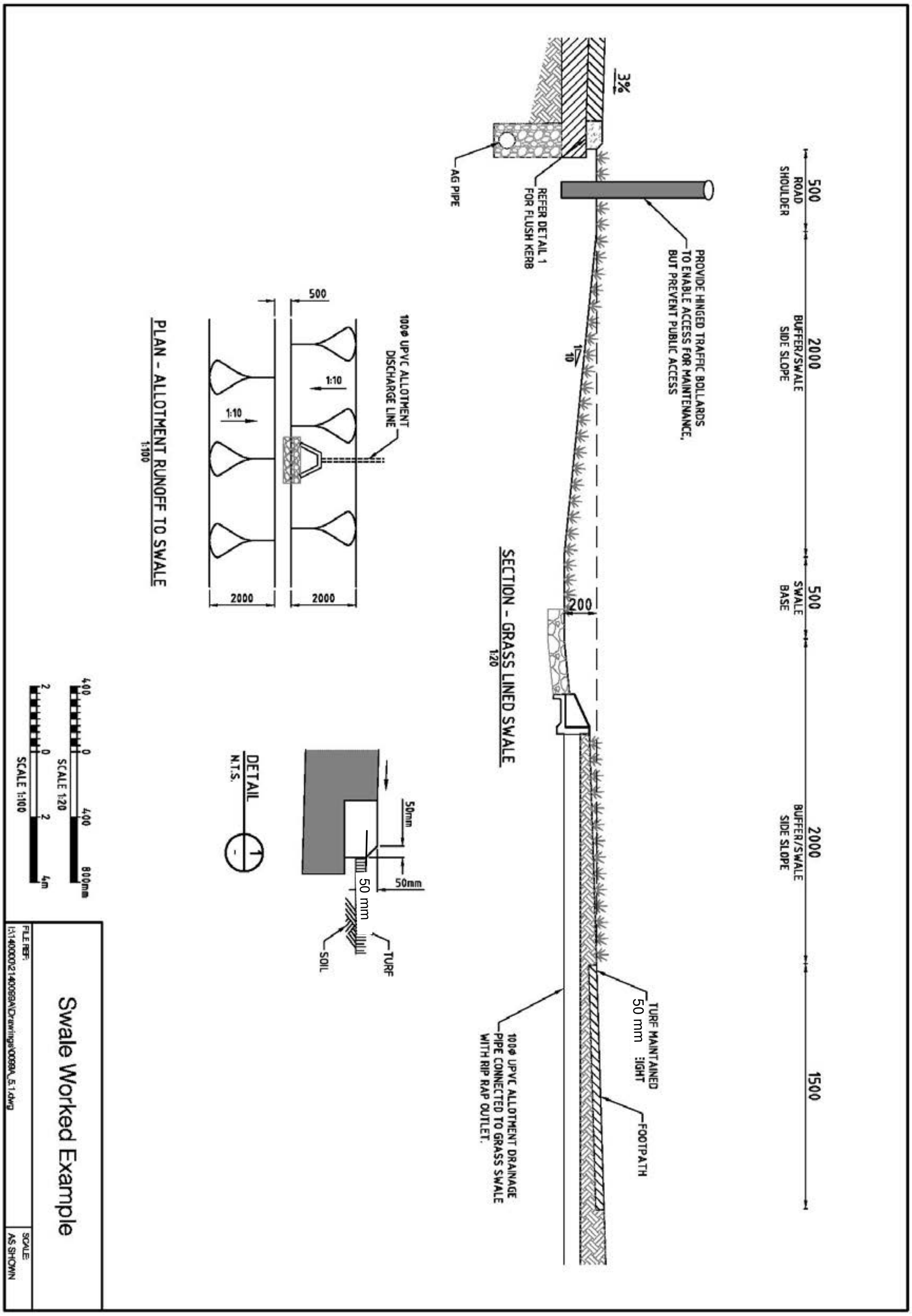
The sheet overleaf shows the results of the design calculations.

Swales

CALCULATION SUMMARY

	CALCULATION TASK		OUTCOME		CHECK
1	**Identify design criteria**				
	conveyance flow standard (ARI)		5	year	
	vegetation height		50	mm	✓
2	**Catchment characteristics**				
		Upper area	4,800	m^2	
		total area	12,000	m^2	
		slope	3 and 6	%	
	Fraction impervious				
		f_{imp}	0.66		✓
3	**Estimate design flow rates**				
	Time of concentration				
	estimate from flow path length and velocities		10	minutes	✓
	Identify rainfall intensities				
		station used for IFD data:	Hobart		
		major flood – 100 year ARI	140	mm/hr	
		minor flood – 5 year ARI	67	mm/hr	
	Peak design flows				
		Q_{minor}	0.14	m^3/s	
		Q_{100}	0.36	m^3/s	✓
4	**Swale design**				
		Manning's n below vegetation height	0.3		
		Manning's n at capacity	0.04		✓
5	**Inlet details**				
		adequate erosion and scour protection?	rock pitching		
		flush kerb setdown?	50	mm	✓
6	**Velocities over vegetation**				
		Velocity for 5 year flow (<0.5m/s)	0.09	m/s	
		Velocity for 100 year flow (<1.0m/s)	0.49	m/s	
		Safety: Vel x Depth (<0.4)	0.13	m^2/s	✓
7	**Overflow system**				
		spacing of overflow pits	not required		
		pit type			✓
8	**Plant selection**		*turf*		✓

7.6.10 Construction drawings



7.7 References

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Chapter 8 Constructed Wetlands

Definition:

A constructed wetland is a man-made copy of a natural wetland system used for the treatment of stormwater runoff.

Purpose:

- Removal of fine and coarse sediments.
- Efficient removal of hydrocarbons and other soluble or fine particulate contaminants from biological uptake.
- To provide extended detention.
- Provide flow retardation for frequent (low ARI) rainfall events.

Implementation considerations:

- In addition to playing an important role in stormwater treatment, wetlands can also have significant community benefits. They provide habitat for wildlife and a focus for recreation, such as walking paths and resting areas. They can also improve the Aesthetics of a development and be a central feature in a landscape.
- Wetlands can be constructed on many scales, from house block scale to large regional systems. In highly urban areas they can have a hard edge form and be part of a streetscape or forecourts of buildings. In regional settings they can be over 10 hectares in size and provide significant habitat for wildlife.

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8.2 Verifying size for treatment .... 8—6

8.3 Design procedure: constructed wetlands .... 8—8

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8.5 Maintenance requirements .... 8—25

8.6 Worked example .... 8—28

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8.1 Introduction

Constructed wetland systems are shallow extensively vegetated water bodies that use enhanced sedimentation, fine filtration and pollutant uptake processes to remove pollutants from stormwater. Water levels rise during rainfall events and outlets are configured to slowly release flows, typically over three days, back to dry weather water levels.

Wetlands generally consist of an inlet zone (sediment basin to remove coarse sediments), a macrophyte zone (a shallow heavily vegetated area to remove fine particulates and uptake of soluble pollutants) and a high flow bypass channel (to protect the macrophyte zone). They are designed to remove stormwater pollutants associated with fine to colloidal particulates and dissolved contaminants. Figure 8.1 shows an example layout of a wetland system.

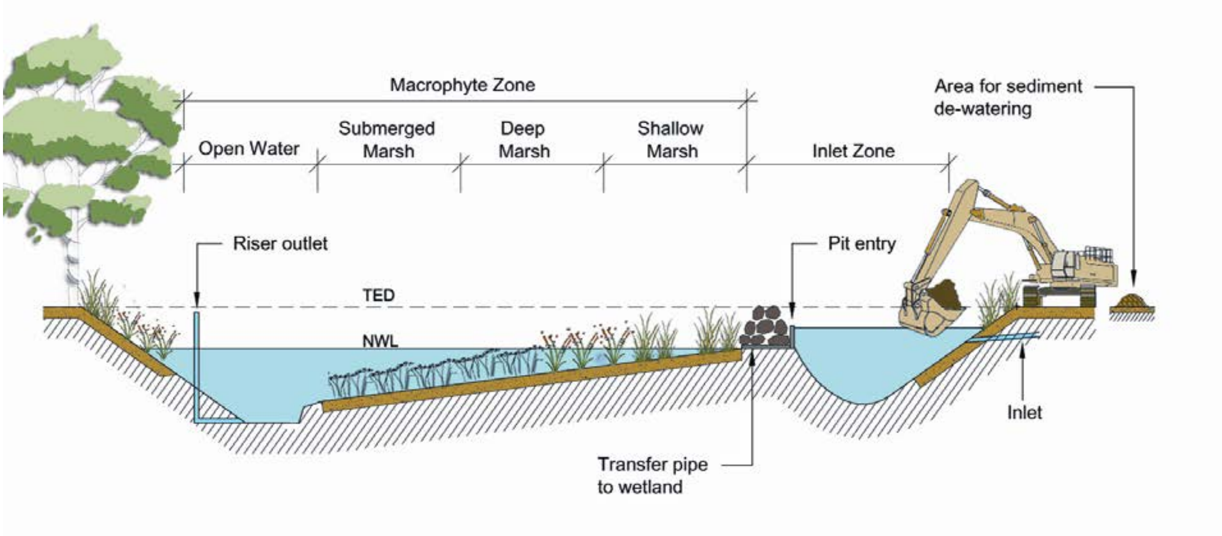


Figure 8.1. Layout of a constructed wetland system

Simulations using computer models are often undertaken to optimise the relationship between detention time, wetland volume and the hydrologic effectiveness of the constructed wetland to maximise treatment. The relationship between detention time and pollutant removal efficiency is influenced by the settling velocity of the target particle size. Standard equations for settling velocities often do not apply for very fine particulates owing to the influence of external factors such as wind and water turbulence. It is recommended that detention periods should notionally be about 72 hours.

The key operational design criteria for constructed wetlands may be summarised as follows:

- Promote sedimentation of particles larger than 125 µm within the inlet zone.
- Discharge water from the inlet zone into the macrophyte zone for removal of fine particulates and dissolved contaminants through the processes of enhanced sedimentation, filtration, adhesion and biological uptake.
- Ensure that the required detention period is achieved for all flow though the wetland system through the incorporation of a riser outlet system.

- Ensure adequate flood protection of the macrophyte zone from scouring during "above-design" conditions by designing for by-pass operation when inundation in the macrophyte zone reaches the design maximum extended detention depth.

Poor design of constructed wetlands has led to many urban wetlands and ponds becoming a long term liability to the community. Common problems encountered include:

- accumulation of litter in some sections of the wetland;
- accumulation of oil and scum at "dead zones" in the wetland;
- infestation of weeds or dominance of certain species of vegetation;
- mosquito problems;
- algal blooms;
- scouring of sediment and banks, especially during high flows.

Many of the above problems can be minimised or avoided by good engineering design principles. Poor wetland hydrodynamics and lack of appreciation of the stormwater treatment chain are often identified as major contributors to wetland management problems. A summary of desired hydrodynamic characteristics and design elements is presented below.

Table 8-1. Desired Wetland Hydrodynamic Characteristics and Design Elements

Hydrodynamic Characteristics	Design Issues	Remarks
Uniform distribution of flow velocity	Wetland shape, inlet and outlet placement and morphological design of wetland to eliminate short-circuit flow paths and "dead zones".	Poor flow pattern within a wetland will lead to zones of stagnant pools which promote the accumulation of litter, oil and scum as well as potentially supporting mosquito breeding. Short circuit flow paths of high velocity will lead to the wetland being ineffective in water quality improvement.
Inundation depth, wetness gradient, base flow and hydrologic regime	Selection of wetland size and design of outlet control to ensure compatibility with the hydrology and size of the catchment.	Regular flow throughput in the wetland promotes flushing of the system and maintains a dynamic system, avoiding problems associated with stagnant water e.g. algal blooms, mosquito breeding, oil and scum accumulation

Hydrodynamic Characteristics	Design Issues	Remarks
	Morphological and outlet control design to match botanical layout design and the hydrology of the wetland.	etc. Inadequate attention to the inundation depth, wetness gradient of the wetland and the frequency of inundation at various depth range would lead to dominance of certain plant species, especially weed species which, over time, results in a deviation from the intended botanical layout of the wetland. Recent research findings indicate that regular wetting and drying of the substrata of the wetland can prevent releases of phosphorus from the sediment deposited in the wetland.
Uniform vertical velocity profile	Selection of plant species and location of inlet and outlet structures to promote uniform velocity profile	Preliminary research findings have indicated that certain plant species have a tendency to promote stratification of flow conditions within a wetland leading to ineffective water pollution control and increase the potential for algal bloom.
Scour protection	Design of inlet structures and erosion protection of banks	Owing to the highly dynamic nature of stormwater inflow, measures are to be taken to “protect” the wetland from erosion during periods of high inflow rates.

In many urban applications, wetlands can be constructed in the base of retarding basins, thus reducing the land required for stormwater treatment. In these situations, the wetland systems will occasionally become inundated to greater depths than the extended detention depth. However, the inundation is relatively short (hours) and is unlikely to affect the vegetation provided there is a safe pathway to drain following flood events that does not scour vegetation or banks.

Key design issues to be considered are:

1. verifying size and configuration for treatment
2. determining design flows
3. design of Inlet Zone (see Design Procedure for Sedimentation Basin, Chapter 4)
4. Macrophyte Zone Layout
   - zonation
   - longitudinal and cross sections
5. hydraulic Structures:
   - Macrophyte Zone outlet structures
   - connection to Inlet Zone
   - bypass weir and channel
6. Recommended plant species and planting densities
7. Provision for maintenance

8.2 Verifying size for treatment

The curves are based on the performance of the system at the reference site with varying extended detention depths and were derived using the MUSIC (eWater, 2009). To estimate an equivalent performance at other locations in Tasmania, the hydrologic design region relationships should be used to convert the treatment area into an equivalent treatment area, refer to Chapter 2. In preference to using the curves, local data should be used to model the specific treatment performance of the system.

The curves were derived assuming the systems receive direct runoff (ie. no pretreatment) and have the following characteristics:

- the Inlet Zone forms part of the wetland system sized to retain 125 μm sediment for flows up to the 1 year ARI peak discharge and with provision for high flow bypass
- notional detention period of 72 hours.

The curves in Figure 8.2 to 8.4 can be used to check the expected performance of the wetland system for removal of TSS, TP and TN. The x-axis on the curves is a measure of the size of the surface of the wetland (measured as the permanent pool area), expressed as a percentage of the contributing *impervious* catchment.

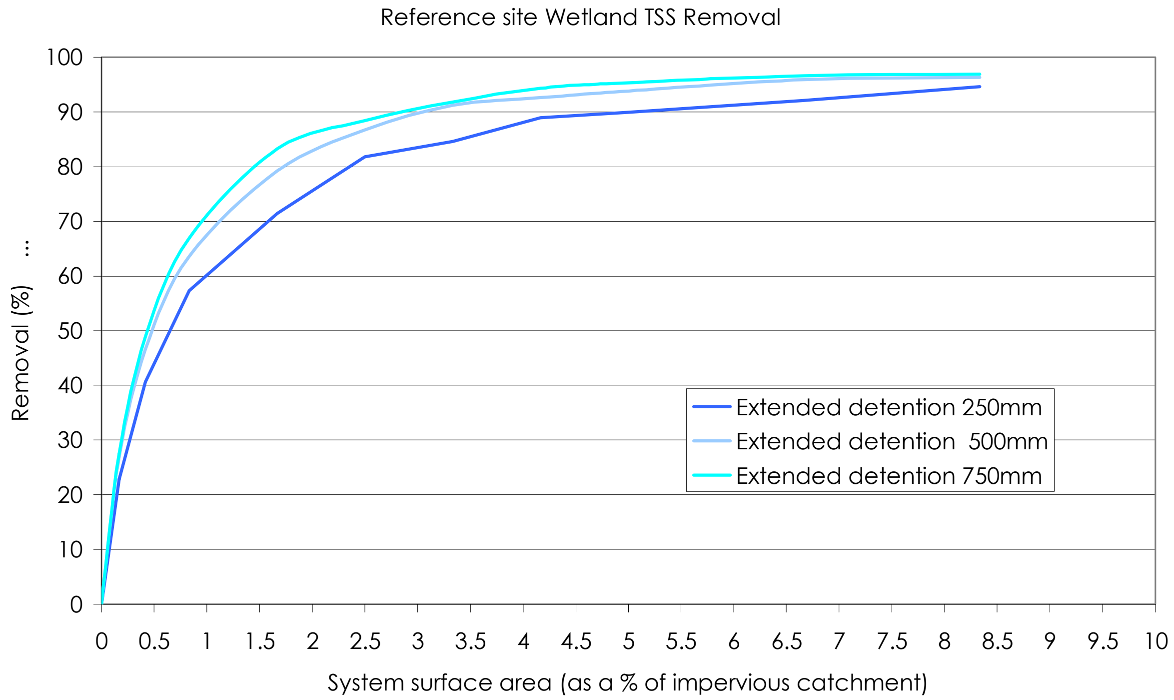


Figure 8.2. TSS removal in wetland systems with varying extended detention

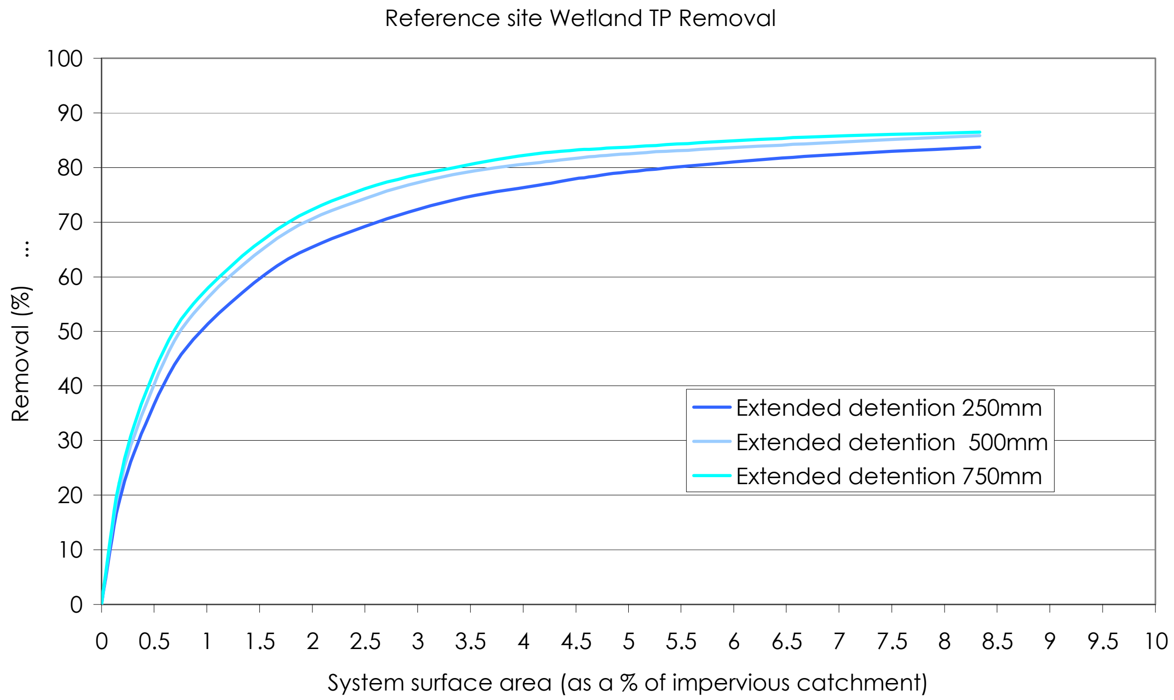


Figure 8.3 TP removal in wetland systems with varying extended detention

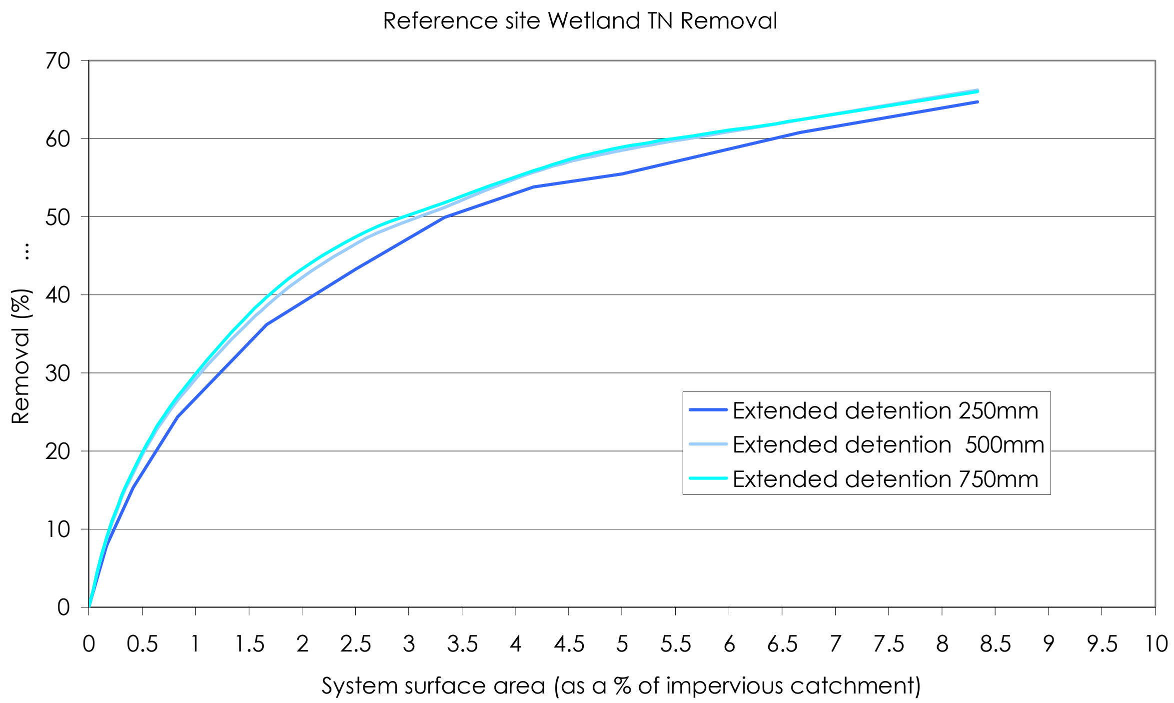


Figure 8.4. TN removal in wetland systems with varying extended detention

8.3 Design procedure: constructed wetlands

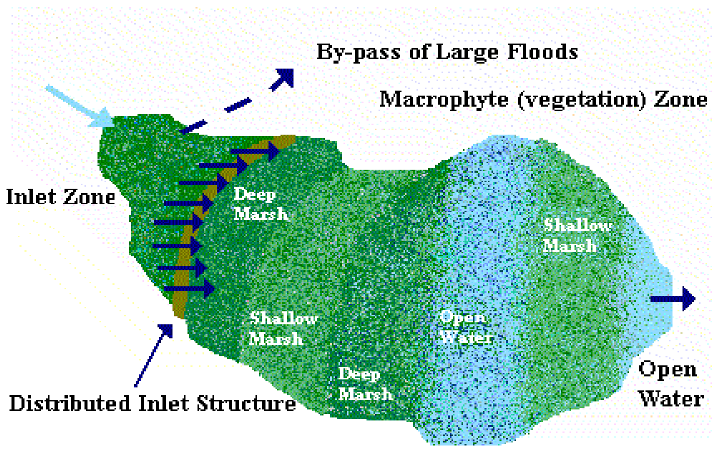


Figure 8.5. Elements of a constructed wetland system

Analyses to be undertaken during the detailed design phase of the inlet zone and the macrophyte zone of a constructed wetland system include the following:

- Design of the inlet zone as a sedimentation basin to target sediment of 125 µm or larger. Design considerations include:
  - Inlet zone to operate as a flow regulator into the macrophyte zone during normal operation

  - Inlet zone to operate for by-pass of the macrophyte zone during above-design conditions
  - Design the connection between the inlet zone and the macrophyte zone with appropriate design of inlet conditions to provide for energy dissipation and distribution of inflow into the macrophyte zone
  - Provision for sediment and debris removal.
- Configure the layout of the macrophyte zone to provide an extended detention volume in a manner such that the system's hydraulic efficiency can be optimised. Design considerations include:-
  - Suitable extended detention depths are between 0.25 m and 0.75 m, depending on the desired operation of the wetland and target pollutant.
  - Bathymetric design the of the macrophyte zone to promote a sequence of ephemeral, shallow marsh, marsh and submerged marsh systems in addition to a small open water system in the vicinity of the outlet structure.
  - Particular attention to the placement of the inlet and outlet structures, the aspect ratio of the macrophyte zone and flow control features to promote a high hydraulic efficiency within the macrophyte zone.
  - A key design consideration is the location and depth of permanent pools within the macrophyte zone.
  - Provision to drain the macrophyte zone if necessary should also be considered.
  - Design of the macrophyte zone outlet structure to provide for a 72 hour notional detention time for a wide range of flow depth. The outlet structure should include measures to trap debris to prevent clogging.
- Landscape design will be required including:-
  - Macrophyte zone vegetation (including littoral zone)
  - Terrestrial vegetation

The following sections detail the design steps required for constructed stormwater wetland systems.

8.3.1 Estimating design flows

The hydrologic design objectives for the inlet zone are as follows:

i. Capacity to convey stormwater inflows up to the peak 1 year ARI discharge into the macrophyte zone

ii. Capacity to convey above-design stormwater inflows to the by-pass system. Design discharge capacity for the by-pass system corresponds to the following, e.g.:

a. the minor system capacity (2 or 5 year ARI) if overland flow path does not direct overland flow into the wetland

b. 100 year ARI peak discharge if the wetland system forms part of the major drainage system.

8.3.2 Minor and major flood estimation

A range of hydrologic methods can be applied to estimate design flows. If the typical catchment areas are relatively small, the Rational Method Design Procedure is considered to be a suitable method for estimating design flows. However, the use of the Rational Design Procedure should strictly be used to size inlet hydraulic structures only and that a full flood routing computation method should be used in sizing the outlet hydraulic structures (e.g. outlet pipe, spillway and embankment height, etc.).

8.3.3 Inlet Zone

The Inlet Zone of a constructed stormwater wetland serves two basic functions, i.e. (i) the pre-treatment of inflow to remove gross pollutants and coarse to medium sized sediment; and (ii) the hydrologic control of inflows into the macrophyte zone and bypass of floods during "above-design" operating conditions. The Inlet Zone typically comprise a relatively deep open water body (> 1.5 m) that operates primarily as a sedimentation basin. It may often be necessary that a Gross Pollutant Trap (GPT) be installed at the inlet to this zone such that litter and large debris can be captured at the interface between the incoming waterway (or pipe) and the open water of the Inlet Zone.

For more information and guidance on the design of the Inlet Zone, the reader is referred to Chapter 3 Sediment Basins.

8.3.4 Macrophyte Zone Layout

8.3.4.1 Size and Dimensions

To optimise hydraulic efficiency, i.e. reduce short circuits and dead zones, it is desirable to adopt a high length to width ratio. The ratio of length to width varies depending on the size of the system and the site characteristics. To simplify the design and earthworks smaller systems tend to have length to width ratios at the lower end of the range. This can often lead to poor hydrodynamic conditions within the macrophyte zone. Persson *et al* (1999) used the term hydraulic efficiency to define the expected hydrodynamic characteristics for a range of configurations of stormwater detention systems.

Engineers Australia (2006) present expected hydraulic efficiencies of detention systems for a range of notional shapes, aspect ratios and inlet/outlet placements within stormwater detention systems and recommends that constructed wetlands systems should not be less than 0.5 and should be designed to promote hydraulic efficiencies greater than 0.7. (see Figure 8.6)

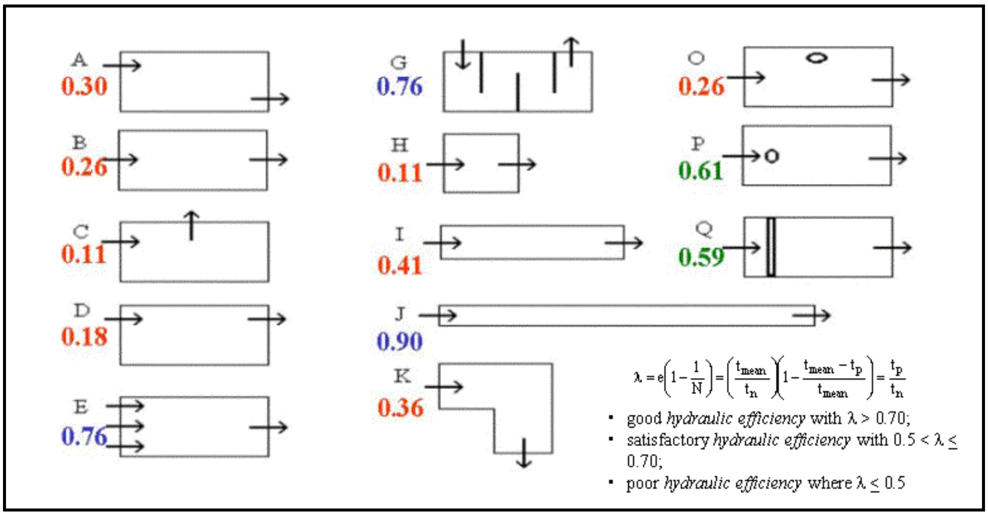


Figure 8.6. Hydraulic Efficiency - λ- A measure of Flow Hydrodynamic Conditions in Constructed Wetlands and Ponds; Range is from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment (Persson et al, 1999)

Figure 8.6 provides some guidance on what is considered to be good design with the higher values (of λ) representing structures with good sediment retention properties, where a value of λ greater than 0.5 should be a design objective. If the basin configuration yields a lower value, modification to the basin configuration should be explored to increase the λ value (e.g. inclusion of baffles, islands or flow spreaders).

The shape of the structure has a large impact on the effectiveness of the basin to retain sediments and generally a length to width ratio of at least 3:1 should be aimed for. In addition, the location of the inlet and outlet, flow spreaders and internal baffles impact the hydraulic efficiency of the basin for stormwater treatment. These types of elements are noted in Figure 4.3 as the figure "o" in diagrams O and P (which represent islands in a waterbody) and the double line in diagram Q which represents a structure to distribute flows evenly.

> **DESIGN ADVICE –**
>
> Consideration of maintenance access to a structure is also required when developing the shape of a wetland as this can impact the allowable width (if access is from the banks) or the shape if access ramps into a basin are required. An area for sediment de-watering should also be accommodated and it is required to drain back into the basin. This too may impact on the footprint area required for a sedimentation system.

8.3.4.2 Zonation

It is good design practice to provide a range of habitat areas within wetlands to support a variety of plant species and ecological niches. The wetland is broadly divided into four macrophyte zones and an open water zone. The bathymetry across the four macrophyte zones is to vary gradually over the depth range outlined below, ranging from 0.2 m above the permanent pool level to 0.5 m below the permanent pool level (refer to Appendix B for guidance on selection of plant species). The depth of the open water zone in the vicinity of the outlet structure is to be 1.0 m below the permanent pool level.

To ensure optimal hydraulic efficiency of the wetland for the given shape and aspect ratio, the wetland zones are arranged in bands of equal depth running across the flow path. Appropriate bathymetry coupled with uniform plant establishment ensures the cross section has equivalent hydraulic conveyance, thus preventing short-circuiting.

8.3.4.3 Long Section

In defining a long section for the macrophyte zone, it is necessary to provide areas for habitat refuge. For this reason it is desirable to have areas of permanent pools interconnected to prevent fauna being isolated in areas that dry out. This also reduces the piping required to drain the wetland for maintenance purposes.

An example bathymetry of a wetland system is shown in Figure 8.7. It illustrates gradual changes in depth longitudinally to create different vegetation areas as well as consistent zone banding across the wetland.

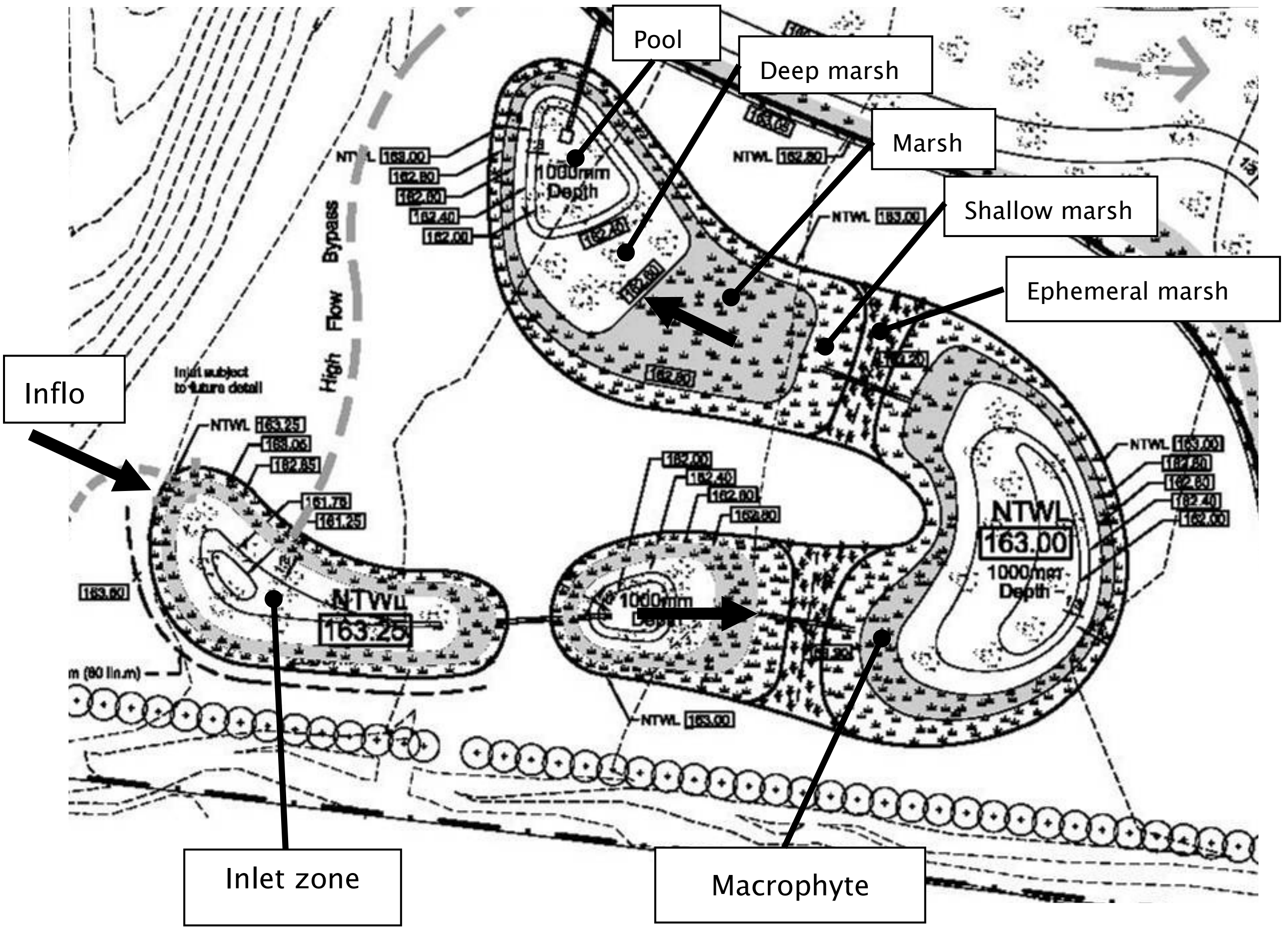


Figure 8.7. Example bathymetry of a constructed wetland system (GbLA, 2004)

8.3.4.4 *Cross Sections*

The batter slopes on approaches and immediately under the permanent water level have to be configured with consideration of public safety (e.g. Figure 8.8).

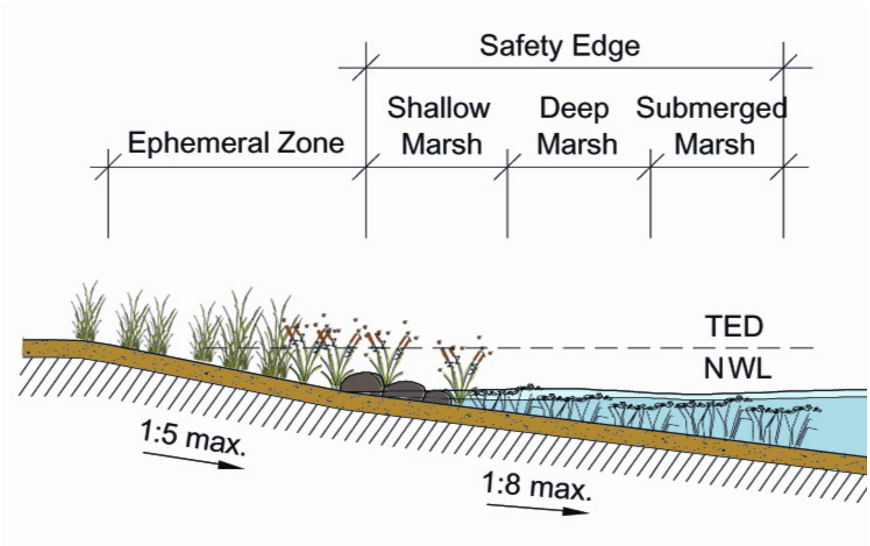


Figure 8.8. Example of edge design to a constructed wetland system

It is recommended that a gentle slope to the water edge and extending below the water line be adopted before the batter slope steepens into deeper areas. An alternative to the adoption of a flat batter slope is to provide a 3 m "safety bench" that is less than 0.2 m deep below the permanent pool level be built around the wetland.

Safety requirements for individual wetlands may vary from site to site, and it is recommended that an independent safety audit be conducted for each design. Safety guidelines are also provided by some local authorities (e.g. Royal Life Saving Society of Australia *Guidelines for Water Safety in Urban Water Developments*) and these should be followed.

8.3.5 Macrophyte Zone Outlet Structure

The macrophyte zone outlet structure forms two purposes. The first is to control discharges from the extended detention storage to ensure the wetland maintains a notional detention time of 72 hours. The outlet structure also needs to include features to allow the permanent pool to be drained for maintenance.

8.3.5.1 Maintenance Drain

The permanent pool of the wetland should be able to be drained with a maintenance drain operated manually. A suitable design flow rate is one which can draw down the permanent pool within 12 hours, i.e. overnight

The orifice discharge equation is considered suitable for sizing the maintenance drain on the assumption that the system will operate under inlet control with its discharge characteristics determined as follows, ie.

$$A_o = \frac{Q}{C_d\sqrt{2gh}}$$

Equation 8.1

C_d	=	Orifice Discharge Coefficient (0.6)
H	=	Depth of water above the centroid of the orifice (m)
A_o	=	Orifice area (m^2)
Q	=	required flow rate to drain the volume of the permanent pool in 12 hours

8.3.5.2 Riser Outlet – Size and Location of Orifices

The riser is designed to provide a uniform notional detention time over the full range of the extended detention depth[1]. The target maximum discharge may be computed as the ratio of the volume of the extended detention to the notional detention time, ie.

[1] It should be noted that detention time is never a constant and the term notional detention time is used to provide a point of reference in modelling and determining the design criteria for riser outlet structures.

Target Maximum Discharge (m^3/s) = extended storage volume (m^3)/ detention time (s)

The placement of outlet orifices and determination of appropriate diameters is designed iteratively by varying outlet diameters and levels, using the orifice equation (Equation 9.1) applied over discrete depths along the length of a riser up to the maximum detention depth. This can be performed with a spreadsheet as illustrated in the worked example.

As the outlet orifices can be expected to be small, it is important that the orifices are prevented from clogging by debris. Some form of debris guard is recommended as illustrated in the images below.

An alternative to using a debris guard is to install the riser within a pit which is connected to the permanent pool of the macrophyte zone via a submerged pipe culvert. This connection should be adequately sized such that there is minimal water level difference between the water within the pit and the water level in the macrophyte zone. With the water entering into the outlet pit being drawn from below the permanent pool level, floating debris are prevented from entering the outlet pit while heavier debris would normally settle onto the bottom of the permanent pool.

8.3.5.3 Riser Outlet – Pipe Dimension

While conservative, it is desirable to size the riser pipe such that it has the capacity to accommodate the 1 year ARI peak discharge operating as a “glory hole” spillway. Under normal operation, this flow would be by-passed around the macrophyte zone when this zone is already operating at design capacity. Nevertheless, it is good practice to provide a level of contingency in discharge capacity for the riser outlet to prevent any overtopping of the embankment of the macrophyte zone. A minimum of 0.3 m freeboard for the embankment (ie. crest level of embankment above top of extended detention) is often required.

Significant attenuation of the peak 1 year ARI inflow can be expected and some routing of the inflow hydrograph through the storage provided by the macrophyte zone is recommended.

The weir equation can be used in defining the required perimeter (and thus dimension) of the riser outlet. A weir coefficient of 1.7 (sharp-crested weir) is recommended, ie.

$$P = \frac{Q_{des}}{C_w \cdot H^{1.5}}$$

Equation 8.2

P = Perimeter of the outlet pit

H = Depth of water above the crest of the outlet pit

Q_{des} = Design discharge (m^3/s)

8.3.5.4 Discharge Pipe

The discharge pipe of the wetland conveys the outflow of the macrophyte zone to the receiving waters (or existing drainage infrastructure). The conveyance capacity of the discharge pipe is to be sized to match the higher of the two discharges, ie. maximum discharge from the riser or the maximum discharge from the maintenance drain.

8.3.6 Connection to Inlet Zone

The pipe that connects the inlet zone to the macrophyte zone must have sufficient capacity to convey a 1 year ARI flow, assuming the macrophyte zone is at the permanent pool level, without resulting in any flow in the bypass system. The configuration of the hydraulic structure connecting the inlet zone to the macrophyte zone would normally consist of an overflow pit connected to one or more pipes through the embankment separating these two zones.

Typical specifications of water and embankment levels are as follows:-

- Bypass spillway level = top of extended detention in the macrophyte zone
- Permanent Pool level in inlet zone = 0.3 m above permanent pool level in macrophyte zone

Velocity checks are to be conducted for when the wetland is full and when it is near empty. Velocities should ideally be less than 0.05 m/s.

8.3.7 High-flow route and by-pass design

To protect the integrity of the macrophyte zone of the wetland, it is necessary to consider the desired above-design operation of the wetland system. This is generally provided for with a high flow route that by-passes the macrophyte zone during flow conditions that may lead to scour and damage to the wetland vegetation. As outlined in **Section 8.3.1**, a function of the inlet zone is to provide hydrologic control of inflow into the macrophyte zone. A by-pass weir is to be included in the design of the inlet zone, together with a by-pass floodway (channel) to direct high flows around the macrophyte zone.

Ideally, the bypass weir level should be set at the top of the extended detention level in the macrophyte zone. This would ensure that a significant proportion of catchment inflow will bypass the Macrophyte Zone once it has reached its maximum operating extended detention level. The width of the spillway is to be sized to safely pass the maximum discharge conveyed into the inlet zone or the 100 year ARI discharge (as defined in **Section 8.3.1**) with the maximum water level above the crest of the weir to be defined by the top of embankment level (plus a suitable freeboard provision).

8.3.8 Vegetation specification

Vegetation planted in the macrophyte zone (ie. marsh and pool areas) is designed to treat stormwater flows, as well as add Aesthetic value. Dense planting of the littoral berm zone will inhibit public access to the macrophyte zone, minimising potential damage to the plants and the safety risks posed by water bodies. Terrestrial planting may also be recommended to screen areas and provide an access barrier to uncontrolled areas of the stormwater treatment system.

Plant species for the wetland area will be selected based on the water regime, microclimate and soil types of the region, and the life histories, physiological and structural characteristics, natural distribution, and community groups of the wetland plants. Refer to the Appendix B for a list of suggested plant species suitable for constructed wetland systems in Tasmania and recommended planting densities. The distribution of species within the wetland will relate to their structure, function, relationship and compatibility with other species. Planting densities should ensure that 70-80 % cover is achieved after two growing seasons (2 years).

8.3.9 Designing to avoid mosquitos

Mosquitos are a natural component of wetland fauna and the construction of any water body will create some mosquito habitat. To reduce the risk of high numbers of mosquitoes designs should function as balanced ecosystems with predators controlling mosquito numbers. Design considerations that should be addressed include:

- providing access for mosquito predators to all parts of the water body (do not have stagnant isolated area of water)
- providing an area of permanent water (even during long dry periods) where mosquito predators can seek refuge
- maintaining natural water level fluctuations that disturb the breeding cycle of some mosquito species
- providing a bathymetry such that regular wetting and drying is achieved and water draws down evenly so isolated pools are avoided
- providing sufficient gross pollutant control at the inlet such that human derived litter does not accumulate and provide breeding habitat
- ensuring maintenance procedures do not result in wheel ruts and other localised depressions that create isolated pools when water levels fall.

Local agency's guidelines should also be consulted with regard to approaches for avoiding excessive mosquito populations.

8.3.10 Design Calculation Summary

Overleaf is a design calculation summary sheet for the key design elements of a construction wetland to aid the design process.

Constructed Wetand — CALCULATION SUMMARY

	CALCULATION TASK		OUTCOME		CHECK
1	**Identify design criteria**				
	Design ARI Flow for Inlet Zone			year	
	Target Sediment Size for Inlet Zone			mm	
	Notional Detention Period for Macrophyte Zone			hrs	
	Design ARI Flow for Bypass Spillway			year	
	Extended Detention Volume			m^3	☐
2	**Catchment characteristics**				
		Residential		Ha	
		Commercial		Ha	
	Fraction impervious				
		Residential			
		Commercial			☐
3	**Estimate design flow rates**				
	Time of concentration				
	estimate from flow path length and velocities			minutes	☐
	Identify rainfall intensities				
		station used for IFD data:			
		100 year ARI		mm/hr	
		1 year ARI		mm/hr	☐
	Design runoff coefficient				
		C_1			
		C_{100}			☐
	Peak design flows				
		Q_1		m^3/s	
		Q_{100}		m^3/s	☐
4	**Inlet zone**				
		refer to sedimentation basin calculation checksheet			☐
5	**Macrophyte Zone Layout**				
		Extend Detention Depth		m	
		Area of Macrophyte Zone		m^2	
		Aspect Ratio		L:W	
		Hydraulic Efficiency			
		Length		m	
		Top width (including extended detention)		m	
		Cross Section Batter Slope		V:H	☐
6	**Macrophyte Zone Outlet Structures**				
	Maintenance Drain				
		Diameter of Maintenance Valve		mm	
		Drainage time		hrs	☐
	Riser				
		Linear Storage-Discharge Relationship for Riser			☐
	Discharge Pipe				
		Discharge Capacity of Discharge Pipe		m^3/s	☐
7	**Connection between Inet Zone and Macrophyte Zone**				
		Discharge Capacity of Connection Culvert		m^3/s	☐
8	**Bypass Weir**				
		Discharge Capacity of Bypass Weir		m^3/s	☐

8.4 Checking tools

This section provides a number of checking aids for designers and referral authorities. In addition, advice on construction techniques and lessons learnt from building wetland systems are provided.

Checklists are provided for:

- design assessments
- construction (during and post)
- operation and maintenance inspections
- asset transfer (following defects period).

8.4.1 Design assessment checklist

The checklist below presents the key design features that should be reviewed when assessing a design of a constructed wetland system. These considerations include configuration, safety, maintenance and operational issues that should be addressed during the design phase.

Where an item results in an "N" when reviewing the design, referral should be made back to the design procedure to determine the impact of the omission or error.

In addition to the checklist, a proposed design should have all necessary permits for its installations. The referral agency should ensure that all relevant permits are in place. These can include permits to clear vegetation, to dredge, create a waterbody, divert flows or disturb fish or platypus habitat.

Land and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the asset transfer checklist (see 8.4.4).

Wetland Design Assessment Checklist					
Wetland Location:					
Hydraulics	Minor Flood: (m^3/s)		Major Flood: (m^3/s)		
Area	Catchment Area (ha):		Wetland Area (ha)		
Treatment				**Y**	**N**
Treatment performance verified from curves?					
Inlet Zone				**Y**	**N**
Inlet pipe/structure sufficient for maximum design flow (Q_5 or Q_{100})?					
Scour protection provided at inlet?					
Configuration of inlet zone (aspect, depth and flows) allows settling of particles >125µm?					
Bypass weir incorporated into inlet zone?					
Bypass weir and channel sufficient to convey >Q1 <= maximum inlet flows?					
Bypass weir crest at macrophyte permanent pool level + extended detention depth?					
Bypass channel has sufficient scour protection?					
Structure from inlet zone to macrophyte zone enables energy dissipation/flow distribution?					
Structure from inlet zone to macrophyte zone enables isolation of the macrophyte zone for maintenance?					
Inlet zone permanent pool level above macrophyte permanent pool level?					
Maintenance access allowed for into base of inlet zone?					
Public access to inlet zone prevented through vegetation or other means?					
Gross pollutant protection measures provided on inlet structures (both inflows and to macrophyte zone)					
Macrophyte Zone				**Y**	**N**
Extended detention depth >0.25m and <0.75m?					
Vegetation bands perpendicular to flow path?					
Vegetation bands of near uniform depth?					
Sequencing of vegetation bands provides continuous gradient to open water zones?					
Vegetation appropriate to selected band?					
Aspect ratio provides hydraulic efficiency >0.5?					
Velocities from inlet zone <0.05 m/s or scouring protection provided?					
Batter slopes from accessible edges shallow enough to allow egress?					
Maintenance access provided into areas of the macrophyte zone (especially open water zones)?					
Public access to macrophyte zones restricted where appropriate?					
Safety audit of publicly accessible areas undertaken?					
Freeboard provided above extended detention depth?					
Outlet Structures				**Y**	**N**
Riser outlet provided in macrophyte zone?					
Orifice configuration allows for a linear storage-discharge relationship for full range of the extended detention depth?					
Riser diameter sufficient to convey Q_1 flows when operating as a “glory hole” spillway?					
Maintenance drain provided?					
Discharge pipe from has sufficient capacity to convey the maintenance drain flows or Q_1 flows (whichever is higher)?					
Protection against clogging of orifice provided on outlet structure?					

8.4.2 Construction advice

This section provides general advice for the construction of wetlands. It is based on observations from construction projects around Australia.

Protection from existing flows

It is important to have protection from upstream flows during construction of a wetland system. A mechanism to divert flows around a construction site, protect from litter and debris is required. This can be achieved by constructing a high flow bypass channel initially and then diverting all inflows along the channel until the wetland system is complete.

High flow contingencies

Contingencies to manage risks associated with flood events during construction are required. All machinery should be stored above acceptable flood levels and the site stabilised as best as possible at the end of each day as well as plans for dewatering following storms made.

Erosion control

Immediately following earthworks it is good practice to revegetate all exposed surfaces with sterile grasses (e.g. hydroseed). These will stabilise soils, prevent weed invasion yet not prevent future plants from establishing. Site runoff may be prevented by establishing the inlet zone as temporary sediment basin in the early stages of construction. Common soil and water management practices for any potential runoff are crucial.

Inlet erosion checks

It is good practice to check the operation of inlet erosion protection measures following the first few rainfall events. It is important to check for these early in the systems life, to avoid continuing problems. Should problems occur in these events the erosion protection should be enhanced.

Tolerances

Tolerances are important in the construction of wetlands (eg base, longitudinal and batters). Levels are particularly important for a well distributed flow path and establishing appropriate vegetation bands. It is also important to ensure that as water levels reduce (e.g. for maintenance) that areas drain back into designated pools and distributed shallow pools across the wetland are avoided. Generally plus or minus 50mm is acceptable.

Transitions

Pay attention to the detail of earthworks to ensure smooth transitions between benches and batter slopes. This will allow for strong edge vegetation to establish and avoid local ponding, which can enhance mosquito breeding habitat.

Inlet zone access

An important component of an inlet zone is accessibility for maintenance. If excavators can reach all parts of the inlet zone an access track may not be required to the base of the inlet zone, however an access track around the perimeter of the inlet zone is required. If sediment collection is performed using earthmoving equipment a stable ramp will be required to the base of the inlet zone (maximum slope 1:10).

Inlet zone base

It is recommended that the inlet zone be constructed with a hard (i.e. rock) bottom. This is important if maintenance invoves driving into the basin. It also serves an important role in determining the levels that excavation should extend to (ie how deep to dig).

Dewatering collected sediments

An area should be constructed that allows for dewatering of removed sediments from an inlet zone. This area should be located such that water from the material drains back into the inlet zone. Material should be allowed to drain for a minimum of overnight before disposal.

Timing for planting

Timing of vegetation planting is influenced by season (and potential irrigation requirements) as well as timing in relation to the phases of development. Temporary sediment controls should always be used prior to planting as lead times from earthworks to planting are often long.

Vegetation establishment

During the establishment phase water levels should be controlled carefully to prevent seedlings from being desiccated or drowned. This is best achieved with the use of maintenance drains. Once established, water levels can be raised to operational levels.

Bird protection

Bird protection (e.g. nets) should be considered for newly planted areas of wetlands, birds can pull out young plants and reduce plant densities.

8.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Wetlands

INSPECTED BY:
DATE:
TIME:
WEATHER:
CONTACT DURING VISIT:

SITE: ____________________

CONSTRUCTED BY: ____________________

DURING CONSTRUCTION									
Items inspected	Checked		Satisfactory	Unsatisfactory		Checked		Satisfactory	Unsatisfactory
Preliminary works	Y	N				Y	N		
1. Erosion and sediment control plan adopted					18. Concrete and reinforcement as designed				
					19. Inlets appropriately installed				
2. Limit public access					20. Inlet energy dissipation installed				
3. Location same as plans					21. No seepage through banks				
4. Site protection from existing flows					22. Ensure spillway is level				
5. All required permits in place					23. Provision of maintenance drain(s)				
Earthworks					24. Collar installed on pipes				
6. Integrity of banks					25. Low flow channel rocks are adequate				
7. Batter slopes as plans					26. Protection of riser from debris				
8. Impermeable (eg. clay) base installed					27. Bypass channel stabilised				
9. Maintenance access to whole wetland					28. Erosion protection at macrophyte outlet				
10. Compaction process as designed					**Vegetation**				
11. Placement of adequate topsoil					29. Vegetation appropriate to zone (depth)				
12. Levels as designed for base, benches, banks and spillway (including freeboard)					30. Weed removal prior to planting				
					31.Provision for water level control during establishment				
13. Check for groundwater intrusion									
14. Stabilisation with sterile grass					32. Vegetation layout and densities as designed				
Structural components									
15. Location and levels of outlet as designed					33. Provision for bird protection				
16. Safety protection provided					34. By-pass channel vegetated				
17. Pipe joints and connections as designed									
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					9. Check for uneven settling of banks				
2. Confirm structural element sizes					10. Evidence of stagnant water, short circuiting or vegetation scouring				
3. Check batter slopes									
4. Vegetation planting as designed					11. Evidence of litter or excessive debris				
5. Erosion protection measures working					12. Provision of removed sediment drainage area				
6. Pre-treatment installed and operational									
7. Maintenance access provided					13. Evidence of debris in high flow bypass				
8. Public safety adequate					14. Macrophyte outlet free of debris				

COMMENTS ON INSPECTION

ACTIONS REQUIRED

1.									
2.									
3.									

8.4.4 Asset transfer checklist

Asset Handover Checklist		
Asset Location:		
Construction by:		
Defects and Liability Period		
Treatment	**Y**	**N**
System appears to be working as designed visually?		
No obvious signs of under-performance?		
Maintenance	**Y**	**N**
Maintenance plans provided for each asset?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
Asset inspected for defects?		
Asset Information	**Y**	**N**
Design Assessment Checklist provided?		
As constructed plans provided?		
Copies of all required permits (both construction and operational) submitted?		
Proprietary information provided (if applicable)?		
Digital files (eg drawings, survey, models) provided?		
Asset listed on asset register or database?		

8.5 Maintenance requirements

Wetlands treat runoff by filtering it through vegetation and providing extended detention to allow sedimentation. In addition, they have a flow management role that needs to be maintained to ensure adequate flood protection for local properties and protection of the wetland ecosystem.

Maintaining vibrant vegetation and adequate flow conditions in a wetland are important maintenance considerations. Weeding, planting and debris removal are the dominant tasks. In addition the wetland needs to be protected from high loads of sediment and debris and the inlet zone needs to be maintained in the same way as sedimentation basins (see Chapter 3).

The most intensive period of maintenance is during the plant establishment period (first two years) when weed removal and replanting may be required. Requirements of plant establishment are discussed further in Appendix B Plant Lists. It is also the time when large loads of sediments could impact on plant growth particularly in developing catchments with poor building controls.

Other components of the system that will require careful consideration are the inlet points. Inlets can be prone to scour and build up of litter. Occasional litter removal and potential replanting may be required as part of maintaining an inlet zone.

Maintenance is primarily concerned with:

- Maintenance of flow to and through the system
- Maintaining vegetation
- Preventing undesired vegetation from taking over the desirable vegetation
- Removal of accumulated sediments
- Litter and debris removal

Vegetation maintenance will include:

- Removal of noxious plants or weeds
- Re-establishment of plants that die

Similar to other types of stormwater practices, debris removal is an ongoing maintenance function. Debris, if not removed, can block inlets or outlets, and can be unsightly if located in a visible location. Inspection and removal of debris should be done regularly, but debris should be removed whenever it is observed on the site.

Inspections are also recommended following large storm events to check for scour.

8.5.1 Operation & maintenance inspection form

The form below should be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time. Inspections should occur every 3-months for the first year and then 6-monthly thereafter. More detailed site specific maintenance schedules should be developed for major wetland systems and include a brief overview of the operation of the system and key aspects to be checked during each inspection. An example is presented as part of the worked example in 8.6.

Wetland Maintenance Checklist			
Inspection Frequency: 3 monthly	Date of Visit:		
Location:			
Description:			
Site Visit by:			
Inspection Items	**Y**	**N**	**Action Required (details)**
Sediment accumulation at inflow points?			
Litter within inlet or macrophyte zones?			
Sediment within inlet zone requires removal (record depth, remove if >50%)?			
Overflow structure integrity satisfactory?			
Evidence of dumping (building waste, oils etc)?			
Terrestrial vegetation condition satisfactory (density, weeds etc)?			
Aquatic vegetation condition satisfactory (density, weeds etc)?			
Replanting required?			
Settling or erosion of bunds/batters present?			
Evidence of isolated shallow ponding?			
Damage/vandalism to structures present?			
Outlet structure free of debris?			
Maintenance drain operational (check)?			
Resetting of system required?			
Comments:			

8.6 Worked example

8.6.1 Worked example introduction

A sedimentation basin and wetland system is proposed to treat runoff from a residential and commercial area located in Hobart. The wetland will consist of an inlet zone designed to treat the larger pollutant sizes. Flow will than pass through into a macrophyte zone where a riser outlet will be used to control the system detention period to settle out finer sediment particles. A by-pass channel will enable large flood events to by-pass the macrophyte zone during periods when the macrophyte zone is already operating at its design level. This worked example focuses only on the macrophyte zone component of the system with the design of the inlet zone (sedimentation basin) and by-pass channel contained in an earlier worked example. An illustration of the site and proposed layout of the wetland is shown below.

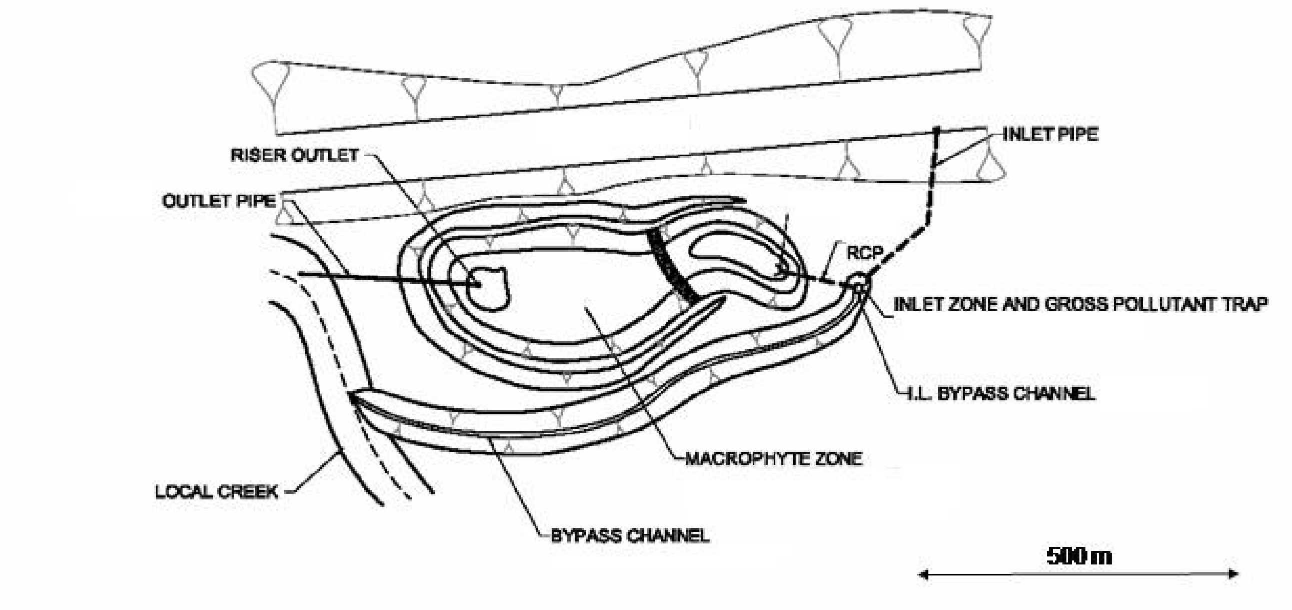


Figure 8.9. Layout of Proposed Wetland System

The contributing catchment area of the proposed wetland is 10 Ha (with percentage imperviousness of 50%). The site is flat with the maximum fall of less than 0.5 m across the site. Stormwater from the catchment is conveyed by conventional stormwater pipes and discharges into the constructed wetland via a single DN1000 diameter pipe. There are no site constrains with regard to the size of the wetland, as construction can extend into an adjacent park if required.

8.6.2 Design Considerations

The design criteria for the wetland system are to:

- Promote sedimentation of particles larger than 125 μm within the inlet zone.
- Optimise the relationship between detention time, wetland volume and the hydrologic effectiveness of the system to maximise treatment given the wetland volume site constraints. This is equivalent to a hydrologic effectiveness of 85% for a notional detention period of 72 hours.
- Ensure that the required detention period is achieved for all flow though the wetland system through the incorporation of a riser outlet system.
- Provide for by-pass operation when the inundation of the macrophyte zone reaches the design maximum extended detention depth.

This worked example focuses on the design of the macrophyte zone of the wetland system. Analyses to be undertaken during the detailed design phase of the macrophyte zone of the wetland system include the following:

- Configure the layout of the macrophyte zone to provide an extended detention depth of 0.5m in a manner such that the system *hydraulic efficiency* can be optimised. This includes particular attention to the placement of the inlet and outlet structures, the aspect ratio of the macrophyte zone and the need to use bathymetry and other flow control features to promote a high hydraulic efficiency within the macrophyte zone. A key design consideration is the extended detention depth for the macrophyte zone.
- Design the provision to drain the macrophyte zone if necessary.
- Design the connection between the inlet zone and the macrophyte zone with appropriate design of inlet conditions to provide for energy dissipation and distribution of inflow into the macrophyte zone.
- Design the bathymetry of the macrophyte zone to promote a sequence of ephemeral, shallow marsh and marsh system in addition to a small open water system in the vicinity of the outlet structure.
- Design of the macrophyte zone outlet structure to provide for a 72 hour notional detention time, including debris trap.

In addition, a landscape design will be required and they include:

- Macrophyte zone vegetation (including edge vegetation)
- Terrestrial vegetation.

8.6.2.1 Confirming Macrophyte Zone Area

As a basic check of the adequacy of the size of the wetland, reference is made to the performance curves presented in 8.2 Verifying size for treatment. According to Figures 8.2 – 8.4, the required wetland size to satisfy stormwater quality best practice environment management objectives (based on 0.5 m extended detention depth) at the reference site is the larger of 2.1% (for 80% reduction in TSS); 1.1% (for 45% reduction of TP) and 3.0% (for 45% reduction of TN) of the impervious area, (ie. 3.0 % of the impervious area is the critical size).

The required wetland area is as follows:

Catchment area = 10Ha (50% impervious)

Therefore Impervious Area = 5 Ha

Required Wetland Area (0.5 m extended detention) is:

Impervious Area (m^2) x treatment area required (%) = 50000 x 0.03

= 1500 m^2

An adjustment factor is then applied to that size to determine the size wetland required in the area of interest. The wetland adjustment factor equation should be applied (see Section 2.4).

In this worked example the wetland has been sized to require an extended detention volume of 885m^3*. An extended detention depth of 0.5m has been adopted requiring a surface area of 1710m^2*.

* These figures are for the worked example only. The appropriate region and corresponding adjustment factor must be identified then calculated for each individual project (see Section 2.4).

DESIGN NOTE – The values derived from the Figures 8.2 to 8.4 will only be valid if the design criteria for the proposed installation are similar to those used to create the Figures. Site specific modelling using programs such as MUSIC (eWater, 2009) may yield a more accurate result.

8.6.3 Design calculations

8.6.3.1 Estimating Design Flows

See Appendix E Design Flows – tc for a discussion on methodology for calculation of time of concentration.

Step 1 – Calculate the time of concentration.

With the catchment area being relatively small, the Rational Method Design procedure is considered to be an appropriate method for computing the design flows.

Catchment area = 10 Ha

t_c ~ 10 min (ARR 1998 methods)

Using a time of concentration of 10 minutes, the design rainfall intensities from the IFD chart relevant to the catchment location are –

I_1 = 38.2 mm/hr *

I_{100} = 130 mm/hr *

* These figures are for the worked example only. The appropriate region and corresponding rainfall intensities must be selected for each individual project.

Step 2 – Calculate design run-off coefficients (using the method outlined in Australian Rainfall and Runoff Book VIII (Engineers Australia, 2003)).

Where – Fraction impervious (f) = 0.5

Rainfall intensity ($^{10}I_1$) = 38.2mm/hr (from the relevant IFD chart)

Calculate $C^1{}_{10}$ (pervious run-off coefficient)

$C^1{}_{10} = 0.1 + 0.0133\ (^{10}I_1 - 25) = 0.275$

Calculate C_{10} (10 year ARI run-off coefficient)

$C_{10} = 0.9f + C^1{}_{10}\ (1-f)$

$C_{10} = 0.587$

Step 3 – Convert C_{10} to values for C_1 and C_{100}

Where – $C_y = F_y \times C_{10}$

From Table 1.6 in Australian Rainfall and Runoff – Book VII;

$C_1 = 0.8 \times C_{10} = 0.47$

$C_{100} = 1.2 \times C_{10} = 0.70$

Step 4 – Calculate peak design flow (calculated using the Rational Method).

$$Q = \frac{CIA}{360}$$

Where –	C	is the runoff coefficient (C_1 and C_{100})
	I	is the design rainfall intensity mm/hr (I_1 and I_{100})
	A	is the catchment area (Ha)

Q_1 = 0.47 m^3/s (466 L/s)

Q_{100} = 2.52 m^3/s (2528 L/s)

8.6.3.2 Inlet Zone

The procedure for the design of the inlet zone follows that presented in Procedure 1 for sediment basin. In this worked example, design computation for the by-pass weir and the connection to the macrophyte zone will be presented.

8.6.3.3 Macrophyte Zone Layout

Size and Dimensions

The wetland has been sized to require an extended detention volume of 885m^3. An extended detention depth of 0.5m has been adopted requiring a surface area of 1710m^2.

In this case it has been chosen to adopt a length (L) to width (W) ratio of 6 to 1. This aspect ratio represents a shape configuration in between Case G and Case I in **Figure 8.6** and the expected hydraulic efficiency is 0.6. This is lower than ideal for a wetland, however the space constraints of the site limit the available area for the macrophyte zone.

*Aspect Ratio is **6(L) to 1(W)**; Hydraulic Efficiency **~ 0.6***

To calculate the dimensions

L = 6W

Wetland Area = 6W x W = 1710

*Notional Macrophyte Zone dimension is ~**102 m (L) x 17 m (W)***

Zonation

The wetland is broadly divided into four macrophyte zones and a open water zone. The bathymetry across the four macrophyte zones is to vary gradually over the depth range outlined below, ranging from 0.2 m above the permanent pool level to 0.5 m below the permanent pool level. The depth of the open water zone in the vicinity of the outlet structure is to be 1.0 m below the permanent pool level.

Zone	**Depth Range (m)**	**Proportion of Macrophyte Zone Surface Area (m)**
Open Water	>1.0 ***below*** permanent pool	10%
Submerged Marsh	0.5 – 1.0 ***below*** permanent pool	10%
Deep Marsh	0.35 – 0.5 ***below*** permanent pool	25%
Marsh	0.2 – 0.35 ***below*** permanent pool	25%
Shallow Marsh	0.0 – 0.2 ***below*** permanent pool	25%
Littoral (edges)	+0.5 – 0.0 ***above*** permanent pool	5%

To ensure optimal hydraulic efficiency of the wetland for the given shape and aspect ratio, the wetland zones are arranged in bands of equal depth running across the flow path. The appropriate bathymetry coupled with uniform plant establishment ensures the cross section has equivalent hydraulic conveyance, thus preventing short-circuiting.

Wetland consist of four macrophyte zones arranged in bands of equal depth running across the flow path

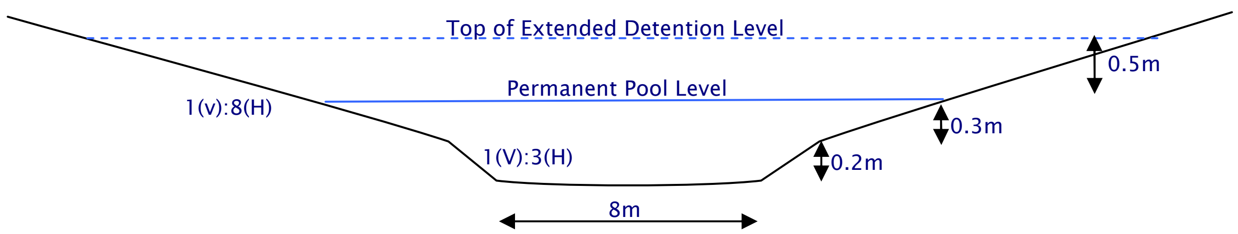
Long Section

Hobart has a relatively low mean annual rainfall, with much of the rainfall falling in winter and spring. The region also has high summer evaporation. It is therefore likely that water losses during summer will be high and it will be necessary to provide areas of habitat refuge. For this reason it is desirable to have areas of permanent pool interconnected to prevent fauna being isolated in areas that dry out. The proposed long section is for the bed of the wetland to gradually deepen over the four macrophyte zones (i.e. excluding edges). This profile also facilitates draining of the wetland.

Long section of the macrophyte zone is to be gradually deepening over the four macrophyte zones ranging from the permanent pool level (shallow marsh) to 1.0 m below the permanent pool (submerged marsh zone).

Cross Sections

The batter slopes on approaches and immediately under the permanent water level have to be configured with consideration of public safety. A batter slope of 1(V):8(H) from the top of the extended detention depth to 0.3 m beneath the water line before steepening into a 1(V):3(H) slope is recommended as a possible design solution (see illustration below). The safety requirements for individual wetlands may vary from site to site, and it is recommended that an independent safety audit be conducted of each design.



Typical Cross Section of Macrophyte Zone

Cross section of macrophyte zone is trapezoidal in shape with a ***base width of 8 m*** *and a* ***top width of 22.0 m.***

8.6.3.4 Macrophyte Zone Outlet Structure

Maintenance Drain

A maintenance drain will be provided to allow drainage of the system. Valves will be operated manually to drain the inlet zone and macrophyte zone independently.

The mean flow rate for the maintenance drain is selected to drawdown the permanent pool over a notional 12 hour period and is computed as follows:

Permanent Pool Volume ~ 500 m^3 (assuming approximate 0.25 nominal depth)

Q = Volume/Time to drain

Q = 500/(12 x 3600) = 0.012 m^3/s

It is assumes the valve orifice will operate under inlet control with its discharge characteristics determined by the orifice equation (Equation 9.1), i.e.

$$A_o = \frac{Q}{C_d\sqrt{2gh}}$$

Q = 500/(12 x 3600) = 0.012 m^3/s

C_d = 0.6

H = 0.33 m (one third of permanent pool depth)

Giving A_o = 0.0079 m^2 corresponding to an orifice diameter of 100mm – adopt **100mm**

> *Pipe valve to allow draining of the permanent pool for maintenance to be* ***at least 100 mm diameter.***

Riser Outlet – Size and Location of Orifices

The riser is designed to provide a uniform notional detention time over the full range of the extended detention depth[2].

Target Q_{max} = extended storage volume/detention time

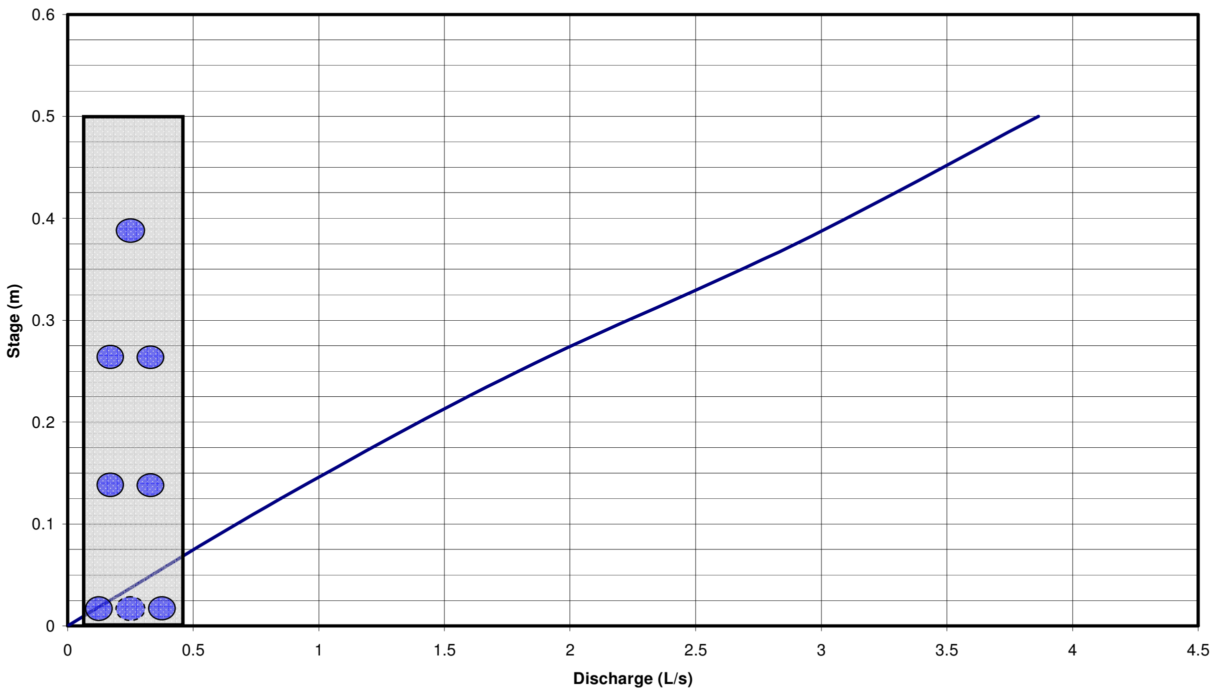
= 750/(72 x 3600) = 0.0029 L/s

Outlet orifices along the riser is located at 0.125 m intervals along the length of riser, ie. at 0 m, 0.125 m, 0.250 m and 0.375 m above the permanent pool level. A standard orifice diameter of 20 mm was selected and the numbers required at each level were determined

[2] It should be noted that detention time is never a constant and the term notional detention time is used to provide a point of reference in modelling and determining the design criteria for riser outlet structures.

iteratively using a spreadsheet (as shown below) and applying the orifice equation (Equation 8.1) applied over discrete depths along the length of the riser up to the maximum detention depth. The results of the design are summarised in the table below. The stage-discharge relationship of the riser is plotted in the chart and shows that the riser maintains a linear stage discharge relationship.

			Q1	Q2	Q3	Q4	total	Notional
Orifice Positions (invert level)			0	0.125	0.25	0.375	Flow	Detention Time
Orifice Diameter (mm)			20	20	20	20	(L/s)	(hrs)
Number			3	2	2	1		
	Water Depth	Volume						
Q1	0	0	0				0	
Q2	0.125	220	0.850				0.850	71.92
Q3	0.25	468	1.228	0.567			1.794	72.38
Q4	0.375	743	1.514	0.818	0.567		2.899	71.15
	0.5	1045	1.754	1.009	0.818	0.283	3.865	75.10



As the wetland is relatively small and the required orifices are small, it is necessary to include measures to prevent blocking of the orifices.

The riser is to be installed within an outlet pit with a culvert connection to the permanent pool of the macrophyte zone. The connection is via a 300 mm diameter pipe. The pit is accessed via a locked screen on top of the pit.

The riser pipe should not be smaller than the pipe conveying the outflow from the wetland to the receiving waters (see next section).

*Outlet Riser consist of 4 rows of orifices of **20 mm diameter** located as follows:-*		
	Depth above permanent pool	*Number of 20 mm diameter orifice*
	0.000 m	*3*
	0.125 m	*2*
	0.250 m	*2*
	0.375 m	*1*

Riser Outlet – Pipe Dimension

As designed, high flows would by-pass around the macrophyte zone when this zone is already operating at design capacity (ie. when water level in the macrophyte zone reaches the top of its extended detention. A notional riser pipe diameter of 150 mm is thus sufficient.

*Riser pipe to be **150 mm diameter***

Discharge Pipe

The discharge pipe of the wetland conveys the outflow of the macrophyte zone to the receiving waters (or existing drainage infrastructure). Under normal operating conditions, this pipe will need to have sufficient capacity to convey larger of the discharges from the riser or the maintenance drain.

- The maximum discharge from the riser = 2.9 L/s
- The maximum discharge through the maintenance pipe occurs for depth of 1.0 m (ie. depth of open water zone). From, the maximum discharge through the 90 mm diameter valve is computed to be 17 L/s.

The required pipe diameter for the outlet should be thus larger than 90 mm. Since the diameter of the riser has been selected to be 150 mm, it is appropriate to also use this dimension for the outlet pipe connecting the wetland to the adjoining creek.

*Outlet pipe for wetland for discharge to receiving waters (or existing drainage infrastructure) is to be **150 mm** diameter.*

8.6.3.5 Connection to Inlet Zone

The configuration of the hydraulic structure connecting the inlet zone to the macrophyte zone consist of an overflow pit (in the inlet zone) and a pipe with the capacity to convey the 1 year ARI peak discharge of 0.47 m^3/s.

Design specifications:

Bypass spillway level = top of extended detention in the macrophyte zone

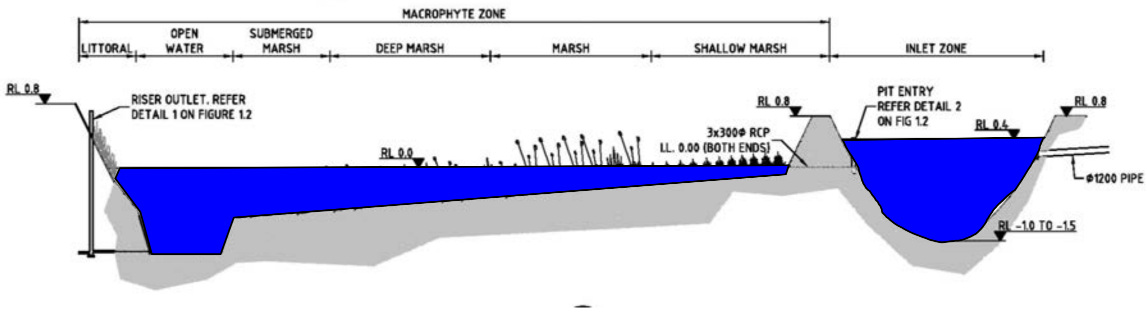
Permanent Pool level in inlet zone = 0.3 m above permanent pool level in macrophyte zone

In designing the culvert connecting the inlet zone to the macrophyte zone, the following conditions apply:

Headwater Level = 0.5 m above macrophyte permanent pool level

Tail water level = permanent pool level

Design Flow = 0.47 m^3/s



Assume culvert under outlet control; $K_e = 0.5$, $K_{out} = 1$, $n = 0.015$

Try 1 by 450 mm dia Capacity = 0.41 m^3/s ....Too small

Try 3 Nos 300 mm dia Capacity = 0.55 m^3/s ....OK

Culverts connecting inlet zone to macrophyte zone is to be **3 Nos. 300 mm diameter**.

Velocity checks are to be conducted for when the wetland is full and when it is at permanent pool level. For the velocity checks the maximum inflow corresponding to the 1 year ARI peak discharge is used, i.e. 0.47 m^3/s.

Flow check V = Q/A

When full

A = 15 x 0.5 = 7.5m^2, V = 0.06 m/s... no risk of scour

When at permanent pool level

A = 15 x 0.1 = 1.5m^2, V = 0.3 m/s...no risk of scour

8.6.3.6 High-flow route and by-pass design

The bypass weir level at the inlet zone is set to match the top of the extended detention level in the macrophyte zone. The width of the spillway is to be sized to safely pass the 100 year ARI discharge with a water level over the weir of 0.3 m (ie. top of wetland embankment).

The 100 year ARI peak discharge = 2.52 m^3/s.

Crest Level = 0.5 m above macrophyte permanent pool

Freeboard (top of wetland embankment) = 0.3 m

k_w = 1.7 (Sharp Crested Weir)

Therefore $L = Q/k_w.H^{1.5}$

Q = 2.52 m^3/s (100 year ARI flow from contributing catchment)

H = 0.30 m

L = 8.6 m

The spillway length is to be 9.0 m set at a crest level 0.5 m above the permanent pool level of the macrophyte zone.

8.6.3.7 Vegetation Specifications

The vegetation specification and recommended planting density for the macrophyte zone are summarised in the table below.

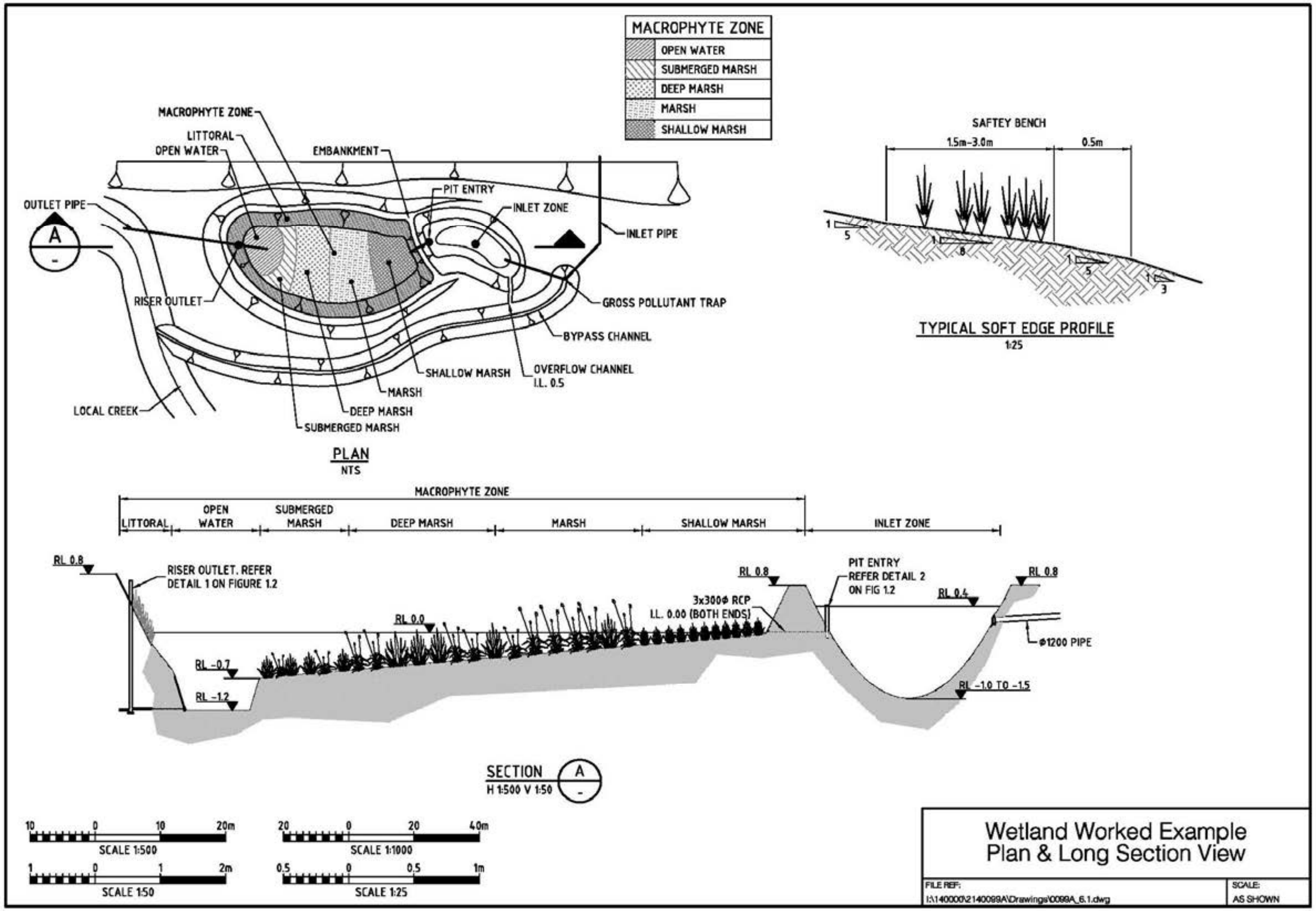
Zone	**Plant Species**	**Planting Density (plants/m²)**
Littoral berm	*Persicaria decipens*	3
Ephemeral marsh	*Blechnum minus*	6
Shallow March	*Cyperus lucidus*	6
Marsh	*Bolboschoenus caldwellii*	4
Deep Marsh	*Juncus ingens*	8

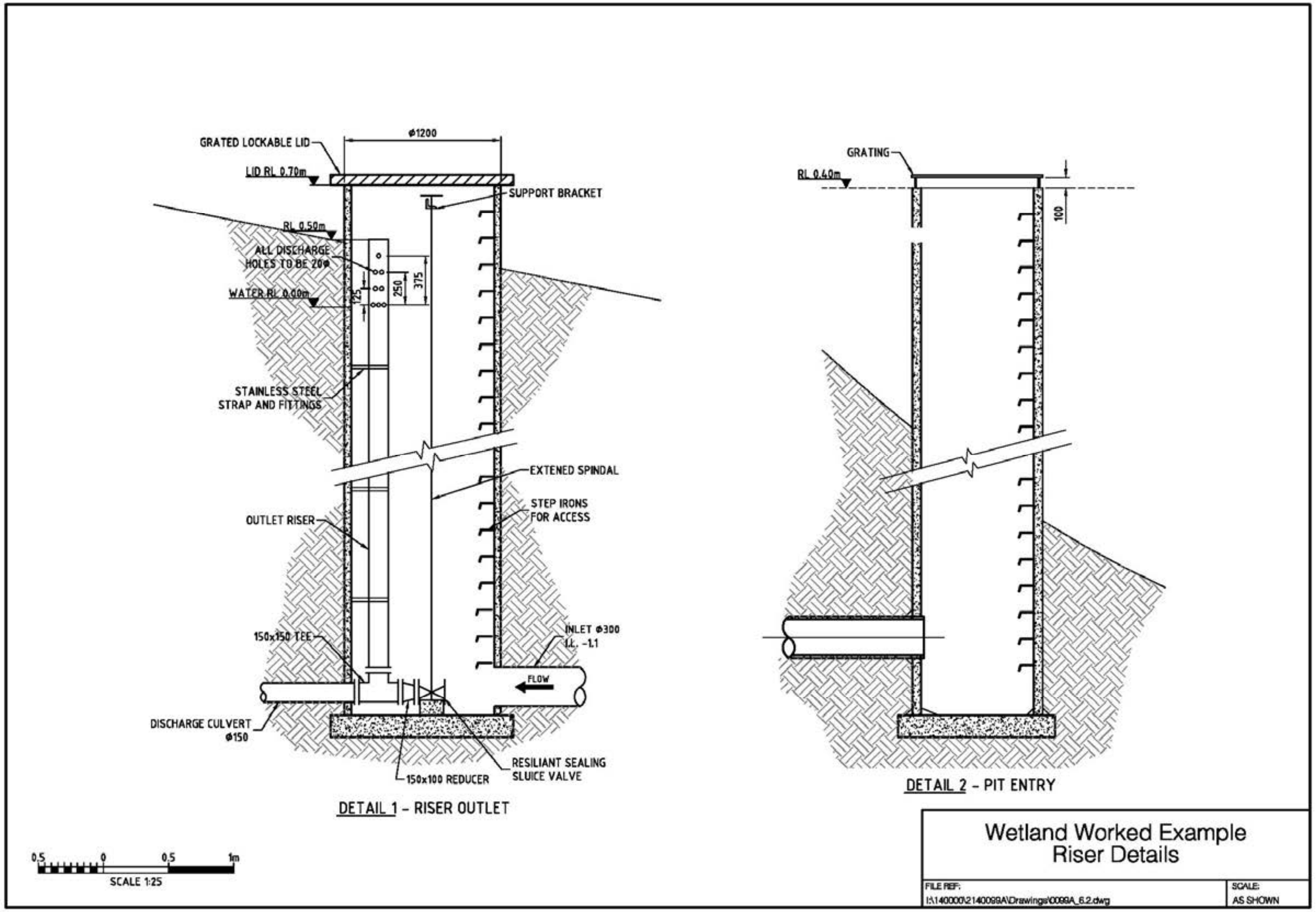
The reader is referred to Appendix B Plant Lists for further discussion and guidance on vegetation establishment and maintenance.

8.6.4 Design Calculation Summary

Constructed Wetand		CALCULATION SUMMARY		
	CALCULATION TASK	OUTCOME		CHECK
1	**Identify design criteria**			
	Design ARI Flow for Inlet Zone	1	year	
	Target Sediment Size for Inlet Zone	0.125	mm	
	Notional Detention Period for Macrophyte Zone	72	hrs	
	Design ARI Flow for Bypass Spillway	100	year	
	Extended Detention Volume	750	m^3	
2	**Catchment characteristics**			
	Residential	7	Ha	
	Commercial	3	Ha	
	Fraction impervious			
	Residential	0.4		
	Commercial	0.7		
3	**Estimate design flow rates**			
	Time of concentration			
	estimate from flow path length and velocities	10	minutes	
	Identify rainfall intensities			
	station used for IFD data:	Hobart		
	100 year ARI	130	mm/hr	
	1 year ARI	38.2	mm/hr	
	Design runoff coefficient			
	C_1	0.47		
	C_{100}	0.7		
	Peak design flows			
	Q_1	0.47	m^3/s	
	Q_{100}	2.520	m^3/s	
4	**Inlet zone**			
	refer to sedimentation basin calculation checksheet			
5	**Macrophyte Zone Layout**			
	Extend Detention Depth	0.5	m	
	Area of Macrophyte Zone	1710	m^2	
	Aspect Ratio	6(L):1(W)	L:W	
	Hydraulic Efficiency	0.6		
	Length	102	m	
	Top width (including extended detention)	17	m	
	Cross Section Batter Slope	1(V):8(H)	V:H	
6	**Macrophyte Zone Outlet Structures**			
	Maintenance Drain			
	Diameter of Maintenance Valve	90	mm	
	Drainage time	12	hrs	
	Riser			
	Linear Storage-Discharge Relationship for Riser			
	Discharge Pipe			
	Discharge Capacity of Discharge Pipe	0.75	m^3/s	
7	**Connection between Inet Zone and Macrophyte Zone**			
	Discharge Capacity of Connection Culvert	0.55	m^3/s	
8	**Bypass Weir**			

8.6.5 Construction drawings




GRATED LOCKABLE LID
ø1200
LID RL 0.70m
SUPPORT BRACKET
RL 0.50m
ALL DISCHARGE HOLES TO BE 20ø
WATER RL 0.00m
125
250
375
STAINLESS STEEL STRAP AND FITTINGS
EXTENED SPINDAL
STEP IRONS FOR ACCESS
OUTLET RISER
150x150 TEE
INLET ø300 I.L. -1.1
FLOW
DISCHARGE CULVERT ø150
RESILIANT SEALING SLUICE VALVE
150x100 REDUCER
DETAIL 1 - RISER OUTLET
GRATING
RL 0.40m
100
DETAIL 2 - PIT ENTRY
0.5
0
0.5
1m
SCALE 1:25
Wetland Worked Example
Riser Details
FILE REF:
SCALE:
AS SHOWN

8.6.6 Example inspection and maintenance schedule

An example inspection and maintenance schedule for a constructed wetland showing local adaptation to incorporate specific features and configuration of each individual wetlands. The following is an inspection sheet developed for the Hobart wetland which was developed from the generic wetland maintenance inspection form.

HOBART WETLANDS – MAINTENANCE FORM		
Location		
Description	Constructed wetland and sediment forebay	
SITE VISIT DETAILS		
Site Visit Date: ______________		
Site Visit By: ______________		
Weather ______________		
Purpose of the Site Visit	Tick Box	Complete Sections
Routine Inspection	☐	Section 1 only
Routine Maintenance	☐	Section 1 and 2
Cleanout of Sediment	☐	Section 1, 2 and 3
Annual Inspection	☐	Section 1, 2, 3 and 4
SECTION 1 INSPECTION		
Depth of Sediment:	______________ m	
Cleanout required if Depth of Sediment ≥ 1.0 m	Yes/No	
Any weeds or litter in wetland (If Yes, complete Section 2 Maintenance)	Yes/No	
Any visible damage to wetland or sediment basin? (If Yes, completed Section 4 – Condition)	Yes/No	
Inspection Comments:		
SECTION 2 MAINTENANCE		
Are there weeds in the wetland?	Yes/No	
Were the weeds removed this site visit?	Yes/No	
Is there litter in the wetland or forebay?	Yes/No	
Was the litter collected this site visit?	Yes/No	

SECTION 3 CLEANOUT OF SEDIMENT		
Have the following been notified of cleanout date?	Yes	No
Coordinator – open space and/or drainage	☐	☐
Local Residents	☐	☐
Other (specify ………………………………………….)	☐	☐
Method of Cleaning (excavator or eductor)		
Volume of Sediment Removed (approximate estimate)	m^3	
Any visible damage to wetland or sediment forebay? (If yes, complete Section 4 Condition)	Yes/No	

SECTION 4 CONDITION					
Component	Checked?		Condition OK?		Remarks
	Yes	**No**	**Yes**	**No**	
Inlet weir or pipes					
Outlet riser/s and weir/s					
Sediment forebay					
Bypass channel (if constructed)					
Wetland vegetation					
Wetland banks and batter slopes					
Wetland floor					
Wetland diversion bunds (if constructed)					
Retaining walls					
Surrounding landscaping					
Comments:					

8.7 References

Engineers Australia, 2006, *Australian Runoff Quality Australian Runoff Quality: A guide to Water Sensitive Urban Design, Editor-in-Chief, Wong, T.H.F.*

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Chapter 9 Ponds

Definition:

An open body of water that has either occurred naturally or is man-made.

Purpose:

- To promote particle sedimentation, adsorption of nutrients by phytoplankton and ultra violet disinfection.
- They can be used as storages for reuse schemes and urban landform features for recreation as well as wildlife habitat.

Implementation considerations:

- In areas where wetlands are not feasible (e.g. very steep terrain), ponds can be used for a similar purpose of water quality treatment. In these cases, ponds should be designed to settle fine particles and promote submerged macrophyte growth.
- Fringing vegetation, while Aesthetically pleasing, contributes little to improving water quality. Nevertheless, it is necessary to reduce bank erosion.
- Ponds still require pretreatment such as a sediment basins that need maintaining more regularly than the main open waterbody.
- Ponds are well suited to steep confined valleys where storage volumes can be maximised.
- Some limitations for ponds can be site specific for example, proximity to airports, as large numbers of flocking birds can cause a disturbance to nearby air traffic.
- They also require regular inspection and maintenance to ensure that their Aesthetic value is not diminished.

Ponds are popular landscape features in urban areas

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9.2 Introduction .... 9—3

9.3 Verifying size for treatment .... 9—4

9.4 Design procedure for ponds and lakes .... 9—4

9.5 Checking tools .... 9—19

9.6 Maintenance requirements .... 9—24

9.7 Worked example .... 9—26

9.8 References .... 9—41

9.1 Preface

Several sections of this chapter (specifically those in relation to algal growth) make reference to data and studies for sites in Victoria. These sections have not been altered specifically for Tasmania as the theory behind them is applicable to all areas or is derived from studies carried out in Victoria.

9.2 Introduction

Ponds and lakes are artificial bodies of open water usually formed by a simple dam wall with a weir outlet structure or created by excavating below natural surface levels. The depth of water in these waterbodies are typically greater than 1.5 m and there is usually a small range of water level fluctuation although newer systems may have riser style outlets allowing for extended detention and longer temporary storage of inflows. Aquatic vegetation has an important function for water quality management in ponds and lakes. Emergent macrophytes are normally restricted to the margins because of water depth, although submerged plants may occur in the open water zone. The submergent plants will enhance treatment of stormwater inflow and prevent ingress of weed species. Ponds are seldom used as "stand-alone" stormwater treatment measures and are often combined with constructed wetlands as a treatment forebay to the open waterbody. In many cases, these systems ultimately become the ornamental waterbody that require water quality protection.

Ponds and lakes often form part of a flood retarding system and design requirements are generally associated with hydraulic structures for flow conveyance and flood attenuation. These are not covered in this document and only design elements associated with the water quality function of the system is presented.

There have been cases where water quality problems in ornamental ponds and lakes are caused by poor inflow water quality, especially high organic load, infrequent waterbody "turnover" and inadequate mixing. Detailed modelling may be necessary to track the fate of nutrients and consequential algal growth in the waterbody during periods of low inflow (and thus long detention period). As a general rule, it is recommended that the turnover period for lakes between 20 and 50 days (depending on water temperatures) at least 80% of the time (see Appendix D for more information).

If these turnover times can not be met, it may be necessary to introduce a lake management plan to reduce the risk of algal blooms during the dry season.

This design procedure outlines design elements for large waterbodies associated with the design of a constructed lake, an associated wetland forebay (or inlet zone) and water re-circulation scheme (if required) to maintain water quality in the pond. Further investigations need to be undertaken to finalise the design from that presented in the Worked Example to address issue such as the embankment stability and detailed design of structural elements. These are discussed in the worked example.

9.3 Verifying size for treatment

The curves shown in Figure 9.1 describe the pollutant removal performance expected for constructed pond and lake systems in Hobart (reference site) for suspended solids, total phosphorus and total nitrogen. The curves were derived assuming the systems receive direct runoff (ie. no other WSUD elements upstream) and have the following characteristics:

- The mean depth is 2.0 m
- Outflow from the system is via an overflow weir.

These curves can be used, together with the adjustment factors derived from the hydrologic regionalisation procedure discussed in Chapter 2, to check the expected performance of the wetland system for removal of TSS, TP and TN.

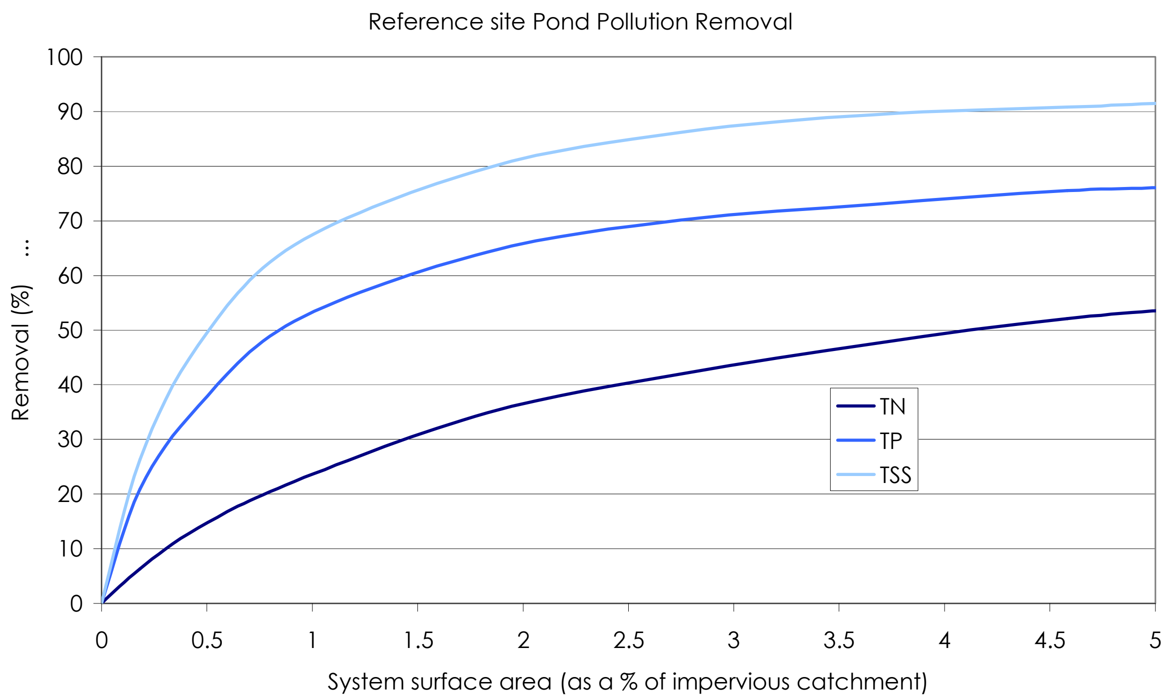


Figure 9.1. TSS, TP and TN removal in pond systems

9.4 Design procedure for ponds and lakes

Design considerations include the following:

1. Computations to ensure that the pond volume is not excessively large or too small in comparison to the hydrology of the catchment.
2. Configuring the layout of the pond and inlet zone such that the system *hydraulic efficiency* can be optimised, including a transition structure between the inlet zone and the open waterbody
3. Design of hydraulic structures

a. inlet structure to provide for energy dissipation of inflows up to the 100 year ARI peak discharge

b. Design of the pond outlet structure for the pond

4. Landscape design

   a. Edge treatment

   b. Recommended plant species and planting density

5. Maintenance provisions

The figure below summarises the pond/lake design elements. The following sections detail the design steps required for constructed stormwater wetland systems.

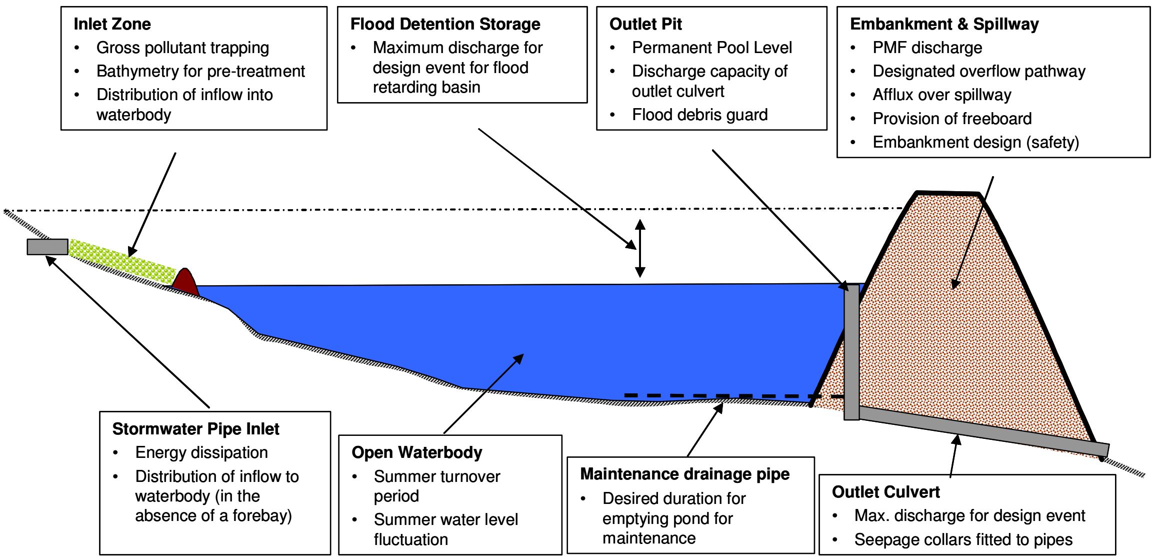


Figure 9.2. Pond/Lake design elements and design considerations

9.4.1 Hydrology

The hydrologic operation of a pond or lake is to safely convey stormwater inflows up to the peak 100 year ARI discharge into the pond or lake system with discharge from the pond or lake being via a combination of pipe (low flow) culvert and overflow spillway.

9.4.1.1 *Flood Estimation*

A range of hydrologic methods can be applied to estimate design flows. If the typical catchment areas are relatively small, the Rational Method Design Procedure is considered to be a suitable method for estimating design flows. However, the use of the Rational Design Procedure should strictly be used to size inlet hydraulic structures only and that a full flood routing computation method should be used in sizing the outlet hydraulic structures (e.g. outlet pipe, spillway and embankment height, etc.)

9.4.1.2 Waterbody residence time

As discussed earlier, a combination of inflow water quality, organic load and water circulation characteristics influence the water quality in the pond. Water quality problems for large lakes exhibiting relatively small upstream catchments typically arise because the water body receives insufficient water inflows to circulate and/or displace the water stored in the lake. Under long residence times blue green algae blooms can occur.

Experience with management of many open waterbodies suggests that many incidences of algal blooms in waterbodies are preceded by extended periods of no or minimal inflows. Waterbody residence time (or turnover frequency) analysis can often be a very useful indicator as to whether the waterbody is of significant risk of water quality problems (especially associated with algal growth). Appendix D discusses the risk of algal growth in more detail.

Turnover analysis can be undertaken using probabilistic monthly evaporation and rainfall data or daily historical rainfall data, with the latter providing a more rigorous analysis. Average residence times are calculated by modelling continuous simulation of flows into and out of a lake. Estimates of daily outflows are then summed (in arrears) to give an estimate of the average residence time of the lake for each day of the simulation

Seasonal distribution of rainfall and the relative volume of the waterbody to the mean annual runoff will determine the range of residence periods for the waterbody. For example, a small waterbody with a large catchment will have small residence times simply by the fact that the volume of the waterbody is a small fraction of the mean annual runoff volume of the catchment. On the other hand, the residence times of a larger waterbody will be more sensitive to seasonality of rainfall and thus a higher risk of long water detention periods and associated water quality problems.

A cumulative probability distribution of waterbody residence time can be derived using the modelled outflows from a lake. Figure 9.3 shows the results of an example residence time analysis for a waterbody in Melbourne.

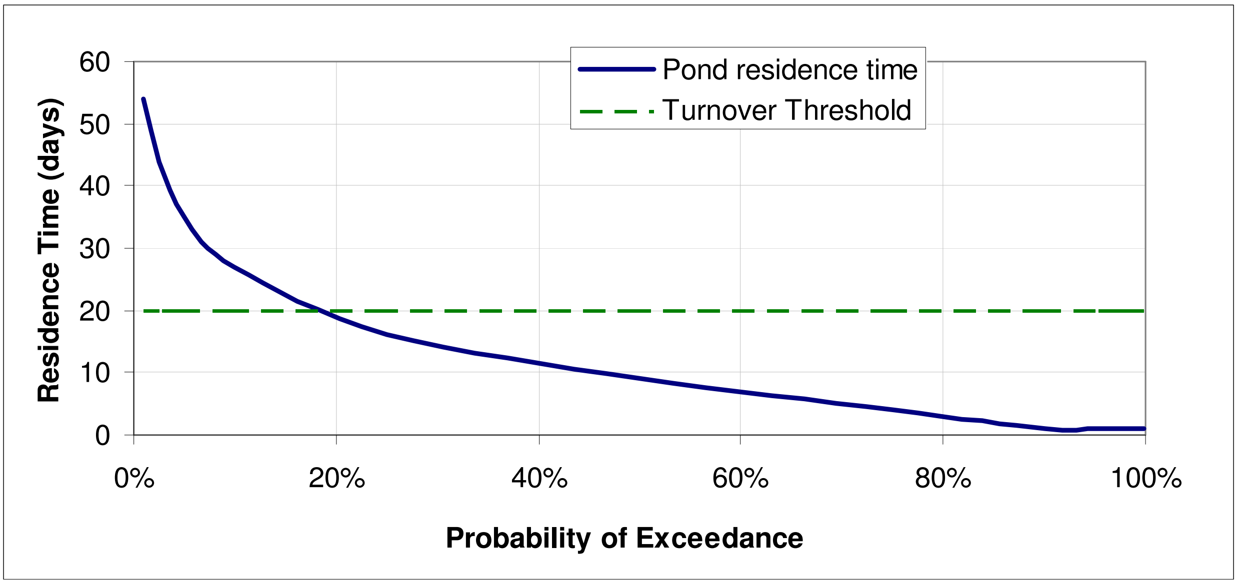


Figure 9.3. Results of residence time analysis for a waterbody in Melbourne

Algal growth can occur rapidly under favorable conditions. Nuisance growths (blooms) of cyanobacteria (Blue-green algae) can occur in both natural and constructed water bodies. In constructed water bodies it is important to ensure that designs include measures to restrict cyanobacterial growth. Cyanobacterial blooms can have adverse effects on aquatic ecosystem function, Aesthetics and public amenity. Some species of cyanobacteria are of particular concern because of their potential to produce toxins.

Many factors influence cyanobacterial growth including (Tarczynska *et al*, 2002; Mitrovic *et al*, 2001; Sherman *et al*, 1998; Reynolds, 2003):

- light intensity
- water temperature
- nutrient concentration
- hydrodynamics
- stratification
- catchment hydrology
- zooplankton grazing
- parasitis

Excessive growth of cyanobacterial species is considered an Alert Level 1 Algal Bloom when concentrations reach 15000 cells/mL (Government of Victoria, 'Blue-Green Algae' *Information Brochure*). Appendix D discusses these issues in more detail.

Assuming adequate light and nutrient availability, a model of algal growth can be developed using a simple relationship between time and growth rate at various temperatures (see Appendix D). This simple model can be used to determine how long it will take for an algal population to reach bloom proportions (15,000 cells/mL) and hence inform the development of guidelines on water body hydraulic detention time.

Modelling conducted and based on reasonable assumptions suggest the following times under ideal conditions for blooms to occur depending on mixing conditions (Appendix D).

Figure 9.4 and 9.5 were derived assuming a 'best practice' design of a pond. This includes shallow depth, have a flat bottom and are generally well mixed. A reasonable assumption is that the hydrodynamic conditions in a best management practice design varies somewhere between fully mixed and diurnally, partially mixed.

The curves represent three temperature zones relating to summer water temperature as follows:

- ► 15°C Use for upland sites in the Eastern and Western Ranges.
- ► 20°C Use for lowland sites south of the Great Dividing Range.
- ► 25°C Use for lowland sites north of the Great Dividing Range.

The modeling approach taken is considered to be reasonably conservative. For example it adopts:

- Non-limiting conditions for nutrient and light availability
- Growth rates for a known nuisance species (*Anabaena circinalis*)
- Summer temperature values (the main risk period)
- High starting population concentrations (50 cells/mL)

As a result, a probabilistic approach to the use of detention time criteria is recommended. A 20% exceedance is suggested as an acceptable risk to compensate for the occurrence of all other risk factors being favorable for algal growth. The 20% exceedance of a specific detention time objective does not indicate that a bloom will occur; just that detention time (for a given temperature range) is long enough for exponential growth to achieve a bloom alert level of 15,000 cells/mL if all other risk factors were favourable. The 20% exceedance value is an interim value chosen as a relatively conservative estimate of the general variation in ecological factors in the Australian environment.

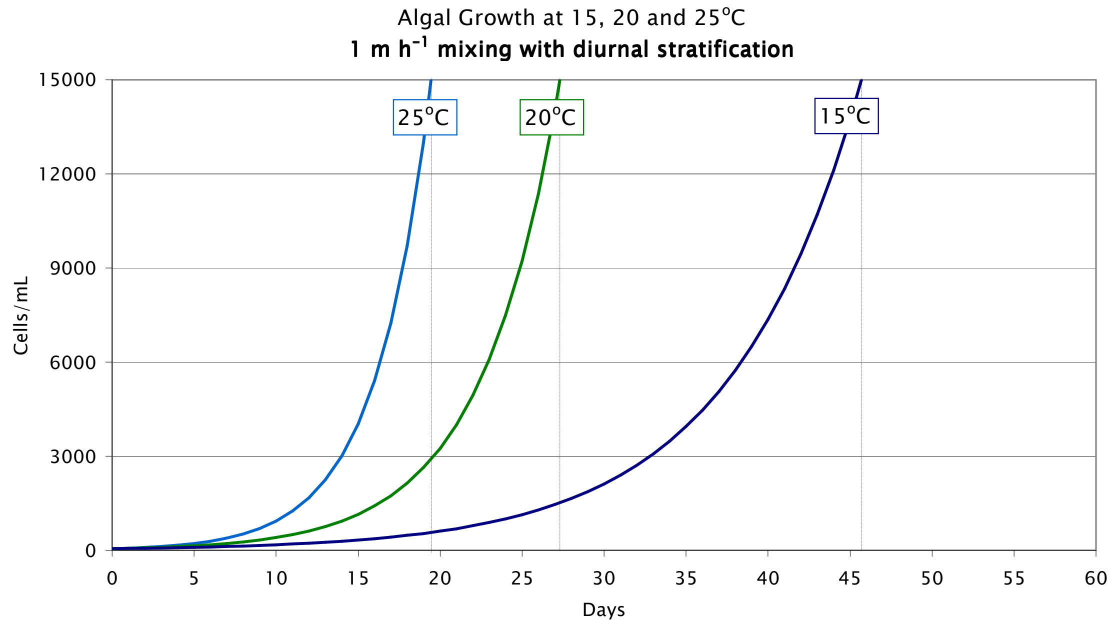


Figure 9.4. Growth curves illustrating modelled times for cyanobacterial populations to reach bloom proportions under different temperature conditions and 1 m h^{-1} mixing conditions with diurnal stratification. Based on growth rates of *A. circinalis* measured *in situ* (Westwood and Ganf, 2004) adjusted for temperature, Q_{10} 2.9, and assuming 50 cells/mL starting concentrations.

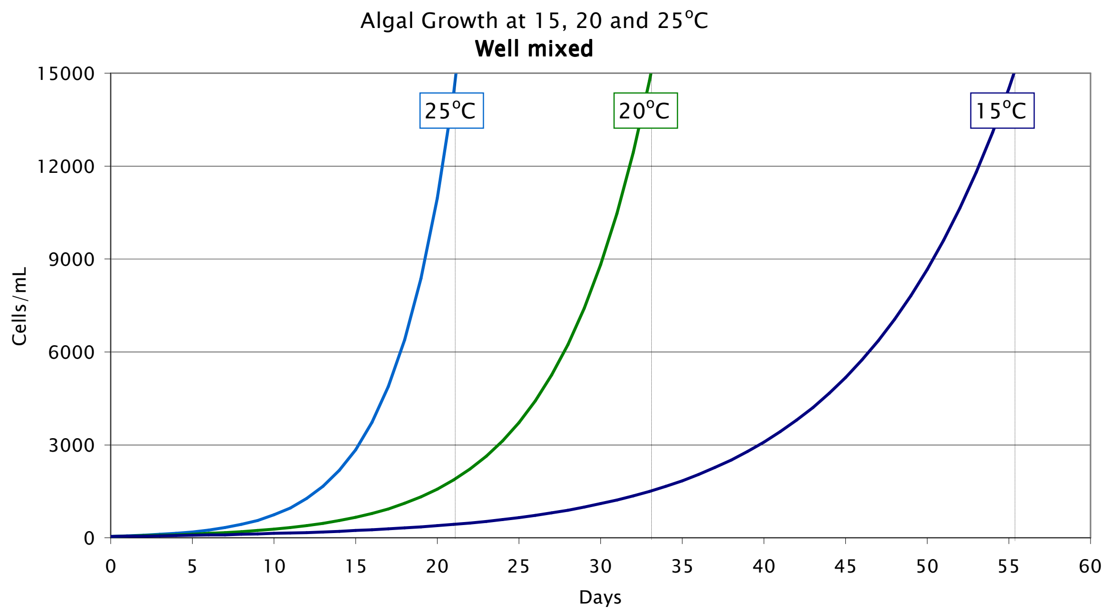


Figure 9.5. Growth curves illustrating modelled times for cyanobacterial populations to reach bloom proportions under different temperature conditions and well mixed conditions. Based on growth rates of *A. circinalis* measured *in situ* (Westwood and Ganf, 2004) adjusted for temperature, Q_{10} 2.9, and assuming 50 cells/mL starting concentrations.

9.4.1.3 Turnover design criteria

The following guideline detention times are recommended. For water bodies with summer water temperatures in the following ranges, the 20%tile detention times should not exceed:

- 50 days (15°C)
- 30 days (20°C)
- 20 days (25°C)

These values are broadly consistent with literature detention time values considered to be protective against the risk of cyanobacterial blooms (Reynolds 2003, Wagner-Lotkowska *et al* 2004) and consistent with current industry experience.

9.4.1.4 Lake Water Level Fluctuation Analysis

Water level fluctuation analysis is another important analysis that needs to be undertaken as they may have a significant influence on the landscape design of the lakes edge. As in the waterbody turnover analysis, lake water level analysis can be undertaken using probabilistic monthly evaporation and rainfall data or daily historical rainfall data, with the latter providing a more rigorous analysis. A variety of models can be used to predict water levels from continuous simulations (e.g. MUSIC). A typical analysis may be to determine the 10%tile, 50%tile and 90%tile water depths in a lake during summer.

The results of an analysis to ascertain the relationship between lake volume and probabilistic summer water levels for a proposed lake in Shepparton in Victoria are plotted below.

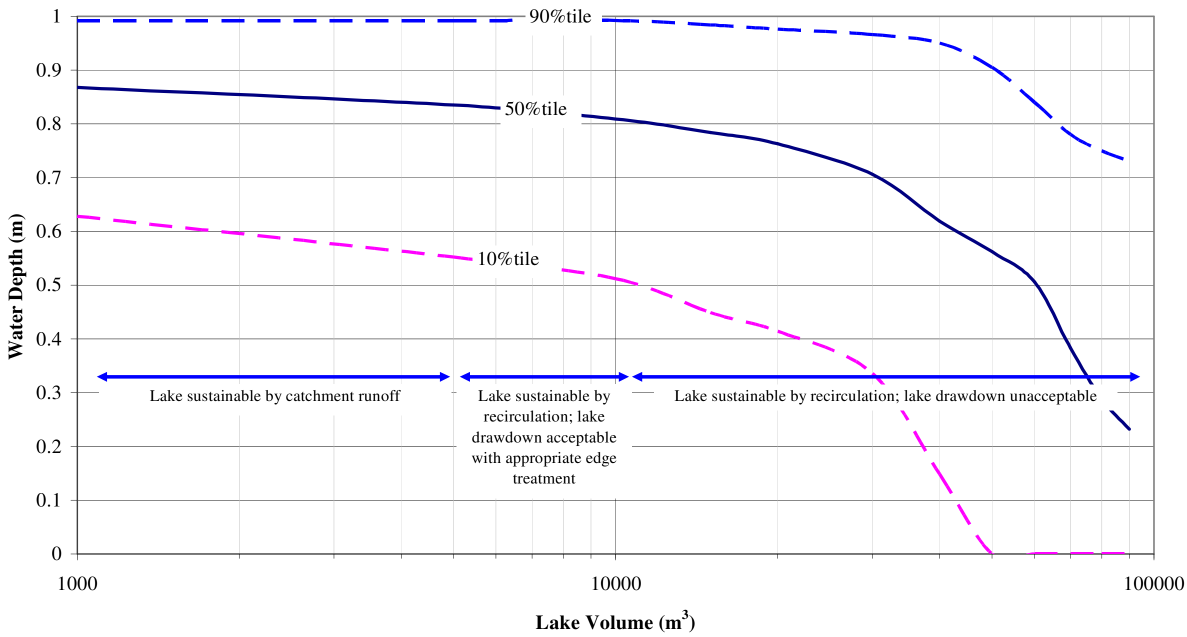


Figure 9.6. Analysis of Probabilistic Summer Water Depth with different Lake Volume for a proposed lake in Shepparton in Victoria

9.4.1.5 Option for a larger waterbody

Often much larger open waterbodies are proposed by landscape and urban designers as ornamental lakes while serving the function of stormwater quality improvement. This can mean further design and operation considerations necessary to maintain a healthy waterbody, to provide an acceptable low level of risk of algal growth.

If an analysis indicates that a waterbody is at significant risk of algal blooms (i.e. the *Turnover Design Criteria* are not met) a lake turnover strategy will need to be developed. In addition, a lake management plan may be required and involve more detailed modeling using such models as the Cooperative Research Centre for Freshwater Ecology's Pond Model.

A re-circulating pump can be used to increase the turnover of a waterbody during the drier months such that it has an acceptable residence time (in accordance with the *Turnover Design Criteria*). The required pump rate is estimated as the lake volume divided by the required maximum residence time.

To re-circulate the lake water, it is necessary to pass the water through a wetland system to reduce nutrient levels in the water column and limit the growth of planktonic algae. The wetland should be designed in accordance to the design procedure for constructed wetlands (see Chapter 8) with a permanent pool that extends over the majority of the wetland area. The combined permanent pool and extended detention volume should be size to provide a recommended detention period of 5 days for the re-circulating pump rate to ensure adequate nutrient removal.

9.4.2 Pond Layout

9.4.2.1 Size and Dimensions

To optimise hydraulic efficiency, i.e. reduce short circuits and dead zones, it is desirable to adopt a high length to width ratio and to avoid zones of water stagnation. The ratio of length to width varies depending on the size of the system and the site characteristics while inlet and outlet conditions as well as the general shape of the pond can influence the presences and extent of water stagnation zones. To simplify the design and earthworks, smaller systems tend to have length to width ratios at the lower end of the range. This can often lead to poor hydrodynamic conditions.

Persson *et al* (1999) used the term hydraulic efficiency to define the expected hydrodynamic characteristics for a range of configurations of stormwater detention systems. Engineers Australia (2003) present expected hydraulic efficiencies of detention systems for a range of notional shapes, aspect ratios and inlet/outlet placements within stormwater detention systems and recommends that such systems should not have a λ value of less than 0.5 and should be designed to promote hydraulic efficiencies greater than 0.7 (see Figure 9.7).

λ is estimated from the configuration of the basin according to Figure 9.7.

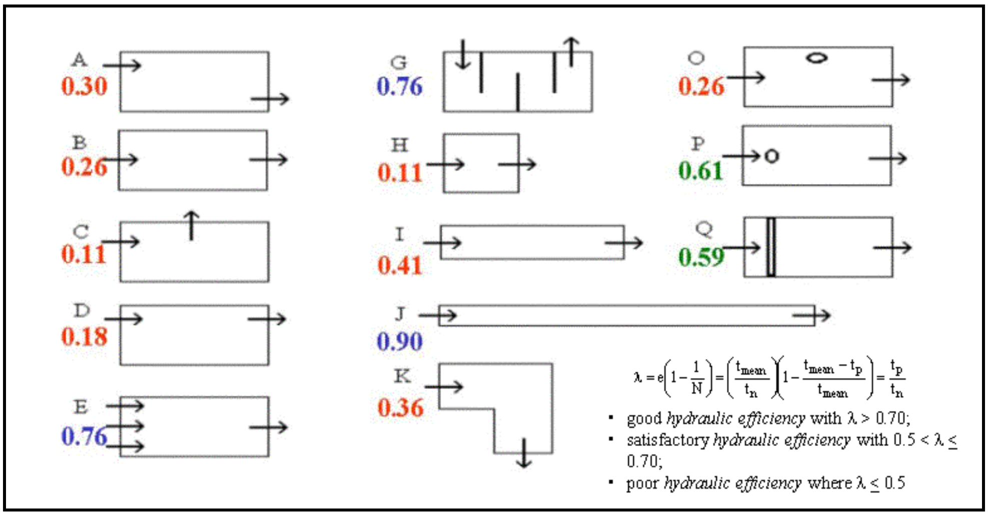


Figure 9.7. Hydraulic Efficiency - λ - A measure of Flow Hydrodynamic Conditions in Constructed Wetlands and Ponds; Range is from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment (Persson et al., 1999)

The numbers in Figure 9.7 represent the values of λ that are used to estimate the turbulence parameter 'n' for Equation 9.2 (see next section).

Higher values (of λ) represent ponds with good sediment retention properties, where a value of λ greater than 0.5 should be a design objective. If the pond configuration yields a lower value, modification to the configuration should be explored to increase the λ value (e.g. inclusion of baffles, islands or flow spreaders).

There can often be multiple inlets into the waterbody and the locations of these inlets to the outlet structure can influence the hydraulic efficiency of the system. Inlet structures design that reduce localized water eddies and promote good mixing of water within the immediate vicinity of the inlet may be necessary and the use of an inlet zone is a common approach to inlet design.

The shape of the pond also has a large impact on the effectiveness to retain sediments and generally a length to width ratio of at least 3:1 should be aimed for. In addition, the location of the inlet and outlet, flow spreaders and internal baffles impact the hydraulic efficiency of the basin for stormwater treatment. These types of elements are noted in Figure 9.7 as the figure "o" in diagrams O and P (which represent islands in a waterbody) and the double line in diagram Q which represents a structure to distribute flows evenly.

9.4.2.2 Inlet zone (or forebay)

It is good design practice to provide pre-treatment of stormwater to ponds and lakes for removal of sediment, organic matter and nutrients. The inlet zone can take many forms, ranging from systems that function as a sedimentation basin to that of a shallow ephemeral wetland. They are a transitional zone into the deeper waters of a pond.

Some inlet zones are constructed with a porous embankment at its transition with the deeper water zone to promote a wider distribution of inflow water across the open water body.

The bathymetry across the inlet zone is to vary gradually from 0.2 m above the permanent pool level to 0.3 m below the permanent pool level over a distance of between 10 m to 20 m.

There is generally little need for any hydraulic structures separating an inlet zone of a pond to the open water section, although a designer may consider the use of a porous embankment to promote better flow distribution into the open water zone. A low flow vegetated swale should be provided to convey dry weather flow and low flows to the open waterbody.

The notional required inlet zone area can be computed by the use of sedimentation theory (see Chapter 3), targeting the 125 μm sediment (settling velocity of 11 mm/s) operating at the 1 year ARI peak discharge.

The specification of the required area (A) of a sedimentation basin may be based on the expression by Fair and Geyer (1954), formulated for wastewater sedimentation basin design:

$$R = 1 - \left(1 + \frac{1}{n}\frac{v_s}{Q/A}\right)^{-n}$$

Equation 9.1

where	R	=	fraction of target sediment removed
	v_s	=	settling velocity of target sediment
	Q/A	=	rate of applied flow divided by basin surface area
	n	=	turbulence or short-circuiting parameter

The above expression for sedimentation is applied with 'n' being a turbulence parameter. Figure 9.7 provides guidance on selecting an appropriate 'n' value (according to the configuration of the basin). 'n' is selected using the following relationship:

$$\lambda = 1 - 1/n; \quad n = \frac{1}{1-\lambda}$$

Equation 9.2

Equation 9.1 is strictly applicable for systems with no permanent pool, and will generally over-estimate the required area of a sedimentation basin. This equation is thus often considered to provide an upper limit estimate of the required size for sedimentation basins.

Good practice in the design of inlet zone will include a permanent pool to reduce flow velocities and provide storage of settled sediment.

The presence of a permanent pool reduces flow velocities in the sedimentation basin and thus increases detention times. Owing to the outlet structure being located some distance above the bed of a sedimentation basin, it is also not necessary for sediment particles to settle to the bed of the basin to effectively retain the sediments. It is envisaged that sediments need only settle to an effective depth which is less than the depth to the bed of the sediment. This depth is considered to be approximately 1 m below the permanent pool level.

Equation 9.1 can thus be re-derived to account for the effect of the permanent pool storage as follows:

$$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)}\right]^{-n}$$

Equation 9.3

where d_e is the extended detention depth (m) above the permanent pool level

d_p is the depth (m) of the permanent pool

d^* is the depth below the permanent pool level that is sufficient to retain the target sediment (m) - adopt 1.0 or d_p whichever is lower.

list the typical settling velocities of sediments.

Table 9-1 Settling velocities under ideal conditions

Classification of particle size	Particle diameter (μm)	Settling velocities (mm/s)
Very coarse sand	2000	200

Coarse sand	1000	100
Medium sand	500	53
Fine sand	250	26
Very fine sand	125	11
Coarse silt	62	2.3
Medium silt	31	0.66
Fine silt	16	0.18
Very fine silt	8	0.04
Clay	4	0.011

9.4.2.3 Cross Sections

Batter slopes on approaches and immediately under the water line have to be configured with consideration of public safety. Both hard and soft edge treatments can be applied to compliment the landscape of the surrounding area of a pond or lake. A soft edge treatment approach will involve a gentle slope to the water edge and extending below the water line be adopted before the batter slope steepen into deeper areas. This is illustrated in

.

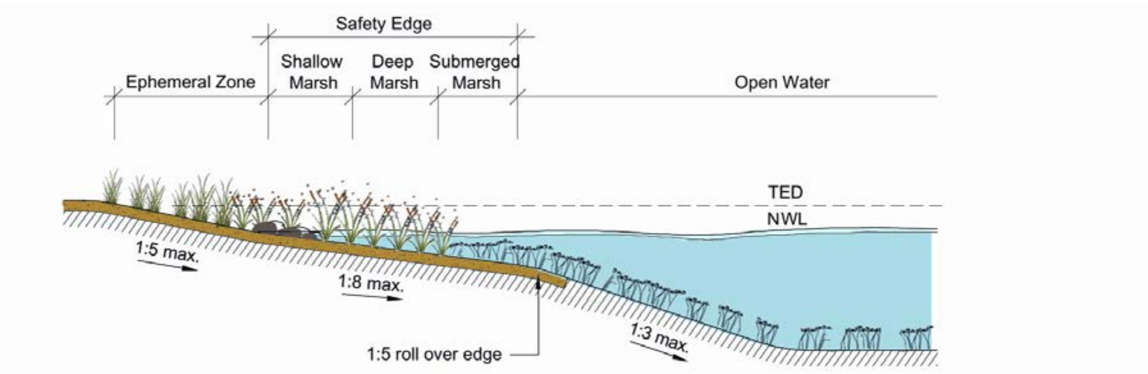


Figure 9.8 Illustration of a soft edge treatment for ponds and lakes (GbLA, 2004)

An alternative to the adoption of a flat batter slope beneath the water line is to provide a 3 m "safety bench" that is less than 0.2 m deep below the permanent pool level be built around the waterbody.

Figure 9.9 shows an option for a hard edge detail, using a vertical wall and has an associated handrail for public safety. This proposal uses rock to line the bottom of the pond to prevent vegetation (particularly weed) growth.

The safety requirements for individual ponds and lakes may vary from site to site, and it is recommended that an independent safety audit be conducted of each design.

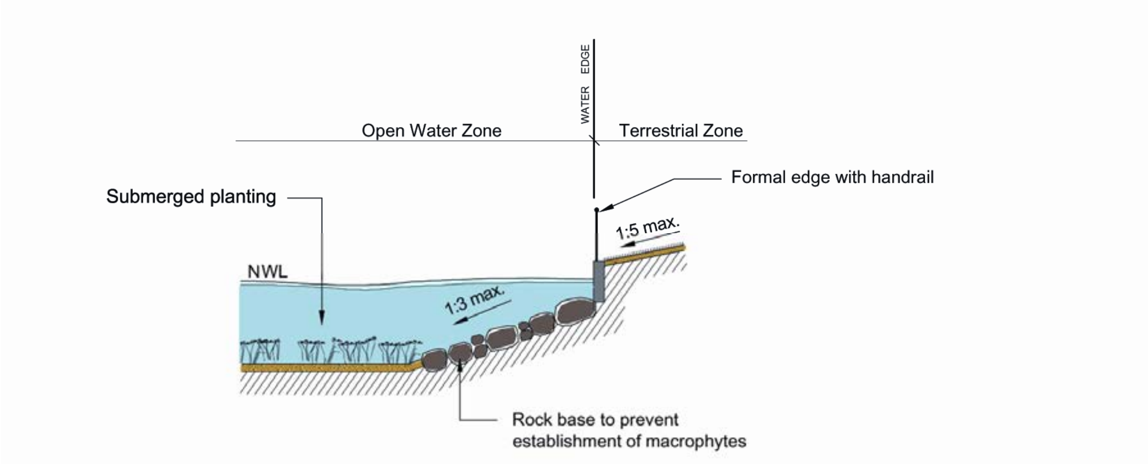


Figure 9.9 Illustration of hard edge treatment for open waterbodies (GbLA, 2004)

9.4.3 Hydraulic Structures

Hydraulic structures are required at the inlet and outlet of a pond or lake. Their function is essentially one of conveyance of flow with provisions for (i) energy dissipation at the inlet structure(s) and (ii) extended detention (if appropriate) at the outlet.

9.4.3.1 Inlet Structure

Discharge of stormwater into the open waterbody of a pond or lake may be via an inlet zone or direct input. In both cases it will be necessary to ensure that inflow energy is adequately dissipated so as not to cause localised scour in the vicinity of the pipe outfall. Design of stormwater pipe outfall structures are common hydraulic engineering practice.

Litter control is normally required at the inlet structure and it is generally recommended that some form of gross pollutant trap be installed as part of the inlet structure. There are a number of proprietary products for capture of gross pollutants and these are discussed in Chapter 7 in Australian Runoff Quality (Engineers Australia, 2006). The storage capacity of gross pollutant traps should be sized to ensure that maintenance (clean-out) frequency is not greater than 1 every 3 months.

9.4.3.2 Outlet Structure

The outlet structure of a pond or lake can be configured in many ways and is dependent on the specified operation of the system during periods of high inflows. Many ponds form part of a flood retarding basin in which case the outlet structure consist of two components, i.e. outlet pit and outlet culvert. The computation of the required outlet culvert is an essential element of the retarding basin design and will be based on flood routing computation as outlined in ARR. The main function of the inlet pit is to maintain the desired permanent pool level and to provide a means of connecting the maintenance pipe to the outlet culvert. Design considerations of the outlet pit include the following:

- Ensure that the crest of the pit is set at the permanent pool level of the lake or pond

- Ensuring that the dimension of the pit provides discharge capacity that is greater than the discharge capacity of the outlet culvert or pipe
- Protection against clogging by flood debris

The dimension of an outlet pit is determined by considering two flow conditions, weir and orifice flow (Equations 9.4 and 9.5)

A blockage factor is also used to account for any debris blockage. Assuming the pit is 50% blocked is recommended. Generally it will be the discharge pipe from the inlet zone (and downstream water levels) that controls the maximum flow rate from the area, it is therefore less critical if the outlet pit is oversized to allow for blockage.

1. Weir flow condition – when free overall conditions occur over the pit (usually when the extended detention storage of the retarding basin is not fully engaged), ie.

$$P = \frac{Q_{des}}{B \cdot C_w \cdot H^{1.5}}$$

Equation 9.4

P = Perimeter of the outlet pit

B = Blockage factor (0.5)

H = Depth of water above the crest of the outlet pit

Q_{des} = Design discharge (m^3/s)

C_w = weir coefficient (1.7)

2. Orifice flow conditions – when the inlet pit is completely submerged (corresponding to conditions associated with larger flood events), ie.

$$A_o = \frac{Q_{des}}{B \cdot C_d \sqrt{2gH}}$$

Equation 9.5

C_d = Orifice Discharge Coefficient (0.6)

B = Blockage factor (0.5)

H = Depth of water above the centroid of the orifice (m)

A_o = Orifice area (m^2)

Q_{des} = Design discharge (m^3/s)

Whichever conditions calculates the largest required pipe should be adopted. It is important that an outlet pit is prevented from blockage by debris. Design consideration needs to include means of preventing blockage of the outlet structure.

Outlet culvert of pipe capacity is estimated using the orifice equation (10.5) without a blockage factor.

9.4.3.3 Maintenance Drain

The waterbody should be able to be drained for maintenance with manual operation. A suitable design flow rate is one which can draw down the permanent pool within seven days although, depending on the volume of the waterbody, this may not be realistic.

The orifice discharge equation (Equation 9.5) is considered suitable for sizing the maintenance drain (without blockage factor) on the assumption that the system will operate under inlet control.

9.4.4 High-flow route design

The provision of a high-flow route is standard design practice to ensure that overflow from the dam embankment can be safely conveyed either by the use of a spillway or ensuring that the embankment is designed to withstand overtopping. This issue requires specialised design inputs and are not discussed in this document.

9.4.5 Vegetation specification

Vegetation planted along the littoral zone of a pond or lake serves the primary function of inhibiting public access to the open waterbody. Terrestrial planting beyond the littoral zone may also be recommended to screen areas and provide an access barrier to uncontrolled areas of the stormwater treatment system.

Plant species for the inlet zone area will be predominantly of ephemeral wetland species. The reader is referred to the Appendix B for a list of suggested plant species suitable for the inlet zone and littoral zones in Victoria and recommended planting densities.

9.4.6 Design Calculation Summary

Overleaf is a design calculation summary sheet for the key design elements of a construction wetland to aid the design process.

Ponds and Lakes — CALCULATION SUMMARY

	CALCULATION TASK		OUTCOME		CHECK
1	**Identify design criteria**				☐
	Design ARI Flow for inlet hydraulic structures			year	
	Design ARI Flow for outlet hydraulic structures				
	Design ARI for emergency hydraulic structures			year	
	80%tile turnover period			days	
	Probabilistic summer water level – 10%tile			m	
	Probabilistic summer water level – 90%tile			m	
	Flood Detention Storage Volume (from flood routing analysis)			m^3	
	Outlet pipe dimension (from flood routing analysis)			mm	
2	**Catchment characteristics**				☐
		Residential		Ha	
		Commercial		Ha	
	Fraction impervious				☐
		Residential			
		Commercial			
3	**Estimate design flow rates**				
	Time of concentration				
	estimate from flow path length and velocities			minutes	☐
	Identify rainfall intensities				
		station used for IFD data:			
		Design Rainfall Intensity for inlet structure(s)		mm/hr	☐
	Design runoff coefficient				
		inlet structure(s)			☐
	Peak design flows				☐
		Inlet structure(s)		m^3/s	
		Outlet structure(s)		m^3/s	
4	**Forebay Zone Layout**				☐
		Area of Forebay Zone		m^2	
		Aspect Ratio		L:W	
		Hydraulic Efficiency			
5	**Lake Residence Time**				☐
		Is wetland forebay for recirculation required			
		Area of wetland forebay for water recirculation		m^2	
		Detention time during reciculation of wetland forebay		days	
		Lake water recirculation pump rate		L/s	
6	**Pond Layout**				☐
		Area of Open Water		m^2	
		Aspect Ratio		L:W	
		Hydraulic Efficiency			
		Length		m	
		Width		m	
		Cross Section Batter Slope		V:H	
7	**Hydrualic Structures**				
	Inlet Structure				☐
		Provision of energy dissipation			
	Outlet Structure				☐
		Pit dimension		L x B	
				mm diam	
		Discharge capacity of outlet pit		m^3/s	
		Provision of debris trap			
	Maintenance Drain				☐
		Diameter of Maintenance Valve		mm	
		Drainage time		days	

9.5 Checking tools

This section provides a number of checking aids for designers and referral authorities. In addition, advice on construction techniques and lessons learnt from building lake systems are provided.

Checklists are provided for:

- Design assessments
- Construction (during and post)
- Operation and maintenance inspections
- Asset transfer (following defects period).

9.5.1 Design assessment checklist

The checklist below presents the key design features that should be reviewed when assessing a design of a lake. These considerations include configuration, safety, maintenance and operational issues that should be addressed during the design phase.

Where an item results in an "N" when reviewing the design, referral should be made back to the design procedure to determine the impact of the omission or error.

In addition to the checklist, a proposed design should have all necessary permits for its installations. The referral agency should ensure that all relevant permits are in place. These can include permits to clear vegetation, to dredge, create a waterbody, divert flows or disturb fish or platypus habitat.

Land ownership and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the asset transfer checklist (see 9.5.4).

Pond and Lake Design Assessment Checklist			
Lake Location:			
Hydraulics	Minor Flood: (m^3/s)	Major Flood (m^3/s):	
Inlet Zone		Y	N
Inlet pipe/structure sufficient for maximum design flow (Q_5 or Q_{100})?			
Scour protection provided at inlet structures?			
Configuration of forebay zone (aspect, depth and flows) allows even distribution of inflow into open water zone?			
Maintenance access provided?			
Public access to forebay zone managed through designated pathways?			
Gross pollutant protection measures provided on inlet structures?			
Open Water Zone		Y	N
Depth of open water > 1.5 m?			
Aspect ratio provides hydraulic efficiency >0.5?			
Depth of permanent water >1.5m?			
20% probability of exceedance in accordance with guidelines (i.e. 20, 30 or 50 days)			
Edge treatment – Batter slopes from accessible edges shallow enough to allow egress?			
Edge treatment – provision of littoral zone planting with 1:8 batter slopes to 0.2 m below the waterline ?			
Edge treatment – vertical fall to shallow bench?			
Maintenance access provided?			
Public access to open zones restricted to designated pathways with appropriate safety considerations?			
Embankment height > flood detention depth?			
Lake turnover management plan developed (if turnover is inadequate)?			
Probabilistic summer water level fluctuation within desired range and edge treatment developed to suit?			
Outlet Structures		Y	N
Outlet pit set at permanent water level?			
Discharge capacity of outlet pit > computed discharge capacity of outlet pipe? (checked against weir flow and orifice flow operating conditions)			
Maintenance drain provided?			
Protection against clogging of outlet pit provided?			

9.5.2 Construction advice

This section provides general advice for the construction of lakes. It is based on observations from construction projects around Australia.

Protection from existing flows

It is important to have protection from upstream flows during construction of a lake or pond system. A mechanism to divert flows around a construction site, protect from litter and debris is required. This can be achieved by constructing a high flow bypass channel initially and then diverting all inflows along the channel until the pond system is complete.

High flow contingencies

Contingencies to manage risks associated with flood events during construction are required. All machinery should be stored above acceptable flood levels and the site stabilised as best possible at the end of each day as well as plans for dewatering following storms made.

Erosion control

Immediately following earthworks it is good practice to revegetate all exposed surfaces with sterile grasses (e.g. hydroseed). These will stabilise soils, prevent weed invasion yet not prevent future planting from establishing.

Inlet erosion checks

It is good practice to check the operation of inlet erosion protection measures following the first few rainfall events. It is important to check for these early in the systems life, to avoid continuing problems. Should problems occur in these events the erosion protection should be enhanced.

Inlet zone access

An important component of an inlet zone (or forebay) is accessibility for maintenance. Should excavators be capable of reaching all parts of the inlet zone an access track may not be required to the base of the inlet zone, however an access track around the perimeter of the inlet zone is required. If sediment collection is by using earthmoving equipment, then a stable ramp will be required into the base of the inlet zone (maximum slope 1:10).

Inlet zone base

To aid maintenance it is recommended to construct the inlet zone either with a hard (i.e. rock or concrete) bottom or a distinct sand layer. These serve an important role for determining the levels that excavation should extend to during sediment removal (i.e. how deep to dig) for either systems cleaned from the banks or directly accessed. Hard bases are also important if maintenance is by driving into the basin.

Dewatering collected sediments

An area should be constructed that allows for dewatering of removed sediments from an inlet zone. This area should be located such that water from the material drains back into the inlet zone. Material should be allowed to drain for a minimum of overnight before disposal.

Timing for planting

Timing of vegetation planting is dependent on a suitable time of year (and potential irrigation requirements) as well as timing in relation to the phases of development. Temporary sediment controls should always be used prior to planting as lead times from earthworks to planting are often long.

Vegetation establishment

During the establishment phase water levels should be controlled carefully to prevent seedlings from being desiccated or drowned. This is best achieved with the use of maintenance drains. Once established, water levels can be raised to operational levels. This issue is further discussed in Appendix B.

Bird protection

Bird protection (e.g. nets) should be considered for newly planted area of wetlands, birds can pull out young plants and reduce plant densities.

Trees on embankments

Consideration should be given to the size of trees planted on embankments as root systems of larger trees can threaten the structural integrity of embankments.

9.5.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Ponds and Lakes

INSPECTED BY:
DATE:
TIME:
WEATHER:
CONTACT DURING VISIT:

SITE: ____________________

CONSTRUCTED BY: ____________________

DURING CONSTRUCTION									
Items inspected	Checked		Satisfactory	Unsatisfactory	**Structural components**	Checked		Satisfactory	Unsatisfactory
Preliminary works	Y	N			15. Location and levels of outlet as designed	Y	N		
1. Erosion and sediment control plan adopted					16. Safety protection provided				
					16. Safety protection provided				
2. Limit public access					17. Pipe joints and connections as designed				
3. Location same as plans					18. Concrete and reinforcement as designed				
4. Site protection from existing flows					19. Inlets appropriately installed				
5. All required permits in place					20. Inlet energy dissipation installed				
Earthworks					21. No seepage through banks				
6. Integrity of banks					22. Ensure spillway is level				
7. Batter slopes as plans					23. Provision of maintenance drain(s)				
8. Impermeable (eg. clay) base installed					24. Collar installed on pipes				
9. Maintenance access for inlet zone					26. Protection of riser from debris				
10. Compaction process as designed					**Vegetation**				
11. Placement of adequate topsoil (edges)					29. Vegetation appropriate to zone (depth)				
12. Levels as designed for base, benches, banks and spillway (including freeboard)					30. Weed removal prior to planting				
					31.Provision for water level control during establishment				
13. Check for groundwater intrusion									
14. Stabilisation with sterile grass					32. Vegetation layout and densities as designed				
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					9. Check for uneven settling of banks				
2. Confirm structural element sizes					10. Evidence of stagnant water or short circuiting				
3. Check batter slopes									
4. Vegetation planting as designed					11. Evidence of litter or excessive debris				
5. Erosion protection measures working					12. Provision of removed sediment drainage area				
6. Pre-treatment installed and operational									
7. Maintenance access provided					13. Outlet free of debris				
8. Public safety adequate									

COMMENTS ON INSPECTION

ACTIONS REQUIRED

1.									
2.									
3.									
4.									
5.									

9.5.4 Asset transfer checklist

Asset Handover Checklist		
Asset Location:		
Construction by:		
Defects and Liability Period		
Treatment	Y	N
System appears to be working as designed visually?		
No obvious signs of under-performance?		
Maintenance	Y	N
Maintenance plans provided for each asset?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
Asset inspected for defects?		
Asset Information	Y	N
Design Assessment Checklist provided?		
As constructed plans provided?		
Copies of all required permits (both construction and operational) submitted?		
Proprietary information provided (if applicable)?		
Digital files (eg drawings, survey, models) provided?		
Asset listed on asset register or database?		

9.6 Maintenance requirements

Pond and lakes treat runoff by providing extended detention and allowing sedimentation to occur. In addition, they have a flow management role that needs to be maintained to ensure adequate flood protection for local properties.

The majority of lake maintenance is associated with the inlet zone (and GPT if installed). Weeding, planting and debris removal are the dominant tasks. In addition, if artificial turnover of the lake is required (because of long residence times) a mechanical system will need to be employed and will require specific maintenance.

Edge vegetation will also require maintenance including weed removal and replanting. Other components of the system that will require careful consideration are the inlet points. Inlets can be prone to scour and build up of litter. Occasional litter removal and potential replanting may be required as part of maintaining an inlet zone.

Maintenance is primarily concerned with:

- Maintenance of flow to and through the system
- Maintaining vegetation
- Removal of accumulated sediments and litter and debris

Similar to other types of stormwater practices, debris removal is an ongoing maintenance function. Debris, if not removed, can block inlets or outlets, and can be unsightly if located in a visible location. Inspection and removal of debris should be done regularly, but debris should be removed whenever it is observed on the site.

Inspections are also recommended following large storm events to check for scour.

9.6.1 Operation & maintenance inspection form

The form below should be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

Pond Maintenance Checklist			
Inspection Frequency: 3 monthly		**Date of Visit:**	
Location:			
Description:			
Site Visit by:			
Inspection Items	**Y**	**N**	**Action Required (details)**
Litter within inlet or open water zones?			
Sediment within inlet zone requires removal (record depth, remove if >50%)?			
Overflow structure integrity satisfactory?			
Evidence of dumping (building waste, oils etc)?			
Terrestrial vegetation condition satisfactory (density, weeds etc)?			
Replanting required?			
Submerged/floating vegetation requires removal/harvesting (if >50%)?			
Settling or erosion of bunds/batters present?			
Damage/vandalism to structures present?			
Outlet structure free of debris?			
Maintenance drain operational (check)?			
Comments:			

9.7 Worked example

9.7.1 Worked example introduction

As part of a residential development in Hobart, a permanent waterbody is proposed to treat runoff from a residential area of 110 Ha (45% catchment imperviousness) and provide landscape amenity as an integral component of the public open space. The residential development is to have a number of stormwater quality improvement measures within the streetscape. A pond is expected to reduce the nitrogen load from the catchment by 10%.

This pond is to be nested within the site of a flood retarding basin. The site for the retarding basin is 4.2 Ha in area and is quadrangle in shape as shown in Figure 9.10. A combination of active and passive open space (urban forestry, pond etc.) functions are to be incorporated into the site.

Stormwater is conveyed by stormwater pipes (up to the 10 year ARI event) and by designated floodways (including roadways) for events larger than the 10 year ARI event. There are four sub-catchments discharging into the retarding basin. During the design 100 year ARI event, the maximum discharge from the retarding basin is 4.1 m^3/s.

9.7.2 Design Considerations

Design considerations include the following:

1. Verifying the size of the pond (depth and area).
2. Computation to ensure that the pond volume is not excessive large in comparison to the hydrology of the catchment.
3. Configuring the layout of the pond such that the system *hydraulic efficiency* can be optimised, including the transition structure between the inlet zone and the open waterbody
4. Design of hydraulic structures
    a. inlet structure to provide for energy dissipation of inflows up to the 100 year ARI peak discharge
    b. Design of the pond outlet structure for the pond and retarding basin.
5. Landscape design
    a. Edge treatment
    b. Recommended plant species and planting density
6. Maintenance provisions

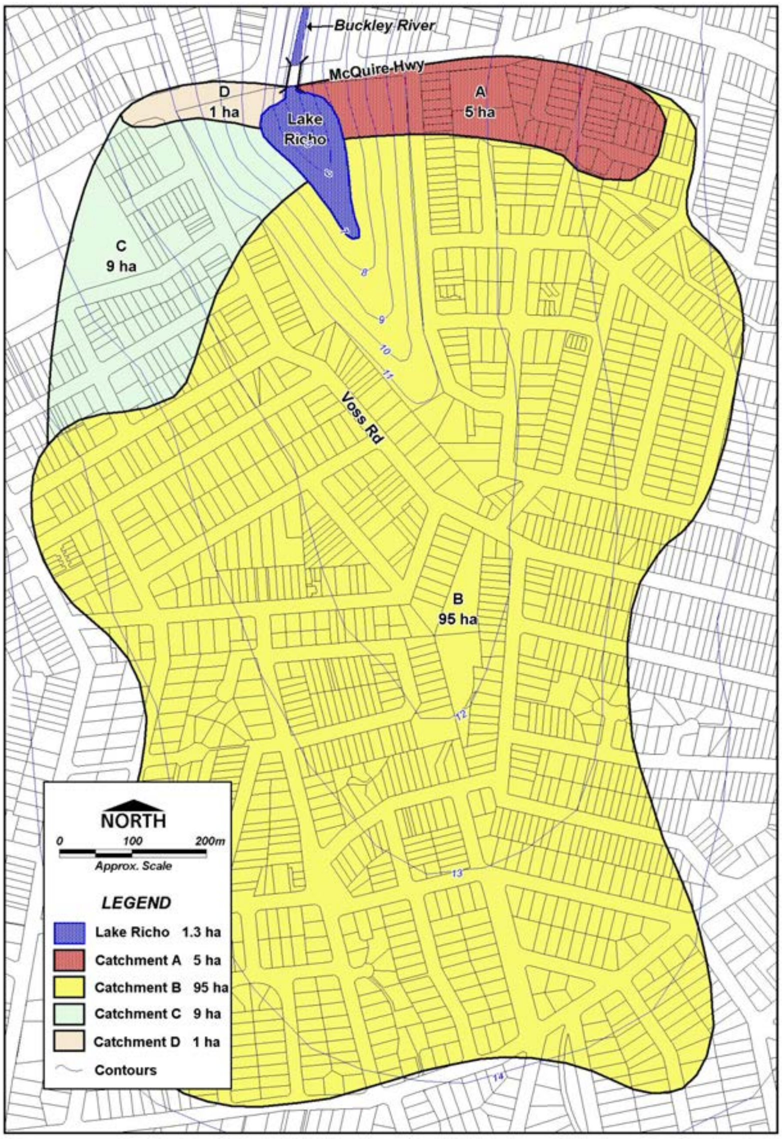


Figure 9.10 Proposed site for Retarding Basin and Pond

9.7.3 Confirming Pond Area

As a basic check of the adequacy of the size of the lake, reference is made to the performance curves presented in 9.3 Verifying size for treatment According to Figure 9.1, the required lake area necessary to reduce TN load by 10% is approximately 0.35% of the impervious area of the catchment.

The required lake area computed from the simple procedure presented in Chapter 3 is as follows:

Catchment area = 110Ha (45% impervious)

Therefore Impervious Area = 50 Ha

Required Lake Area (2.0m mean depth) is:

Impervious Area (m^2) x treatment area required (%) = 500000 x 0.0035

= 1750 m^2

Note: This area should be converted to a site-specific area using the appropriate adjustment factor / hydrologic region relationship.

DESIGN NOTE – The values derived from Figure 9.1 will only be valid if the design criteria for the proposed installation are similar to those used to create the Figures. Site specific modelling using programs such as MUSIC (eWater, 2009) may yield a more accurate result.

The proposed lake area is 3000 m^2, the proposed permanent pool level is 5.5 m-AHD with a maximum depth of 2.5 m and a depth range between 1.5 m and 2.5 m. The volume of the proposed lake waterbody is approximately 6 ML (ie. 0.3 Ha area x 2 m depth). The layout of the proposed waterbody is shown in Figure9.11.

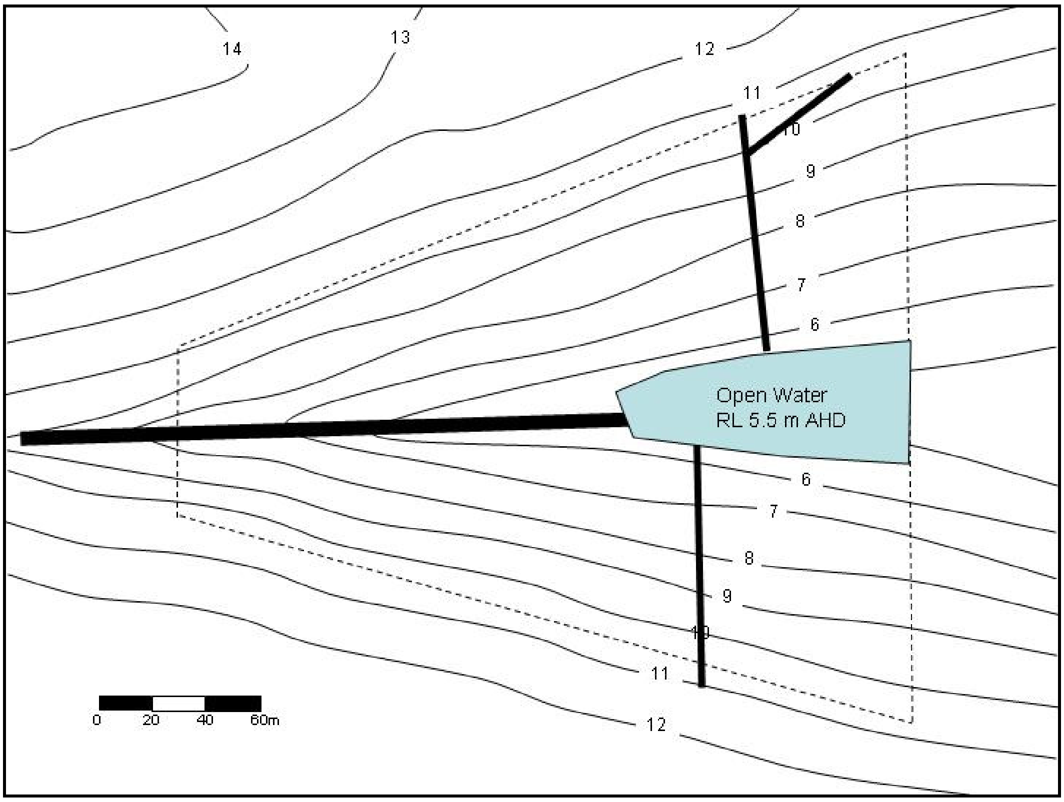


Figure9.11 Layout of Proposed Pond

- Proposed pond area is 3000 m² is confirmed as larger than the expected size required to achieve the 10% reduction in TN proposed
- Permanent Pool Level is set at 5.5 m-AHD
- Lake volume ~ 6 ML

9.7.4 Design calculations

9.7.4.1 Lake Hydrology

Waterbody residence time analysis

Waterbody residence time analysis should be undertaken using a continuous simulation approach with the use of historical rainfall data with historical potential evaporation data or probabilistic monthly potential evaporation estimates (see Section 9.3.1). A simplified approach may be undertaken as a preliminary assessment of the adequacy of waterbody turnover in the first instance. This is outlined below.

The statistics of the monthly rainfall and areal potential evapo-transpiration data are summarised in Table 9-2 below.

**** For the purposes of this worked example, the figures in Table 9.2 are adopted as being the actual data for this area.**

Table 9-2 Meteorological Data

	Jan	Feb	Mar	Apr	May	Jun	
Mean Rainfall (mm)	35.2	33.5	43	65.3	88.9	100.1	
Median Rainfall (mm)	25.9	25.8	36.9	61.9	82.1	96.5	
Decile 9 Rainfall (mm)	74.7	72.6	83.6	110.2	145.6	153.7	
Decile 1 Rainfall (mm)	9.4	5.9	12.1	25.6	36	56.4	
Mean No. of Raindays	8.7	8	11.4	14.7	18.4	19.6	
Monthly Areal PET (mm)	150	120	100	85	40	30	
	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Mean Rainfall (mm)	108.5	107.6	85.1	70.4	53.2	44.7	835.5
Median Rainfall (mm)	102.8	102.6	81.4	67.5	48.9	38.6	834.9
Decile 9 Rainfall (mm)	167.2	165.1	126.6	111.9	93.4	81.8	1001.4
Decile 1 Rainfall (mm)	57	55.5	53.4	29.2	22	13.8	656.6
Mean No. of Raindays	21.1	21.2	18.5	16.2	13	11.3	182.1
Monthly Areal PET (mm)	30	45	70	100	125	135	1000

From the above meteorological data, a simple assessment can be made of the waterbody residence times for the 10%ile, 50%ile and 90%ile summer meteorological conditions. This can be done by computing the ratio of net summer inflow to the pond volume and subsequently dividing the number of days over the summer period (92 days) with this ratio.

	Summer Rainfall (mm)	Net Summer Inflow (ML)[1]	Net Summer Inflow / Lake Volume	Summer Probabilistic Residence Time (days)
10%tile	29.1	13.3	2.2	~41 days
50%tile	90.3	43.6	7.3	~ 13 days
90%tile	229	112	18.7	~5 days

[1]Catchment Inflow (~Rainfall x Impervious Area)-net evaporation (~[Evaporation - Rainfall] x Lake Area)

The 20%tile residence time can be estimated by interpolating between the 10%tile value and the 50%tile value. The interpolation is best undertaken using a log-normal probability plot.

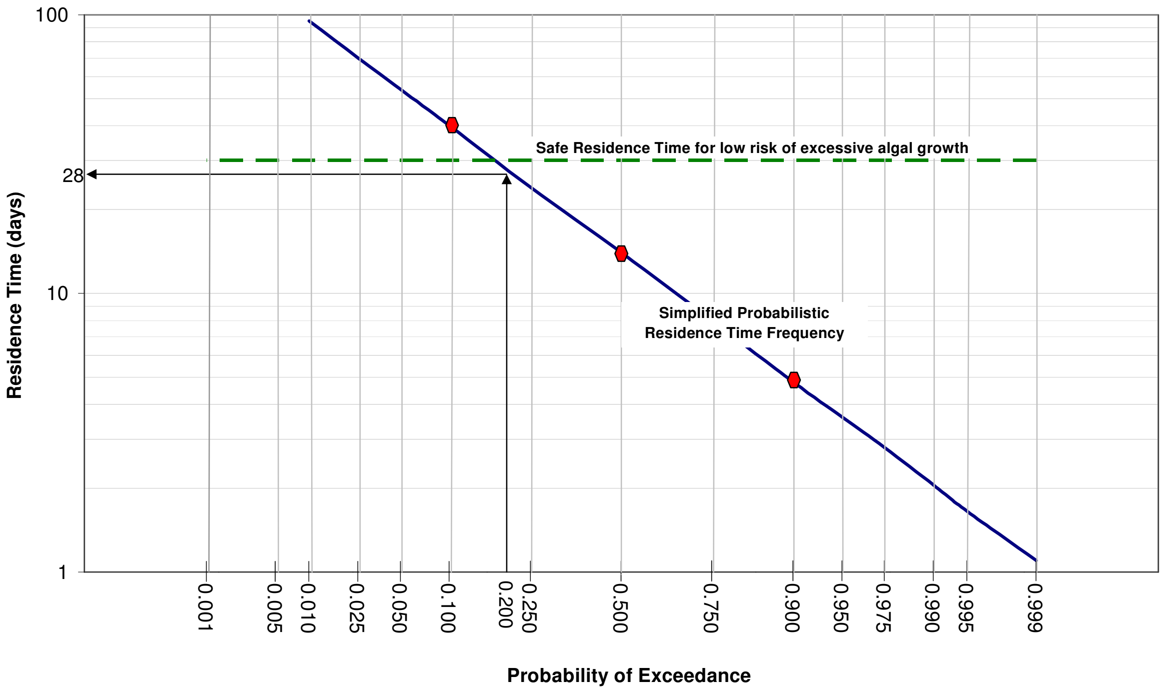


Figure 9.12 Simplified Log-normal probability plot of summer pond probabilistic residence time

The analysis undertaken indicated that the proposed pond has a 20 percentile probabilistic residence time of approximately 28 days. This is just within the guidelines for sustainable ecosystem health of a waterbody of 30 days and it is advisable that a continuous simulation of pond residence time be undertaken to confirm that the pond has a low risk of euthrophication.

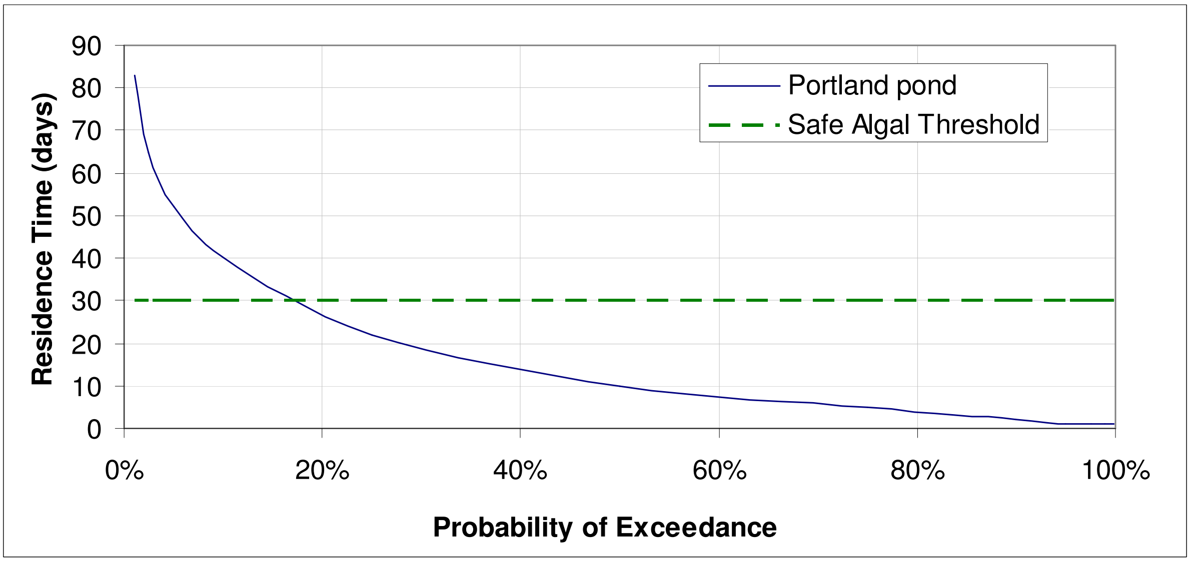


Figure 9.13 Plot of probabilistic residence time determine from continuous simulation using 25 years of rainfall record

❒ There is no significant risk of summer algal bloom with the proposed pond volume.

Probabilistic Summer Water Levels

Water level fluctuation over the summer period is influenced by catchment inflow and evaporation from the lake waterbody. As is the case for the waterbody turnover analysis, a rigorous approach to determination of the probabilistic summer water level fluctuation is through a continuous simulation approach using a daily timestep.

A simplified approach to determine if water level fluctuation is excessive within the water body can be undertaken by examining the 10%tile monthly water balance. Figure 9.14 shows the plot of the 10% catchment inflow to the lake and the average monthly evaporative losses from the lake. The adoption of the average monthly evaporative losses are not expected to significantly under-estimate the evaporative loss corresponding to a 10%tile hydrologic scenario.

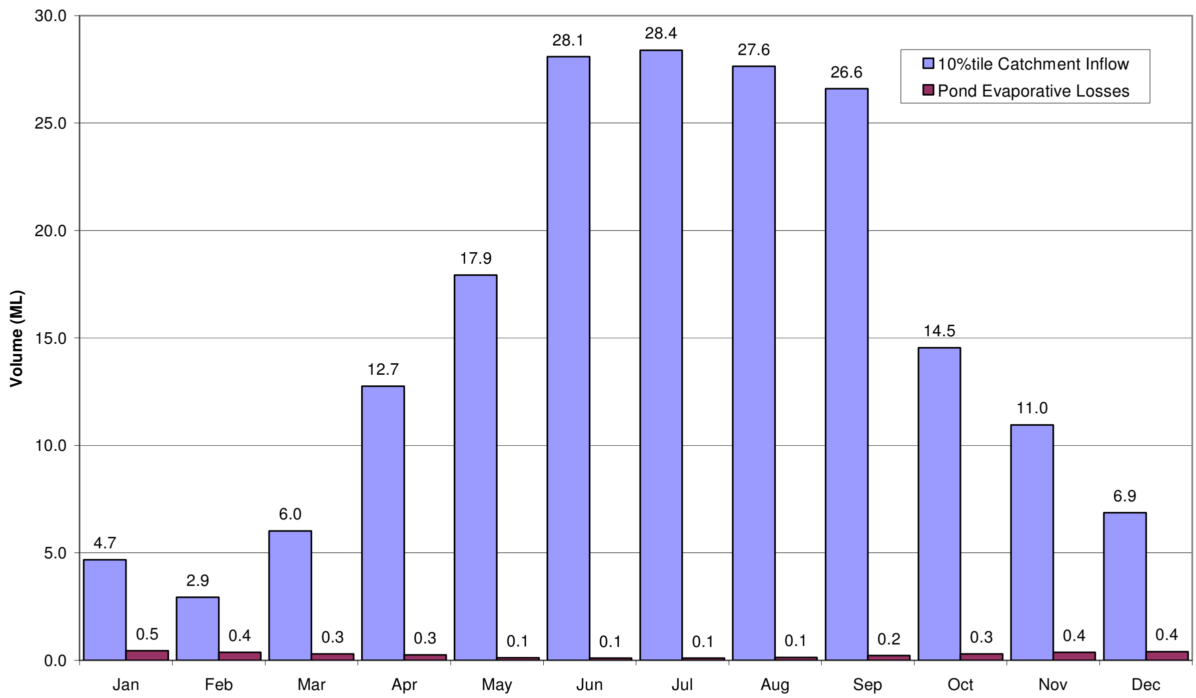


Figure 9.14 Lake water budget (10%ile catchment rainfall)

The analysis shows that monthly catchment inflow exceeds evaporative losses in all months indicating that even for the 10%tile rainfall scenario, the lake level can be expected to be at full level at least on one occasion each month. The maximum fluctuation in water level (corresponding to the January/February period) can be conservatively computed to be the sum of the expected evaporation losses of these two months, e.g. approximately 250 mm.

Lake water level fluctuation is not expected to be a significant Aesthetic issue for the proposed lake.

Estimating Design Flows

Times of concentration have been assessed by assuming pipe and overland flow velocities of 1 m/s and estimating flow paths. In smaller catchments, a minimum time of concentration of 6 minutes has been adopted to allow for lot scale impacts. The characteristics of each catchment are summarised in

Rainfall intensities were estimated using IFD intensities for Portland and are also summarised below.

Table 10.3 Catchment Characteristics, Rainfall Intensities and Design Discharges

Sub Catchment	Area (Ha)	Flow Path Length (m)	t_c (min)	I_1	C_1	Q_1	I_{10}	C_{10}	Q_{10}	I_{100}	C_{100}	Q_{100}
A	5	220	7	34	0.59	0.28	63	0.74	0.65	144	0.88	1.78
B	95	1400	30	17	0.39	1.73	30	0.49	3.84	64	0.59	9.93
C	9	200	7	34	0.59	0.50	63	0.74	1.17	144	0.88	3.20
D	1	150	7	34	0.59	0.06	63	0.74	0.13	144	0.88	0.36

Runoff coefficients for the 1 year, 10 year and 100 year ARI events for the catchments (each with a 0.45 fraction impervious) were calculated in accordance to the procedure in AR&R 1998 (Book 8) and are also summarised.

9.7.4.2 Open Water Zone Layout

Size and Dimensions

The open water zone will be quadrangular in shape to conform to the natural terrain of the site. The general dimension is a mean width of 30 m and 100 m along the long axis, giving an aspect ration of 3(L) to 1(w). With the largest of the catchment discharging into the lake from one end of the longer axis, the expected hydraulic efficiency of the open water body can be up to of the order of 0.34 unless the outlet from sub-catchment B can be designed such that outflow is uniformly distributed across the 20 m wide foreshore of the pond. This can be achieved by designing a vegetated swale transition between the pipe outfall and the forebay of the pond.

*Aspect Ratio is **3(L) to 1(W)**;*

*Hydraulic Efficiency ~**0.76** with distributed inflow*

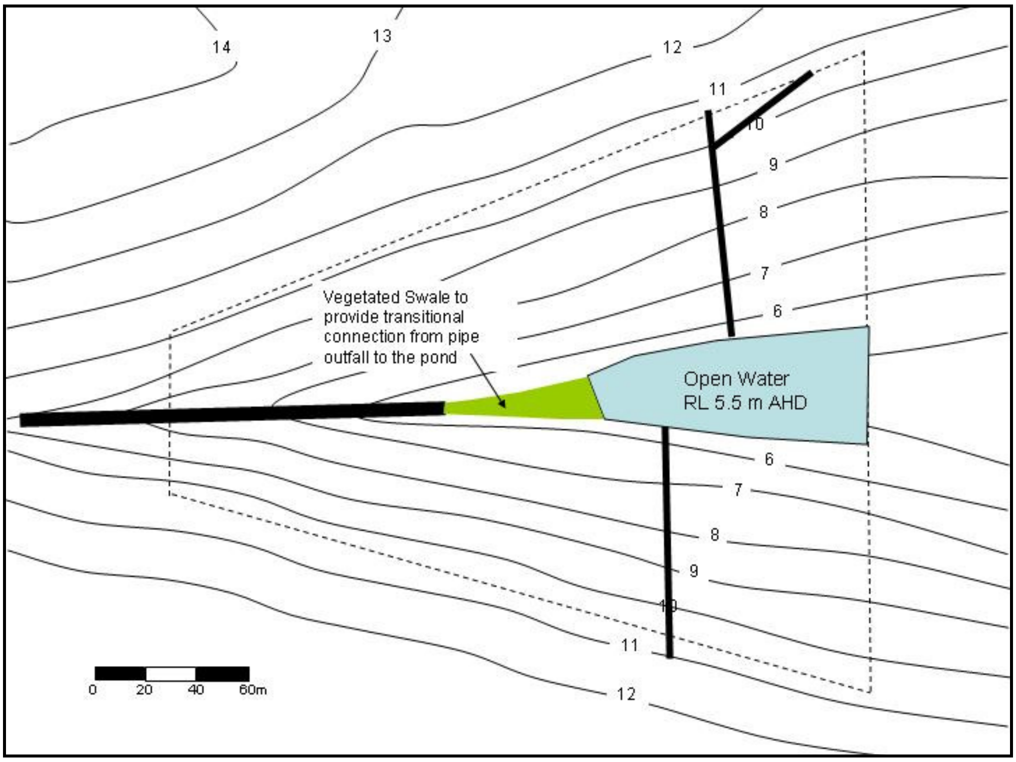
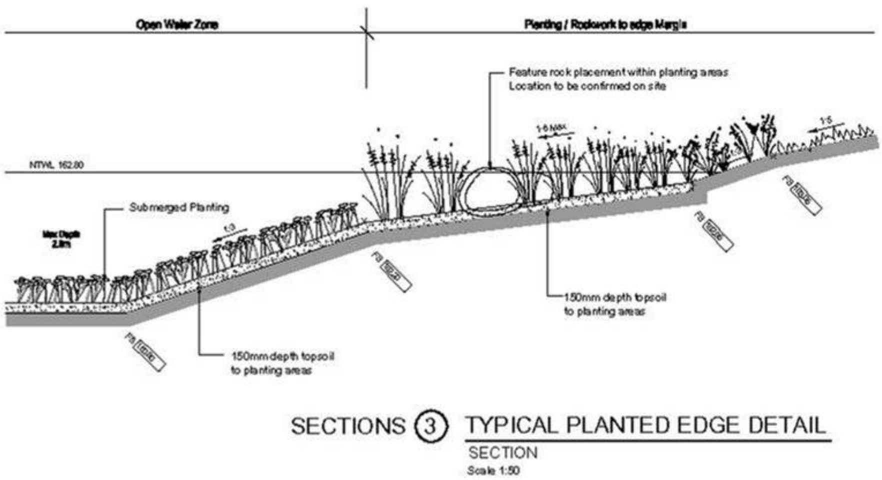


Figure 9.15 Vegetated swale recommended to provide flow transition from pipe outfall to foreshore of pond

Section

The long and cross sections of the pond will follow the natural terrain with limited requirement for earthworks to form the bathymetry of the pond.

The batter slopes on approaches and immediately under the permanent water level have to be configured with consideration of public safety. A batter slope of 1(V):8(H) from the littoral zone to 0.3 m beneath the water line before steepening into a 1(V):3(H) slope is



recommended as a possible design solution (see illustration below).

Cross section of littoral zone to below the water line consists of a 1:8 batter slope to 0.3 m below the permanent pool level

9.7.4.3 Pond Outlet Structure

Maintenance Drain

A maintenance drain will be provided to allow drainage of the system. Valves will be operated manually to drain the permanent waterbody. The drawdown period should be of the order of 24 hours if practical.

The mean flow rate for the maintenance drain is selected to drawdown the permanent pool over 24 hours is computed as follows:

Permanent Pool Volume ~ 6,000 m^3

Q = 6000/(1 x 24 x 3600) = 0.07 m^3/s = 70 L/s

It is reasonable to assume that the valve orifice will operate under inlet control with its discharge characteristics determined by the orifice Equation 9.5, i.e.

$$A_o = \frac{Q}{C_d \sqrt{2gh}}$$

Q = 0.07 m^3/s

C_d = 0.6

H = 0.67 m (two thirds of maximum permanent pool depth)

Giving A_o = 0.02 m^2 corresponding to an orifice diameter of 161 mm

adopt **200 mm maintenance pipe**

*Pipe valve to allow draining of the permanent pool for maintenance to be **at least 200 mm diameter.***

Outlet Pit

The outlet pit is to be set at a crest level at the nominated permanent pool level of 5.5 m AHD. The discharge capacity of the outlet pit must be at least equal but preferably higher than the design retarding basin outflow.

During the 100 year ARI operation of the retarding basin, the outlet pit will be completely submerged and the required dimension of the outlet pit to discharge 4.1 m^3/s can be computed using the orifice flow Equation 9.5, i.e.

$$A_o = \frac{Q_{des}}{BC_d \sqrt{2gH}}$$

B = Blockage factor

C_d = Orifice Discharge Coefficient (0.6)

H = 3.5 m

A_o = Orifice area (m^2)

Q_{des} = 4.1 m^3/s

The computed plan area of the overflow pit is 1.65 m^2. The nominal pit dimension to ensure adequate discharge capacity is 2.0 m x 1.0 m.

Outlet pit dimension is 2.0 m x 1.0 m

9.7.4.4 High-flow route and spillway design

The spillway weir level is set at RL 11.0 m AHD and the retarding basin embankment height is approximately 7 m. It will be necessary to design the spillway with adequate capacity to safely convey peak discharges up to the Probable Maximum Flood (PMF). This requires specialist hydrological engineering input involving flood estimation and flood routing calculations.

The spillway needs to be designed to safely convey discharges up to the Probable Maximum Flood.

9.7.4.5 Vegetation Specifications

The vegetation specification and recommended planting density for the littoral and open water zone are summarised in the table below.

Zone	**Plant Species**	**Planting Density (plants/m²)**
Littoral berm	*Persicaria decipens*	3
Open water zone	*Vallisneria spiralis*	4

The reader is referred to Appendix B for further discussion and guidance on vegetation establishment and maintenance.

9.7.4.6 *Design Calculation Summary*

Ponds and Lakes — **CALCULATION SUMMARY**

	CALCULATION TASK	OUTCOME		CHECK
1	**Identify design criteria**			✓
	Design ARI Flow for inlet hydraulic structures	10	year	
	Design ARI Flow for outlet hydraulic structures	100		
	Design ARI for emergency hydraulic structures	PMF	year	
	80%tile summer turnover period	>>110	days	
	Probabilistic summer water level - 10%tile	7.2	m	
	Probabilistic summer water level - 90%tile	7.5	m	
	Flood Detention Storage Volume (from flood routing analysis)	150000	m^3	
	Outlet pipe dimension (from flood routing analysis)	750	mm	
2	**Catchment characteristics**			✓
	Residential	110	Ha	
	Commercial	0	Ha	
	Fraction impervious			✓
	Residential	0.45		
	Commercial	N/A		
3	**Estimate design flow rates**			
	Time of concentration			
	estimate from flow path length and velocities	7 to 30	minutes	✓
	Identify rainfall intensities			
	station used for IFD data:	Portland		
	Design Rainfall Intensity for inlet structure(s)	30 to 63	mm/hr	✓
	Design runoff coefficient			
	inlet structure(s)	0.49 to 0.74		✓
	Peak design flows			✓
	Inlet structure(s)	0.13 to 3.84	m^3/s	
	Outlet structure(s)	4.100	m^3/s	
4	**Forebay Zone Layout**			✓
	Area of Forebay Zone	15 to 125	m^2	
	Aspect Ratio	2(L):1(W)	L:W	
	Hydraulic Efficiency	0.4		
5	**Lake Residence Time**			✓
	Is wetland forebay for recirculation required	Y		
	Area of wetland forebay for water recirculation	10000	m^2	
	Detention time during reciculation of wetland forebay	5	days	
	Lake water recirculation pump rate	17	L/s	
6	**Pond Layout**			✓
	Area of Open Water	22000	m^2	
	Aspect Ratio	2(L):1(W)	L:W	
	Hydraulic Efficiency	0.76		
	Length	200	m	
	Width	50 to 150	m	
	Cross Section Batter Slope	1(V):8(H)	V:H	
7	**Hydrualic Structures**			
	Inlet Structure			✓
	Provision of energy dissipation	Y		
	Outlet Structure			✓
	Pit dimension	1 x 1	L x B	
			mm diam	
	Discharge capacity of outlet pit	4.1	m^3/s	
	Provision of debris trap	Y		
	Maintenance Drain			✓
	Diameter of Maintenance Valve	200	mm	
	Drainage time	7	days	

9.7.5 Example Maintenance Schedule

The following is an example inspection sheet developed for a lake at Portland showing local adaptation to incorporate specific features and configuration of individual lakes. The following inspection sheet was developed from the generic lake maintenance inspection form.

PORTLAND LAKE – MAINTENANCE FORM		
Location		
Description	Constructed lake and sediment forebay	
SITE VISIT DETAILS		
Site Visit Date:	____________________	
Site Visit By:	____________________	
Weather	____________________	
Purpose of the Site Visit	Tick Box	Complete Sections
Routine Inspection	☐	Section 1 only
Routine Maintenance	☐	Section 1 and 2
Cleanout of Sediment	☐	Section 1, 2 and 3
Annual Inspection	☐	Section 1, 2, 3 and 4
SECTION 1 INSPECTION		
Gross Pollutant Load cleanout required?	Yes/No	
Depth of Sediment in Forebay:	________________	m
Cleanout required if Depth of Sediment ≥ 1.0 m	Yes/No	
Any weeds or litter in wetland (If Yes, complete Section 2 Maintenance)	Yes/No	
Any visible damage to wetland or sediment basin? (If Yes, completed Section 4 – Condition)	Yes/No	
Inspection Comments:		
SECTION 2 MAINTENANCE		
Are there weeds in the wetland forebay and littoral zone?	Yes/No	

Were the weeds removed this site visit?	Yes/No	
Is there litter in the lake or forebay?	Yes/No	
Was the litter collected this site visit?	Yes/No	
SECTION 3a CLEANOUT OF GROSS POLLUTANTS		
Have the following been notified of cleanout date?	Yes	No
Coordinator – open space and/or drainage	☐	☐
Local Residents	☐	☐
Other (specify ....)	☐	☐
Method of Cleaning (excavator or eductor)		
Volume of Gross Pollutant and Sediment Removed (approximate estimate)	m^3	
Any visible damage to gross pollutant trap? (If yes, complete Section 4 Condition)	Yes/No	
SECTION 3b CLEANOUT OF SEDIMENT		
Have the following been notified of cleanout date?	Yes	No
Coordinator – open space and/or drainage	☐	☐
Local Residents	☐	☐
Other (specify ....)	☐	☐
Method of Cleaning (excavator or eductor)		
Volume of Sediment Removed (approximate estimate)	m^3	
Any visible damage to wetland or sediment forebay? (If yes, complete Section 4 Condition)	Yes/No	

SECTION 4 CONDITION					
Component	Checked?		Condition OK?		Remarks
	Yes	No	Yes	No	
Inlet structures					
Outlet structures					
Sediment forebay					
Spillway and spillway channel					
Forebay and littoral zone vegetation					
Banks and batter slopes					
Forebay bunds or porous embankment (if constructed)					
Retarding Basin embankment					
Surrounding landscaping					
Comments:					

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Chapter 10 Infiltration Measures

Definition:

A sub-surface water filtration system designed to allow water to infiltrate into surrounding soils.

Purpose:

- To encourage stormwater to infiltrate into surrounding soils.
- To reduce runoff as well as provide pollutant retention on site.
- To provide some detention and retention functionality

Implementation considerations:

- They are highly dependant on local soil characteristics and are best suited to sandy soils with deep groundwater.
- All infiltration measures require significant pretreatment of stormwater before infiltration to avoid clogging of the surrounding soils and to protect groundwater quality.
- Generally these measures are well suited to highly permeable soils, so that water can infiltrate at a sufficient rate. Areas with lower permeability soils may still be applicable, but larger areas for infiltration and detention storage volumes are required.
- Infiltration measures are required to have sufficient set-back distances from structures to avoid any structural damage, these distances depend on local soil conditions.

Infiltration measures can also be vegetated and provide some landscape amenity to an area. These systems provide improved pollutant removal through active plant growth improving filtration and ensuring the soil does not become 'clogged' with fine sediments.

Infiltration systems are best suited to sandy soils with deep groundwater

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10.2 Verifying size for treatment .... 10—4

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10.1 Introduction

Stormwater infiltration systems are designed to encourage stormwater to infiltrate into surrounding soils via a controlled system and are particularly suited to reducing the magnitude of peak storm discharges from impervious areas (Figure 10.1 illustrates an infiltration system during operation).

This chapter outlines the engineering design of such systems following the selection of a required detention storage volume associated with infiltration. Australian Run-off Quality (Engineers Australia, 2003) provides a detailed discussion of procedures for sizing stormwater infiltration systems.

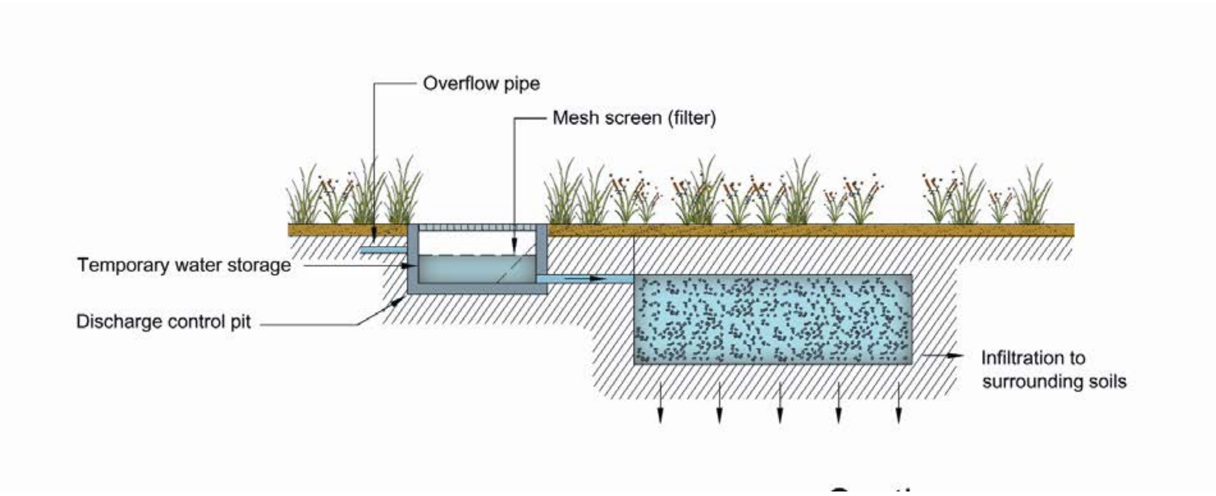


Figure 10.1 Operation of a gravel filled 'soak-away' pit style infiltration system

Not all areas are suited to infiltration systems and the following criteria should be taken into consideration when designing these systems.

- These systems are highly dependent on local soil characteristics and are best suited to sandy soils with deep groundwater.
- All infiltration measures require significant pretreatment of stormwater before infiltration to avoid clogging of the surrounding soils and to protect groundwater quality. Pre-treatment to remove sediments is a vital component in the treatment train and infiltration systems should be positioned as the final element of a treatment train, with its primary function being the discharge of treated stormwater into the surrounding soils and groundwater system.
- Careful consideration of the type of runoff source area from which the runoff originates is important to ensure the continued effective operation of these schemes. Poor consideration of catchment pollutant types and characteristics and site conditions is often the main cause for deteriorating infiltration effectiveness over time because of clogging and lack of appropriate maintenance.

Soils with low hydraulic conductivities do not necessarily preclude the use of infiltration systems even though the required infiltration/storage area may become unfeasible.

Additionally, these soils are likely to render them more susceptible to clogging and require enhanced pretreatment.

Low infiltration rates also lead to the detention of water for long periods of time, which may also promote algal growth that increases the risk of clogging of the infiltration media. It is therefore recommended that soil saturated hydraulic conductivities exceeding 1 x 10^{-5} m/s (36 mm/hr) are most suited for infiltration systems.

Other key factors influencing the operation of an infiltration system are the relationship between infiltration rate, the volume of runoff discharged into the infiltration system, depth to groundwater or bedrock and the available detention storage, i.e.

- The infiltration rate is a product of the infiltration area and the hydraulic conductivity of the in-situ soil, i.e. $Q_{inf} = A \times K_h$ (m^3/s). It follows that different combinations of infiltration area and hydraulic conductivity can produce the same infiltration rate.
- The volume of runoff discharged into an infiltration system is a reflection of the catchment area of the system and the meteorological characteristics of the catchment.
- The detention storage provides temporary storage of inflow to optimise the volume of runoff that can be infiltrated.

The *Hydrologic Effectiveness* of an infiltration system defines the proportion of the mean annual runoff volume that infiltrates. For a given catchment area and meteorological condition, the hydrologic effectiveness of an infiltration system is determined by the combined effect of the soil hydraulic conductivity, infiltration area and available detention storage.

As outlined in Australian Runoff Quality (Engineers Australia, 2006), there are four basic types of detention storages used for promoting infiltration, these being:

- Single size gravel or crushed concrete trenches
- Upstand slotted pipes forming “leaky wells”
- “milk-crate” type trenches or “soakaways”
- Infiltration basins.

10.2 Verifying size for treatment

Figure 10.2 shows relationships between the hydrologic effectiveness, infiltration area and detention storage for a range of soil hydraulic conductivities using Hobart meteorological conditions. These charts can be used to verify the selected size of a proposed infiltration system.

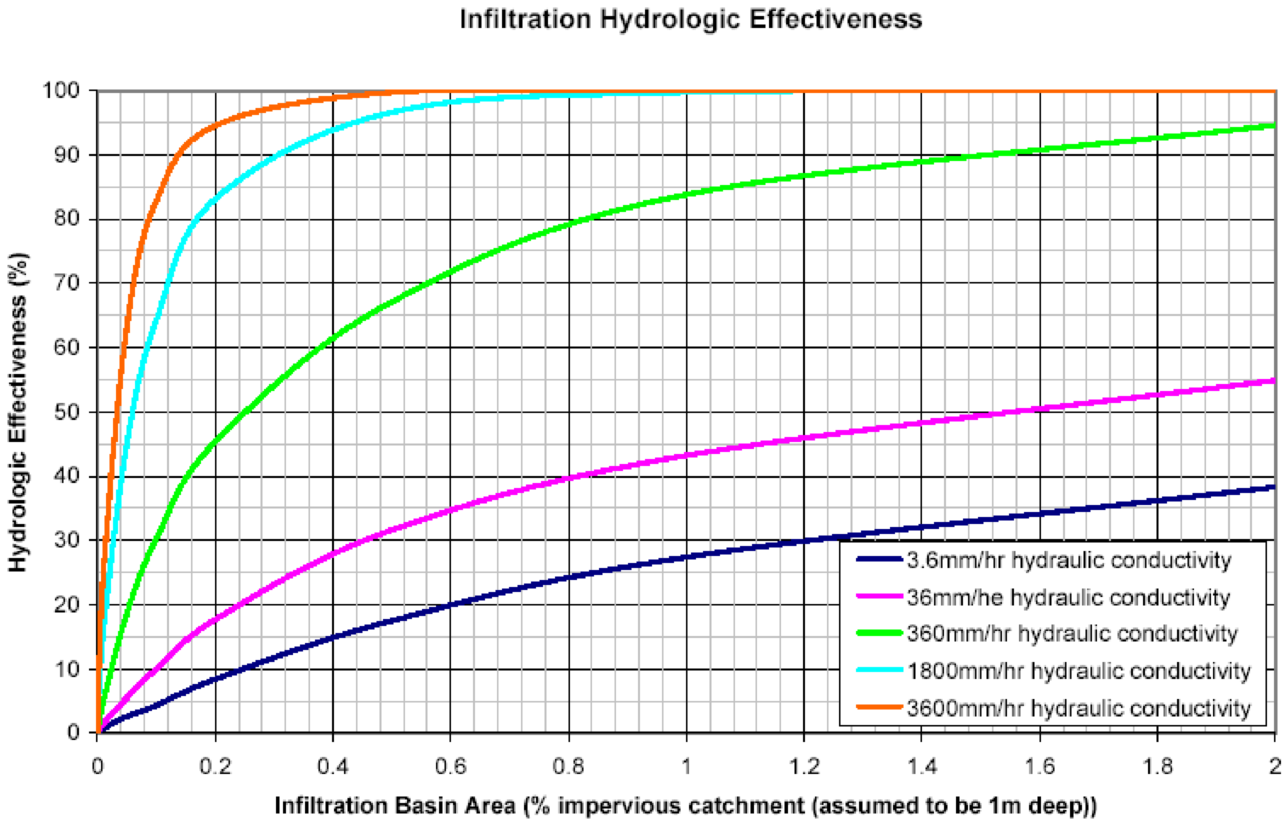


Figure 10.2 Hydrologic Effectiveness of Detention Storages for Infiltration Systems at the reference site

10.3 Design procedure: infiltration measures

10.3.1 Checking Field Conditions

Key factors influencing a site's capability to infiltrate stormwater are the soil permeability, soil reactivity to frequent wetting, presence of groundwater and its environmental values and site terrain.

10.3.1.1 Site terrain and soil salinity

A combination of poor soil conditions (e.g. sodic and dispersive soils), steep terrain and shallow saline groundwater can render the use of infiltration systems inappropriate. Dryland salinity is caused by a combination of factors, including leaching of infiltrated water and salt at "break-of-slope" terrain and the tunnel erosion of dispersive soils. Soil with high sodicity is generally not considered to be suited for infiltration as a means of managing urban stormwater.

Infiltration into steep terrain can result in the stormwater re-emerging onto the surface at some point downstream. The likelihood of this pathway for infiltrated water is dependent on the soil structure, with duplex soils and shallow soil over rock being situations where re-emergence of infiltrated water to the surface is most likely to occur. This occurrence does not necessarily preclude infiltrating stormwater, unless leaching of soil salt is associated with this process. The provision for managing this pathway will need to be taken into consideration at the design stage.

10.3.1.2 Hydraulic Conductivity

It is essential that field hydraulic conductivity tests be undertaken to confirm assumptions of soil hydraulic conductivity adopted during the concept design stage. Field soil hydraulic conductivity can be determined using the falling head augerhole method of Jonasson (1984). The range of soil hydraulic conductivities typically determined from a 60-minute falling head period is as follows:

Sandy soil: K_{60} = 5×10^{-5} m/s (180 mm/hr)

Sandy clay: K_{60} = between 1×10^{-5} and 5×10^{-5} m/s (36 to 180 mm/hr)

Medium clay: K_{60} = between 1×10^{-6} and 1×10^{-5} m/s (3.6 to 36 mm/hr)

Heavy clay: K_{60} = between 1×10^{-8} and 1×10^{-6} m/s (0.036 to 3.6 mm/hr)

where k_{60} is the 60-minute value of hydraulic conductivity.

Saturated hydraulic conductivity (K_{sat}) is the hydraulic conductivity of a soil when it is fully saturated. K_{60} is considered to be a reasonable estimate of K_{sat} for design purposes and can be measured in the field.

Soil is inherently non-homogeneous and field tests can often misrepresent the areal hydraulic conductivity of a soil into which stormwater is to be infiltrated. Field experience has suggested that field tests of "point" soil hydraulic conductivity can often under-estimate the areal hydraulic conductivity of clay soils and over-estimate in sandy soils. To this end, Australian Runoff Quality (Engineers Australia, 2006) recommends that moderation factors for hydraulic conductivities determined from field test be applied as shown in Table 11.1.

Table 10-1 Moderation factors to convert point to areal conductivities (after Engineers Australia, 2003)

Soil Type	**Moderation Factor (U)** **(to convert "point" K_h to areal K_h)**
Sandy soil	0.5
Sandy clay	1.0
Medium and Heavy Clay	2.0

10.3.1.3 Groundwater

Two groundwater issues need to be considered when implementing an infiltration system. The first relates to the environmental values of the groundwater (i.e. the receiving water) and it may be necessary to achieve a prescribed water quality level before stormwater can be discharged into them. A second design consideration is to ensure that the base of an infiltration system is always above the groundwater table and consideration of the seasonal variation of groundwater levels is essential if a shallow groundwater table is likely to be encountered. This investigation should include groundwater mounding (i.e. higher levels in the immediate vicinity of the infiltration system) that in shallow groundwater areas could cause problems with nearby structures.

10.3.2 Estimating design flows

10.3.2.1 Design Discharges

Two design flows are required for infiltration systems:

- Peak inflow to the infiltration system for design of an inlet structure.
- Major flood rates for design of a by-pass system.

Infiltration systems can be subjected to a range of performance criteria including that of peak discharge attenuation and volumetric runoff reduction.

Design discharge for the by-pass system is often set at the 100-year ARI event or the discharge capacity of the stormwater conveyance system directing stormwater runoff to the infiltration system. Consultation with the relevant local authority is important to determine their criteria or requirements for the discharge design rainfall ARI.

10.3.2.2 Minor and major flood estimation

A range of hydrologic methods can be applied to estimate design flows. With typical catchment areas discharging to infiltration measures being relatively small, the Rational Method Design Procedure is considered to be a suitable method for estimating design flows.

Figure 10.3 shows an assumed shape of an inflow hydrograph that can be used to estimate the temporary storage volume for an infiltration system. The flow rate shown on the diagram represents a linear increase in flow from the commencement of runoff to the time of concentration, then this peak flow rate is maintained for the storm duration. Following the storm duration the flow rate decreases linearly over the time of concentration. This is a simplification of an urban hydrograph for the purposes of design.

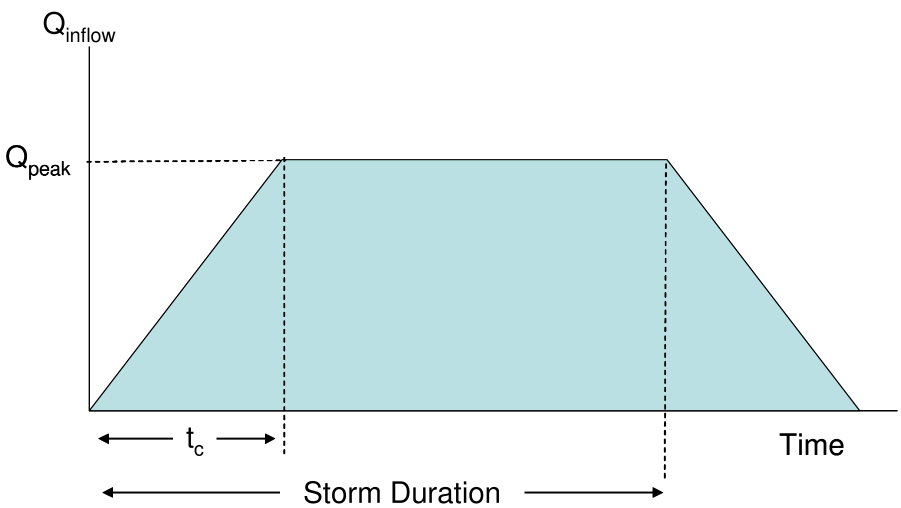


Figure 10.3 Generalised shape of inflow hydrograph

10.3.3 Location of Infiltration Systems

Infiltration systems should not be placed near building footings to remove the influence of continually wet subsurface or greatly varying soil moisture contents on the structural integrity of these structures. Australian Runoff Quality (Engineers Australia, 2006) recommends minimum distances from structures in Table 10.2 (and property boundaries to protect possible future buildings in neighbouring properties) for different soil types.

Table 10-2 Minimum set-back distances (adapted from Engineers Australia, 2006)

Soil Type	Saturated Hydraulic Conductivity	Minimum distance from structures and property boundaries
Sand	> 5×10^{-5} m/s (180 mm/hr)	1.0 m
Sandy Clay	1×10^{-5} to 5×10^{-5} m/s (36 to 180 mm/hr)	2.0 m
Weathered or Fractured Rock	1×10^{-6} to 1×10^{-5} m/s (3.6 to 36 mm/hr)	2.0 m
Medium Clay	1×10^{-6} to 1×10^{-5} m/s (3.6 to 36 mm/hr)	4.0 m
Heavy Clay	1×10^{-8} to 1×10^{-6} m/s (0.036 to 3.6 mm/hr)	5.0 m

Identifying suitable sites for infiltration systems should also include avoidance of steep terrain and areas of shallow soils overlying largely impervious rock (non-sedimentary rock and some sedimentary rock such as shale). An understanding of the seasonal variation of the groundwater table is also an essential element in the design of these systems.

10.3.4 Source Treatment

Treatment of source water for the removal of debris and sediment is essential and storm runoff should never be conveyed directly into an infiltration system. Pre-treatment measures include the provision of leaf and roof litter guards along the roof gutter, sediment sumps, vegetated swales, bioretention systems or sand filters.

10.3.5 Sizing the detention storage

10.3.5.1 Storage Volume

The required storage volume of an infiltration system is defined by the difference in inflow and outflow volumes for the duration of a storm. The inflow volume is a product of rainfall, contributing area and the runoff coefficient connected to the infiltration system, i.e.

$$\text{Inflow volume (for storm duration D, m}^3\text{)} = C \times I \times A \times D/1000$$

Equation 10.1

where C is the runoff coefficient as defined in ARR Book VIII

I is the probabilistic rainfall intensity (mm/hr)

A is the contributing area connected to the infiltration system (m^2)

D is the storm duration (hours)

Outflow from the infiltration system is via the base and sides of the infiltration system and depends on the area and depth of the infiltration system. In computing the infiltration from the walls of an infiltration system, Australian Runoff Quality (Engineers Australia, 2006) suggests that pressure is hydrostatically distributed and thus equal to half the depth of water over the bed of the infiltration system, i.e.

$$\text{Outflow volume (for storm duration D, } m^3) = [(A_{inf}) + (P \times d/2)] \times U \times K_h \times D/1000$$

where K_h is the "point" saturated hydraulic conductivity (mm/hr)

A_{inf} is the infiltration area (m^2)

P is the perimeter length of the infiltration area (m)

d is the depth of the infiltration system (m)

U is the "point" soil hydraulic conductivity moderating factor (see Table 11.1)

D is the storm duration (hours)

Approximations of the required storage volumes of an infiltration system can be computed as follows:

$$\text{Required Storage } (m^3) = \{(C \times I \times A) - [(A_{inf}) + (P \times d/2)] \times U \times K_h\}\ D/1000$$

Equation 10.2

Computation of the required storage will need to be carried out for the full range of probabilistic storm durations, ranging from 6 minutes to 72 hours. The critical storm event is the one which results in the highest required storage. A spreadsheet application is the most convenient way of doing this.

10.3.5.2 Emptying Time

Emptying time is defined as the time taken to fully empty a detention storage associated with an infiltration system following the cessation of rainfall. This is an important design consideration as the computation procedure associated with Figure 10.3. assumes that the storage is empty prior to the commencement of the design storm event.

Australian Runoff Quality (Engineers Australia, 2006) suggests an emptying time of the detention storage of infiltration systems to vary from 12 hours to 84 hours, depending on the average recurrence interval of the design event with the former being more appropriate for frequent events (1 in 3 month ARI) and the latter to less frequent events of 50 years or longer ARI.

Emptying time is computed simply as the ratio of the volume of water in temporary storage (dimension of storage x porosity) to the infiltration rate (hydraulic conductivity x infiltration area).

10.3.6 Hydraulic Structures

Two checks of details of the inlet hydraulic structure are required for infiltration systems, i.e. provision of energy dissipation and by-pass of above design discharges. By-pass can be achieved in a number of ways, most commonly a surcharge pit, an overflow pit or discharge into an overflow pipe connected to a drainage system. Details of designing a surcharge pit are described in Chapter 4, 5 and 7.

10.3.7 Design calculation summary

Overleaf is a design calculation summary sheet for the key design elements of an infiltration system to aid the design process.

Infiltration System — CALCULATION SUMMARY

	CALCULATION TASK	OUTCOME		CHECK
1	**Identify design criteria**			☐
	Design ARI event to be infiltrated (in its entirety)		year	
	OR			
	Design Hydrologic Effectiveness		%	
	ARI of Bypass Discharge		year	
2	**Site characteristics**			☐
	Catchment Area connected to infiltration system		m^2	
	Impervious Area connected to infiltration system		m^2	
	Site hydraulic conductivity		mm/hr	
	Areal hydraulic conductivity moderating factor			
3	**Estimate design flow rates**			
	Time of concentration			
	estimate from flow path length and velocities		minutes	☐
	Identify rainfall intensities			
	station used for IFD data:			
	Design Rainfall Intensity for inlet structure(s)		mm/hr	
	Design Rainfall Intensity for overflow structure(s)		mm/hr	☐
	Design runoff coefficient			
	inlet structure(s)			☐
	Peak design flows			☐
	Inlet structure(s)		m^3/s	
	Bypass structure(s)		m^3/s	
4	**Detention Storage**			☐
	Volume of detention storage		m3	
	Dimensions		L:W	
	Depth		m	
	Emptying Time		hrs	
5	**Provision of Pre-treatment**			☐
	Receiving groundwater quality determined			
	Upstream pre-treatment provision			
6	**Hydrualic Structures**			
	Inlet Structure			☐
	Provision of energy dissipation			
	Bypass Structure			☐
	Weir length		m	
	Afflux at design discharge		m	
	Provision of scour protection			
	Discharge Pipe			☐
	Capacity of Discharge Pipe		m^3/s	

10.4 Checking tools

This section provides a number of checking aids for designers and referral authorities. In addition, advice on construction techniques and lessons learnt from building infiltration systems are provided.

Checklists are provided for:

- Design assessments
- Construction (during and post)
- Operation and maintenance inspections
- Asset transfer (following defects period).

10.4.1 Design assessment checklist

The checklist below presents the key design features that should be reviewed when assessing the design of an infiltration system. These considerations include configuration, safety, maintenance and operational issues that should be addressed during the design phase.

Where an item results in an "N" when reviewing the design, referral should be made back to the design procedure to determine the impact of the omission or error.

In addition to the checklist, a proposed design should have all necessary permits for its installation. The referral agency should ensure that all relevant permits are in place. These can include permits to clear vegetation, to dredge, create a waterbody, divert flows or disturb fish or platypus habitat.

Land ownership and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the asset transfer checklist.

<table>
<tr><th colspan="6">Infiltration Design Assessment Checklist</th></tr>
<tr><td>Bioretention location:</td><td colspan="5"></td></tr>
<tr><td>Hydraulics</td><td colspan="2">Minor Flood: (m^3/s)</td><td colspan="3">Major Flood: (m^3/s)</td></tr>
<tr><td>Area</td><td>Catchment Area (ha):</td><td></td><td>Infiltration Area (ha)</td><td></td><td></td></tr>
<tr><td colspan="4">Treatment</td><td>Y</td><td>N</td></tr>
<tr><td colspan="4">Pretreatment system sufficient to protect groundwater?</td><td></td><td></td></tr>
<tr><td colspan="4">Infiltration storage volume verified from curves?</td><td></td><td></td></tr>
<tr><td colspan="4"></td><td></td><td></td></tr>
<tr><td colspan="4">Inlet zone/hydraulics</td><td>Y</td><td>N</td></tr>
<tr><td colspan="4">Station selected for IFD appropriate for location?</td><td></td><td></td></tr>
<tr><td colspan="4">Overall flow conveyance system sufficient for design flood event?</td><td></td><td></td></tr>
<tr><td colspan="4">Velocities at inlet and within infiltration system will not cause scour?</td><td></td><td></td></tr>
<tr><td colspan="4">Bypass sufficient for conveyance of design flood event?</td><td></td><td></td></tr>
<tr><td colspan="6"></td></tr>
<tr><td colspan="4">Basin</td><td>Y</td><td>N</td></tr>
<tr><td colspan="4">Maximum ponding depth will not impact on public safety?</td><td></td><td></td></tr>
<tr><td colspan="4">Maintenance access provided to base of infiltration (where reach to any part of a basin >6m)?</td><td></td><td></td></tr>
<tr><td colspan="6"></td></tr>
</table>

10.4.2 Construction advice

This section provides general advice for the construction of infiltration systems. It is based on observations from construction projects around Australia.

Building phase damage

Protection of infiltration media and vegetation is critical during building phase, uncontrolled building site runoff is likely to cause excessive sedimentation, introduce litter and require replacement of media.

Traffic and deliveries

Ensure traffic and deliveries do not access infiltration areas during construction. Traffic can compact the filter media, cause preferential flow paths and clogging of the surface, deliveries and wash down material can also clog filtration media. Infiltration areas should be fenced off during building phase and controls implemented to avoid washdown wastes.

Timing for engagement

It is critical to ensure that the pretreatment system for an infiltration device is fully operational before flows are introduced into the infiltration media. This will prolong the life of the infiltration system and reduce the risk of clogging.

Inspection wells

It is good design practice to install inspection wells at numerous locations in an infiltration system. This allows water levels to be monitored during and after storm events and infiltration rates can be confirmed over time.

Clean drainage media

Ensure drainage media is washed prior to placement to remove fines and prevent clogging.

10.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Infiltration measures

INSPECTED BY:
DATE:
TIME:
WEATHER:
CONTACT DURING VISIT:

SITE: ______________________

CONSTRUCTED BY: ______________________

DURING CONSTRUCTION									
Items inspected	Checked		Satisfactory	Unsatisfactory		Checked		Satisfactory	Unsatisfactory
Preliminary works	Y	N			**Structural components**	Y	N		
1. Erosion and sediment control plan adopted					10. Location and levels of overflow points as designed				
2. Traffic control measures					11. Pipe joints and connections as designed				
3. Location same as plans					12. Concrete and reinforcement as designed				
4. Site protection from existing flows					13. Inlets appropriately installed				
Earthworks					14. Observation wells appropriately installed				
5. Excavation as designed					**Infiltration system**				
6. Side slopes are stable					15. Correct filter media used				
Pre-treatment					16. Fines removed from filter media				
7. Maintenance access provided					17. Inlet and outlet as designed				
8. Invert levels as designed									
9. Ability to freely drain									
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of surface				
2. Traffic control in place					7. No surface clogging				
3. Confirm structural element sizes					8. Maintenance access provided				
4. Filter media as specified					9. Construction generated sediment and debris removed				
5. Confirm pre-treatment is working									

COMMENTS ON INSPECTION

ACTIONS REQUIRED

1.
2.
3.
4.
5.
6.

Inspection officer signature: ______________________

10.4.4 Asset transfer checklist

Asset Handover Checklist		
Asset Location:		
Construction by:		
Defects and Liability Period		
Treatment	**Y**	**N**
System appears to be working as designed visually?		
No obvious signs of under-performance?		
Maintenance	**Y**	**N**
Maintenance plans provided for each asset?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
Asset inspected for defects?		
Asset Information	**Y**	**N**
Design Assessment Checklist provided?		
As constructed plans provided?		
Copies of all required permits (both construction and operational) submitted?		
Proprietary information provided (if applicable)?		
Digital files (eg drawings, survey, models) provided?		
Asset listed on asset register or database?		

10.5 Maintenance requirements

Maintenance for infiltration systems is focused on ensuring the system does not clog with sediments and that an appropriate infiltration rate is maintained. The most important consideration during maintenance is to ensure the pretreatment is operating as designed.

In addition to checking and maintaining the pretreatment, the form below can be used during routine maintenance inspections:

Infiltration Maintenance Checklist			
Inspection Frequency: 3 monthly	Date of Visit:		
Location:			
Description:			
Site Visit by:			
Inspection Items	**Y**	**N**	**Action Required (details)**
Sediment accumulation in pretreatment zone requires removal?			
Erosion at inlet or other key structures?			
Evidence of dumping (eg building waste)?			
Evidence of extended ponding times (eg. algal growth)?			
Weeds present within device?			
Clogging of drainage points (sediment or debris)?			
Damage/vandalism to structures present?			
Surface clogging visible?			
Drainage system inspected?			
Resetting of system required?			
Comments:			

10.6Infiltration system worked example

10.6.1 Introduction

An infiltration system is to be installed to treat stormwater runoff from a residential allotment in Venus Bay. As discussed in Australian Runoff Quality (Engineers Australia, 2003), pre-treatment of stormwater prior to discharge into the ground via infiltration is essential to ensure sustainable operation of the infiltration system and protection of groundwater. Suspended solids and sediment are the key water quality constituents requiring pre-treatment prior to infiltration.

Roof run-off is directed into a rainwater tank for storage and to be used as an alternative source of water. Overflow from the rainwater tank can be discharged directly into the gravel trench for infiltration into the surrounding sandy soil without further "pre-treatment". Stormwater runoff from paved areas will be directed to a pre-treatment vegetated swale and then into a gravel trench for temporary storage and infiltration. An illustration of the proposed allotment stormwater management scheme is shown in Figure 10.4.

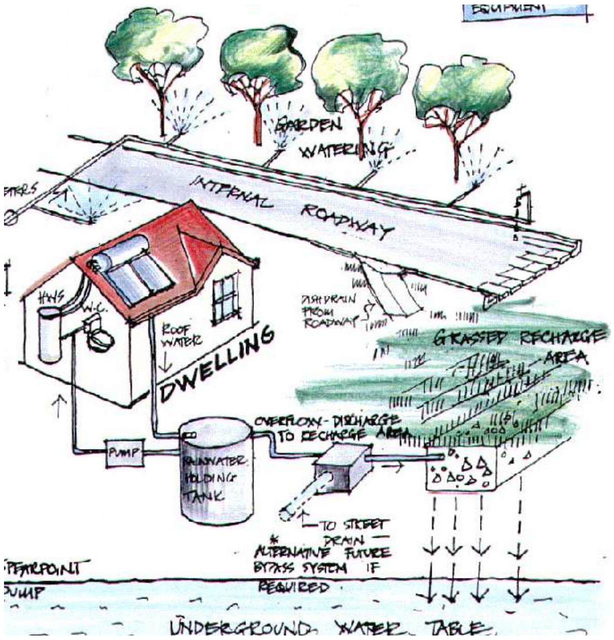


Figure 10.4 Illustration of Allotment Stormwater Management Scheme

[source: Urban Water Resource Centre, University of South Australia; http://www.unisa.edu.au/uwrc/ham.htm]

The allotment in question in this worked example is 1000 m^2 in area on a rectangular site with an overall impervious surface area of 500 m^2. The site layout is shown in Figure 10.5.

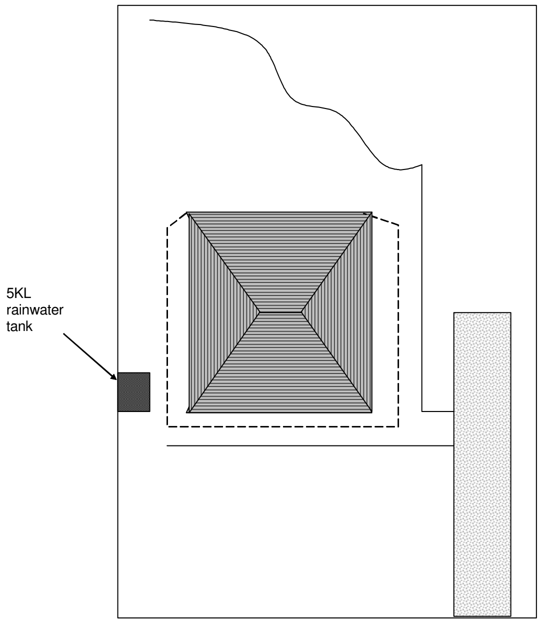


Figure 10.5 Site Layout

Of the impervious surfaces, roof areas make up a total of 210 m^2, while on-ground impervious surfaces make up the remaining 290 m^2. There is no formal stormwater drainage system, with stormwater runoff discharging into a small table drain in the front of the property.

The design objective of the infiltration system is retention of stormwater runoff from the allotment for events up to, and including, the 2-year ARI event. Stormwater flows in excess of the 2-year ARI peak discharges are directed towards the road table drain at the front of the property.

Roof runoff is directed to a 5kL rainwater tank. In this worked example, the design of the infiltration system involves an assumption that the 5kL tank will be full in the event of a 2-year storm event.

The design criteria for the infiltration system are to:

- Provide pre-treatment of stormwater runoff.
- Determine an appropriate size of infiltration system.
- Ensure that the inlet configuration to the infiltration system includes provision for by-pass of stormwater when the infiltration system is operating at its full capacity.

This worked example focuses on the design of the infiltration system and associated hydraulic structures. Analyses to be undertaken during the detailed design phase of the infiltration trench will be based on the procedure outlined in Australian Runoff Quality (Chapter 10 – Infiltration Systems).

10.6.1.1 Design Objectives

The design objectives are summarised as follows:

- Size infiltration trench to retain the entire runoff volume from the critical (volume) 2 year ARI storm event.
- Design the inlet and outlet structures to convey the peak 2-year ARI flow from the critical (flow rate) storm event. Ensure the inlet configuration includes provision for stormwater bypass when the infiltration system is full.
- Configure the layout of the infiltration trench and associated inlet/bypass structures.
- Pre-treat stormwater runoff.
- Design appropriate ground cover and terrestrial vegetation over the infiltration trench.

10.6.1.2 Site Characteristics

The property is frequently uninhabited and the 5kL tank will be full for a more significant proportion of time than typical installations. It is assumed that the 5kL tank will be full at the commencement of the design event.

The site characteristics are summarised as follows:

- Catchment area 210m^2 (roof)
  290 m^2 (ground level paved)
  500 m^2 (pervious)
  1000 m^2 (Total)
- Landuse/surface type pervious area is grassed or landscaped with garden beds.
- Overland flow slope Lot is 25m wide, 40m deep, slope = 3%
- Soil type sandy clay
- saturated hydraulic conductivity (K_h) = 360mm/hr

10.6.2 Checking Field Conditions

Boreholes were drilled at 2 locations within the site and the results are as follows:

- ❒ Field tests found the soil to be suitable for infiltration, consisting of fine sand with a saturated hydraulic conductivity of between 360 mm/hr to 1800 mm/hr.
- ❒ The moderating factor to convert this to the representative areal hydraulic loading is 0.5.

10.6.3 Estimating design flows

See Appendix E Design Flows – t_c for a discussion on methodology for calculation of time of concentration.

Step 1 – Calculate the time of concentration.

The catchment area is 1000m^2

Min t_c = 6 minutes

Rainfall Intensities for the area of study (for the 2 and 100 year average recurrence intervals) are estimated using ARR (1998) with a time of concentration of = 6 minutes and are:

I_2 = 56.4 mm/hr*

I_{100} = 155 mm/hr*

* These figures are for the worked example only. The appropriate region and corresponding rainfall intensities must be selected for each individual project.

Step 2 – Calculate design runoff coefficients (using the method outlined in Australian Rainfall and Runoff Book VIII (Engineers Australia, 2003)).

Where – Fraction impervious (f) = 0.5

Rainfall intensity ($^{10}I_1$) = 25.6mm/hr (from the relevant IFD chart)

Calculate C^1_{10} (pervious runoff coefficient)

$$C^1_{10} = 0.1 + (0.7-0.1) \times (^{10}I_1 - 25)/(70 - 25) = 0.11$$

Calculate C_{10} (10 year ARI runoff coefficient)

$$C_{10} = 0.9f + C^1_{10}(1-f)$$

$$C_{10} = 0.50$$

Step 3 – Convert C_{10} to values for C_2 and C_{100}

Where – $C_y = F_y \times C_{10}$

Runoff coefficients as per Table 1.6 Book VIII ARR 1998

ARI (years)	Runoff Coefficient, *Cy*
2	0.43
100	0.60

Step 4 – Calculate peak design flow (calculated using the Rational Method).

$$Q = \frac{CIA}{360}$$

Where – C is the runoff coefficient (C_1, C_{10} and C_{100})

I is the design rainfall intensity mm/hr (I_1 , I_{10} and I_{100})

A is the catchment area (Ha)

Q2 = 0.007m^3/s

Q_{100} = 0.026 m^3/s

Design Discharges	Q_2	= 0.007 m^3/s
	Q_{100}	= 0.026 m^3/s

10.6.4 Location of Infiltration Systems

With a sandy soil profile, the minimum distance of the infiltration system from structures and property boundary is 1 m. As the general fall of the site is to the front of the property, it is proposed that the infiltration system be sited near the front of the property with paved area runoff directed to grassed buffers and a feature vegetated landscaped area adjacent to the infiltration system. Overflow from the infiltration system will be directed to the table drain of the street in front of the property.

> The infiltration system is to be located near the front of the property set back by at least 1 m from the property boundary.

10.6.5 Source Treatment

Roof runoff is directed to a rainwater tank. Although the tank may often be full, it nevertheless serves a useful function as a sedimentation basin. This configuration is considered sufficient to provide the required sediment pretreatment for roof runoff.

Stormwater runoff from paved areas is directed to a combination of grass buffer areas and a landscape vegetated area which is slightly depressed to provide for trapping of suspended solids conveyed by stormwater. Stormwater overflow from the landscaped area into a grated sump pit and then into the infiltration system.

> Pre-treatment for sediment removal is provided by the following:
>
> - connection of roof runoff into a rainwater tank;
>
> - paved area runoff is conveyed to a combination of grassed buffer areas and a landscaped vegetated depression.

10.6.6 Sizing the detention storage

Estimating the required storage volume of the infiltration system involves the computation of the difference in the volumes of stormwater inflow and infiltration outflow according to Figure 10.6. A gravel-filled trench will be used, with a depth of 1 m proposed.

Figure 10.6 shows the spreadsheet developed to undertake the calculations to determine the required dimension of a gravel-filled soakaway trench for the range of probabilistic 2-year ARI storm durations. By varying the size (and perimeter) of the infiltration system, at least 100% of required storage is provided for all storm durations.

Calculation of Dimensions of Soakaways

Location	**Venus Bay**						
Catchment Area	1000	m^2	Infiltration Area		16	m^2	
Volumetric Runoff Coefficient	0.55		Perimeter of Infiltration Area		20	m	
Soil K_h	360	mm/hr	Emptying Time		1	hours	OK
Moderating Factor	0.5						
Width of Infiltration Area	2	m					
Length of Infiltration Area	8	m					
Depth of Storage	1	m					
Porosity	0.35						

Storm Duration	Storm Mean Intensity	Volume in	Volume out	Storage Volume Required	Percentage of Storage provided	
(minutes)	(mm/hr)	(m^3)	(m^3)	(m^3)	%	
6	56.39	3.101	0.468	2.633	213%	OK
12	42.29	4.652	0.936	3.716	151%	OK
18	34.87	5.754	1.404	4.350	129%	OK
30	26.71	7.345	2.340	5.005	112%	OK
45	21.27	8.774	3.510	5.264	106%	OK
60	17.97	9.884	4.680	5.204	108%	OK
90	14.11	11.641	7.020	4.621	121%	OK
120	11.84	13.024	9.360	3.664	153%	OK
180	9.22	15.213	14.040	1.173	477%	OK
240	7.72	16.984	18.720	0.000		OK
300	6.72	18.480	23.400	0.000		OK
360	6.01	19.833	28.080	0.000		OK
480	5.03	22.132	37.440	0.000		OK
600	4.39	24.145	46.800	0.000		OK
720	3.92	25.872	56.160	0.000		OK
840	3.53	27.181	65.520	0.000		OK
960	3.22	28.336	74.880	0.000		OK
1080	2.98	29.502	84.240	0.000		OK
1200	2.77	30.470	93.600	0.000		OK
1320	2.59	31.339	102.960	0.000		OK
1440	2.44	32.208	112.320	0.000		OK
2160	1.83	36.234	168.480	0.000		OK
2880	1.48	39.072	224.640	0.000		OK
3600	1.24	40.920	280.800	0.000		OK
4320	1.07	42.372	336.960	0.000		OK

Figure 10.6 Spreadsheet for calculating required storage volume of infiltration system

As shown above, the storm duration that provides the lowest percentage of required storage (above 100%) is a storm duration of 45 minutes (the dimensions of the infiltration device in the spreadsheet have been altered until the storage is greater than 100% for each storm duration). The critical storm duration is 45 minutes and the storage volume requirement 5.3 m^3. With a porosity of a gravel-filled trench estimated to be 0.35, the required dimension of the soakaway is 2 m (w) by 8 m (L) by 1m (d). The proposed layout of the infiltration system is shown below.

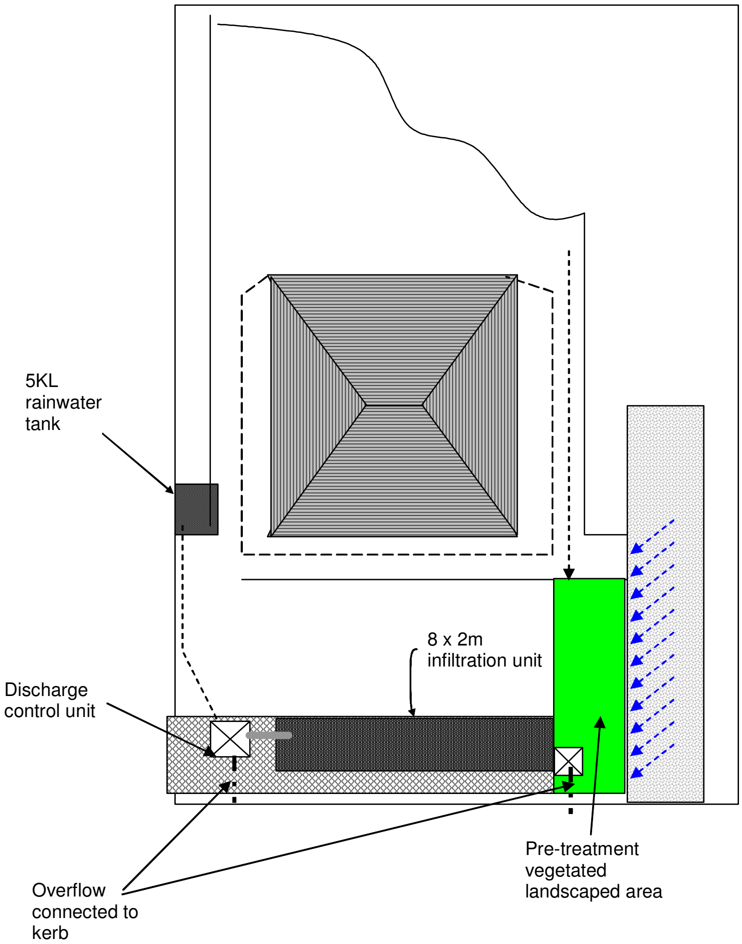


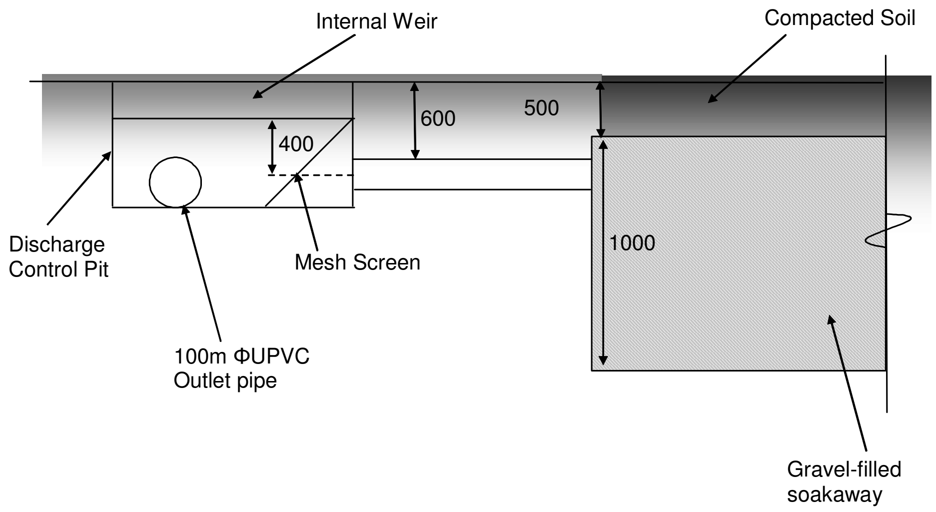
Figure 10.7 Layout of Stormwater Infiltration System

10.6.7 Hydraulic Structures

10.6.7.1 Inlet design

- Peak 2yr ARI design flow = 0.007 m^3/s (calculated previously) with approximately 0.003 m^3/s discharging from the rainwater tank overflow and 0.004 m^3/s from other paved areas
- There are two inlets to the infiltration system, i.e. one from the rainwater tank and the second from the driveway (see Figure 10.7). These inlets are to be designed to discharge flows up to 0.004 m^3/s each into the infiltration trench with overflows directed to the table drain on the street in front of the property.

Pipe connections from the inlet pits to the infiltration system and street table drain are computed using the orifice flow equation



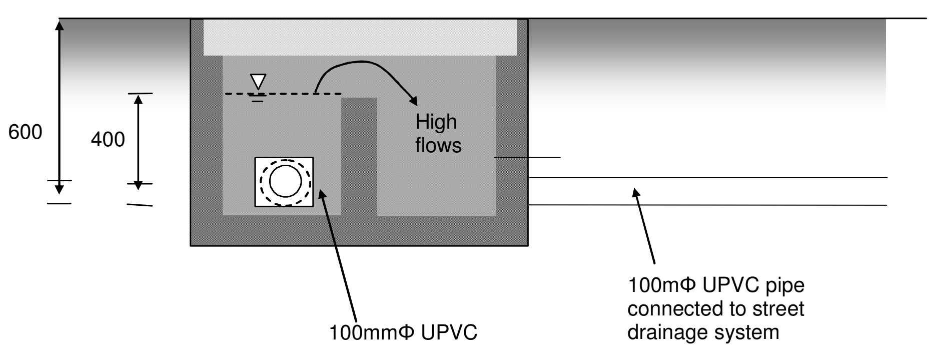
$$A_o = \frac{Q}{C_d\sqrt{2gh}}$$

Equation 10.3

C_d	=	Orifice Discharge Coefficient (0.6)
H	=	Depth of water above the centroid of the orifice (m)
A_o	=	Orifice area (m^2)

- For pipe connections to the infiltration system, adopt h = 0.40 m; Q = 0.004m^3/s

  This gives an orifice area of 0.002 m^2, equivalent to a 55 mm diameter pipe ➔adopt 100 mm diameter uPVC pipe.



10.6.7.2 Bypass Design

An overflow weir (internal weir) separates two chambers in the inlet pits; one connecting to the infiltration system and the second to convey overflows (in excess of the 2 year ARI event) to the street table drain. The overflow internal weirs in discharge control pits are to be sized to convey the peak 100 yr ARI flow, i.e.

Q_{100} = 0.5 x 0.026 m^3/s (two inlet pits) = 0.013m^3/s

- The weir flow equation is used to determine the required weir length:-

$$L = \frac{Q}{C_w \Delta H^{1.5}}$$

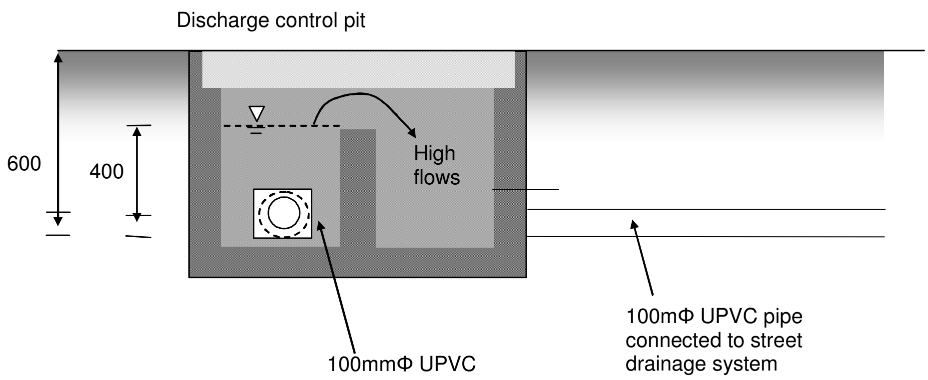
Equation 10.4

Adopting C =1.7 and H = 0.05 gives L = 0.7

Overflow weir will provide at least 150m freeboard during the peak 100 yr ARI flow

- For pipe connection to the street table drain, adopt h=0.40 m; Q = 0.013 m^3/s

  This gives an orifice area of 0.008 m^2, equivalent to a 100 mm diameter pipe ➔ adopt 100 mm diameter uPVC pipe.



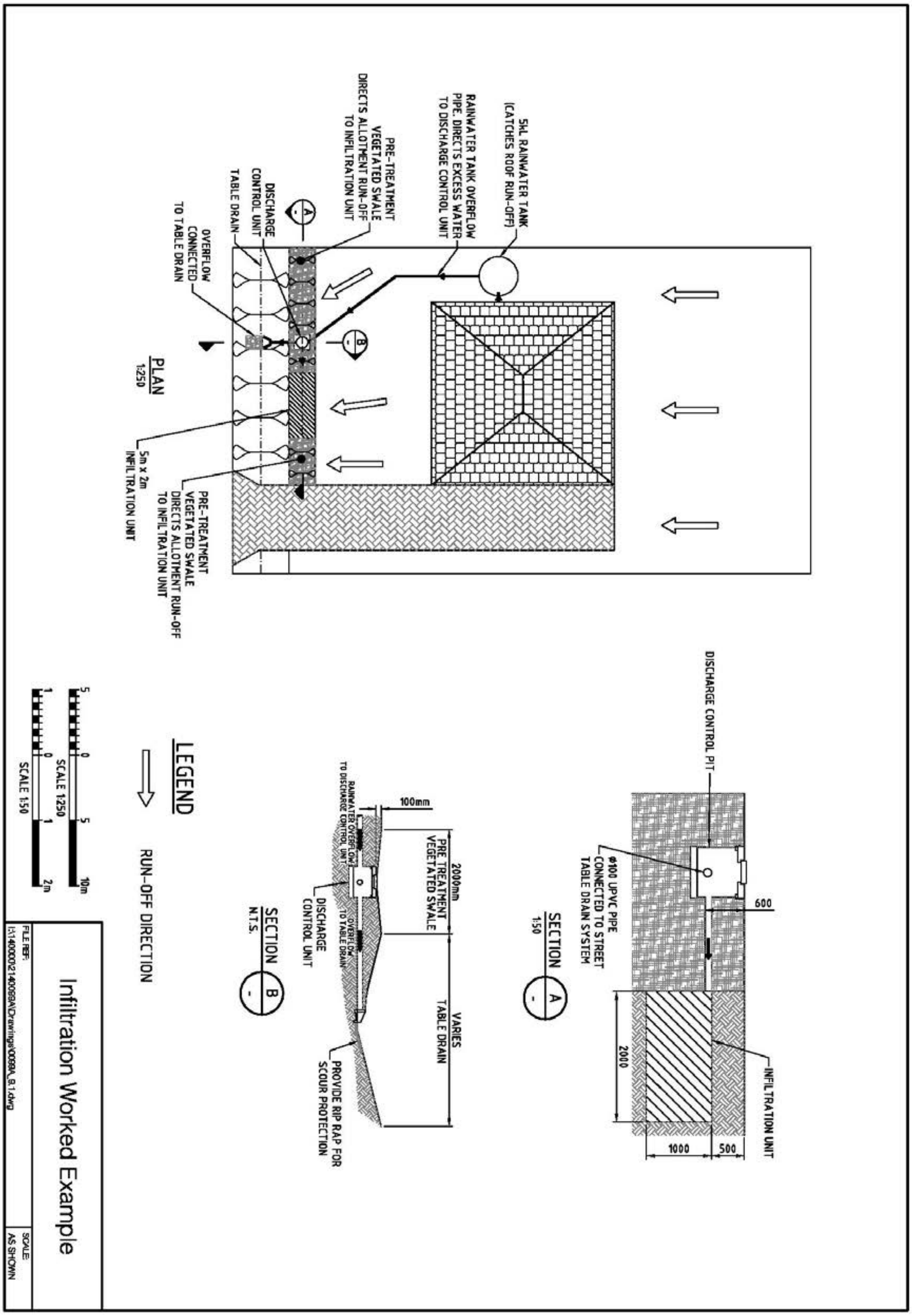
10.6.8 Design Calculation Summary

Infiltration System

CALCULATION SUMMARY

	CALCULATION TASK	OUTCOME		CHECK
1	**Identify design criteria**			✓
	Design ARI event to be infiltrated (in its entirety)	2	year	
	OR			
	Design Hydrologic Effectiveness	N/A	%	
	ARI of Bypass Discharge	100	year	
2	**Site characteristics**			✓
	Catchment Area connected to infiltration system	1000	m^2	
	Impervious Area connected to infiltration system	500	m^2	
	Site hydraulic conductivity	360	mm/hr	
	Areal hydraulic conductivity moderating factor	0.5		
3	**Estimate design flow rates**			
	Time of concentration			
	estimate from flow path length and velocities	6	minutes	✓
	Identify rainfall intensities			
	station used for IFD data:	Hobart		
	Design Rainfall Intensity for inlet structure(s)	56.4	mm/hr	
	Design Rainfall Intensity for overflow structure(s)	155	mm/hr	✓
	Design runoff coefficient			
	inlet structure(s)	0.43 to 0.60		✓
	Peak design flows			✓
	Inlet structure(s)	0.007	m^3/s	
	Bypass structure(s)	0.026	m^3/s	
4	**Detention Storage**			✓
	Volume of detention storage	5.3	m3	
	Dimensions	8 m x 2 m	L:W	
	Depth	1	m	
	Emptying Time	1	hrs	
5	**Provision of Pre-treatment**			✓
	Receiving groundwater quality determined	Y		
	Upstream pre-treatment provision	Y		
6	**Hydrualic Structures**			
	Inlet Structure			✓
	Provision of energy dissipation	Y		
	Bypass Structure			✓
	Weir length	0.70	m	
	Afflux at design discharge	0.05	m	
	Provision of scour protection	Y		
	Discharge Pipe			✓
	Capacity of Discharge Pipe	0.013	m^3/s	

10.6.9 Construction drawings



10.7 References

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Chapter 11 Rainwater Tanks



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11.1 Introduction

The core sustainability objective of using rainwater tanks is to conserve mains water. In addition to conserving mains water, rainwater tanks help to protect urban streams by reducing stormwater runoff volumes, particularly from small storms, and associated stormwater pollutants from reaching downstream waterways. Rainwater and stormwater harvesting on individual allotments are some initiatives that can be implemented to deliver a potable water conservation objective.

Another important household initiative to conserve water is the use of water efficient plumbing fittings and appliances. These are often adopted as a first priority in water conservation initiatives as they are easy to adopt, have high cost effectiveness and broader environmental benefits such as reduced wastewater discharges. Recent research has found that the adoption of water efficient showerheads and dual flush toilets can reduce indoor water use by 15 – 20% (11 – 15% of total internal and external water use). Following improving the efficiency of water use within a household, finding supplementary sources for water is fundamental to further reducing demand on mains water. The use of rainwater tanks to collect roof runoff is an accepted means of supplementing mains water supplies which is simpler to implement than other potential alternative water sources such as greywater or surface stormwater.

There are presently no quantitative performance targets (e.g. size of tank, targeted reductions in potable demand) in any existing local government and state authority policies and guidelines regarding the use of rainwater tanks.

This design procedure focuses on factors associated with selecting and using a rainwater tank. Variables that need to be considered in sizing a rainwater tank include the size or area of roof directed to a tank, the quantity and nature of the demand and the rainfall pattern of a particular area.

11.2 Rainwater tank considerations

The use of rainwater tanks to reduce demand on reticulated potable water supplies and stormwater runoff volume need to consider a number of issues. These are:

- *Supply and demand* – conditions such as a low roof area to occupancy ratio (e.g. high density development) and low annual rainfall regions (e.g. northern Victoria) can result in large rainwater tank volumes to provide a “reliable” supplementary water supply to the end-uses connected to a tank.
- *Water quality* – the quality of water from rainwater tanks needs to be compatible with the water quality required by the connected “end-use”. There are a number of ways in which the water quality in rainwater tanks can be affected and it is important to understand these so that appropriate management measures can be implemented.
- *Stormwater quality benefits* – the quantity of the stormwater that is reused from a tank system reduces the quantity of runoff and associate pollutants discharging into a

stormwater system. The benefits, in terms of pollutant reduction, should be considered as part of a stormwater treatment strategy.

- *Cost* – the cost of rainwater tanks needs to be considered against alternative demand management initiatives and alternative water sources.
- *Available space* – small lots with large building envelopes may preclude the use of external, above ground, rainwater tanks.
- *Competing uses for stormwater runoff* – there may be situations where a preferred beneficial use for stormwater runoff (such as irrigation of a local public park, oval, or golf course) may provide a more cost effective use of runoff from roofs than the use of rainwater tanks on individual allotments.
- *Maintenance* – most rainwater tanks will need to be maintained by the householder or a body corporate (or similar).

These issues are further discussed below.

11.2.1 Supply and Demand Considerations

Supply and demand considerations are matters that should be examined during the concept investigation phase of a project. Nevertheless, a number of key considerations are discussed below to ensure that they are sufficiently addressed before implementing a rainwater harvesting scheme.

Low Roof Area to Occupancy Ratio

This situation is most likely to arise on projects with medium and high density residential dwellings (i.e. where the ratio of roof area to the number of occupants in the dwelling is low). In these situations, it is probably most important to maximise the use of water efficient fittings and appliances to reduce the demand on the reticulated water supply so that the additional supply opportunities that are presented by a rainwater tank are maximised.

A smaller ratio of roof area to number of occupants (ie. increasing density) has the effect of increasing the size of rainwater tank required to deliver a given *reliability*[3] of supply to the connected end-uses. With high density, multi-storey developments (> people/100 m^2 of roof), there is a diminishing opportunity for the effective use of roof water as a means of supplementary supply for internal uses. In these situations, it may be most practical to capture rainwater centrally for external uses (landscaping and car washing).

Increasing the number of end-uses connected to a tank (e.g. laundry and garden in addition to toilets) will reduce the reliability of the supply. While the reliability decreases with increasing end-uses, the total use of available rainwater increases because there is a greater frequency of drawdown and reduced frequency of overflow from the tank during storm events.

[3] The reliability of water supply is simply the percentage of water demand that is met by that supply.

The reliability of supply for internal uses may not necessarily be a concern if potable water is available to supplement a supply (e.g. a mains water top-up or switch-over mechanism).

Low Rainfall Regions

The effectiveness of rainwater tanks as a supplementary water source is reduced in low rainfall regions.

The use of rainwater tanks on projects in low annual rainfall regions will need to be considered carefully for viability as a cost effective alternative water source if mains water is available. Other potential water sources such as reclaimed water and/or greywater re-use may need to be given greater consideration in these regions as these water sources are independent of local climatic conditions and can provide a higher reliability of supply.

11.2.2 Water Quality

Water quality is an important consideration with all roof water systems, especially in urban and industrial areas. Possible pathways for contamination of roof water are:

- Atmospheric pollution settling onto roof surfaces.
- Bird and other animal droppings can pollute the water with bacteria and gastro-intestinal parasites.
- Insects, lizards and other small animals can become trapped and die in a tank.
- Roofing materials and paints. Lead based paints in particular should never be used on roofs where water is collected for potable water uses. Tar-based coatings are also not recommended, as they may affect the water's taste. Zinc can be a significant pollutant in some paints and galvanized iron or zincalume roofs (particularly when new) should not be collected for potable use. Care should also be taken to avoid dissimilar metals that may accelerate corrosion and affect water quality.
- Detergents and other chemicals from roofs painted with acrylic paints can dissolve in the runoff. Runoff from roofs made of fibrous cement should be discarded for an entire winter due to the leaching of lime.
- Chemically treated timbers or lead flashing should not be used in roof catchments and rainwater should not be collected from parts of the roof incorporating flues from wood burners.
- Overflows or discharge pipes from roof mounted appliances, such as evaporative air-conditioners or hot water systems, should not discharge onto a roof catchment or associated gutters feeding a rainwater tank.

The presence of these contamination pathways will vary from project to project and will be largely dependant on:

- The proximity of the project to areas of heavy traffic, incinerators, smelters or heavy industry, and users of herbicide and pesticides (e.g. golf course, market gardens).
- Roofing materials and roof mounted appliances.

- The provision of a well sealed rainwater tank with a first flush device and with inlet and overflow points provided with mesh covers to keep out materials such as leaves and to prevent the access of mosquitoes and other insects.

The quality of roof water collated from relevant Australian studies is summarised and further discussed in Australian Runoff Quality (Engineers Australia, 2006).

Water quality requirements of an end-use connected to a rainwater tank will be the determinant of whether or not additional water quality treatment needs to be provided between the tank and the end-use. For all non-potable uses (e.g. toilet flushing, washing machines, garden watering etc.) available monitoring data indicates that typically there are low levels of risk to consumers if additional water quality treatment (e.g. disinfection) is not provided (Coombes, 2002). One exception in this regard is where a rainwater tank is connected to the hot water system where there is a heightened potential of human ingestion of rainwater (e.g. when showering, children in the bath). If connected to the hot water system, some disinfection is required which may include providing hot water at a certain temperature (to allow for complete pasteurisation) or other disinfection methods (e.g. chlorination).

11.2.3 Stormwater quality benefits

Using collected rainwater reduces the total volume of stormwater runoff from a site and therefore reduces pollutant discharges. The percent reduction of stormwater from a site can be estimated based on the reuse demand, reuse reliability and mean annual rainfall.

$$\text{Percent reduction} = \frac{\text{Reliability} * \text{Reuse Demand (kL)}}{\text{Rainfall Volume (m}^3\text{)}}$$

Where, Reliability = proportion of demand met by supply

Reuse Demand = avg. ann. demand per person * Number of occupants

Rainfall Volume = mean Annual Rainfall (m) * Contributing Roof Area (m^2)

For example, the percent reduction in stormwater from a rainwater tank that provides 70% reliability for a house in Bendigo (MAR 570 mm) with 3 occupants and a roof area of 120 m^2 is calculated as follows:

$$\text{Stormwater reduction} = (70\%)*(8\ \text{kL/p/yr}*3\ \text{people})/(0.57\ \text{m}*120\text{m}^2) = 25\%$$

Therefore, the reduction in stormwater runoff and hence TSS, TP and TN loads from the roof due to reuse from the rainwater tank is 25%.

Additionally, rainwater tanks provide some treatment of water that is not removed from the tank for reuse (ie water that is stored for some period and then spills when the tank overflows). The dominant process is the settlement of suspended solid loads. The reduction in pollutant loads in water that is spilt from rainwater tanks is likely to be small compared with the reduction due to the removal from the system.

11.2.4 Cost Considerations

Typically the cost of a rainwater tank installation for supplementary water source ranges from $600 to $2000 for residential detached or semi-detached dwellings. Three cost components are normally involved, i.e. the cost of the tank, installation and plumbing works and the cost of a pump. Costs may increase with higher density development as space constraints could require more specialised tanks to be fitted (see next section for more discussion) unless communal use of a centralised rainwater tank can be facilitated. Local authorities in some areas also offer rebates for the installation of rainwater tanks.

The typical payback period of a rainwater supply system purely through a reduction in domestic water charges is of the order of 35 years under current two part tariff metered water pricing and will often not be able to justify the use of rainwater as an alternative source of water to mains water. This is mostly due to the present pricing of mains water not reflecting the true environmental and social cost of the water resource. There has been an emergence of terms such as 'total resource cost' and 'total community cost' in addition to the more commonly used terms of 'life cycle cost' and 'whole of life cost' in recent analysis of the value of water to more holistically reflect the beneficial outcomes associated with water conservation practices through the adoption of alternative water sources and associated matching of their respective water quality with fit-for-purpose usage. Researchers (Coombes, 2002) have shown that when such 'total resource cost' issues, and the potential benefits of rainwater capture/reuse in regard to reduced stormwater flows, are considered that more positive economic benefits can apply.

11.2.5 Available Space Considerations

Small allotments with large building envelopes are becoming more common as dwindling land stocks require the provision of smaller lots to meet increasing demand. However, the public's desire for 'traditional' sized houses remains strong and as a consequence front and back yards are being reduced to allow large houses to be built onto progressively smaller allotments. This phenomenon imposes a potential constraint on the use of rainwater tanks where tanks are installed external to a building and above ground (as is conventional practice). Competing demand for the use of external areas raises the potential for resistance to the imposition of rainwater tanks on small allotments with large building envelopes. This can be overcome by burying tanks or placing them underneath houses but these have techniques associated cost implications for construction and maintenance.

Rainwater tank designs have advanced markedly in recent times with slim line rainwater tank designs reducing tank footprints. Modular rainwater tank systems are also now being developed. These systems can be interconnected to form boundary fences or potentially walls for a garden shed or carport. Rainwater tanks in buildings can also provide energy benefits through thermal inertia of the stored water moderating temperature variations within households. Examples of slim line and modular rainwater tanks are shown in Figure 11.1.

Figure 11.1 Slim Line Tank and Modular Rainwater Tank system

The final decision on the acceptability of using rainwater tanks on small lots is likely to be influenced by the size of tank required (which is influenced by the available roof area and the water conservation outcome to be attained from a rainwater tank), the compatibility of commercially available tank systems with the built form and the available area for a tank. This decision needs to be made on a case by case basis.

11.2.6 Competing uses for Stormwater Runoff

There may be situations, especially on larger precinct-wide projects, where there may be one or more competing uses for stormwater runoff generated from roof areas and ground level impervious surfaces. Rainwater tanks may not provide the optimal strategy from a sustainability perspective, especially when comparing the life cycle cost and resource use outcomes of a centralised stormwater harvesting scheme with a de-centralised rainwater harvesting scheme. These issues need to be investigated thoroughly during the concept design stage of a project.

A common example of such competing uses is that associated with residential development adjoining public open spaces and golf courses. In development scenarios such as this, it is often more cost effective (from both capital and asset maintenance cost perspectives) to implement a precinct-wide stormwater harvesting scheme and supply the water for public open space watering.

11.2.7 Maintenance Considerations

Whilst the maintenance of a rainwater tank based mains supply augmentation system is not particularly arduous for a property owner, it is nevertheless an additional requirement for households that normally would have their water supply sourced from a reticulated water supply system. This may have possible long-term impacts on the sustainability of a rainwater tank supply scheme, especially if homeownership changes, though with more realistic water pricing policies and appropriate education practices, the impacts of this consideration should be minimal. Further discussion on the maintenance elements of a rainwater tank/reuse system is provided in Section 11.5.

11.3 Australian standards for installation of rainwater tank systems

Rainwater tanks need to be installed in accordance with the **National Plumbing and Drainage Code (AS/NZS3500.3:2003)** and the **Tasmanian Plumbing Regulations 2004**.

While not strictly a standard, rainwater should only be sourced from roof sources, and flows from roads, footpaths, and other common areas at ground level, are addressed through separate stormwater treatment processes. If supply is supplemented by an interconnection with a reticulated water supply, backflow prevention via either an air gap or proprietary device is required in accordance with Australian Standard AS 3500.1.2 (1998) and the requirements of the local water supply authority. For treatment and usage it is suggested that[4]:

- The collection system should incorporate a first flush device or "filter sock" to divert or filter initial run-off from a roof.
- The tank system should be connected to the toilet, hot water, laundry and garden irrigation fixtures, and there should be no direct supply from the mains water to these services.
- There should be no connection to other indoor fixtures from the rainwater tank unless measures are undertaken to make the supply fit for consumption.
- The tank is enclosed and inlets screened, in order to prevent the entry of foreign matter and to prevent mosquito breeding.
- Overflow from a rainwater tank should be directed to a detention device, swale or stormwater drain.

[4] Donovan, I., (2003). Water Sensitive Planning Guide for the Sydney Region, Upper Parramatta River Catchment Trust.

11.4Design procedure: rainwater tanks

Design considerations when evaluating a rainwater tank system include the following:

- selection of end-uses
- determination of size of tank required/desired
- hydraulic fixtures
- water filter or first flush diversion
- mains water top-up supply
- on-site detention provisions
- maintenance provisions.

11.4.1 Selection of end-uses

Water consumption in a household varies depending on the type and location of the house. Typical water consumption figures for residential areas expressed on a per capita basis are summarised in Table 11-1.

It is important to not overlook the effect of using water efficient appliances on reducing water demand when sizing rainwater tanks. Consumption of water for toilet flushing has reduced significant since the mandatory introduction of dual flush toilets over a decade ago. Table 11-2 lists the likely reduction in indoor household water demands resulting from the adoption of such water efficient appliances.

Table 11-1 Typical Household Water Consumption in Melbourne [adapted from Water Resources Strategy for the Melbourne Area Committee, 2001]

Water Uses	Per Person Usage (kL/person/yr)	Percentage of total usage
Garden	32	35%
Kitchen	5	5%
Laundry	14	15%
Toilet	18	19%
Bathroom	24	26%
Total	92	
Hotwater	24	26%

Table 11-2 Estimation of Reduction in Water Demand by Water Efficient Appliances [adapted from NSW Department of Infrastructure Planning and Natural Resources, 2004]

Water Uses	Conventional Demand (kL/person/yr)	Reduced Demand with Water Efficient Appliances and Fittings (kL/person/yr)
Shower	20.8	13.5
Bath	3.2	3.2
Hand Basin	2.2	1.2
Toilet	12.8	7.3
Washing Machine	17.0	11.9
Kitchen Sink	4.4	2.3
Dishwashing	1.1	0.6
Total	61.5	40.0

The most obvious water uses for rainwater are toilet and garden supply as they avoid the requirement for treatment to potable standards. Replacement of mains potable water for toilet flushing is considered to be the more effective of the two because of its consistent demand pattern and thus a higher reliability of water supply can be achieved for a given rainwater tank size. Whilst having a higher water demand, water usage for garden watering is seasonal and the demand pattern is “out-of-phase” with the supply pattern (ie. high garden watering demand occurs during low rainfall periods) and thus a larger rainwater tank storage may be required to achieve comparable reductions in potable water usage compared with toilet flushing.

Following the use of rainwater for toilet flushing and garden watering, the next appropriate use of rainwater is in the laundry (e.g. washing cold tap). Supplementing the supply for hot water is also an effective option. Hot water usage constitutes approximately 30% of household indoor usage. The quality of water delivered from a rainwater tank via a hot water system is improved by the combined effects of high temperature pasteurisation, pressure in the pump and the instantaneous heat differentials between the rainwater tank and a hot water service.

11.4.2 Tank Size and Supply Reliability

The supply reliability of a rainwater tank is directly influenced by three factors,

1. supply characteristics – as defined by the size of the catchment (ie. roof area connected to the rainwater tank) and the rainfall pattern of a region (mean annual rainfall and seasonal pattern).

2. demand characteristics – as defined by the type of uses. If indoor use, this is dependent on household occupancy and if for garden watering, demand is dependent on garden design and climatic conditions of the region.
3. storage size

Owing to the intermittent nature of the supply of rainwater, the most appropriate analytical approach for assessing the reliability of supplies is a continuous simulation (modelling) approach using long records of rainfall data. Australian Runoff Quality (Engineers Australia, 2003) provides a detailed discussion on appropriate modelling techniques for determining a relationship between tank size and rainwater supply reliability.

For any assessments evaluating more widespread usage of rainwater, rigorous assessments using models such as PURRS, AQUACYCLE and UVQ are recommended (see Chapter 5 and 13 of Australian Runoff Quality for guidance in this regard). Mass balance analyses using long-term rainfall data and demand patterns may also be completed using spreadsheet software.

Table 11-3 Sources of further information on the use of rainwater tanks

Source	Web Address
Aust. Environmental Health Council	http://enhealth.nphp.gov.au/council/pubs/ecpub.htm
Gold Coast City Council	http://www.goldcoast.qld.gov.au/attachment/goldcoastwater/GuidelinesTankInstall.pdf
Lower Hunter & Central Coast Regional Environmental Management Strategy	http://www.lhccrems.nsw.gov.au/pdf_xls_zip/pdf_wsud/4_Rainwatertanks.pdf
Sydney Water	http://www.sydneywater.com.au/everydropcounts/garden/rainwater_tanks_installation.cfm
Department of Environment & Heritage, South Australia	http://www.environment.sa.gov.au/sustainability/pdfs/rainwater_final.pdf
Your Home Consumers Guide [A joint initiative of the Australian government and the design and construction industries	http://www.greenhouse.gov.au/yourhome/technical/fs22_2.htm

Inlet Filter

Some form of filter is strongly recommended on all flows being directed to a rainwater tank. This filter will provide a primary treatment role in regard to removing leaf litter and some sediment that would otherwise enter the tank, and possibly contribute to water quality degradation. Such a filter can also serve to isolate the tank from access by vermin and mosquitoes.

First Flush Diverter

Diversion of the 'First Flush' from a roof is also a recommended practice, as this can minimise the ingress to the tank of fine particulates, bird/animal faeces and other potential contaminants. Current research does not enable the specification of a definitive First Flush, with values between 0.25 and 1.0mm of runoff typically being quoted.

Proprietary devices are available, that often provide a joint 'Filter/First Flush' diversion role. Alternatively, a first-flush device can be made from readily available PVC fittings.

Maintenance Drain

Periodic removal of sludge and organic sediments that accumulate in the base of a rainwater tank may be necessary is build up is excessive, and as such a suitable outlet should be provided. It has been suggested that this sludge layer, and biofilms that develop on the walls of a tank, may play a role in the natural purification processes occurring in the tank therefore removing a sludge layer should only occur when build up impedes the tank operation.

Mains Top up

Most rainwater tanks will require an automatic top-up system to ensure uninterrupted supply to the household. This top-up should occur as a slow 'trickle' such that there are benefits in regard to reducing peak flow rates in the mains supply system (which, if properly planned, can enable smaller mains infrastructure to be installed in a 'greenfields' situation).

The volume/rate of top-up should be such that there is always at least one day's supply contained within the tank. Top-up should also occur when tank levels are drawn down to a depth of 0.3m, or one day's capacity, whichever is the greater, to both guarantee supply and to minimise sludge/sediment resuspension.

A final consideration with any top-up system is that there is a requirement for an 'air gap' between the top-up supply entry point and the full supply level in the tank in order to ensure there is no potential for backflow of water from the tank into the potable supply system. A suitable air gap is of the order of 100mm. Backflow prevention should also be installed at the property stop-tap.

Alternatively, a proprietary device to automatically switch from tank supply to the mains supply when the tank is low may be installed. This negates the need for moving parts inside the tank or the potential for a top-up mechanism failure causing the mains supply to run into the tank when not required. A failure such as this may not be noticed by the property owner until the next water bill arrives as the mains water could be running through the full tank and into the overflow stormwater connection.

Overflow

Rainwater tank overflows should be directed to the stormwater collection system. In areas with suitable soils and slopes, discharge to a lot scale infiltration trench may also be possible (see Procedure 8 for more detail in this regard).

Overflows should also be located below the mains top-up supply point in order to prevent the potential for backflow.

Pump

The supply to the household from the rainwater tank can occur via a pressure pump system, or alternatively a solar panel/pump/header tank system may be implemented, if low heads are acceptable. Careful selection of a suitable pump system is recommended to minimise operational costs and noise issues.

On-site Detention

In some situations, rainwater tanks can be configured with an active ‘detention’ zone located above the ‘capture and reuse’ zone. This system reduces the effective yield from a tank, but may deliver greater downstream stormwater conveyance benefits through the delivery of lower peak flows for low to moderate ARI events. In such applications, it is important to ensure that the potable supply top-up is located above the ‘detention’ zone, not just the ‘capture and reuse’ zone.

11.5Inspection and maintenance

Rainwater tanks are low maintenance, not no maintenance systems. Good maintenance practice is necessary and should include:

- Routine inspection (every 6 months) of roof areas to ensure that they are kept relatively free of debris and leaves. Roof gutters should be inspected regularly and cleaned if necessary. There are special gutter designs available for limiting the amount of debris and litter that can accumulate in the gutter to be subsequently transported to the rainwater tank. These special gutters cost about twice normal guttering but require little maintenance. Leaf screens may also be installed in most standard gutters and provide a cheap method of preventing excessive leaf litter accumulating in the guttering.
- It may be necessary to prune surrounding vegetation and overhanging trees which may otherwise increase the deposition of debris on the roof.
- First flush devices should be cleaned out once every 3 to 6 months, or as required.
- All screens at inlet and overflow points from the tank should be inspected regularly to check for fouling, at least every 6 months.
- Tanks should be examined for the accumulation of sludge at least every 2 to 3 years. If sludge is covering the base of the tank and affecting its operation (i.e. periodically resuspending, or reducing storage capacity) it should be removed by siphon, flushed from the tank or by completely emptying of the tank. Professional tank cleaners can be used.
- Any pumping system should be maintained in accordance with the manufacturer’s specifications.

11.6 References

Coomes, P.J. (2002), 'Rainwater tanks revisited: new opportunities for urban water cycle management'. PhD thesis. University of Newcastle, Callaghan, NSW.

Coombes, P.J. and Kuczera, G. (2003), 'Analysis of the Performance of Rainwater Tanks in Australian Capital Cities', 28th International Hydrology and Water Resources Symposium, Engineers Australia, Wollongong.

Engineers Australia, 2006, *Australian Runoff Quality Australian Runoff Quality: A guide to Water Sensitive Urban Design, Editor-in-Chief, Wong, T.H.F.*

eWater, 2009, Model for Urban Stormwater Improvement Conceptualisation (MUSIC) User Manual, Version 4.0, September.

Chapter 12 Aquifer Storage and Recovery

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12.1 Introduction

Aquifer storage and recovery (ASR) is a means of enhancing water recharge to underground aquifers through either pumping or gravity feed. Stored water can then be pumped from below ground during dry periods for subsequent reuse and can therefore be a low cost alternative to large surface storages. In the stormwater context, it may also be used as a method to store excess water produced from urbanisation during wet periods (e.g. winter) and which can then harvested during long dry periods to reduce reliance on mains supply for uses such as irrigation.

Both stormwater and treated wastewater are potential sources for an ASR system. This chapter focuses on stormwater ASR systems, although many of the concepts are the same for both systems. Stormwater ASR systems are designed to harvest increased flows attributed to urbanisation. Harvesting urban runoff and diverting it into underground groundwater systems also requires that the quality of the injected water is sufficient not to degrade the existing and potential future beneficial uses of the groundwater supplies. The level of treatment is dependent on the quality of the groundwater. In most instances, the range of management measures described in this manual will provide sufficient treatment prior to injection.

The viability of an ASR scheme is highly dependent on the underlying geology of an area and the presence and nature of the aquifers. There are a range of possible aquifers that can accommodate an ASR scheme including fracture unconfined rock and confined sand and gravel aquifers. Detailed geological investigations are required to establish the feasibility of any ASR scheme. This Chapter provides an overview of the main elements of an ASR system and directs readers to more specific guidance documents.

Broad requirements of ASR systems include:

- protecting or improving groundwater quality where ASR is practiced
- ensure that the quality of recovered water is fit for its intended use
- protecting aquifers and aquitards (fractured rock) from being damaged by depletion or excessive pressure (from over-injection)
- avoiding problems such as clogging or excessive extraction of aquifer sediments
- ensuring reduced volumes of surface water downstream of the harvesting point are acceptable and consistent with a catchment management strategy.

In addition to the physical requirements of an ASR system, they also require permits to divert water, to install treatment measures, to inject into groundwater as well as extraction for the intended use. A thorough investigation of the required permits should be undertaken. The Victorian Smart Water fund plans to develop Best Practice

Guidelines for ASR in Victoria, commencing in 2004. Further information on this project can be found at www.smartwater.com.au.

The following material has been reproduced from the Code of Practice for Aquifer Storage and recovery (SA EPA, 2004) with the permission of the author, to provide an overview of the main components of an ASR system.

12.2 Components of an ASR system

An ASR scheme that harvests stormwater typically contains the following structural elements (see Figure 12.1):

- a diversion structure from a stream or drain
- a control unit to stop diversions when flows are outside an acceptable range of flows or quality
- some form of treatment for stormwater prior to injection
- a wetland, detention pond, dam or tank, part or all of which acts as a temporary storage measure (and which may also be used as a buffer storage during recovery and reuse)
- a spill or overflow structure incorporated in wetland or detention storage
- well(s) into which the water is injected (may require extraction equipment for periodic purging)
- an equipped well to recover water from the aquifer (injection and recovery may occur in the same well)
- a treatment system for recovered water (depending on its intended use)
- systems to monitor water levels, and volumes injected and extracted
- systems to monitor the quality of injectant, groundwater and recovered water
- sampling ports on injection and recovery lines
- a control system to shut down recharge in the event of unfavourable conditions.

Figure 12.1 presents a schematic of the major elements of an ASR scheme.

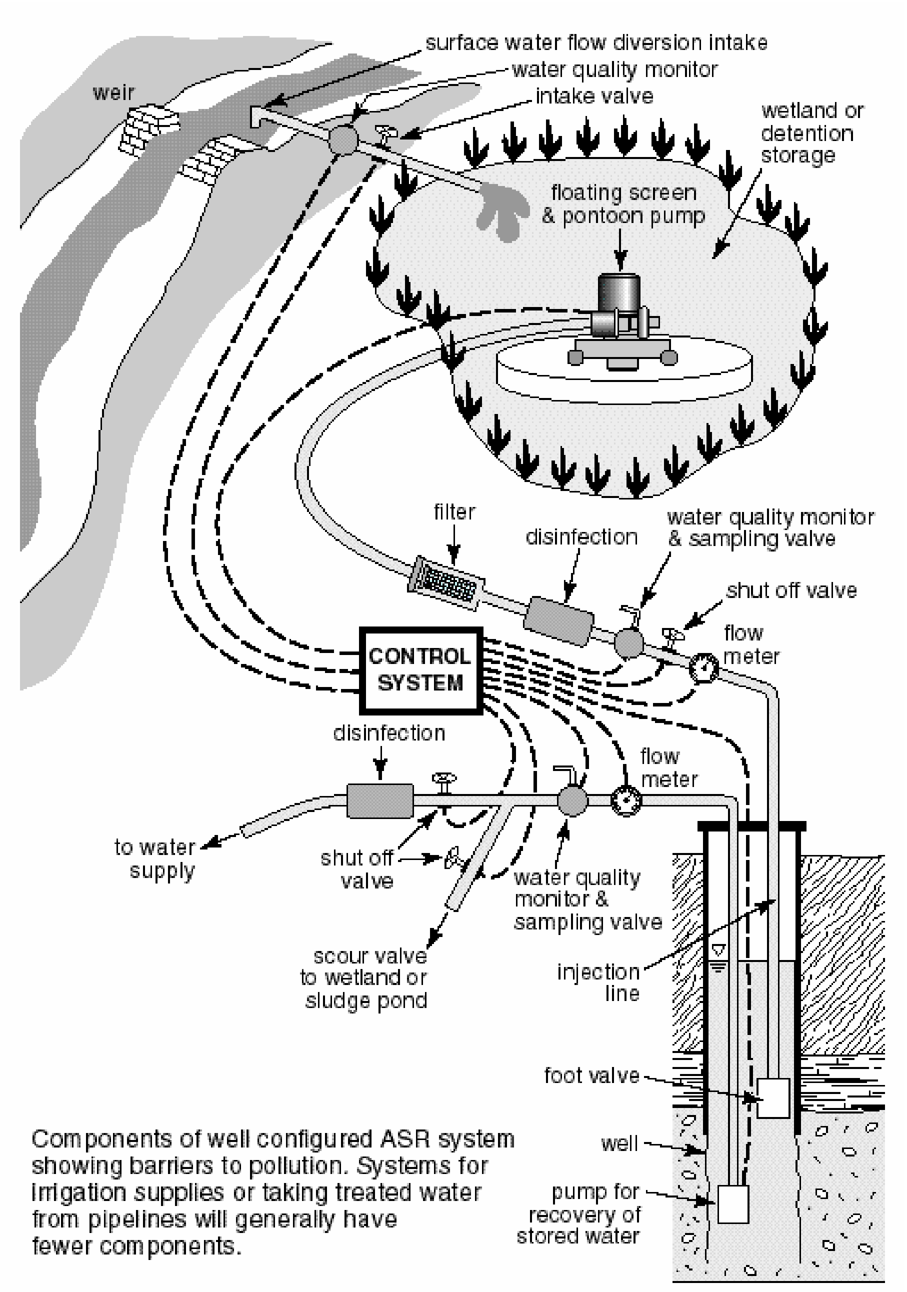

Figure 12.1 Components of a well-configured ASR system (diagram CSIRO Land and Water)

12.3Treatment and pollution control

For stormwater ASR systems, water quality treatment will be required prior to injection into groundwater. The level of treatment depends on the quality of the groundwater (beneficial uses) and local regulation should be checked. Many of the treatments described in earlier chapters will provide sufficient treatment for an ASR system. These systems also have the added benefit of reducing the risk of ‘clogging’ the ASR injection well because of efficient fine sediment removal.

12.3.1 Knowledge of pollutant sources in the catchment upstream

Each ASR scheme must identify potential pollution sources within a catchment and plan risk management strategies, including pollution contingency plans. For urban stormwater harvesting, treatment measures described in this manual are considered a minimum requirement.

Comparisons with native groundwater quality and its environmental values will indicate the requirements for treatment of water detained for injection. An evaluation of the pollutants that may be present within the injectant water needs to be carried out on a catchment basis. Pollutants will vary according to whether the catchment drains urban residential, urban industrial, rural or a combination of any of these catchment types.

The concentrations of pollutants typically have seasonal or within-event patterns, and heavy pollutant loadings can be avoided by being selective in the timing of diversions. Knowledge of the potential pollutant profile helps to define water quality sampling and analysis costs when determining the viability of the ASR project.

12.3.2 Aquifer Selection

The quality of water to be injected must be no worse than the quality of water already in the aquifer, and better if possible. As discussed earlier, the aquifer may already be providing beneficial uses to others and the quality and flow requirements of these users needs to be considered in the aquifer selection. This may exclude using aquifers containing high quality groundwater for ASR schemes.

Factors to consider when choosing a suitable aquifer include:

- environmental values of the aquifer (beneficial uses)
- sufficient permeability of the receiving aquifer
- salinity of aquifer water greater than injection water
- possible damage to confining layers due to pressure increases
- adverse effects of reduced pressure on other groundwater users
- higher recovery efficiencies of porous media aquifers
- impacts on other aquifer users
- aquifer mineral dissolution, if any, and potential for well aquitard collapse.

12.3.3 Pretreatment prior to injection

Many of the treatment measures describe in earlier chapters of the Manual are suitable as pretreatments for ASR schemes. In general, methods that have long detention times are advantageous to reduce pathogenic microorganisms in addition to other pollutants.

An advantage of using treatment with large storages (e.g. wetlands) is the dilution effect should an isolated pollution event occur, thus reducing the risk of aquifer contamination.

12.3.4 Injection Shutdown System

Controls should be incorporated to shut down an injection pump or valve if any of the following exceed the criteria for the environmental values of the aquifer:

- standing water level in the well
- injection pressure
- electrical conductivity (salinity)
- turbidity
- temperature
- pH
- dissolved oxygen concentrations
- volatile organics
- other pollutants likely to be present in injectant water that can be monitored in real time.

12.3.5 Maintenance and Contingency Plans

Protection of the treatment and detention system from contamination is a necessary part of the design in ASR systems. This includes constructing treatment systems away from flood-prone land, taking care with or avoiding the use of herbicides and pesticides within the surrounding catchment, planting non-deciduous vegetation, and preventing mosquitoes and other pests breeding in the storage pond.

Contingency plans should be developed to cater for the possibility of contaminated water being inadvertently injected into the aquifer. These include how to determine the duration of recovery pumping (to extract contaminated water), what sampling intervals are needed and how to manage recovered water.

12.3.6 Recovered Water Post-treatment

For drinking water supplies, recovered water may need to be treated, e.g. using ultraviolet disinfection. For some other forms of supply, such as irrigation via drippers, it may be necessary to insert a cartridge filter.

12.3.7 Discharge of Well Development/Redevelopment Water

In the development of wells for use in an ASR system, the well needs to be “developed”, that is, it needs to be purged for a period of time to remove poor quality water that may have been created as part of the construction of the well. Usually this water is high in fine sediment and as such must not be disposed of to a water body or a watercourse unless it is of suitable quality. It may be used on site, possibly for irrigation, discharged to the sewer (with the approval of the relevant authority), or returned to a treatment system.

12.3.8 Groundwater Attenuation Zones

In some cases the impact of certain ground water pollutants can be diminished over time due to natural processes within the aquifer. Chemical, physical and microbiological processes can occur to ameliorate the harm or potential harm caused by these pollutants.

12.4 Quality of water for injection and recovery

The selection of a storage aquifer and the quality of water that can be injected will be determined by a Water Quality Policy if the relevant agency (e.g. EPA, water authorities).

Designated environmental values of the recovered water, such as raw water for drinking, stock water, irrigation, ecosystem support and groundwater ecology are determined from:

- ambient groundwater quality, with reference to the National Water Quality Management Strategy (Australian Drinking Water Guidelines 1996, NHMRC & ARMCANZ; Australia & New Zealand Guidelines for Fresh and Marine Water Quality 2000, ANZECC & ARMCANZ)
- local historical and continuing uses of those aquifers

Artificial recharge should improve or at least maintain groundwater quality.

12.5 Domestic scale ASR

It is also possible to install an ASR scheme at the domestic scale. Generally they are subject to the same considerations as larger scale design, however being smaller systems they are likely to be shallower and therefore additional considerations are required.

It is recommended that domestic scale ASR in shallow aquifers not be undertaken in locations where water tables are already shallow (less than 5 m) or in areas where:

- saline groundwater ingress to sewers occurs
- water tables could rise to within 5 m of the soil surface as a result of ASR in areas of expansive clay soils
- other structures such as cellars or basements could be adversely impacted by rising water tables

- dryland salinity is an issue in the local catchment

The water recharged must be of the highest possible quality, equivalent to roof runoff after first flush bypass, such as overflow from a rainwater tank, and must be filtered to prevent entry of leaves, pine needles and other gross pollutants into a well.

Runoff from paved areas must not be admitted, unless this has first passed through a treatment measure (as described in previous chapters) to reach the required quality for injection.

An inventory should be made of other potential pollutants in the well's catchment and strategies devised to ensure these are excluded from the well, or are treated and removed before water enters the well.

The aquifer pressure must at all times be below ground level. To achieve this, injection should be by gravity drainage into the well, rather than by using a pressurised injection system, and there should be an overflow facility, e.g. to a garden area where excess water discharges to or to the urban stormwater drainage system.

12.6 Additional information

This chapter provides a brief introduction into ASR and the considerations required to assess feasibility. Considerably more investigations and consultation are required to determine the functional details of a possible ASR system.

There are some Australian guidelines available for ASR systems (particularly from South Australia where there is considerable experience with these systems) as well a Victorian Guideline for ASR being developed as part of the Smart water fund. Some relevant guides and websites for further information are listed below.

- Environment Protection Authority (South Australia), 2004, Code of Practice for Aquifer Storage and Recovery (www.environment.sa.gov.au/epa/pdfs/cop_aquifer.pdf)
- Dillon, PJ & Pavelic, P 1996, 'Guidelines on the quality of stormwater and treated wastewater for injection into aquifers for storage and reuse', *Research Report No 109*, Urban Water Research Association of Australia.

12.7 Web sites

Aquifer Storage Recovery
www.asrforum.com

International Association of Hydrogeologists—Managing Aquifer Recharge (IAH-MAR)
www.iah.org/recharge/

CSIRO water reclamation project in Australia
www.clw.csiro.au/research/catchment/reclamation/

Smart water fund

www.smartwater.com.au

Environment Protection and Heritage Council (EPHC)
www.ephc.gov.au/index.html

Department of Water, Land and Biodiversity Conservation (regarding licensing requirements)
www.dwlbc.sa.gov.au

Chapter 13 Other Measures

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13.1 Introduction

There are a range of ‘other’ stormwater management and treatment measures that can be considered as part of the available toolkit for the WSUD practitioner. These ‘other’ measures are either proprietary devices or non-mainstream measures. As such no detailed design procedures have been prepared for them. The following sections of this chapter provide general guidance on the characteristics of these additional techniques for review and further consideration by interested designers of a WSUD oriented project.

The techniques that are discussed include the following:

- subsurface wetlands
- proprietary products
- porous pavements
- use of natural areas including reforestation and revegetation.

13.2 Sub-surface wetlands

The discussion presented in this section relates to sub-surface flow wetlands (commonly referred to as reed beds) in which the flow to be treated passes through a porous media such as sand or gravel which under lies the wetland which supports emergent type vegetation. The purpose of the vegetation is to provide some oxygen to the root zone.

Sub-surface wetlands are typically applied in wastewater treatment systems where there is a relatively consistent influent flow rate. To date in Australia, there have been few, if any applications of these techniques in the stormwater field, though there are obvious overlaps between a porous media, planted bioretention system and a vertical subsurface flow wetland.

One of the major issues associated with the use of subsurface flow wetlands in a stormwater treatment context relates to the highly episodic nature of stormwater events. A subsurface wetland would require considerable volumes of balancing/detention storage above it to attenuate stormwater inflows. There may also be problems with the subsurface wetlands excessively drying under prolonged low rainfall conditions with associated losses of algal and microbial slime layers.

Figure 13.1 provides an example of indicative cross-sections of both free surface and subsurface wetlands (source Queensland Department of Natural Resources and Mines 2000). The ‘free surface’ wetland illustrated in Figure 8.1 is addressed in Chapter 8 of this Manual.

In the context of WSUD, smaller reed bed systems may be better suited to the treatment of greywater for reuse in areas with limited rainwater supply or where maximum rainwater harvest has already been utilised for internal applications and an external demand (garden watering) exists.

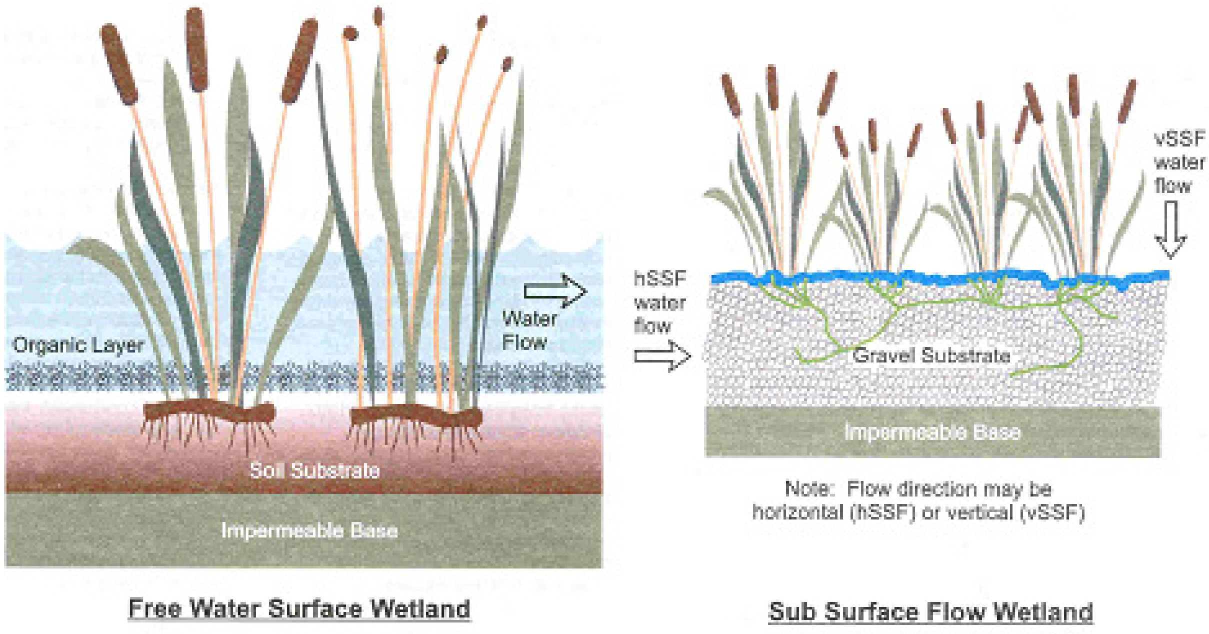

Figure 13.1 Major Types of Constructed Wetlands

Table 13-1 Advantages and disadvantages of sub-surface wetlands

Advantages	Disadvantages
Significant ability to treat high organic loads (see note 1)	Intermittent stormwater flows may adversely affect treatment
High cold weather tolerance	Higher capital cost, associated with media supply
Greater treatment per unit area when compared to free surface wetlands	They can be prone to blockage, particularly at the inlet zones (see note 2)
Mosquitoes and odours are generally not a problem	They are limited to smaller pollutant loadings.
There are no public safety issues as the wetland is not a body of open water	
Re-suspension of sediment due to wind, birdlife etc. is eliminated (unlike surface wetlands)	
Horizontal flow paths through porous media require only mild hydraulic gradients (hence long detention times)	
There are minimal harvesting needs.	

Note 1 –

The environment within a subsurface wetland is mostly anoxic or anaerobic. Some oxygen is supplied to the root zone that is likely to be used up in the biomass growing there rather than penetrate too far into the water column and, for this reason, subsurface wetlands are effective in denitrification.

Note 2 –

A frequently reported problem with subsurface wetlands is blockage of the inlet zones which then leads to short circuiting and surface flow. Attention needs to be given to good inflow distribution and the placement of larger aggregate within this inlet zone. Inlet apertures need to be large enough to avoid being blocked by algal growth and designs should aim to facilitate regular inspections for maintenance purposes.

Primary design criteria for subsurface flow wetlands are as follows:

- detention time
- organic loading rate
- hydraulic loading rate
- media size
- bed depth
- aspect ratio.

Typical overseas design criteria for wastewater subsurface wetlands (based on Crites and Tchobanoglous 1998) are provided in Table 13-2.

Table 13-2 Typical Design Criteria & Expected Effluent Quality for Subsurface Wetlands

Item	Value
Detention Time	3-4 days
BOD loading	0.01 kg/m^2/d
SS loading (see note)	0.04 kg/m^2/d
Water Depth	up to 0.6 m
Media Depth	up to 0.75 m
Harvesting	Limited
Expected Effluent Quality:	
BOD	less than 20 mg/L
SS	less than 20 mg/L
TN	less than 10 mg/L
TP	less than 5 mg/L

Note: For wetland length to width ratio greater than 4:1, the influent suspended solids loading may be a concern. To avoid entry zone blockages, suspended solid loadings should not exceed 0.08 kg/m^2/d. (Bavor, et al 1989)

13.3 Proprietary stormwater treatment devices

In the development of a WSUD treatment train for a site, there is an extensive array of proprietary products that are available for consideration. Such products usually take a primary treatment role, removing gross pollutants and litter before other devices (as described in the earlier design procedures in this report) address the fine sediment, nutrient and pathogen content of urban stormwater. However, there are also products available for sedimentation, spill controls, oil separation and fine filtration.

Given the diversity of forms and configurations of these proprietary devices and, in some cases the confidential nature of their design and performance data, it is not possible in this document to provide any more than general guidance as to the issues that should be considered when selecting such devices. We also provide some guidance as to those factors which should be considered when reviewing performance values often ascribed to such devices by suppliers.

Where proprietary GPTs are used in isolation and not as part of a treatment train, an important consideration is the nature of captured material storage of the device. Evidence exists that suggests concentrations of some pollutants in the water column are increased when captured material is stored in a wet sump device for extended periods.

13.3.1 Selection Issues

Australian Runoff Quality (Engineers Australia 2006) provides guidance as to those issues which should be considered when selecting a gross pollutant trap, or GPT. Such devices constitute the majority of proprietary products. The following summary of key selection issues has been developed on the basis of the ARQ advice.

A decision of which type (and brand) of proprietary device to select is a trade-off between the life cycle costs of the device (i.e. by combining capital and ongoing costs), expected pollutant removal performance in regard to the values of the downstream waterbody and social considerations.

A life cycle cost approach is recommended. This approach allows the ongoing cost of operation to be considered and the benefits of different devices to be assessed over a longer period. The overall cost of a proprietary device is often determined more by the maintenance costs rather than the initial capital costs.

The expected pollutant removal rate is a function of the amount of runoff treated (i.e. the quantity of flow diverted into a proprietary device compared to that which by-passes) and the pollutant removal rate for flows that go through a proprietary device.

This section highlights some issues that should be considered as part of the decision making process. The issues raised are primarily based on experience with existing proprietary device installations.

13.3.1.1 Life cycle costs

Life cycle costs are a combination of the installation and maintenance costs and provide an indication of the true long-term cost of the infrastructure. It is particularly important to consider life-cycle costs for proprietary devices as maintenance costs can be significant compared to the capital costs of installation.

To determine life cycle costs, an estimated duration of the project (ie. lifespan of the treatment device) needs to be assumed (e.g. 20 or 25 years). If the device is to control pollutants during the development phase only (for example a sediment trap) its lifecycle may be only 3-10 years.

Life cycle costs can be estimated for all devices and then, with consideration to the other influences (expected pollutant removal, social, etc.), the most appropriate device can be selected.

13.3.1.2 Installation costs and considerations

Installation costs include the cost of supply and installation of a proprietary device. These prices should be evident on proposals for proprietary device installations but it should be checked that all installation costs are included. Variables in terms of ground conditions (such as rock or groundwater conditions) or access issues may vary construction costs significantly and cost implications of these should be assessed. The likely occurrence of these issues should be weighed up when estimating an overall installation cost.

Issues that should be checked to be addressed by tenderers include:

- price is for supply and installation (not just supply)
- provision for rock or difficult ground conditions
- proximity to services (and relocation costs)
- required access and traffic management systems for construction.

A true installation cost should then be used when estimating life cycle costs.

As important as obtaining a true installation cost is ensuring the device will suit local conditions. Issues that should be assessed to ensure a proprietary device will suit an area include:

1. the size of the unit;
2. hydraulic impedance caused by the device; and
3. particular construction issues.

More details of the points to consider are outlined on the following pages.

Size of the unit (footprint, depth)

The sizes of proprietary devices vary considerably and this will need to be accommodated by the potential location for the device. Things to consider when assessing the size of a device include:

- required footprint (plan size of device and any required flow diversion);
- depth of excavation (to the bottom of the sump in some cases) – rock can substantially increase installation costs;
- sump volume required (where applicable);
- proximity to groundwater; and
- location of any services that impact construction and likely cost for relocation (e.g. power, water sewer).

Hydraulic impedance/ requirements

Some proprietary devices require particular hydraulic conditions in order to operate effectively, for example some devices require a drop in a channel bed for operation. Requirements such as these can affect which devices may or may not be suitable in a particular area.

Other considerations are possible upstream impacts on flow and a hydraulic gradeline because of the installation of the device. This can increase flooding risks and all devices should be designed to not increase the flooding risk during high flows. Therefore, if a device increases the flooding risk above acceptable limits it may not be considered further.

Other construction issues

For each specific location there will be a number of other considerations and points of clarification that may sway a decision on which device may be the most suitable, these include:

- Does the cost include any diversion structures that will be required?
- Is specialist equipment required for installation (e.g. special formwork, cranes or excavators) and what cost implications do these have?
- Is particular below ground access required, will ventilation and other safety equipment be needed – at what cost?
- Will the device impact on the Aesthetics of an area – will landscape costs be incurred after the device installation – if so how much?
- Will the device be safe from interloper or misadventure access?
- Do the lids/covers have sufficient loading capability (particularly when located within roads) – what is the cost of any increase in load capacity and will it increase maintenance costs?

- Will the device be decommissioned (e.g. after the development phase) and what will this cost – what will remain in the drainage system?
- Are there tidal influences on the structure and how will they potentially affect performance or construction techniques?
- Will protection from erosion be required at the outlet of the device (particularly in soft bed channels), and what cost implications are there?

13.3.2 Maintenance costs and considerations

Maintenance costs can be more difficult to estimate than the installation costs (but are sometimes the most critical variable). Variation in the techniques used, the amount of material removed and the unknown nature of the pollutants exported from a catchment (thus disposal costs) all influence maintenance costs. It is therefore imperative to carefully consider the maintenance requirements and estimate costs when selecting a proprietary device. As part of a tender process, tenderers should be asked to quote annual maintenance prices, based on the relevant site conditions (not just generic estimates).

One important step is to check with previous installations by contacting the owners and asking their frequency of cleaning and annual operation costs (vendors can usually supply contact information).

All maintenance activities should be developed so that they that require no manual handling of collected pollutants because of safety concerns with hazardous material.

Below is a list of maintenance considerations that should be applied to all proprietary devices.

- Is special maintenance equipment required? E.g. large cranes, vacuum trucks or truck-mounted cranes. Does this equipment need to be bought or hired – at what cost?
- Is special inspection equipment needed (e.g. access pits)?
- Are any services required (e.g. wash-down water, sewer access)?
- Are there overhead restrictions such as power lines or trees?
- Does the water need to be emptied before the pollutants – if so how will it be done, where will it be put and what will it cost?
- Can the device be isolated for cleaning (especially relevant in tidal areas)?
- Are road closures required and how much disturbance will this cause?
- Are special access routes required for maintenance (e.g. access roads or concrete pads to lift from) – and what are these likely to cost?
- Is there a need for dewatering areas (e.g. for draining sump baskets) and what implications will this have?

Disposal costs

Disposal costs will vary depending on whether the collected material is retained in wet or free draining conditions in the proprietary device. Handling of wet material is more expensive and will require sealed handling vehicles.

- Is the material in a wet or dry condition and what cost implications are there?
- Are there particular hazardous materials that may be collected and will they require special disposal requirements (e.g. contaminated waste –what cost implications are there?
- What is the expected load of material and what are likely disposal costs?

Occupational health and safety

- Is there any manual handling of pollutants and what will safety equipment cost?
- Is entering the device required for maintenance and operating purposes – will this require confined space entry? What cost implications does this have on the maintenance cycle (for example, minimum of three people on site, safety equipment such as gas detectors, harnesses, ventilation fans and emergency oxygen)?
- Are adequate safety features built into the design (e.g. adequate step irons and inspection ports) or will these be an additional cost?

Miscellaneous considerations

Social considerations can be an important component of the selection of a proprietary device. Consultation with key stakeholders is fundamental to selecting an appropriate proprietary device. Influences on the decision process may include:

- Potential odour concerns at a location
- Likelihood of pests and vermin such as mosquitoes or rats
- Suitability of the proprietary device materials, particularly in adverse environments (e.g. marine)
- Impact on the Aesthetics of an area
- Education and awareness opportunities
- Potential trapping of fauna (e.g. turtles, eels and fish).

These issues should be considered early in the selection process and taken into account when finalising a proprietary device type.

Checklist for selecting proprietary products

The following checklist is reproduced from Engineers Australia (2006) and provides guidance on issue to consider when selecting proprietary stormwater treatment products.

13.3.3 Checklist for selecting a proprietary stormwater treatment device

	YES	NO
1. GENERAL		
• Is there available space for the device (ie. required footprint, access routes, services)?	☐	☐
• Does the location suit catchment treatment objectives (e.g. position in a 'treatment train')?	☐	☐
• Is the pollutant holding chamber suitable (wet or dry retention)?	☐	☐
• Are there sufficient safety precautions (ie. preventing entry, access for cleaning)?	☐	☐
• Is the visual impact satisfactory (and odour potential)?	☐	☐
• Is the treatment flow sufficient to meet treatment objectives?	☐	☐
• Has the flooding impact being demonstrated to be satisfactory?	☐	☐
• Has sufficient consultation taken place with operation staff and affected locals?	☐	☐
• Is the expected pollutant removal rate sufficient to meet treatment objectives (consult with owners of existing installations if required)?	☐	☐
2. INSTALLATION		
• Does the price include installation?	☐	☐
• Are there sufficient contingencies for ground conditions (e.g. rock, shallow water table, soft soils etc.)?	☐	☐
• Have relocation of services being included?	☐	☐

- Are there sufficient access or traffic management systems proposed as part of construction? ☐ ☐

What are the cost implications of the above points?
$_____________

3. MAINTENANCE

- Is the method of cleaning applicable to local conditions (eg, OH&S issues, isolation of the unit from inflows etc.)? ☐ ☐
- Are the maintenance (cleaning) techniques suitable for the responsible organisation (ie. required equipment, space requirements, access, pollutant draining facilities etc.)? ☐ ☐
- Is a maintenance contract included in the proposal? ☐ ☐
- **Is the size of the holding chamber sufficient?** ☐ ☐
- Have disposals cost being accounted for? ☐ ☐

What are the cost implications of the above points?
$_____________

13.3.4 Performance Issues

When considering the adoption of a proprietary device for a particular site, as well as the selection issues addressed above, it is recommended that consideration be given to how the device will perform, especially in respect to the levels of performance which are often attributed to such devices by their suppliers.

In this regard, it is recommended that consideration be given to the following key issues (Auckland Regional Council 2003).

- Whether the operating parameters of the system have been verified.
- Existing or proposed monitoring data.
- Documentation of processes by which pollutants will be reduced (physical, chemical biological).
- Documentation and/or discussion of potential causes of poor performance or failure of the device.

- Key design specifications or considerations.
- Specific installation requirements.
- Specific maintenance requirements.
- Data to support claimed pollutant removal efficiencies. If the device is new or the existing data is not considered reliable, such data should be viewed with caution.

13.4 Porous pavements

A recent Australian review of available data on porous pavements, combined with advice on maintenance and operational issues, is contained in Fletcher et al (2003). The following material has been reproduced from this publication (with the permission of the lead author).

13.4.1 Description

Porous pavements, as their name implies, are a pavement type that promotes infiltration, either to the soil below, or to a dedicated water storage reservoir below it. Porous pavements come in several forms (Figure 13.2), and are either monolithic or modular. Monolithic structures include porous concrete and porous pavement (asphalt). Modular structures includes porous pavers (which may be either made of porous material, or constructed so that there is a gap in between each paver), modular lattice structures (made either of concrete or plastic). Porous pavements are usually laid on sand or fine gravel, underlain by a layer of geotextile, with a layer of coarse aggregate below. Design should ensure that the required traffic load can be carried.

An advantage of modular pavers is their ability to be lifted, backwashed and replaced when blockage occurs. Pavers that are porous from the use of gaps between individual pavers should be carefully chosen with reference to likely catchment inputs, such as leaves and debris that can quickly block the gaps.

Porous pavements should generally be located in areas without heavy traffic loads. In high traffic areas the loads of pollutants can significantly decrease the ability to remain porous. Consideration of the maintenance advantages of modular pavers should also be considered, given that the consequence of blockage with monolithic material

Porous pavement has two main advantages over impervious pavement, in terms of stormwater management:

1. Improvement to water quality, through filtering, interception and biological treatment
2. Flow attenuation, through infiltration and storage.

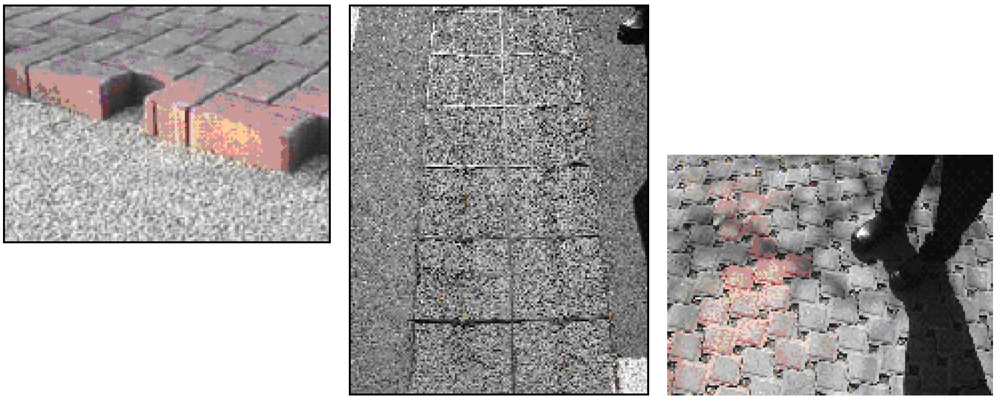

Figure 13.2 Examples of Porous Pavement

13.4.2 Studies of performance

Investigations into the performance of porous pavements have investigated (a) water quality and (b) flow effects.

13.4.2.1 Flow behaviour

Porous pavements can potential reduce peak flow rate, and total flow volume, the individual or combined effect of initial loss, infiltration, storage and evaporation. The level of flow attenuation is dependent in part on (where appropriate) the amount of storage, and the infiltration capacity of the porous pavements, its underlying base material (including any underlying geotextile), and the soil below.

13.4.2.2 Water quality behaviour

Porous pavements act to improve water quality through a number of mechanisms:

- filtering through the pavement media, and underlying material;
- potential biological activity within the pavement and base material; and
- reduction of pollutant loads, as a result of reduced runoff volumes.

Observed behaviour is likely to be a function of the particular storm event (its magnitude and intensity), the input concentration, and the characteristics of the pavement media and underlying filter material.

Importantly, since contaminants such as heavy metals and hydrocarbons are often attached to sediment, the filtering behaviour acts not only to reduce sediment loads, but those of associated contaminants. Because of the ability of porous pavement to provide an initial rainfall loss, runoff from porous pavement is less likely to have the oft-observed ‘first-flush’ effect, where greatly elevated pollutant concentrations are observed in the first part of a storm.

13.4.3 Summary of expected performance

Based on the studies of flow performance reviewed by Fletcher et al (2003), and contingent upon the properties and condition of the porous pavement and its subsoil, a reduction in runoff coefficient from around 0.95 for traditional pavements, to around 0.40 can be expected. However, the expected hydraulic performance of any porous pavement can be easily modelled, either for a single rainfall event (using a spreadsheet-approach), or using a rainfall-runoff model, such as that provided in MUSIC, for a real (or synthetic) rainfall series.

Based on the studies of water quality performance reviewed Fletcher et al (2003), the pollutant removal by porous pavement appears to be relatively consistent. However, this finding should be viewed with some caution, because it may reflect at least in part the lack of studies which have specifically reported on performance relative to input variables, such as inflow concentration, hydraulic loading, and properties of the pavement.

Table 13.3 provides a summary of expected performance of porous pavements, based on the studies reviewed here.

13.4.4 Maintenance

Porous pavements are permeable pavement with an underlying storage reservoir filled with aggregate material. Modular block pavements (including lattice block pavements) or permeable pavements overlie a shallow storage layer (typically 300mm – 500mm deep) of aggregate material that provides temporary storage of water prior to infiltration into the underlying soils. Maintenance activities vary depending on the type of porous pavement. In general, porous pavement should be inspected for cracks and holes, and removal of accumulated debris and sediment should be undertaken every three to six months. Depending on the design of lattice pavements, weeding or grass mowing may need to be undertaken. If properly maintained, and protected from 'shock' sediment loads, porous pavements should have an effective life of at least 20 years (Bond et al, 1999; Pratt, 1999; Schluter et al., 2002 as cited in Fletcher et al, 2003).

Table 13-3 Summary of Expected Porous Pavement Performance

Pollutant	Expected concentration reduction (+ range)	Comments
Total Suspended Solids	80 (70-100)	
Total Nitrogen	65 (60-80)	Will decrease with proportion dissolved
Total Phosphorus	60 (40-80)	Will decrease with proportion dissolved
Hydrocarbons/Oils/Grease	85 (80-99)	Depends on level of microbial activity.
BOD	-	Inadequate data
Pb, Cu, Cd, Zn, Ni	75 (40-90)	Will decrease with proportion dissolved
Litter	-	Litter will simply 'wash off'
Pathogens	-	Inadequate data

Table 13-4 Porous Pavement Maintenance Issues

Design Category	Maintenance Activities and Frequency	Equipment	Design Attributes That Facilitate Maintenance Activities
Modular Block **or** Lattice Pavement **or** **Permeable Pavements**	Maintenance activities for porous pavements should be undertaken every 3 to 6 months and may include: • Inspection of pavement for holes, cracks and excessive amounts of accumulated materials • Removal of accumulated debris and sediment on surface of pavements • Hand weeding largely for Aesthetic purposes • Mowing of grass if used between lattice pavements • Periodical removal of infiltration medium (about every 20 years) and replacement of geo-textile fabric to ensure permeability is maintained to the underlying soils	• High suction vacuum sweeper and high pressure jet hoses • Gloves, spade, hoe • Lawn mower and waste removal vehicle • Bobcat or excavator and waste removal vehicle (such as, tipper truck)	• Separate the upper 300mm of using geo-textile fabric for easy removal and replacement of upper component • Recommended for **low traffic volume** areas only • Recommended for use in **low sediment loading areas** • Invert of system should be at least 1m above impermeable soil layer and seasonal high water Table • Allowance should be made for a 50% reduction in design capacity over a 20 yr lifespan

13.4.5 Capital costs and maintenance costs

The capital cost of porous pavements is disputed, with conflicting estimates given, but consensus is that its cost is similar to that traditional pavement, when the total drainage infrastructure cost is taken into account Landphair et al., (2000). This conclusion is supported by a trial of several types of porous pavements, based on real case studies in the Puget Sound (http://www.psat.wa.gov/Publications /LID_studies/permeable_pavement.htm; 26/08/03). The long-term maintenance costs remain relatively unknown, with no reliable Australian data available.

Some estimates of porous pavement costs were provided at a workshop run by "Water Sensitive Urban Design in the Sydney Region" (www.wsud.org) in March 2003 (no maintenance costs were provided):

- permeable paving allowing infiltration: AU$111/m^2;
- permeable paving over sealed subgrade, allowing water collection: AU$119/m^2;
- permeable paving with concrete block paving: AU$98/m^2 with infiltration, AU$122/m^2 with water collection;
- permeable paving with asphalt: AU$67/m^2 with infiltration or AU$80/m^2 with water collection; and
- permeable paving with concrete block: AU$90/m^2 with infiltration, AU$116/m^2 with water collection.

The Californian Stormwater Quality Association (www.cabmphandbooks.com) have produced a handbook for best practice stormwater management in new development and re-development (http://www.cabmphandbooks.com/Documents/Development /SD-20.pdf). The report draws on research undertaken by Landphair et al., (2000), who reported annual maintenance costs of approximately AU$9,700 per hectare per year. Little information was given on what basis this was calculated. Based on amortized construction and maintenance costs over 20 years, equated to around AU$9 per kg of TSS removed, inc. Landphair et al., also lament the lack of lifecycle cost data for stormwater treatment measures, including porous pavements, and point out that both construction and maintenance costs are very site-specific; whilst some local data may be available, there are not the cost-relationships which allow maintenance costs to be predicted for any given site.

13.4.6 Protection and maintenance of porous pavements

Along with evidence of many successful implementations of porous pavements, there are many instances of failure, because of clogging. *It is absolutely critical that porous pavements are protected from large sediment loads during and shortly after the construction phase.* Failure to do so could see the effective lifespan of the pavement reduced by less than 10% of the predicted lifespan.

13.4.7 Design and supply of porous pavements

There are a number of suppliers of both monolithic and modular porous pavement systems within Australia (although for commercial equality reasons they are not listed here). When seeking information from suppliers on their products, the following information should be sought:

- Cost/ m^2, including supply and installation (taking into account site conditions)
- Required depth of installation and details of the sub-base, geotextile and associated components
- Maintenance requirements and pollutant collection processes for particular pavement
- Independent performance data (infiltration capacity and pollutant removal)
- Potential for application of porous pavement for only part of the paved surface (and impacts on infiltration and pollutant removal performance).

13.5 Use of natural areas including reforestation and revegetation.

Another technique considered worthy of consideration is the use of reforestation and revegetation measures. The following text, largely based on material contained in Auckland Regional Council (2003) should provide initial guidance to practitioners in this regard.

This technique involves the utilisation of existing areas of vegetation, from forested areas to scrub vegetation to pasture areas. The scale of this approach can be made to vary. In a micro sense, redirecting pathway and driveway stormwater runoff onto adjacent grassed or otherwise vegetated areas (also referred to as the minimisation of directly connected impervious areas), illustrates this concept of natural area use. All such opportunities should be considered where redirection can be done without causing problems, such as concentrated flow increasing slope erosion.

For those situations where vegetation already exists, use of that vegetation or enhancement of the vegetation is a good approach. Significant benefits can be gained also by reforesting or revegetating portions of sites that would improve an existing situation or restore a degraded resource.

Reforestation/revegetation includes the planting of appropriate tree and shrub species, coupled with the establishment of an appropriate ground cover around trees and shrubs in order to stabilise soil and prevent an influx of invasive plants and weeds. The practice is highly desirable because, in contrast to many other management approaches, reforestation actually improves in its stormwater performance over time.

Reforestation benefits relate closely to benefits cited in the literature on riparian stream buffer protection, although reforestation is not linear in configuration.

Plant species should be selected carefully to match indigenous species that exist in the area and care should be taken to use species reflective of the combination of environmental factors which characterise the area. This enables species which will flourish in an appropriate site, as well as improving ecological health of streams and natural areas in the wider context.

Reforestation areas need periodic management, at least for the first five years. This will ensure good survival rates for the newly planted stock. The level of management decreases as the plantings mature. During the first 2-3 years, annual spot applications of herbicide may be necessary around the planted vegetation to keep weeds from outcompeting the new trees and shrubs for water and nutrients.

To the extent that vegetation of different types is already established, the already stabilised natural area offers various physical, chemical, and biological mechanisms which should further maximise contaminant removal as well as attaining water quantity objectives.

13.6 References

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Appendix A: Tasmanian Hydrologic Regions

Appendix A: Tasmanian Hydrologic Regions 1

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A.1 Introduction

Achievable treatment objectives for stormwater quality have been defined in Tasmania. These objectives are expressed as reductions in the mean annual pollutant loads that are discharged from typical urban areas with no stormwater treatments installed (eg. 80% reduction in TSS and 45% reduction in TP and TN). A range of stormwater treatment measures are capable of treating urban stormwater to meet treatment objectives. The design of stormwater treatment measures often requires a continuous simulation approach to properly consider the influence of antecedent conditions of the treatment measure during the occurrence of a storm event and the wide range of storm characteristics and hydraulic conditions that the individual treatment measures are to operate in. Computer models such as MUSIC have been developed to enable continuous simulations of complex stormwater management treatment trains to aid in the development of stormwater management strategies and the design (sizing) of stormwater treatment measures.

This report is adapted for Tasmania from Melbourne Water's "WSUD Engineering Procedures: Stormwater" and builds upon earlier work (described in "Hydrologic Regions for Sizing of Stormwater Treatment in Victoria", October 2003) and its purpose is to develop a simple design procedure that can be used in small development projects (eg. single or a small clustered allotment development type) and can serve as a preliminary design procedure. In addition it could be used as a simple design checking tool.

This methodology stems from the theory that sizing stormwater treatment systems could be based around defining simple empirical design equations that would be applicable in their respective designated Hydrologic Region within Tasmania. This report presents results of developing the empirical relationships and the number of regions

A.2 Methodology

After initial consideration of possible design approaches, the following approach was used to develop the different regions and adjustment factors for sizing stormwater treatment measures throughout Tasmania-

1. Select a rating to represent the effectiveness of different design configurations of various stormwater treatment measures. Reduction in total nitrogen is a logical choice as it is commonly the limiting parameter in meeting best practice stormwater quality objectives.

2. Select a reference site for which detailed investigation and design simulations are undertaken to determine the relation between design configurations (eg. area, extended detention depth, permanent pool volume etc.) of a range of stormwater treatment measures and the

corresponding stormwater quality improvement performance. Hobart was selected as a reference site.

3. Define Hydrologic Regions within Tasmania where practitioners wanting to design a stormwater treatment measure at any location in Tasmania could refer to the design requirements developed for the reference site and apply an Adjustment Factor to that size to determine the appropriate dimensions of the treatment measure for their particular site.

For example, it is determined that in order to meet best practice objectives, a wetland at the reference site must be at least 2% of the contributing impervious area of its catchment. A practitioner designing a wetland of similar configuration in Kingston can simply use an empirical equation to calculate the Adjustment Factor that is then applied to the size of a wetland sized for the reference site.

A.3 Determining hydrologic regions

The Hydrologic Regions for WSUD in Tasmania were determined by selecting a set of pluviographic stations with sufficiently long record to enable continuous simulations of the performance of a number of stormwater treatment measures to be undertaken. A total of forty eight stations were selected for analysis. Figure A.1 show the respective spatial locations of the selected stations.

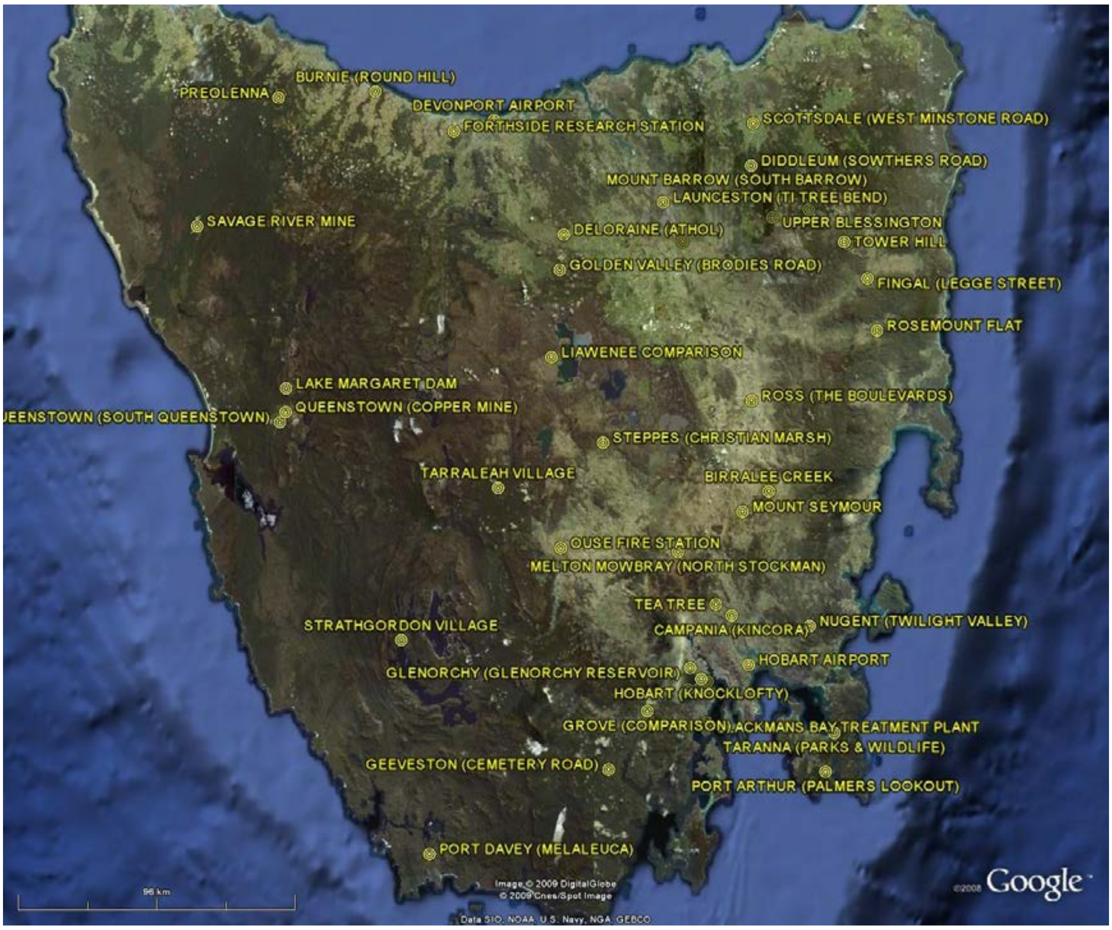

Figure A.1. BOM pluviographic rainfall stations

The mean annual rainfall for the sites selected during the modelled periods ranged from 342 mm to 3173 mm.

Total Nitrogen was selected as the measure for representing the effectiveness of various sized treatment devices. Previous attempts to define the most suitable Hydrologic Region and corresponding predictive equations, the influence of the following factors have been considered in Tasmania and Melbourne:

- mean annual rainfall;
- the ratio of mean summer raindays to mean winter raindays (as a measure of rainday seasonality);

- the ratio of mean summer rainfall to mean winter rainfall (as a measure of rainfall seasonality); and
- site elevation.

The Model for Urban Stormwater Improvement Conceptualisation (MUSIC) developed by eWater was used to simulate the performance of wetlands, bioretention systems, vegetated swales and ponds to size these systems to meet best practice objectives. These sizes were then expressed as the ratio of the size of the treatment area for the reference site. This is thus the Adjustment Factor described in Step 3 in the methodology (see Section 2).

Following extensive testing and analysis of the significance of the above possible influencing factors, it was determined that mean annual rainfall was the most significant influencing factor. For this reason, mean annual rainfall (MAR) has been selected to represent Tasmania with eight Hydrologic Regions.

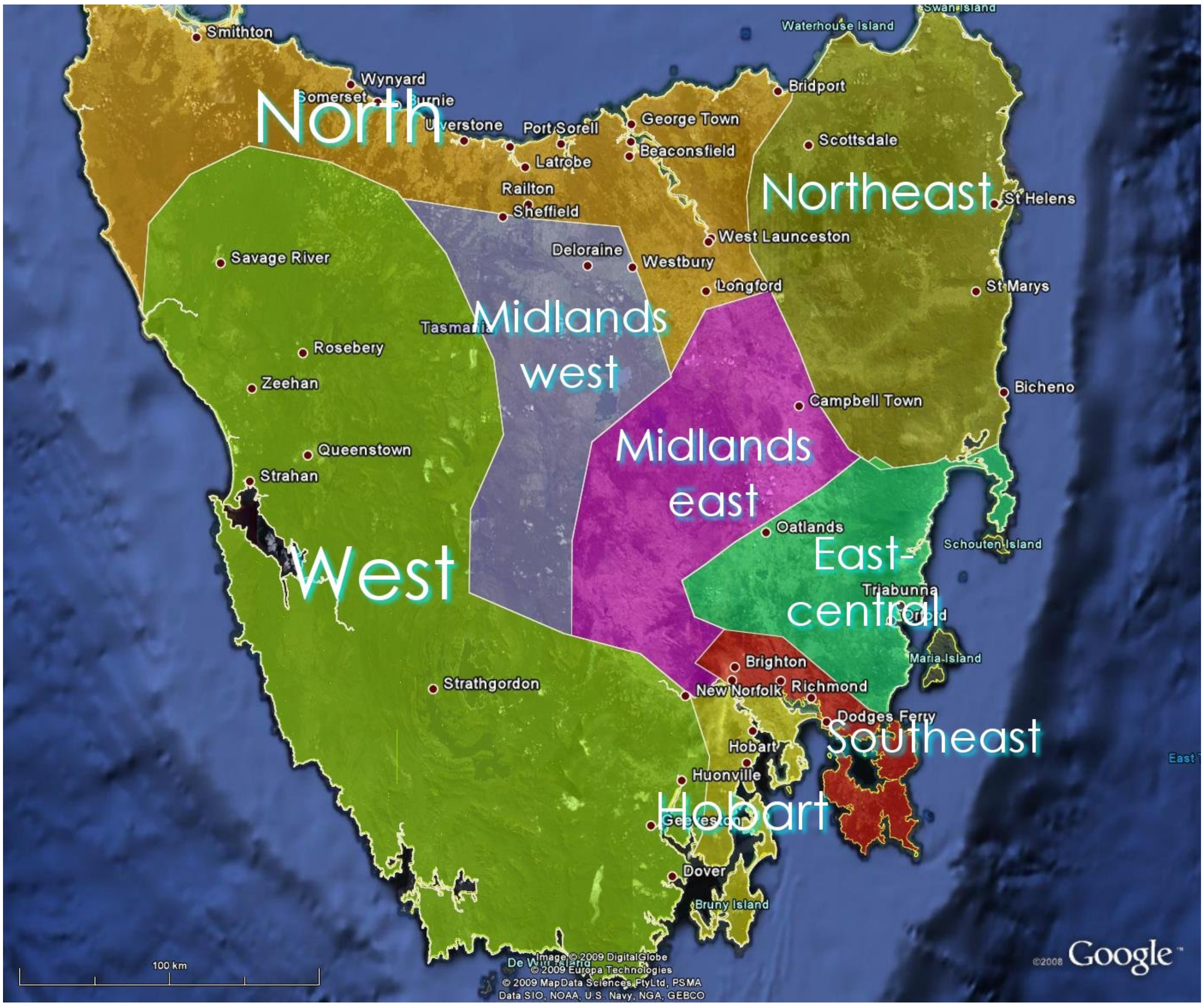

Figure A.2. Tasmanian Stormwater Hydrologic Regions

A.4.1 Hydrological Region Adjustment factors for Tasmania

Adjustment Factors for Tasmania

Region: North

Figure A.3 shows a plot of the *Adjustment Factor*s for bioretention, swales, ponds and wetlands derived against mean annual rainfall for the 'North' Hydrologic Region.. A trend of increasing *Adjustment Factor* with mean annual rainfall is evident for each of the system types.

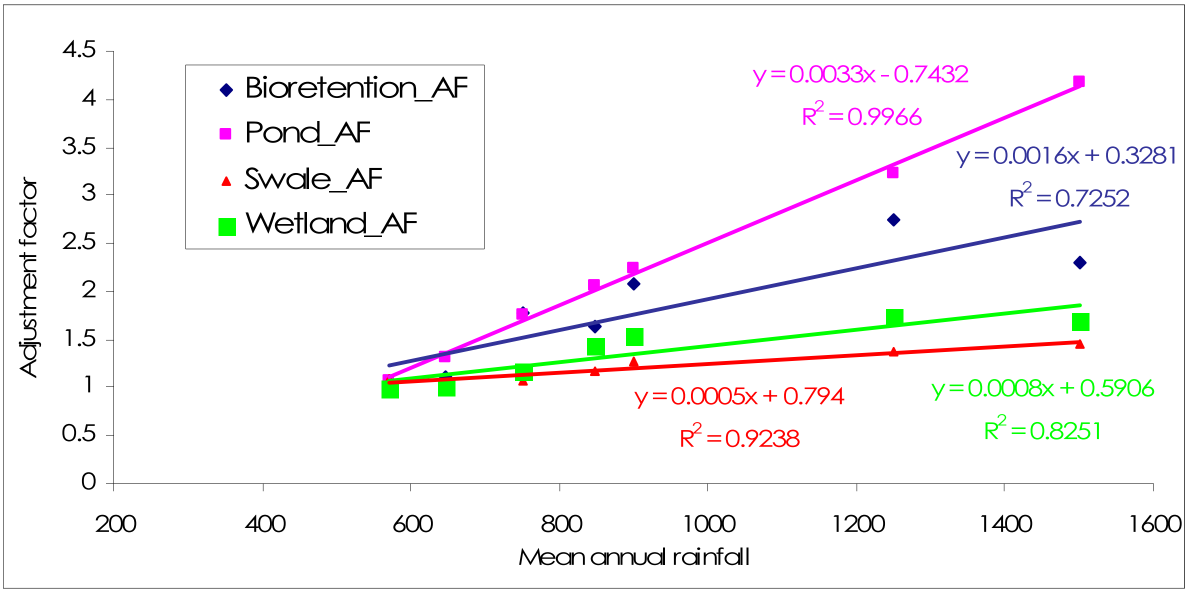

Figure A-1. Plot of *Adjustment Factor* Vs Mean Annual Rainfall (MAR) for the North region

Equations to compute *Adjustment Factors* for each treatment system were obtained by plotting a linear trend (ie 'line of best fit') through for the points on the chart above for each treatment system. The North *Adjustment Factor* equations are shown in the table below.

	North
Bioretention	0.0016(MAR)+0.3281
Pond	0.0033(MAR)-0.7432
Swale	0.0005(MAR)+0.794
Wetland	0.0006(MAR)+0.5906

Region: Northeast

Figure A.4 shows a plot of the *Adjustment Factor*s for bioretention, swales, ponds and wetlands derived against mean annual rainfall for the 'Northeast' Hydrologic Region. A trend of increasing *Adjustment Factor* with mean annual rainfall is evident for each of the system types.

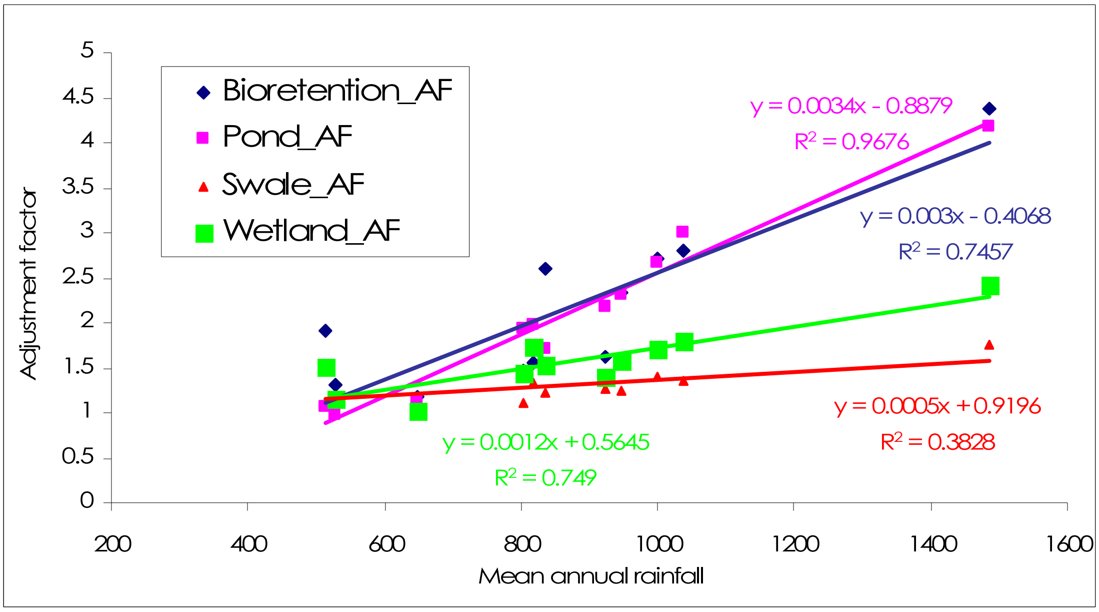

Figure A-2. Plot of *Adjustment Factor* Vs Mean Annual Rainfall (MAR) for the Northeast region

Equations to compute *Adjustment Factors* for each treatment system were obtained by plotting a linear trend (ie 'line of best fit') through for the points on the chart above for each treatment system.

Whilst the coefficient of determination for the swale relationship is lower than others, the lesser slope (m) on the linear curve indicates lower variation in adjustment factor between sites modelled and hence impact of the weaker correlation is not significant.

The Northeast *Adjustment Factor* equations are shown in the table below.

	Northeast
Bioretention	0.0034(MAR)-0.8879
Pond	0.003(MAR)-0.4068
Swale	0.0005(MAR)+0.9196
Wetland	0.0012(MAR)+0.5645

Region: West

Figure A.5 shows a plot of the *Adjustment Factor*s for bioretention, swales, ponds and wetlands derived against mean annual rainfall for the 'West' Hydrologic Region. A trend of increasing *Adjustment Factor* with mean annual rainfall is evident for each of the system types.

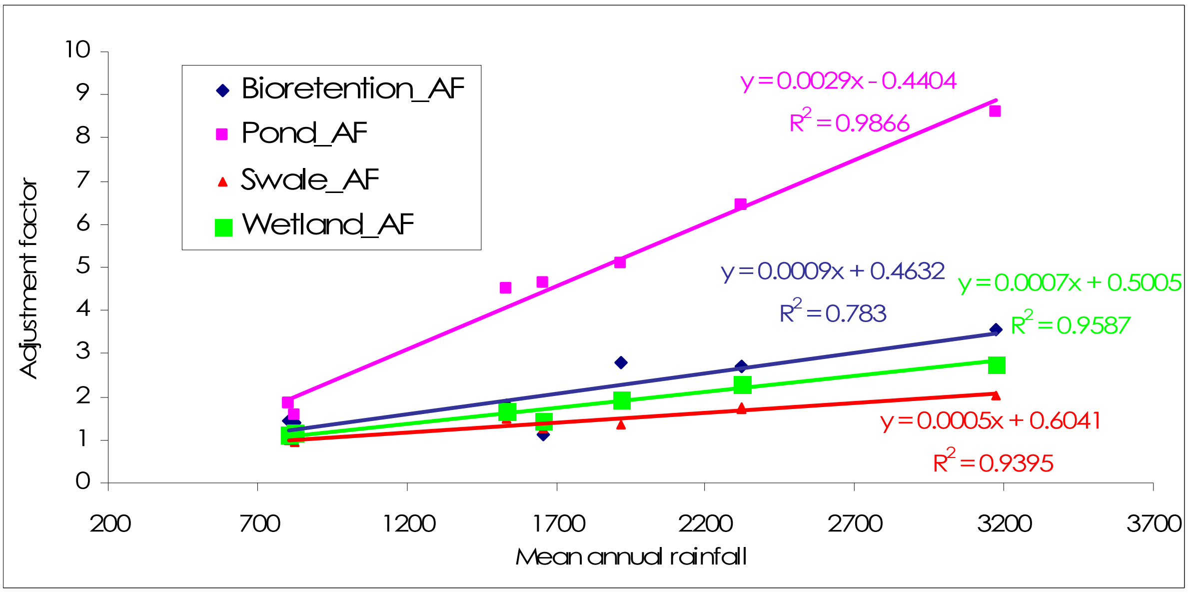

Figure A-3. Plot of *Adjustment Factor* Vs Mean Annual Rainfall (MAR) for the West region

Equations to compute *Adjustment Factors* for each treatment system were obtained by plotting a linear trend (ie 'line of best fit') through for the points on the chart above for each treatment system.

The West *Adjustment Factor* equations are shown in the table below.

	West
Bioretention	0.0009(MAR)+0.4632
Pond	0.0029(MAR)-0.4404
Swale	0.0005(MAR)+0.6041
Wetland	0.0007(MAR)+0.5005

Region: East-central

Figure A.6 shows a plot of the *Adjustment Factor*s for bioretention, swales, ponds and wetlands derived against mean annual rainfall for the 'East-central' Hydrologic Region.. A trend of increasing *Adjustment Factor* with mean annual rainfall is evident for each of the system types.

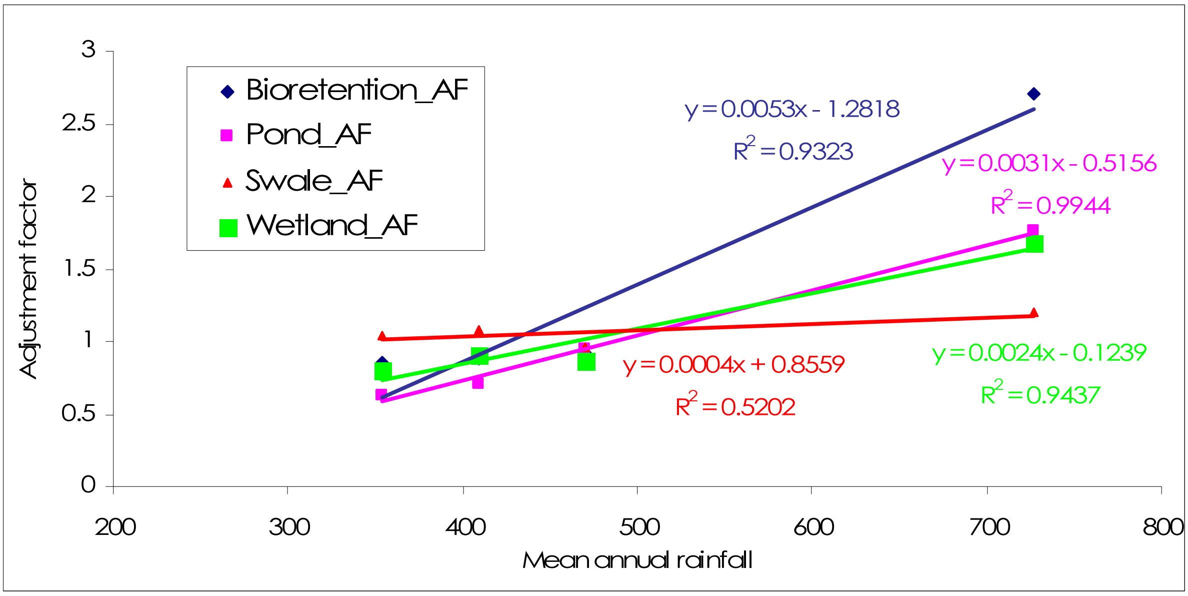

Figure A-4. Plot of *Adjustment Factor* Vs Mean Annual Rainfall (MAR) for the East-central region

Equations to compute *Adjustment Factors* for each treatment system were obtained by plotting a linear trend (ie 'line of best fit') through for the points on the chart above for each treatment system.

Whilst the coefficient of determination for the swale relationship is lower than others, the lesser slope (m) on the linear curve indicates lower variation in adjustment factor between sites modelled and hence impact of the weaker correlation is not significant.

The East-central *Adjustment Factor* equations are shown in the table below.

	East-central
Bioretention	0.0053(MAR)-1.2818
Pond	0.0031(MAR)-0.5156
Swale	0.0004(MAR)+0.8559
Wetland	0.0024(MAR)-0.1239

Region: Southeast

Figure A.7 shows a plot of the *Adjustment Factor*s for bioretention, swales, ponds and wetlands derived against mean annual rainfall for the 'Southeast' Hydrologic Region. A trend of increasing *Adjustment Factor* with mean annual rainfall is evident for each of the system types.

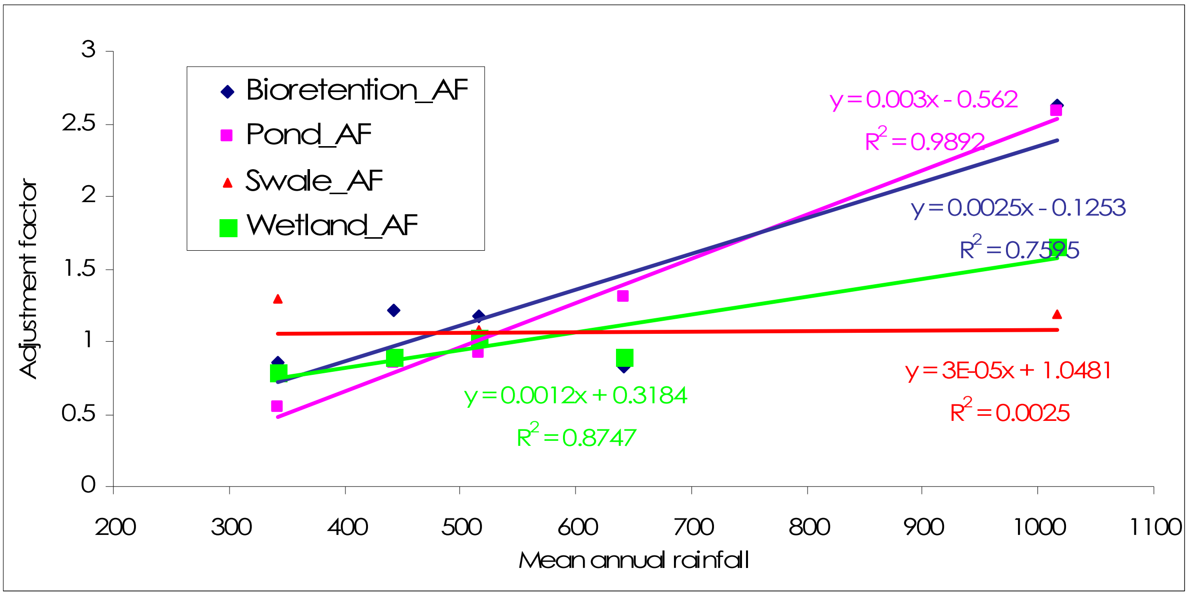

Figure A-5. Plot of *Adjustment Factor* Vs Mean Annual Rainfall (MAR) for the Southeast region

Equations to compute *Adjustment Factors* for each treatment system were obtained by plotting a linear trend (ie 'line of best fit') through for the points on the chart above for each treatment system.

Whilst the coefficient of determination for the swale relationship is lower than others, the lesser slope (m) on the linear curve indicates lower variation in adjustment factor between sites modelled and hence impact of the weaker correlation is not significant.

The Southeast *Adjustment Factor* equations are shown in the table below.

	Southeast
Bioretention	0.0025(MAR)-0.1253
Pond	0.003(MAR)-0.562
Swale	0.00003(MAR)+1.0481
Wetland	0.0012(MAR)+0.3184

Region: Midlands-east

Figure A.8 shows a plot of the *Adjustment Factor*s for bioretention, swales, ponds and wetlands derived against mean annual rainfall for the 'Midlands-east' Hydrologic Region.

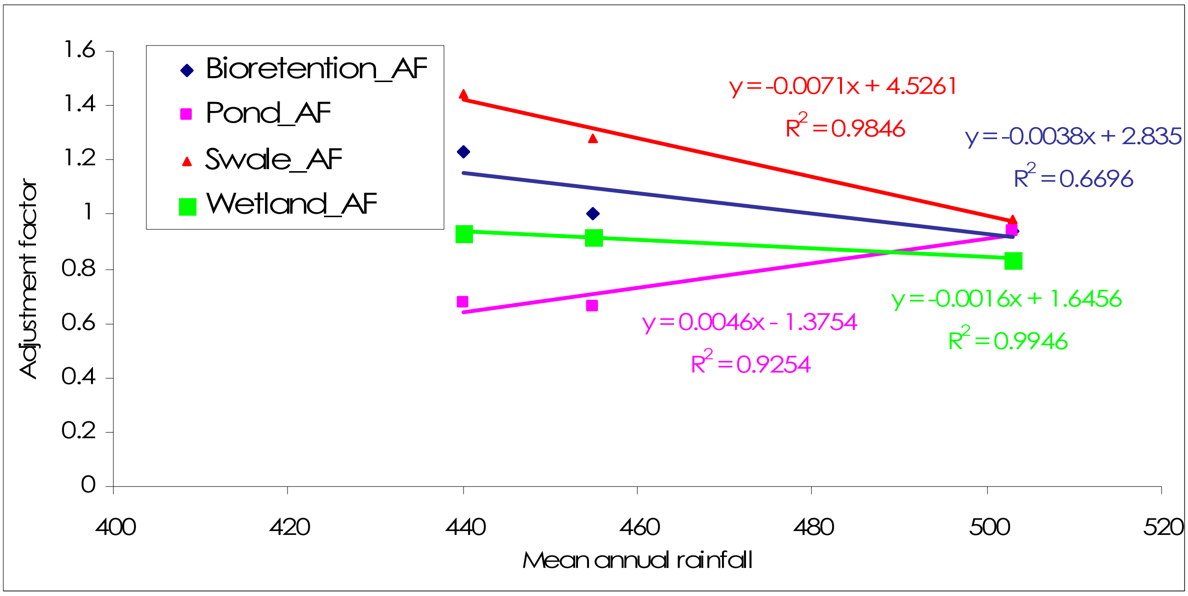

Figure A-6. Plot of *Adjustment Factor* Vs Mean Annual Rainfall (MAR) for the Midlands-east region

Equations to compute *Adjustment Factors* for each treatment system were obtained by plotting a linear trend (ie 'line of best fit') through for the points on the chart above for each treatment system. A trend of decreasing *Adjustment Factor* with mean annual rainfall is evident for swales, bioretention and wetlands while adjustment factor increases with MAR for ponds. This may be due to higher intensity rainfall in areas with lower total annual rainfall.

The Midlands-east *Adjustment Factor* equations are shown in the table below.

	Midlands-east
Bioretention	-0.0038(MAR)+2.835
Pond	0.0046(MAR)-1.3754
Swale	-0.0071(MAR)+4.5261
Wetland	-0.0016(MAR)+1.6456

Region: Midlands-west

Figure A.9 shows a plot of the *Adjustment Factors* for bioretention, swales, ponds and wetlands derived against mean annual rainfall for the 'Midlands-west' Hydrologic Region.

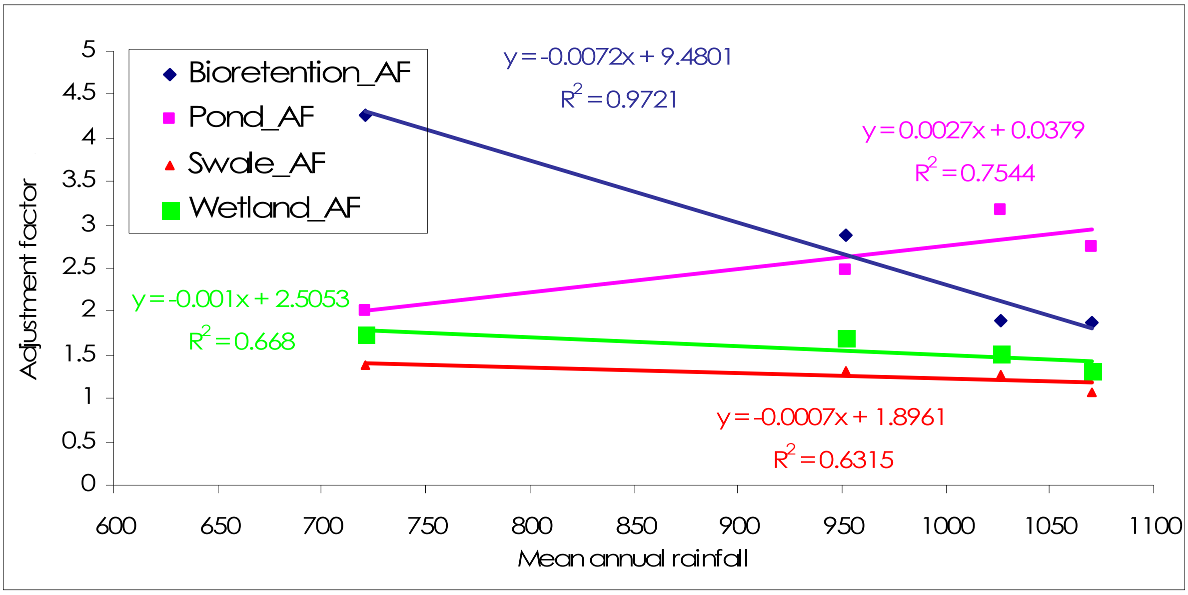

Figure A-7. Plot of *Adjustment Factor* Vs Mean Annual Rainfall (MAR) for the Midlands-west region

Equations to compute *Adjustment Factors* for each treatment system were obtained by plotting a linear trend (ie 'line of best fit') through for the points on the chart above for each treatment system.

A trend of decreasing *Adjustment Factor* with mean annual rainfall is evident for swales, bioretention and wetlands while adjustment factor increases with MAR for ponds. This may be due to higher intensity rainfall in areas with lower total annual rainfall.

The Midlands-west *Adjustment Factor* equations are shown in the table below.

	Midlands-west
Bioretention	-0.0072(MAR)+9.4801
Pond	0.0027(MAR)+0.0379
Swale	-0.0007(MAR)+1.8961
Wetland	-0.001(MAR)+2.5053

Region: Hobart

Figure A.10 shows a plot of the *Adjustment Factor*s for bioretention, swales, ponds and wetlands derived against mean annual rainfall for the 'Hobart' Hydrologic Region. A trend of increasing *Adjustment Factor* with mean annual rainfall is evident for each of the system types.

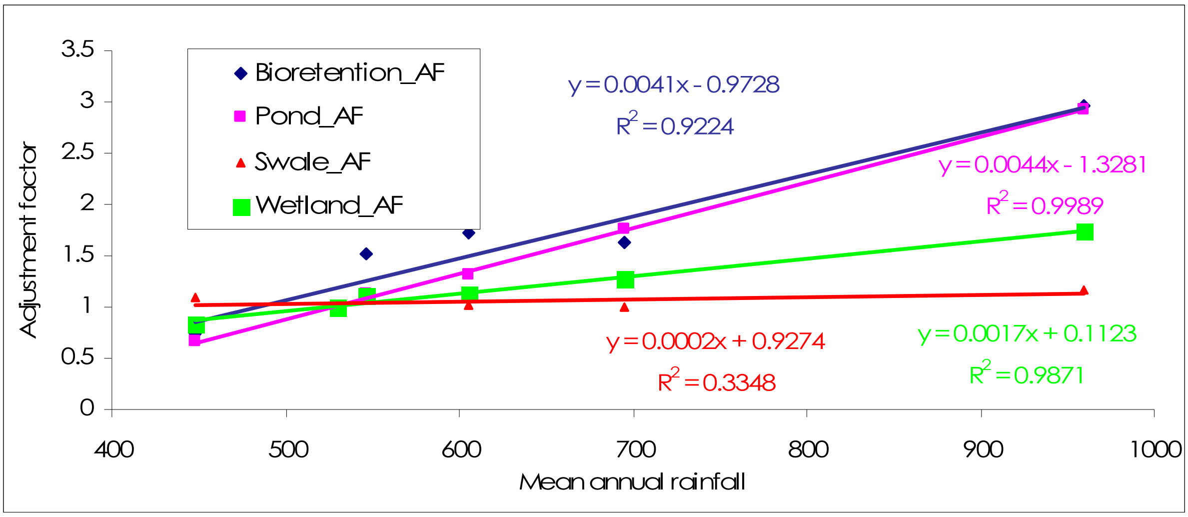

Figure A-8. Plot of *Adjustment Factor* Vs Mean Annual Rainfall (MAR) for the Hobart region

Equations to compute *Adjustment Factors* for each treatment system were obtained by plotting a linear trend (ie 'line of best fit') through for the points on the chart above for each treatment system.

Whilst the coefficient of determination for the swale relationship is lower than others, the lesser slope (m) on the linear curve indicates lower variation in adjustment factor between sites modelled and hence impact of the weaker correlation is not significant.

The Hobart *Adjustment Factor* equations are shown in the table below.

	Hobart
Bioretention	0.0041(MAR)-0.9728
Pond	0.0044(MAR)-1.3281
Swale	0.0002(MAR)+0.9274
Wetland	0.0017(MAR)+0.1123

Adjustment factor verification

In order to verify the predictive ability of the adjustment factor relationships presented above, observed adjustment factors (from the MUSIC modelling exercise) are plotted against adjustment factors calculated using the equations above for each climate station used.

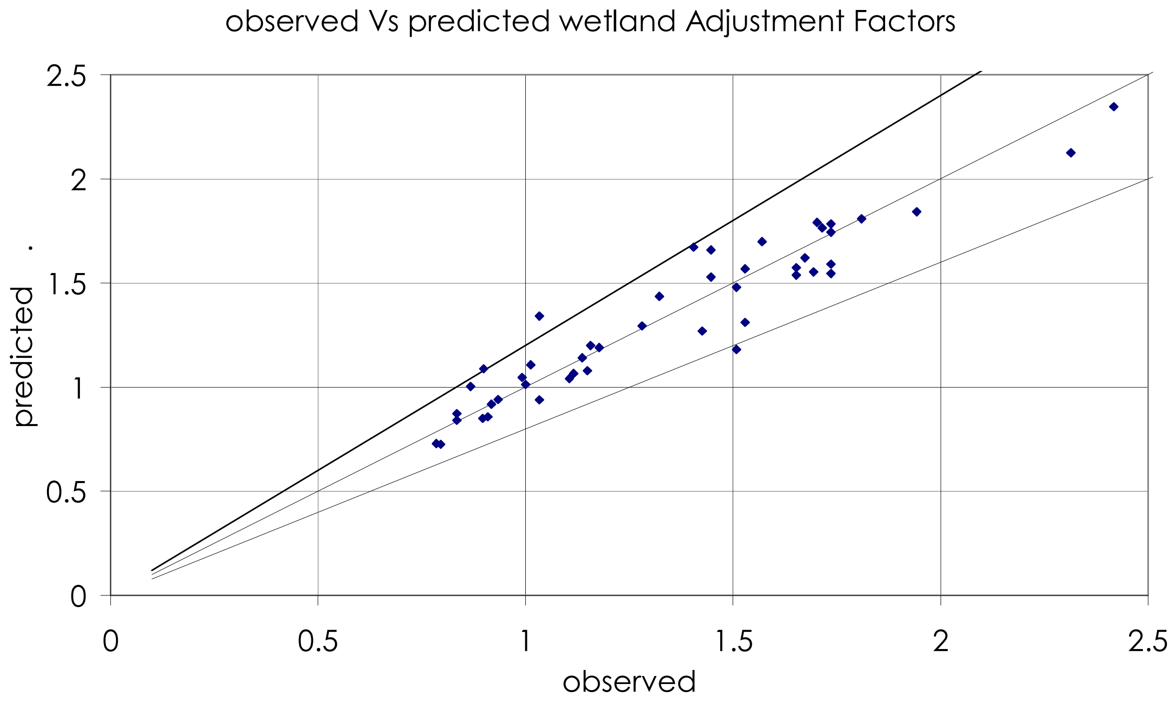

Figure A-9. Observed Vs predicted adjustment factors for wetland systems

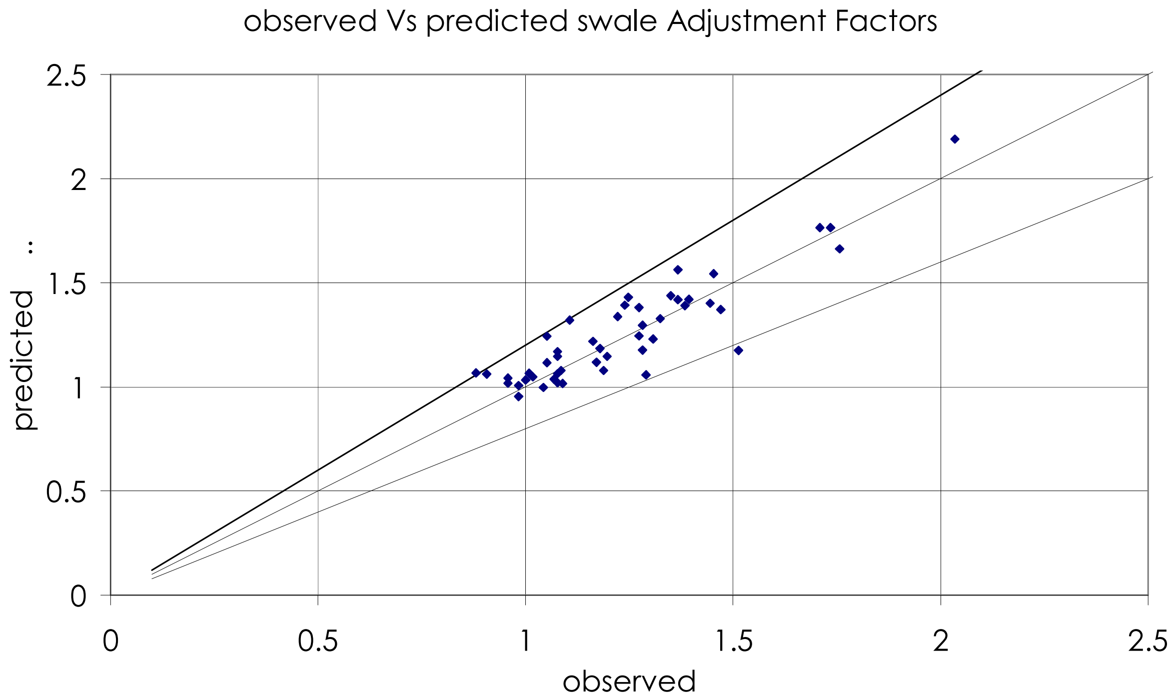

Figure A-10. Observed Vs predicted adjustment factors for grassed swales

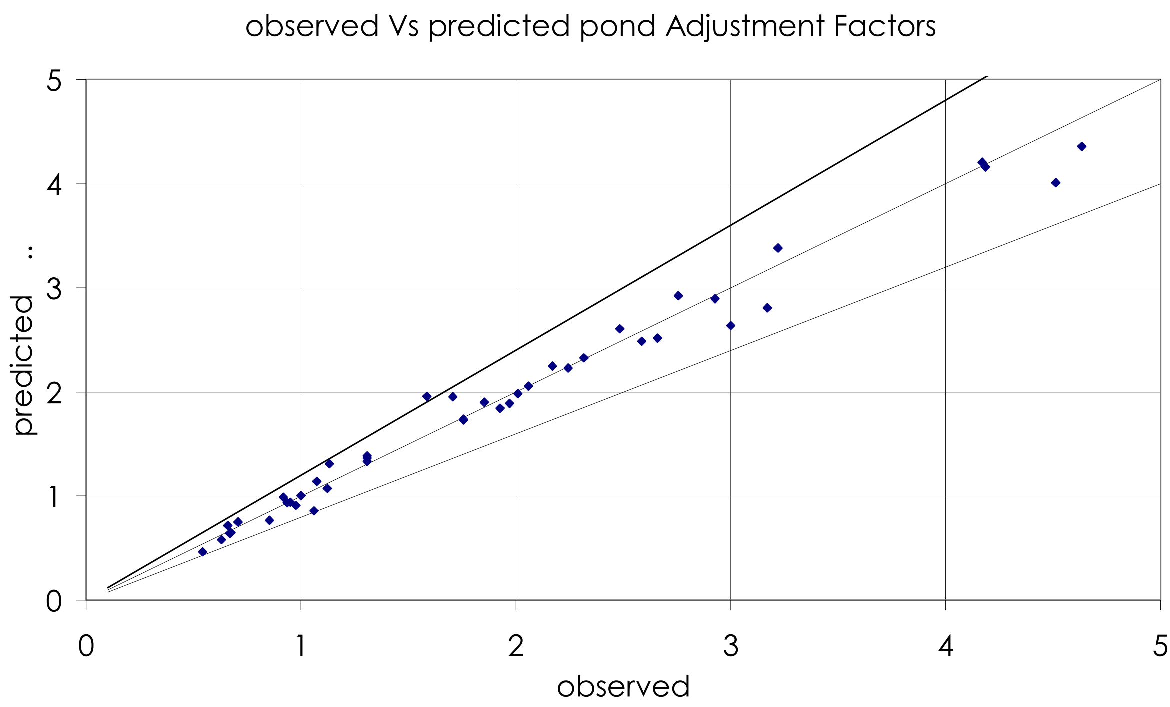

Figure A-11. Observed Vs predicted adjustment factors for ponds

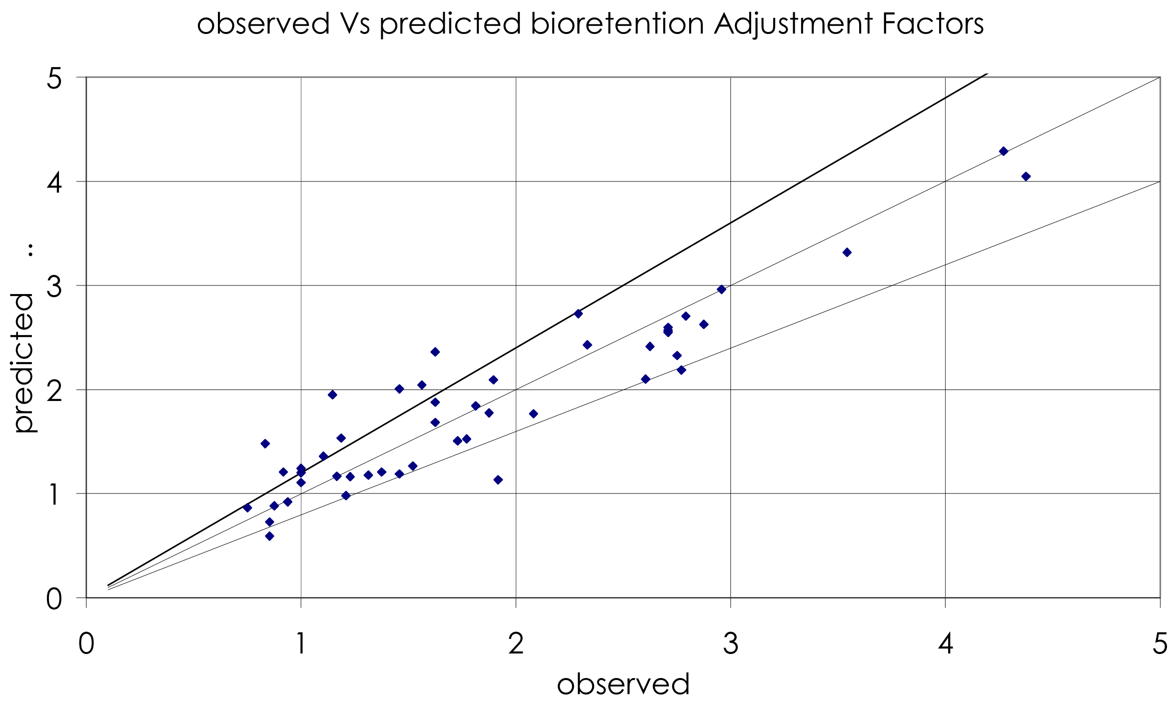

Figure A-12. Observed Vs predicted adjustment factors for bioretention systems

A.5 Adjustment factors for reference rainfall stations

The regional equations and constants for computing *Adjustments Factors* are the result of pooling modelling results for relevant reference pluviographic stations within each hydrologic region. To ensure a systematic application of the procedure, it is recommended that computation of *Adjustment Factors* should exclusively use the regional equations or constants provided instead of individually derived Adjustment Factor values, irrespective of the proximity of the site in question to a reference pluviographic station. This would avoid situations where practitioners get to choose between the adjustment factor computed from the regional approach and that derived for the reference pluviographic station of close proximity to the site in question.

If the option for practitioners to use *Adjustment Factors* derived for individual reference pluviographic stations is to be provided, a consistent approach to define the areal extent of applicability of *Adjustment Factors* derived for individual pluviographic station will need to be developed. This areal extent of applicability for individual reference pluviographic station may vary depending on its proximity to other pluviographic stations and will more than likely be determined in an ad-hoc manner. Furthermore, this option could also introduce debate amongst practitioners about the selection of reference pluviographic stations for the present analysis ahead of others which may be of "more relevant" to their particular sites.

It is recommended that only regional relationships for Adjustment factors be used in WSUD Engineering Procedures for Stormwater Management in Tasmania.

13.6.1 A.6 Recommended Adjustment Factors

The plots comparing the predicted *Adjustment Factors* to those determined from MUSIC modelling indicate that the regional equations and constants derived for the eight hydrologic regions mostly fall within a $\pm$20% band. It is thus reasonable to adopt Adjustment Factor that is 1.1 times (ie. +10%) that predicted by these equations and constants to ensure that predicted size of stormwater treatment measures using this method will not be an under-estimation of what is required. This preserves the opportunity (and incentive) for practitioners to adopt a more rigorous approach (e.g. MUSIC modelling using local rainfall data) to further refine and reduce the size of treatment measures being considered if they so desire. The recommended equations and constants (including + 10% adjustment) for computing the appropriate *Adjustment Factors* for Tasmania are summarised in Table A.1.

Table A.1. Tasmania Adjustment Factors

	Bioretention	Pond	Swale	Wetland
East-central	0.0053(MAR)-1.2818	0.0031(MAR)-0.5156	0.0004(MAR)+0.8559	0.0024(MAR)-0.1239
Hobart	0.0041(MAR)-0.9728	0.0044(MAR)-1.3281	0.0002(MAR)+0.9274	0.0017(MAR)+0.1123
Midlands-east	-0.0038(MAR)+2.835	0.0046(MAR)-1.3754	-0.0071(MAR)+4.5261	-0.0016(MAR)+1.6456
Midlands-west	-0.0072(MAR)+9.4801	0.0027(MAR)+0.0379	-0.0007(MAR)+1.8961	-0.001(MAR)+2.5053
North	0.0016(MAR)+0.3281	0.0033(MAR)-0.7432	0.0005(MAR)+0.794	0.0006(MAR)+0.5906
Northeast	0.0034(MAR)-0.8879	0.003(MAR)-0.4068	0.0005(MAR)+0.9196	0.0012(MAR)+0.5645
Southeast	0.0025(MAR)-0.1253	0.003(MAR)-0.562	0.00003(MAR)+1.0481	0.0012(MAR)+0.3184
West	0.0009(MAR)+0.4632	0.0029(MAR)-0.4404	0.0005(MAR)+0.6041	0.0007(MAR)+0.5005

Appendix B Plant Lists

Suggested plant species for WSUD treatment elements

B.1 Introduction 2

B.2 Bioretention systems, swales and buffer strips 2

B.3 Sediment basins, wetlands and ponds 6

B.4 Additional notes on the tables 12

B.5 References 13

B.1 Introduction

This appendix provides a list of plants that are suitable for different WSUD treatment elements, including:

- Sediment basins
- Bioretention swales
- Bioretention basins
- Swales and buffer strips
- Wetlands
- Ponds.

A table of suggested species is provided in this Appendix and can be used as a guide to select appropriate species to perform a water quality function. Once species are selected from these tables they should be checked for consistency with local recommended species. Indigenous nurseries and/or other relevant agencies (Councils, DEP, DPIWE) should be consulted as part of the plant selection process. Local Landcare and Bushcare groups may also prove invaluable in choosing appropriate local species.

The table includes plants suitable for sediment basins, wetlands, swales, bioretention and ponds. These plant species are principally categorised according to their water depth. Littoral vegetation can be planted around all of the systems. Ponds will have submerged vegetation. Wetlands that have a full depth range will include plants recommended for all of the six zones (littoral, ephemeral marsh, shallow marsh, marsh, deep marsh and pool (submerged marsh species)).

All of the species listed occur in Tasmania. Many species that will also be suitable for planting in WSUD elements will occur on a regional basis.

Rather than solely using plants with a wide distribution, plants can be used that are local to a particular bioregion. Plants that occur in a particular bioregion will be well-adapted to the local conditions and will add and enhance regional biodiversity. Use of locally occurring plants, some of which might be endemic, will encourage regional fauna.

B.2 Bioretention systems, swales and buffer strips

These WSUD elements typically treat stormwater close to its source (surfaces that water runs off). They include bioretention swales, bioretention basins, swales and buffer strips. Swales and buffer strips are typically constructed using local soils whereas soils in bioretention systems require specific hydraulic characteristics and local soils may require adaptation. In some cases imported soils will be required.

Bioretention soils must meet filter media specifications (primarily a prescribed hydraulic conductivity) in addition to supporting plant growth (see Chapters 4 and 5).

Sandy loam soils are commonly used in bioretention systems because they typically have particle size distributions similar to suspended solids in urban stormwater runoff and therefore provide good retention of suspended particles. While sandy loams are usually used, other soil types can be used that suit the local vegetation, if they will support plant growth and are amended to meet the system requirements.

B.2.1 Constructing suitable soil filter media

To ensure the soil/filter media provides for a design hydraulic conductivity and is able to support plant growth the following approach is suggested:

- Identify if local top soil is capable of supporting vegetation growth and if there is enough top soil (some top soils are very shallow) to be used as a base for the filter media (may require active collection of top soil during the construction process). Any topsoil found to contain high levels of salt, extremely low levels of organic carbon (<5%), or any other extremes which may retard plant growth should be rejected. If the top soil is not suitable, a sandy loam soil can be purchased from a supplier for use as a base soil.
- Conduct laboratory tests to estimate the saturated hydraulic conductivity of the top soil/base soil using standard testing procedures (AS 4419-1998).
- If the soil needs to be amended to achieve the desired design saturated hydraulic conductivity either mix in a loose non-angular sand (to increase saturated hydraulic conductivity) or a loose soft clay (to reduce saturated hydraulic conductivity).
- The required content of sand or clay (by weight) to be mixed to the base soil will need to be established in the laboratory by incrementally increasing the content of sand or clay until the desired saturated hydraulic conductivity is achieved (within reasonable bounds). The sand or clay content (by weight) that achieves the desired hydraulic conductivity should then be adopted on-site.
- The base soil should have sufficient organic content to establish vegetation on the surface of the bio-retention system. If the proportion of base soil in the final mix is less than 50% then it may be necessary to add in additional organic material to the mix but should not result in more than 10% organic content (measured in accordance with AS1289 4.1.1).
- The pH of the soil mixture for the filtration layer is to be amended to between 6 and 7, before delivery to the site.

B 2.2 Importance of vegetation

Vegetation is an integral component of the treatment systems. The vegetation needs to fulfil several functions such as:

- Provide a surface area to trap suspended solids and other pollutants as the water flows horizontally through the treatment systems.
- Provide a biologically active root zone to help the removal of pollutants as water infiltrates vertically. This function is crucial for bioretention systems.
- Reduce soil compaction and maintenance of infiltration rate.
- Reduce flow velocities and bind and stabilise the substrate thereby limiting erosion.

Provide a prominent and diverse landscape element in the development and enhance local biodiversity.

B 2.3 Required plant characteristics

Species included in Table B.2 have been specifically selected, based on their life histories, physiological and structural characteristics, to meet the functional requirements of WSUD systems. Other species can be used as long as they can fulfill the functional roles described below.

In general, species that satisfy the roles have the following general features:

- Plants need to be able to tolerate short periods of inundation punctuated by longer dry periods. These dry periods may be reasonably severe due to the free draining nature (relatively low water holding capacity) of bioretention filter media
- Plants can be either prostrate or erect
- Prostrate species would typically be low mat-forming stoloniferous or rhizomatous plants (e.g. *Cynodon dactylon*, *Phyla nodiflora, Dichondra repens*)
- Erect species would typically be rhizomatous plants with simple vertical leaves (e.g. *Juncus spp, Carex spp*)
- Desirable species should have spreading rather than clumped growth forms
- Species should be perennial rather than annual
- Species should have deep, fibrous root systems
- Shrubs and trees should be accompanied by species with the above characteristics as an understorey.

Well-established uniform vegetation is crucial to the successful operation of drainage swale and bioretention systems. As a result, species selection needs to consider both the Aesthetic and functional requirements of the systems.

Swale/bioretention system vegetation can be either single or mixed species designs. Herbaceous groundcover species (e.g. *Phyla nodiflora, Brachyscome multifida, Dichondra repens*) are nearly always best planted as mixtures. Grasses, rushes, sedges and lilies can typically be planted as single (e.g. *Carex appressa*) or mixed species (e.g. *Pennisetum alopecurioides, Dichelachne crinata, Microlaena stipoides*) stands depending on the landscaping requirements. Some of the prostrate shrubs that form scrambling thickets may be better suited to single species planting (e.g. *Hibbertia scandens, Hardenbergia violacea*) in isolated areas for aesthetic impact. These species may also require pruning to ensure even plant cover and to maintain an even root distribution below ground.

Planting density generally varies depending on the species and the type of stock specified. Some lawn and turf species could be established from seed, hydroseeding or established as rolled on turf. Native grasses, rushes, sedges and lilies are typically supplied in small tubes (35-60mm). In drainage swale/bioretention systems this stock should be planted at high densities (12-16plants/m^2). Dicotelydon species (e.g. *Goodenia hederacea, Hibbertia scandens*) are typically supplied in pots (50mm). In drainage swale/bioretention systems this stock should also be planted at high densities (8-10 plants/m^2). These high densities are required to ensure runoff does not establish preferential flow paths around the plants and erode the swale surface. High density planting is also required to ensure a uniform root zone in the bioretention systems.

B 2.4 Plant species selection

Plant species suitable for use in bioretention systems, buffer strips and swales are listed in Table B.2. Most of the species are widespread but some only occur in specific regions or in certain conditions (e.g. substrate type, salinity). Species' ranges should therefore be checked before they are recommended for a particular site.

The plant list in Table B.1 is not exhaustive. A diverse and wide-range of plants can be used for WSUD elements (subject to the characteristics described in Section B.2.3). Table B.1 only includes indigenous plants. Non-indigenous natives and exotics should only be considered when there is a specific landscape need and the species has the appropriate growth form, habit and patterns of wetting and drying.

If non-indigenous natives and exotics are chosen, careful consideration should be given to the potential impacts on downstream drainage systems. For example, *Nandina domestica* (Japanese Sacred Bamboo) and *Phyla nodiflora* (Carpet Weed) are both suitable for use in onsite WSUD elements. Similarly, species that are endemic to particular regions within Victoria (i.e. indigenous but not widespread) can be used.

Plant species should be selected based on a number of factors:

- the objectives, besides treatment function, of the WSUD element (e.g. landscape, Aesthetics, biodiversity, conservation and ecological value)
- the region, climate, soil type and other abiotic factors

- the roughness of the channel (if a conveyance system)
- the extended detention depth.

Species that have the potential to become invasive weeds should be avoided.

The typical heights of the plant species (listed in Table B.2) will help with the selection process. Low-growing and lawn species are suitable for conveyance systems that require a low roughness coefficients. The treatment performance of bioretention systems, in particular, requires dense vegetation to a height equal to that of the extended detention depth. Therefore, a system with a 300 mm extended detention should have vegetation at least 300 mm high. All of the selected plant species are able to tolerate periods of both wetting and drying.

Included in Table B.1 is the recommended planting density for each of the species. The planting densities recommended should ensure that 70-80 % cover is achieved after two growing seasons (2 years).

Although low-growing plants (like grasses, sedges and rushes) are usually used, trees and shrubs can be incorporated into WSUD elements. If using trees and shrubs in bioretention systems, they should be planted in the local soil adjacent to the filter medium, so that the roots do not interfere with the perforated pipes. Shrubs listed provide a wide range of sizes from small to large. Geotechnical advice may be required if using trees in some systems.

B 2.5 Vegetation establishment and maintenance

Conventional surface mulching of swale/bioretention systems with organic material like tanbark etc, should not be undertaken. Most organic mulch floats and runoff typically causes this material to be washed away with a risk of causing drain blockage.

New plantings need to be maintained for a minimum of 26 weeks. Maintenance includes regular watering, weed control, replacement of dead plants, pest monitoring and control, and rubbish removal. Once established lawn, grass and groundcover plantings will need to be mown to maintain the design vegetation height.

B.3 Sediment basins, wetlands and ponds

These WSUD elements typically treat stormwater away from its source. Stormwater may be transported through a conventional drainage system or it may be transported via WSUD elements, so would receive some pre-treatment.

B 3.1 Importance of vegetation

Sediment basins are designed to trap coarse particles (> 125 μm) before the stormwater enters a wetland. Aquatic vegetation is therefore not specified for the sediment basins except in the littoral zone around the edge of the basin. The littoral vegetation is not part of the water quality treatment process in sediment basins so is not essential. However, plants can stabilise banks, so vegetation should be prescribed if erosion is a potential problem. Dense planting of the littoral berm zone also inhibits public access to the treatment elements, minimising the safety risks posed by water bodies. It can also improve the Aesthetics and screen basins, which are typically turbid.

Ponds are principally designed to be open water features providing landscape value. Unless the ponds have hard edges, littoral vegetation should be planted along the edges. These plants will provide habitat for local fauna, will help to stabilise the banks against erosion, and will inhibit weed invasion. Littoral vegetation also plays a treatment role when the water is above normal water level. Dense planting of the littoral zone will also inhibit public access to ponds, minimising the safety risks posed by water bodies.

Submerged plants should be planted in the deep areas of ponds. Submerged plants will be seen occasionally, like after a long dry period, when they surface to flower and seed, or when birds rip up plant fragments, for example. However, in the most part they will be totally submerged and will provide an open water perspective. Submerged plants are crucial for maintaining high water quality and minimising the chance of an algal bloom. They also inhibit weed invasion.

Wetlands are dominated by emergent macrophytes (aquatic plants). Constructed wetlands are designed to trap the fine polluted particles (< 125 μm) where they can be safely stored for long periods (15-20 years). Wetland plants extract nutrients and other dissolved substances, and provide a framework for microbial biofilms. Wetlands therefore clean water through biotic absorption, ingestion and decomposition of pollutants, as well as other chemical transformations resulting from the range of oxidation states.

Vegetation should also be planted along the edges of wetlands. Littoral vegetation will help to filter and treat water during times when the water is above normal water level. Dense planting of the littoral zone will also inhibit public access to the treatment elements, minimising potential damage to the plants and the safety risks posed by water bodies.

B 3.2 Required plant characteristics

Species outlined in Table B.1 include consideration of the wetland zone/depth range and the typical extended detention time (48-72 hrs) and depth (0.5m). Other species may be used to supplement these core species, although they must be selected to suit the particular depth range of the wetland zone and have the structural characteristics to perform particular treatment processes (e.g. distribute flows, enhance sedimentation, maximize surface area for the adhesion of particles and/or provide a substratum for algal epiphytes and biofilms). In general, species that satisfy the roles have the following general features:

- Grow in water as emergent macrophytes (e.g. Marsh species) or tolerate periods of inundation (e.g. Ephemeral Marsh species), typically sedge, rush or reed species
- Desirable species generally have rhizomatous growth forms
- Species should be perennial rather than annual
- Are generally erect species with simple vertical leaves (e.g. Juncus spp, Baumea spp)
- Desirable species should have spreading rather than clumped growth forms
- Species should have a fibrous root systems
- Shrubs and trees (generally only planted in the littoral or ephemeral zones) should be accompanied by species with the above characteristics as an understorey

The locations within a wetland that are best suited to specific wetland plants are determined by the interaction between basin bathymetry, outlet hydraulics and catchment hydrology - the hydrologic regime (Wong *et al*, 1998). Individual species have evolved preferences for particular conditions within the water depth-inundation period spectrum and this must be checked (wetland plant suppliers/nurseries) prior to recommending for a particular wetland zone planting.

B 3.3 Plant species selection

The distribution of the species within the wetland relates to their structure and function. The planting densities recommended should ensure that 70-80 % cover is achieved after two growing seasons (2 years).

Suitable plant species have also been recommended for the littoral zone that will surround the wetlands, ponds and sediment basins. The littoral zone (berms or batters) refer to the perimeter of the treatment elements and extend over a depth range of 0.5 m. Plants that have a drier habit should be planted towards the top of batters, whereas those that are adapted to more moist conditions should be planted closer to the water line.

When selecting plants for wetlands, wetlands should be divided into a series of zones based on their water depth (pools (or submerged marsh), deep marsh, marsh, shallow marsh, ephemeral marsh and littoral zones). The relative size of the zones is determined by the wetland bathymetry. Table B.1 shows the typical permanent depth ranges of the six zones commonly found in wetlands. The zones referred to in Table B.2

correspond with the depth ranges shown in the table below. Some plant species can be used in more than one zone, but plant species are generally categorised into one zone based on their preferred water range.

Table B.1: Depth ranges of wetland macrophyte zones

Zone	Macrophyte Zone Type	Depth[5] (m)
P	Pool – submerged marsh	0.5 – ~ 1
DM	Deep Marsh	0.35 – 0.5
M	Marsh	0.2 – 0.35
SM	Shallow Marsh	0 – 0.2
EM	Ephemeral Marsh	+0.2 – 0
L	Littoral	+0.5 – 0

Table B.2 is just an example of some of the plants that can be used in wetlands. The plant species listed in Table B.2 are recommended as the core species for the zones, but a number of other plants could be used. The species recommended are all thought to satisfy the functional treatment requirements of the zone, and are adapted to the hydrologic conditions of the zone. Indigenous species are generally recommended as they provide habitat for native wetland fauna.

B 3.4 Wetland vegetation establishment

To maximise the success of plant establishment in wetland macrophyte zones the following vegetation establishment program is recommended. The program outlines procedures involved in site preparation, vegetation preparation, planting, and maintenance.

Plant Growth Medium – After establishing a bathymetry of the wetland, a layer of topsoil is required as a substrate for aquatic vegetation. Although there are a few plants that can grow in sub-soils such as heavy clays (e.g. Phragmites), growth is slow and the system would have low species richness, which is deemed undesirable. Wetlands should therefore have a layer of topsoil not less than 200 mm deep (deeper if possible). Topsoil removed from a site during excavation should be stockpiled for subsequent use

[5] Mean water depth at normal water level (NWL) for the summer permanent pool. Natural variation below the NWL is expected to regularly expose the shallow marsh section and much of the marsh section. During events water will temporarily be stored above the NWL and inundate the ephemeral section.

as a growth medium for wetland macrophytes. If the top soil is unsuitable, (will not support plant growth, wetland plants typically prefer silty to sandy loams) it is advisable to purchase appropriate soils from a supplier. If stockpiled topsoil is to be used it is recommended that it be screened to remove any coarse organic mater prior to placement in a wetland. Other topsoil treatment requirements are listed below.

Soil Treatment – The topsoil covering the bed of a system (macrophyte zone an open water zones) should be treated with gypsum or lime (standard on most construction sites). By facilitating flocculation, gypsum will reduce the turbidity of the water column, which will be particularly valuable in the early stages of the wetland system establishment. With lower turbidity, higher levels of light will be able to reach the plants, thereby facilitating their growth and establishment. It is important that the gypsum not be added too far in advance of the vegetation planting; with clear water and no aquatic plants competing for resources, conditions will be favourable for algal growth, thus increasing the threat of an algal bloom. The gypsum should be applied about one week prior to planting at a rate of 0.4 kg/m^2. Subsequent application may be required at intervals depending on pond condition and the amount of exchangeable sodium. Fertilisers should not be applied to the top soil or to terrestrial areas in or around the wetland system, particularly in the early stages of plant establishment, due to the threat posed by algal blooms, particularly cyanobacteria (blue-green algae). The inadvertent addition of nutrients to the wetland system could facilitate the growth of cyanobacteria, particularly when the competing macrophytes and submerged plants are in their early developmental stages, thus raising the likelihood of algal blooms.

Plant Propagation – Plants should be ordered from a vegetation supplier prior to the time of planting to enable the supplier sufficient time to grow the required number of plants and species types and for the plants to grow to a suitable size (maturity) to ensure low mortality rates. The supplier should be made aware of the planned planting layout and schedule.

To ensure successful establishment of wetland plants, particularly in deeper marsh zones, it is strongly recommended more mature tube stock be used (i.e. at least 0.5 metres in height). For shallower zones of a wetland, younger tube stock or seedlings may suffice. As a minimum the following plant stock should be provided by a nursery:

Deep Marsh & Marsh Zone Planting

- 50mm tube stock
- 3-4 shoots or leaves
- 500-600mm height

Shallow Marsh & Ephemeral Marsh Zone Planting

- Preferably 50mm tube stock but 25mm container stock should suffice
- 4-5 shoots or leaves

- 300-400mm height

20mm seedling pots should be avoided as these seedlings are considered to be relatively immature and will result in high loss rates and patchy growth.

Planting Water Level Manipulation - To maximise the chances of successful establishment of the vegetation, water levels within a wetland system should be manipulated in the early stages of vegetation growth. When first planted, vegetation in the deep marsh and pool zones may be too small to exist in their prescribed water depths (depending on the maturity of the plant stock provided). Seedlings intended for the deep marsh sections will need to have at least one third of their form above the water level. This may not be possible if initially planted at their intended depth. Similarly, if planted too deeply, the young submerged plants will not be able to access sufficient light in the open water zones. Without adequate competition from submerged plants, phytoplankton (algae) may proliferate.

Water depth should therefore be controlled in the early establishment phase. Deep marsh zones should have a water depth of approximately 0.2 m for the first 6-8 weeks. This will ensure deep marsh and marsh zones are inundated at shallow depths and the shallow marsh zone remains moist (muddy) which is suitable for plant establishment. After this period, water levels can be raised to normal operating levels.

Planting - Planting in all zones of a wetland should occur at the same time. With water levels controlled as above, deep marsh and marsh zones will be inundated with water and the shallow marsh zone will be moist to allow appropriate growth (however some water over shallow marsh zones may be required). Planting of ephemeral zones will require irrigation at a similar rate as terrestrial landscaping surrounding the wetland.

Establish operating wetland water level - After 6-8 weeks of growth at a controlled water level, wetland plants should be of sufficient stature to endure deeper conditions so the wetland can be filled to its normal operating water level. Therefore, after eight weeks the connection between the inlet pond and the macrophyte zone should be temporarily opened to allow slow filling of the wetland to normal operating water level. Once filled to normal water level, the connection between inlet pond and macrophyte zone should again be closed to allow further plant establishment without exposure to significant water level variations.

Connect Inlet Pond to Macrophyte Zone - The temporary blockage located on the connection between the Inlet Pond and Macrophyte Zone can be removed once the building construction within the wetland catchment has been completed. At this time it will be necessary to de-silt the inlet pond which will have been operating as a sediment basin during the building phase. Planting of the zones disturbed during de-silting will be required.

Vegetation Assessment - Ensure the wetland is operating at the end of the construction landscape management period and the planted macrophytes are established and healthy at the operating water level. If successful the wetland should have a 70-80 % even macrophyte cover after two growing seasons (2 years).

B 3.5 Steps to Choosing Appropriate Vegetation

The following steps should be followed when selecting vegetation for WSUD treatment elements.

1. Determine what soil type is in the local area and if it requires amendment to meet the prescribed hydraulic conductivity (for bioretention systems) and/or amendment to support plant establishment
2. Refer to appended tables to select appropriate species for each macrophyte zone or swale/ bioretention system
3. Ensure species selection is consistent with the local hydrologic regions see Figure B.1
4. Consult local indigenous nurseries and/or other relevant agencies (Councils, DPIWE and local groups) to ensure consistency with local vegetation strategies, avoiding locally invasive or exotic species and selecting for locally indigenous species
5. Where species listed in the tables do not comply with local vegetation strategies seek advice from relevant agencies regarding alternative species with similar characteristics

B.4 Additional notes on the tables

1. The **planting stock** of the different species recommended will require differing degrees of maturity at planting. For example, even though water level management is recommended at planting times, deep marsh species will need to be more advanced stock suitable for planting in deeper water than the species recommended for the shallow marsh zone.
2. **Planting density** indicates the mean number of plants per square metre for the species spatial distribution within the zone. The planting densities recommended are suggested minimums. While planting density can be either increased or decreased depending on budget. Any reduction in planting density has the potential to reduce the rate of vegetation establishment, increase the risk of weed invasion, and increase maintenance costs.
3. The **total number of plants** required for each zone can be calculated:

Number of plants = (Recommended planting density x Section area x Proportion of section planted x Cover density).

Where *the proportion of the section* planted refers to the proportion of the section area that will be planted with the identified species; and

Where *cover density* refers to the proportional cover of that particular plant species in the designated location. The cover density of all of the plant species in a given area typically sums to 1.0.

B 4.1 Key to Plant Species Table

Table B.2 on the following pages, outlines suggested plant species for various WSUD treatment elements. The key to these tables is given below.

Type/ Zone		**Form**	
L	Littoral	E	Erect herbs
EM	Ephemeral Marsh	G	Groundcover
SM	Shallow Marsh	M	Emergent macrophytes
M	Marsh	S	Submerged macrophytes
DM	Deep Marsh	T	Shrubs and trees
P	Pool (submerged marsh)		
F	Forest		

B.5 References

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Table B.2 Plant Species for WSUD systems (indicative)

Suitability	Scientific Name	Common Name		Form	Height(m)	Description	Planting Density (plants/m²)	Requirements	Comments
all	*Agrostis avenacea*	Straw blown grass	L	G	0.25		6-8	well drained	
all	*Amphibromus recurvatus*	Swamp wallaby Grass	L	G	0.5		6-8	well drained	
wet/pond	*Baumea arthrophylla*	Twig rush	M	M	0.3-1.3 (stems)	Aquatic perennial with long rhizomes	6-8	Wet soils to permanent water	Spreads quickly
wet/pond	*Baumea articulata*	Jointed Twig-rush	DM	M	1-2	Tall erect rhizomatous perennial	4	Moist soil to permanent water	Slow growth
wet/pond	*Baumea juncea*	Bare Twig-rush	L	M	0.3-1	Rush-like clump with creeping rhizomes	8	Moist to boggy soils; tolerates occasional dry periods	Slow establishment
wet/pond	*Baumea rubiginosa*	Soft Twig-rush	M	M	0.3-1	Rhizomatous perennial	6-8	Moist soils to prolonged inundation	Slow establishment
wet/pond	*Baumea tetragona*	Square Twig-rush	M	M	0.3-1	Rhizomatous perennial	6-8	Moist soils to prolonged inundation; 0.2-0.4 m depth	Slow establishment
bio/swale	*Blechnum cartilagineum*	Gristle Fern		E	0.5-1.5	Upright tufting fern with short creeping stoloniferous rhizomes, forming spreading patches	2-4	Moist, well drained soils; tolerates drier conditions once established	Aesthetic; readily available
wet/pond	*Blechnum minus*	Soft Water Fern	EM	G	0.5-1.2	Dense, erect clump forming spreading patches from	4-6	Very moist soils; tolerates wet soils	Adaptable

Suitability	Scientific Name	Common Name		Form	Height(m)	Description	Planting Density (plants/m²)	Requirements	Comments
						underground stolons			
wet/pond	*Bolboschoenus caldwellii*	Sea club-rush	M	M	0.3-0.9	Aquatic to semi-aquatic rhizomatous perennial	4-6	Fresh to brackish water on heavy clay to sandy soils	Coastal/saline; rapid establishment
wet/pond	*Bolboschoenus medianus*	Marsh Club-rush	M	M	0.7-2	Aquatic to semi-aquatic rhizomatous perennial	4-6	Moist soils to permanent water	Rapid establishment; spreading
wet/pond	*Brachyscome cardiocarpa*	Swamp Daisy	L	G	0.1-0.3	Tufted perennial herb	2-4	Moist soils	Rapid establishment; aesthetic
all	*Carex appressa*	Tall Sedge	L	M	0.3-0.8	Dense, robust and tough; hairy and sticky;	4-8	Very moist soils, tolerates periods of inundation and dryness	Stabilises banks against erosion; tough; slow-growing; high surface area; dominates zones
wet/pond	*Carex bichenoviana*	Sedge	L	G	0.25-0.5 (stems)	Tufted grass-like sedge with long creeping rhizome	6-8	Moist depressions on heavy clay	May form dense carpets in shady situations
wet/pond	*Carex breviculmis*	Short-stem sedge	L	M	0-0.15	Small but densely tufted sedge	6-8	Moist to wet soils; tolerates dry periods	Very adaptable
all	*Carex fasicularis*	Tassel Sedge	SM	M	0.5-1.5	Coarse, tufted plant	6-8	Moist soils	Aesthetic
wet/pond	*Carex gaudichadiana*	Tufted sedge	SM	M	0.1-0.6	Coarse, tufted plant	6-8	Gravel or mud at water's edge	Aesthetic; tolerates

Suitability	Scientific Name	Common Name		Form	Height(m)	Description	Planting Density (plants/m²)	Requirements	Comments
									drawdown
all	*Carex inversa*	Knob Sedge	L	M	0.3-0.6	Small tufted or spreading clump	10	Moist, well drained soils	Variable species; rapid establishment
wet/pond	*Centella cordifolia*	Swamp Pennywart	L	G	Prostrate	Creeping perennial herb	2-4	Moist to wet soils	Rapid growth; may become invasive
all	*Chrysocephalum apiculatum*	Common Everlasting	L	E	Prostrate- 1.2	Variable, dense spreading perennial herb	2-4	Well drained soils	Aesthetic; widespread
bio/swale	*Correa alba*	White Correa		T	0.5-1.5	Dense, spreading shrub, dwarfed by wind and salt spray	2-4	Well drained soils; tolerates inundation and dry periods once established	Useful for soil binding
bio/swale	*Correa reflexa*	Common Correa		T	Prostrate - 0.15	Very variable - open upright to spreading shrub	2-4	Well drained soils; dry shaded position	Establishes well under trees
wet/pond	*Cyperus gunnii*	Flecked Flat Sedge	EM	M	0.6-1	Densely tufted perennial herb	6-8	Moist to boggy soils	High surface area
wet/pond	*Cyperus lucidus*	Leafy Flat-sedge	SM	M	0.6-1.5	Robust, tufted perennial herb with sharply triangular stems; large; dense	6	Wet soils	Can grow as an aquatic plant; slow spreading
all	*Danthonia sp.*	Wallaby grasses	L	G	0.4-1	Clumping grass	4-8	well drained	

Suitability	Scientific Name	Common Name		Form	Height(m)	Description	Planting Density (plants/m²)	Requirements	Comments
all	*Deschampsia cespitosa*	Tufted Hair Grass	L	G	0.4-1	Clumping grass	4-8		
all	*Dianella longifolia*	Pale Flax-lily	L	E	Prostrate	Tufted perennial clump with short rhizomes	8	Moist, well drained soils	Aesthetic; easy maintenance; ideal under trees
wet/pond	*Dianella tasmanica*	Tasman Flax-lily	L	M	0.6-1.5	Robust tufted perennial; may spread vigorously with strong rhizomes	6	Moist soils, prefers shaded position	Tolerant once established; adaptable (including snow cover); aesthetic
bio/swale	*Dichondra repens*	Kidney Weed		G	Prostrate	Dense spreading herb, forms mats	6-8	Moist, well drained soils; tolerates drying once established	Alternative to grass where foot traffic is light; more vigorous when cultivated; widespread
all	*Distichlis distichophylla*	Australian Salt Grass	L	G	0.4-1		4-8		
wet/pond	*Eleocharis acuta*	Common Spike-rush	SM	M	0.3-0.9	Perennial aquatic herb; slender rhizomes	6-8	Heavy damp soils to 0.20 m depth	High surface area; may spread rapidly in shallow water
wet/pond	*Eleocharis pusilla*	Small Spike-rush	L	G	0.002-0.25	Tiny perennial herb with thread-like rhizomes and stems	6-10	Moist to wet soils	Readily grown; easily controlled
wet/pond	*Eleocharis*	Tall Spike-rush	DM	M	0.5-2	Robust perennial herb with thick woody	6	Aquatic; to depth of 2 m; tolerates occasional	Plant solo, rhizomes can

Suitability	Scientific Name	Common Name		Form	Height(m)	Description	Planting Density (plants/m²)	Requirements	Comments
	sphacelata					rhizome; clumps to big dense stands;		drying	restrict growth of other plants; slow establishment
bio/swale	*Epacris impressa*	Common Heath		E	0.3-0.6, stems 0.6-1.0	Open wiry shrub	2-4	Moist, well drained soils; tolerates limited dry or wet periods once established	
all	*Ehrharta stipoides*	Weeping Ricegrass	L	G	0.3			dry to moist	
bio/swale	*Eucalyptus coccifera*	Tasmanian Snow Gum		T	25	Smooth grey bark (cf. the red bark of E. subcrenulata), fairly broad leaves with a curly tip	<1	Damp alluvial soils; deep subsoils; tolerates inundation and very dry periods once established	Aesthetic; some forms can be used to combat salinity; widespread
bio/swale	*Eucalyptus ovata*	Swamp Gum		T	0.5-10.0		<1	Moist soils; tolerates inundation and dry periods; lake edge	Aesthetic; widespread
wet/pond	*Gahnia filum*	Chaffy Saw-sedge	L		1-1.2	Perennial leafy tussock	4-6	Moist sandy soils; salt tolerant	Aesthetic fruits
wet/pond	*Gahnia grandis*	Giant Saw-sedge	SM	M	>1	perennial sedge	4-6	Moist to wet soil	Aesthetic
wet/pond	*Gahnia sieberiana*	Red-fruited Sword Sedge	L	M	1.5-3	Clumping perennial sedge	4-6	Moist soils; tolerates dry periods once established	Aesthetic; easily grown from seed
wet/pond	*Gahnia trifida*	Cutting Sedge	SM	M	>1	perennial sedge	4-6	Moist to wet soil	Aesthetic
all	*Glyceria australis*	Australian Sweetgrass		G	>0.75			Moist to wet soil, well-drained	

Suitability	Scientific Name	Common Name		Form	Height(m)	Description	Planting Density (plants/m²)	Requirements	Comments
wet/pond	*Goodenia humilis*	Swamp Goodenia	L	M	0.05-.1	Suckering, matting plant	2-4	Moist to wet soil	Aesthetic; very adaptable
wet/pond	*Hemarthria uncinata*	Mat Grass			>0.2				
bio/swale	*Hibbertia prostrata*	Stalked Guinea-flower		E	0.5-1.5	Low erect sub-shrub	4-6	Moist, well drained sandy soils; not clay	Difficult in clay soils
wet/pond	*Isolepis inundata*	Swamp Club-rush	SM	M	0.05-0.3	Tufted perennial rush; small; stoloniferous	6-8	Moist to wet soils; tolerates periodic inundation	Widespread; high surface area; rapid growth
wet/pond	*Isolepis cernua*	Slender Club-rush	SM	M	0.3		6-8	Inundated	
wet/pond	*Isolepis fluitans*	Floating Club-rush	SM	M	0.3				
all	*Isolepis nodosa*	Knobby Club-rush	SM	M	0.5-1	Tall; wiry; rhizomatous; densely tufted perennial rush	6-8	Moist soils; tolerates dry periods once established	binds soils in moist areas; aesthetic
all	*Juncus amabilis*	-	EM	M	0.2-1.2	Rhizomatous tufted perennial rush	8-10	Tolerates inundation and dry periods once established	Widespread
wet/pond	*Juncus australis*	Austral Rush	L	M	0.6-1.2	Rhizomatous tufted perennial rush	6-10	Moist soils; will tolerate short, dry periods	
bio/swale	*Juncus gregiflorus*	-		M	0.5-1.4	Rhizomatous tufted perennial rush	8-10	Moist, well drained soils	
wet/pond	*Juncus kraussii*	Sea Rush	SM	M	0.6-2.3	Rhizomatous perennial	6-10	Brackish to saline	Slow growth; saline; habitat

Suitability	Scientific Name	Common Name		Form	Height(m)	Description	Planting Density (plants/m²)	Requirements	Comments
						rush		conditions	only
all	*Juncus pallidus*	Pale Rush	EM	M	0.5-2.3	Rhizomatous tufted perennial rush	8-10	Grows well with periodic inundation	Rapid growth; adaptable
wet/pond	*Juncus pauciflorus*	Loose-flower Rush	L	M	0.3-1	Rhizomatous perennial rush	6-10	Moist soils; tolerates dryness once established	Adaptable
bio/swale	*Juncus procerus*	-		M	1.0-2.0	Rhizomatous tufted perennial rush	8-10	Moist, well-drained soils in a sheltered position	
wet/pond	*Juncus subsecundus*	Finger Rush	SM	M	0.5-1	Rhizomatous tufted perennial rush	6-10	Heavy, wet soils	Widespread
bio/swale	*Kunzea ambigua*	Tick bush		T	2-3	Dense to open weeping shrub	<1	Adaptable, tolerates dry periods	variable growth
bio/swale	*Lepidosperma gladiatum*	Coastal sword-sedge	Variable sword-sedge	M	0.5-1.0	Leaves wide and flat with dark green blades	6	Moist, well drained sandy soils	Sharp-edged leaves - could be used to manage pedestrian traffic
bio/swale	*Lepidosperma laterale*			M	0.6-1.7	Leaves wide and flat with dark green blades	6	Moist to wet soils but tolerates dry periods	Little maintenance once established
all	*Lepidosperma longitudinale*	Common Sword-sedge	EM	M	0.15-0.5	Sedge with long, flat, dark green blades	6	Moist to wet soils	Aesthetic
wet/pond	*Leptocarpus brownii*	Coarse wire rush	SM	M	>0.4				
wet/pond	*Leptocarpus*	Slender wire	SM	M	>0.4				

Suitability	Scientific Name	Common Name		Form	Height(m)	Description	Planting Density (plants/m²)	Requirements	Comments
	tenax	rush							
bio/swale	*Leptospermum scoparium*	Prickly Tea-tree		T	<2	straggling shrub or small tree	<1	Adaptable; tolerates moisture	Aesthetic
bio/swale	*Leptospermum lanigerum*	Woolly Tea-tree		T	0.5-1.0	Dense shrub to erect small tree	<1	Moist soils	Aesthetic
bio/swale	*Leucopogon australis*	Spike Beard Heath		T	0.2-0.5	Upright shrub	2-4	Well drained damp sandy soils	Strongly perfumed flowers
wet/pond	*Linum marginale*	Native Flax	L	G	0.3-0.8	Slender erect perennial	4-6	Moist, well drained soils	Widespread
all	*Lomandra nana*	Pale Mat-rush	L	M	0.15-0.5	Small tussock with fine blades	6-8	Moist, well drained clay or sandy soils; tolerates dry shaded positions once established	Little maintenance; grows well under trees
all	*Lomandra longifolia*	Sagg / Spiny-headed Mat-rush	L	M	0.5-1.0	Large tussock	4-6	Well drained soils; tolerates dry shaded positions	Grows well under established trees
wet/pond	*Lythrum salicaria*	Purple Loosestrife	L	E	1-2	Erect, hairy perennial	2-4	Moist soils or shallow water	Dies back after summer
all	*Melaleuca ericifolia*	Swamp Paperbark	EM, L		2-9	Erect, open to bushy shrub or small tree	2-4	Moist to wet fertile soils; tolerates dry periods once established;	Very adaptable
bio/swale	*Melaleuca squarrosa*	Scented Paperbark		T	0.5-2.0	Erect, open to compact large shrub or rarely, a small tree	<1	Moist to wet soils	Aesthetic; salt tolerant; grows well in coastal

Suitability	Scientific Name	Common Name		Form	Height(m)	Description	Planting Density (plants/m²)	Requirements	Comments
									areas
bio/swale	*Microlaena stipoides*	Weeping Grass		G, E	0.3-2.0	Highly variable in size	turf or seeds	Moist, well drained soils	Aesthetic; suitable as a lawn grass; widespread
bio/swale	*Myoporum parvifolium*	Creeping Boobialla		G	12-50	Dense matting groundcover	4-6	Well drained soils, tolerates dry periods once established	Adaptable groundcover; layering habit useful for soil binding
wet/pond	*Myriophyllum pedunculatum*	Mat Water-milfoil	P	S	prostrate	Perennial herb, aquatic or fully emergent; stems up to 1 mm diam., prostrate with erect laterals	1	Deep fast-flowing water to shallow brackish or calcareous water	Heterophyllic
wet/pond	*Nymphoides exigua*	Tasmanian Marshwort	SM	M		Single pale-yellow flowers	1	waterlooged soils	Aesthetic
bio/swale	*Patersonia occidentalis*	Long Purple-flag		M	8-30	Compact clumping perennial herb	6-8	Tolerates inundation and dry periods	Aesthetic; may not persist
wet/pond	*Persicaria decipiens*	Slender Knotweed	L	M	Prostrate - 0.6	Glabrous, erect to spreading annual herb	2-4	Semi-aquatic to aquatic	Low surface area; aesthetic
wet/pond	*Phragmites australis*	Common reed	SM	M	>1.5	erect perennial, rapid colonising			Invasive
bio/swale	*Pimelea glauca*	Smooth Rice-flower		T	2-5	Erect, many-branched glabrous shrub	2-4	Well drained soils	Aesthetic

Suitability	Scientific Name	Common Name		Form	Height(m)	Description	Planting Density (plants/m²)	Requirements	Comments
bio/swale	*Pimelea linifolia*	Slender Rice-flower		E	1-4	Variable prostrate erect or clump-forming, depending on habitat	4-6	Well drained soils	Pruning encourages branching
bio/swale	*Poa labillardierei*	Common Tussock Grass		M	2-6	Large, coarse densely tufted tussock	6-8	Adapts to moist or slightly dry soils	Widespread
wet/pond	*Poa tenera*	Slender Tussock Grass	L	G	0.05-0.2	Trailing, sometimes forms open tussocks	6-8	Moist, well drained soils	Very effective when trailing down embankments
wet/pond	*Potamogeton crispus*	Curly Pondweed	P	S	To 4.5	Perennial, rhizomatous aquatic herbs	1	Aquatic; deep permanent water	Growth can be dense
wet/pond	*Potamogeton ochreatus*	Blunt Pondweed	P	S	To 4.5	Annual or perennial, rhizomatous aquatic herbs; submerged floating annuals	1	Aquatic; deep permanent water	Rapid growth; aesthetic; seasonal; salt tolerant (2000 ppm)
wet/pond	*Potamogeton pectinatus*	Fennel Pondweed	P	S	Stems to 3	Perennial, rhizomatous aquatic herbs; submerged	1	Aquatic; deep permanent water	Saline (thrive in >5000 ppm dissolved salt); rarely recommended; not aesthetic; often invasive
wet/pond	*Potamogeton tricarinatus*	Floating Pondweed	P	S	stems to 2.7	Perennial rhizomatous aquatic herb; submerged or attached floating	1	Aquatic; shallow semi-permanent water	Seasonal

Suitability	Scientific Name	Common Name		Form	Height(m)	Description	Planting Density (plants/m²)	Requirements	Comments
bio/swale	*Pultenaea daphnoides*	Large-leaf Bush-pea		T	2-9	Erect branching shrub	2-4	Moist, well drained soils; tolerates dry periods once established	Aesthetic
wet/pond	*Ranunculus inundatus*	River Buttercup	L	G	0.05-0.3	Slender, stoloniferous perennial herb; often forms large mats	2-4	Semi-aquatic to aquatic	Rapid establishment
wet/pond	*Schoenoplectus pungens*	Sharp Club-rush	M	M	0.3-0.6	Robust, tufted rhizomatous herb	4-6	Wet soils to permanent water	Become rare due to urbanisation; rapid establishment
wet/pond	*Schoenoplectus validus*	Lake Club-rush	DM	M	0.8-2	Rhizomatous, robust perennial, grass-like or herb (sedge)	4	Moist soil to permanent water	Rapid establishment
wet/pond	*Schoenus apogon*	Common Bog-rush	L	G	0.05-0.3	Slender perennial tufted herb	8-10	Moist or wet soils	Variable; widespread
bio/swale	*Schoenus lepidosperma*		SM	M	0.1-0.6	Perennial, tufted or with short rhizome	6-8	Moist soils	Tough; spreads to form dense clumps
wet/pond	*Triglochin procerum*	Water -ribbon	M	M	0.2-0.5	Aquatic or amphibious perennial herb with erect or floating leaves	4	Semi-aquatic to aquatic to depth of 1.5m	Aesthetic; spreading
wet/pond	*Vallisneria gigantea*	Ribbonweed	P	S	to 3	Submerged, dioecious tufted stoloniferous perennial with floating flowers	1	Open water; depth of <0.1-4m	Rapid growth; salt tolerant (1500 ppm)

Suitability	Scientific Name	Common Name		Form	Height(m)	Description	Planting Density (plants/m²)	Requirements	Comments
wet/pond	*Villarsia exaltata*	Yellow Marsh Flower	SM	M	0.3, stems to 1.5	Tufted herb; broad basal leaves	6-8	Wet soils to 1 m depth	Leaves float if growing in water
wet/pond	*Villarsia reniformis*	Running Marsh Flower	L	M	0.4	Tufted; stoloniferous if growing in water	6-8	Moist to wet soils	Aesthetic
all	*Viola hederacea*	Native Violet	L	G	Prostrate - 0.15	Stoloniferous herb forming a dense mat	2-4	Moist to wet soil	Rapid growth; aesthetic; prolific growth once established

Appendix C Resource Guide

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C.1 Disclaimer

The documents, products and services represented in this guide are in no way endorsed by the author of this document. They have been selected as useful points of reference to gather the essential information required to construct WSUD elements. The listings are **by no means a complete reference** and suppliers should be sought locally for individual projects.

C.2 Documents

National Management Measures to Control Nonpoint Source Pollution from Urban Areas (US EPA, 2002)

This extensive document provides guidance regarding management measures that may be used to reduce nonpoint source pollution from urban activities. The document also provides technical advice on the best available, most economically achievable means of managing urban runoff and reducing nonpoint source pollution of surface and ground waters from urban sources.

Australian Runoff Quality (Engineers Australia, 2006)

This document contains information on the main stormwater pollutants and the observed data on their loadings. There are also chapters that contain detailed information on the most commonly used WSUD treatment systems and their design, installation, maintenance and life cycle costing. Detail is also provided on urban waterways and the modelling of urban stormwater management systems.

The document can be downloaded from:

http://www.eng.newcastle.edu.au/~ncwe/ncweARQ/arqSummary.htm

Guidelines for Treatment of Stormwater Runoff from the Road Infrastructure (Austroads, 2003)

This document provides information on the treatment of stormwater runoff from road infrastructure. Reference is made to the relevant policies for each state and there is detailed information on the typical contaminants and concentrations from road runoff. A series of worked examples are included for reference.

Stormwater Management Devices : Design Guidelines Manual (Auckland Regional Council, 2003)

This detailed and comprehensive document provides information on the selection, design, construction, maintenance and monitoring of WSUD elements and the impacts of catchment land use and development type on the method of treatment chosen. Each treatment type is discussed in detail.

An extract from the introduction reads – 'The objective of this manual is to provide a commonly accepted design approach for stormwater management practices that will provide both quantity and water quality benefits, thus befitting the wider environmental issues we face'.

The document can be downloaded from www.arc.govt.nz .

Water Sensitive Urban Design Guidelines for the City of Knox (Knox City Council, 2002)

This document is a series of guidelines for the implementation of WSUD principles for the City of Knox. The document covers issues such as:

Summary of issues around urban stormwater management

Outline of the benefits of incorporating WSUD elements

Provision of guidance for the implementation and maintenance of WSUD principles in new and existing environments

Provision of indicative details for adoption by local government in order to include WSUD principles in new and existing developments

Water Sensitive Planning Guide: for the Sydney Region (Upper Parramatta River Catchment Trust, Sydney Coastal Councils Group, Western Sydney Regional Organisation of Council (WSROC) and Lower Hunter Central Coast Regional Environmental Management Strategy (LHCCREMS), 2003)

'This document presents a planning guide that can be used to promote water sensitive urban design and a more integrated approach to urban water cycle management. It is primarily for planners in local government. It can also be a resource for development applicants, including developers, architects and engineers'.

The document can be downloaded from http://www.wsud.org

Water Sensitive Urban Design – A Stormwater Management Perspective (CRCCH, 2002)

Report on the design process, construction activities and monitoring of environmental, social and economic performance indicators for a WSUD stormwater treatment system for a residential development in Victoria.

Stormwater Implementation Project: Statutory Framework and Standards (Association of Bayside Municipalities, 2001)

Report on the development and implementation of stormwater management plans for local authorities through the planing system with the output goals being – 'to develop model planning scheme provisions that provide the necessary detail and statutory force to assess development proposals and to guide selection of appropriate best practice stormwater management techniques for different urban sites, conditions and development scenarios'.

The following product suppliers provide services.

C.3 Pipe Suppliers (plastic and concrete) –

Iplex – (Distribution) 73 – 79 Lilydale Road ROCHERLEA TAS 7248

Telephone: ++ 61 3 6326 8031

Facsimile: ++ 61 3 6326 5277

Vinidex – (Distribution) 15 Thistle Street South Launceston, TAS 7249

Telephone: + 61 3 6344 2521

Facsimile: + 61 3 6343 1100

Humes – Contact via the web page www.humes.com.au

James Hardie (FRC) – Freecall 1800 659 850 or Freefax 1800 639 908

ROCLA – PO Box: Locked Bag 7013, Chatswood, NSW, 2067

ph: 02 9928 3500

fax: 02 9928 3580

C.4 Pre-cast concrete products –

Concrete designs – 9 Lampton Ave Derwent Park TAS 7009 ph: (03) 6273 0463

Humes – 19- 25 Churchill Park Drv Invermay TAS 7248 ph: (03) 6335 6300

James Hardie (FRC) – Freecall 1800 659 850 or Freefax 1800 639 908

C.5 Geosynthetic products –

Maccaferri – (Purple Pig) 8A Lampton Ave Derwent Park TAS 7009 ph: (03) 6272 1055

GeoTas – 14 Chesterman St Moonah TAS 7009 ph: (03) 6273 0511

Bidim Geotextiles – 14 Chesterman St Moonah TAS 7009 ph: (03) 6273 0511

C.6 Sand/soil/blue metal suppliers

HBMI – Southern Outlet Leslie Vale TAS 7054 ph: (03) 6239 6410

Boral Construction Materials Group Ltd – Midlands Hwy Bridgewater TAS 7030

ph: (03) 6268 4177

Hanson Construction Materials Pty Ltd – Flagstaff Gully Rd Lindisfarne

TAS 7015 ph: (03) 6243 8077

Leslie Vale Quarry – 119 Websters Rd Leslie Vale TAS 7054

ph: (03) 6239 6952

C.7 Native plant nurseries

Redbreast Plants – 1709 Channel Hwy Margate TAS 7054

ph: (03) 6267 2871

Clark's Millvale Nursery – 96 Millvale Rd Dromedary TAS 7030

ph: (03) 6263 7132

Plants Of Tasmania Nursery – 65 Hall St Ridgeway TAS 7054

ph: (03) 6239 1583

Pulchella Nursery – Tasman Highway, Buckland, TAS. 7190

ph: (03) 6257 5189

C.8 Treatment systems (GPT's)

Stormwater360

http://www.stormwater360.com.au

ROCLA

http://www.rocla.com.au

Humes

http://www.humes.com.au

Atlantis

http://www.atlantiscorp.com.au/

James Hardie

http://www.jameshardiepipes.com.au/

Appendix D Cyanobacterial growth in constructed water bodies

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Introduction

Algal growth can occur rapidly under favorable conditions in open water bodies. Nuisance growths (blooms) of cyanobacteria (Blue-green algae) can occur in both natural and constructed water bodies. In constructed water bodies it is important to ensure that designs include measures to restrict cyanobacterial growth. Cyanobacterial blooms can have adverse effects on aquatic ecosystem function, Aesthetics and public amenity. Some species of cyanobacteria are of particular concern because of their potential to produce toxins.

D.1 Light

In Australian climatic conditions surface light is rarely a limiting factor for algal growth. Cyanobacterial responses to various light conditions differ between species. Turbidity and mixing conditions within a water body can determine the light environment that algal cells are exposed to by circulating them in and out of the euphotic zone. Typically, cyanobacterial growth rates are reduced under fluctuating light conditions such as those found in well mixed water columns (Mitrovic *et al*, 2003).

Some cyanobacterial species can regulate cell buoyancy and migrate vertically, increasing their exposure to optimum light intensities. Cell buoyancy regulation offers cyanobacteria considerable advantage over other phytoplankton that are distributed evenly throughout the water column (Mitrovic *et al*, 2001). However, this buoyancy advantage is dependent on the mixing regime and degree of turbulence that the cells are exposed to within the water column (Brookes *et al*, 2003).

Depth of light penetration can be reduced by turbidity and therefore limit biomass development. The extent to which turbidity will reduce light availability to cells, and therefore reduce biomass development, depends on the mixing patterns of the water body and the degree of cell buoyancy regulation.

D.2 Temperature

Temperature is an important factor in many cyanobacterial bloom occurrences in Australia. In temperate zones cyanobacterial blooms commonly occur in the warmer months. Cyanobacteria tend to have high optimal growth temperatures compared to green algae and diatoms and achieve maximum growth rates at around 25°C (Chorus and Bartram, 1999).

D.3 Nutrients

Many cyanobacterial blooms are associated with elevated nutrient levels. However, nutrient availability in many aquatic environments is generally adequate to achieve cyanobacterial growth of bloom proportions when other factors such as temperature and hydrodynamics are also favourable. Many of the nuisance species of cyanobacteria are capable of fixing atmospheric nitrogen however, this process requires considerable amounts of energy and may be limited in turbid environments (Chorus and Bartram, 1999).

D.4 Hydrodynamics

A key parameter of aquatic ecosystems is hydraulic detention time (Harris, 1996; Jorgensen, 2003). Long detention times during warm weather in poorly mixed water bodies often leads to persistent stratification of the water column. Periods of stratification of a water body can also facilitate the release of nutrients from the sediments which can act to support algal growth. In lowland rivers and lakes, cyanobacterial blooms are more prevalent during periods of persistent stratification, a condition associated with low flows (Sherman *et al*, 1998). Cyanobacterial species that can regulate their buoyancy, and migrate vertically through the water column, have a competitive advantage over other phytoplankton under stratified conditions (Atlas and Bartha, 1998). Buoyancy regulation allows cell movement between the nutrient rich hypolimnetic waters and the euphotic zone so as to access both high nutrient and optimal light conditions.

In deep water bodies, hydraulic mixing and the breakdown of stratification can slow the growth of cyanobacteria and reduce the prevalence of excessive growth. Hydraulic mixing reduces growth rates by circulating cells below the euphotic zone for long enough to limit light availability, reducing carbohydrate accumulation and exhausting the energy supply required for growth and replication (Brookes *et al*, 2003).

In shallow water bodies, where the ratio of mixing depth: euphotic zone depth is < 3-5, mixing is typically insufficient to reduce growth (Oliver *et al*, 1999). Under such conditions, hydraulic detention time becomes a crucial factor in the control and prevention of excessive algal growth. When the hydraulic detention time is reduced the biomass becomes regulated by the rate at which it is removed from the lake by flushing (Reynolds, 2003).

D.5 Growth Rates

Assuming adequate light and nutrient availability, a model of algal growth can be developed using a simple relationship between time and growth rate at various temperatures. The exponential growth rate equation is $\mu = (1/t) \times \ln(N_t/N_0)$, where μ is the growth rate, t is the number of days, N_t is the final cell concentration and N_0 the starting cell concentration. This simple model can be used to determine how long it will take for an algal population to reach bloom proportions (15,000 cells/mL) and hence inform the development of guidelines on water body hydraulic detention time.

D.6 Common growth rate range

Under favorable growth conditions (20°C and light saturation) laboratory cultures of planktonic cyanobacteria have growth rates of between 0.21 and 0.99 day^{-1}, or 0.3 to 1.4 doublings per day respectively (Chorus and Bartram, 1999). Figure 1 illustrates theoretical growth curves based on growth rates of laboratory grown cultures that have been adjusted to account for a slower growth rate (0.5 normal growth rate) at night (12 out of 24 hours). The graphs are indicative of the range of growth rates both between species and between individual populations of the same species grown in laboratory cultures.

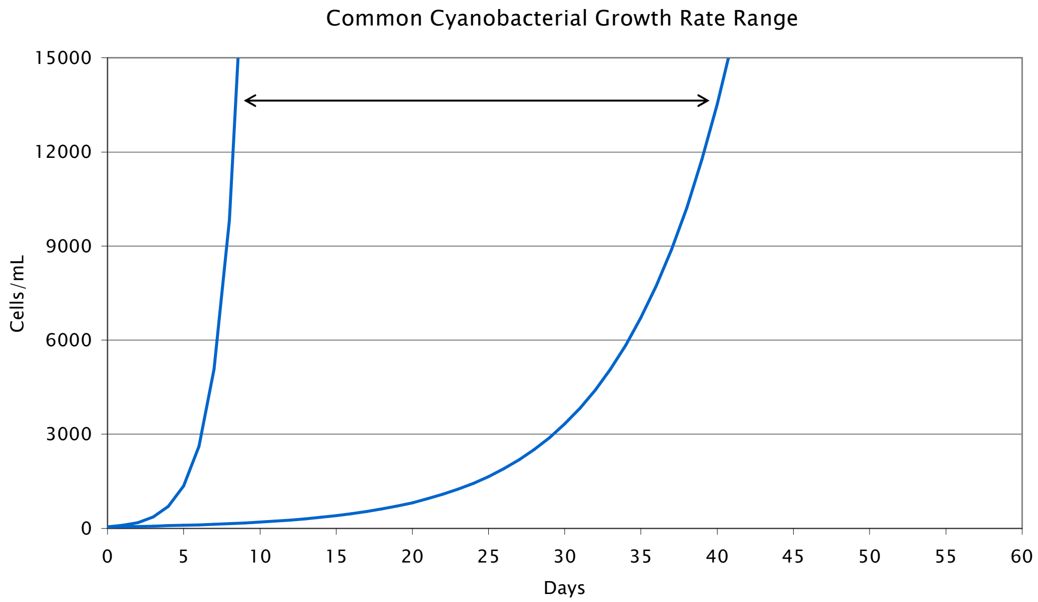

Figure D.1. Common cyanobacterial growth rate range illustrated using theoretical growth curves based on growth rates of laboratory grown cultures (20°C and light saturation) adjusted for a 12h:12h light dark cycle. Growth curves were constructed using an initial algal cell concentration of 50 cells/mL.

These results illustrate the wide range of growths rates that have been recorded for cyanobacteria and suggest that, under ideal conditions at 20°C, laboratory cultured cyanobacteria can achieve bloom conditions in 9 – 41 days depending on the species.

D.7 Laboratory cultures v *in situ* growth rates

Physiological characteristics such as maximum photosynthetic capabilities, photoinhibition levels and flotation rates (speeds of vertical movement) vary considerably between cyanobacterial species and between individual populations within species. Growth rates also decrease with increasing cell or colony sizes (Reynolds 1984). Environmental variables, such as those discussed earlier, influence which species will dominate and the maximum growth rate. Typically, slower *in situ* growth rates occur as a result of these environmental variables. The relationship between laboratory growth rates and *in situ* growth rates is poorly understood. For example, *Microcystis* rarely grows in colonial form when grown in laboratory cultures however, successful growth of colonies in culture have shown much slower growth rates than those recorded previously from unicellular cultures (Reynolds, 1984). As a result, *in situ* growth rates are more desirable to use in models attempting to predict *in situ* conditions.

D.8 Mixing conditions

Westwood and Ganf (2004) measured the *in situ* growth of *Anabaena circinalis* in the Murray River at Morgan (Table D.1). Growth was measured under well mixed and persistently stratified conditions and also under conditions that take into account a range of typical flotation velocities (or mixing conditions) recorded for *A. circinalis* populations (0.01 to 0.40 m h^{-1}).

Table D.1. *In situ* growth rates for *Anabaena circinalis* under various mixing conditions

Species	Hydrodynamic Treatment	Growth rates (day^{-1})	Reference
Anabaena circinalis	Persistent stratification	0.43	Westwood and Ganf (2004)
	1.0 m h^{-1} mixing rate (diurnal stratification)	0.23	
	0.5 m h^{-1} mixing rate (diurnal stratification)	0.15	
	Well mixed	0.19	

Figure D.2 has been constructed based on the *in situ* growth rates of *A. circinalis* recorded by Westwood and Ganf (2004). With starting cell concentrations of 50 cells/mL, the measured growth rates of neutrally buoyant populations under well-mixed conditions suggested the population would take approximately 31 days to reach bloom proportions. Under persistently stratified conditions, bloom proportions would be reached within 14 days. *A. circinalis* populations with flotation velocities of 0.5 and 1.0 m h^{-1}, and under diurnally stratified conditions, would take longer than 25 days to reach bloom proportions.

Water bodies incorporating best practice design features are assumed to be relatively shallow (< 2.5 – 3.0 m), have a flat bottom and be subject to wind mixing. These design features are assumed to prevent persistent stratification and create systems that are well mixed or only diurnally stratified. Where diurnal stratification occurs, mixing rates during the non-stratified period are expected to be relatively fast due to the shallow nature of the water body. As a result, *in situ* growth rates for a fully mixed system and *in situ* growth rates for a partially mixed system with a relatively fast mixing rate, have been adopted. Figure D.2 shows the expected mixing conditions for water bodies that incorporates best management practice design features.

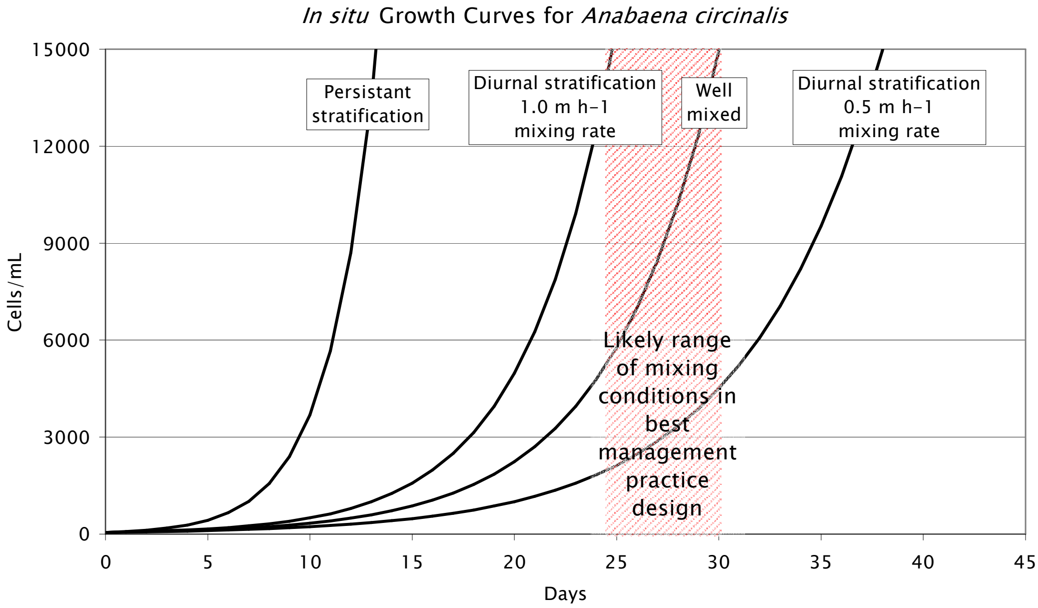

Figure D.2. Growth of *Anabaena circinalis* under various mixing conditions illustrated using growth curves constructed from data collected *in situ* (Westwood and Ganf, 2004) and assuming starting cell concentrations of 50 cells/mL. Area of shading represents the range of mixing conditions likely to be found in best practice design systems.

D.9 Temperature effects

Provided that other factors (e.g. light, nutrients) remain non-limiting, maximum growth rates of cyanobacteria respond directly to changes in temperature. Specific responses to temperature changes differ between species but, typically, growth rates increase with increasing temperature (Reynolds, 1984). The effect of temperature can be accounted for by adjusting growth rates using Q_{10} values[6]. Data presented in Table D.2 indicate that Q_{10} values can vary significantly between species.

[6] Q_{10} is the temperature coefficient (Q_v) that represents the increase in growth rate that occurs with a 10°C increase in temperature.

Table D,2. Q_{10} values for a range cyanobacteria

Genus	Q_{10} Range	Temperature Range (°C)	Reference
Asterionella, Anabaena, Aphanizomenon and *Oscillatoria*	1.8 – 2.9	10 – 20	Reynolds (1984)
Microcystis, Merismopedia and *Oscillatoria*	1.97 – 4.16	15 – 25	Coles and Jones (2000)

D.10 Starting concentration

The theoretical growth rate curves are constructed using initial cell counts of 2 cells/mL and 50 cells/mL which represent typical natural background levels. Webster *et al* (2000) found blooms in the Maude Weir pool forming from initial concentrations of 10 cells/mL. It is clear that the initial starting concentration can influence the time required to reach bloom proportions (although the degree of influence will be depend on the growth rate). For instance for *Anabaena circinalis* in well mixed conditions and 20°C, starting concentrations of 2 and 50 cells/mL result in bloom proportions of 15,000 cells/mL after approximately 33 and 51 days respectively, as illustrated in Figure D.3.

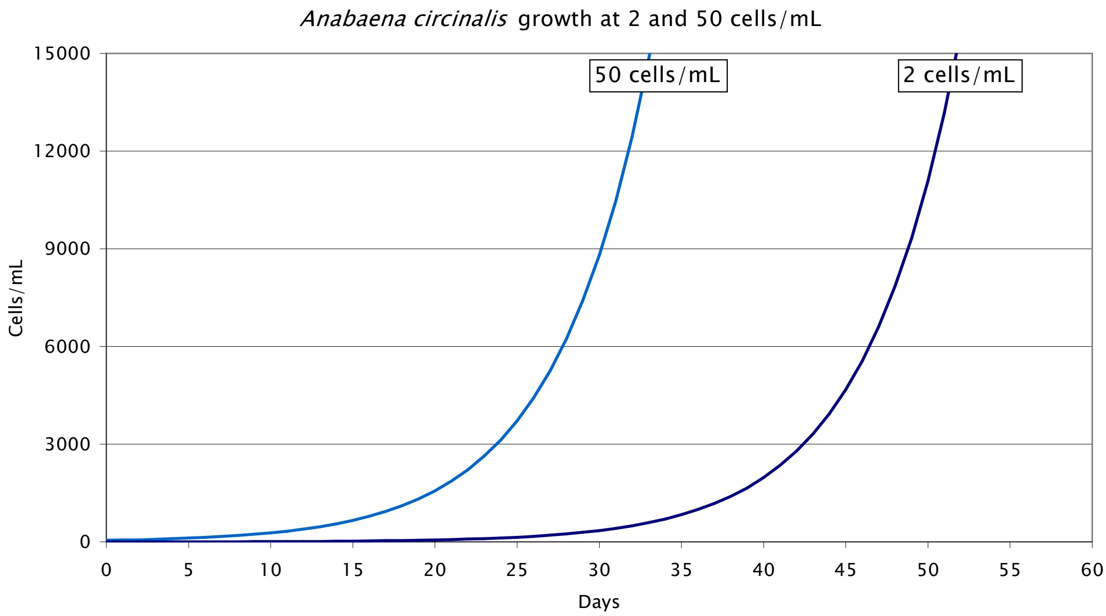

Figure D.3. Cyanobacterial growth curves at starting concentrations of 2 and 50 cells/mL. Constructed using growth rates of *A. circinalis* measured *in situ*, under well mixed conditions (Westwood and Ganf, 2004), adjusted for 20°C (Q_{10} 2.9). The number of days taken to reach bloom proportions varies from 33 to 51 days depending on the starting cell concentration.

D.11 Detention time

Reynolds (2003) recommends that the sensitivity of lakes to eutrophication, in relation to changes in external phosphorus loads, can be classified according to hydraulic detention time. Short detention times weaken the response of lakes to changes in external phosphorus loads. The weakened response of lakes, to changes in phosphorus loads, is due to the biomass becoming regulated by the rate at which it is removed from the lake by flushing, rather than the availability of phosphorus (Reynolds, 2003). The most sensitive lakes being those with a detention time of greater than 30 days. Lakes with a detention time of 3 – 30 days are only slightly sensitive to changes in external phosphorus loads, while lakes with a detention time of less than 3 days are not at all sensitive to changes in phosphorous loads (Reynolds, 2003).

In the Australian climate, designing constructed water bodies with a detention time of less than 3 days is neither practical nor achievable. An upper limit of 30 days may be applied as a general precaution to ensure that water bodies do not lie within the 'very sensitive' category of >30 days detention time. Wagner-Lotkowska *et al* (2004) recommend a hydraulic detention time of less than 30 days for the control of algal blooms in medium sized reservoirs.

Wastewater treatment ponds could be viewed as ideal environments for algal growth (shallow, adequate light, high nutrients). However, experience has shown (e.g. Breen, 1983) that it is rare to get cyanobacteria dominating the phytoplankton community in wastewater treatment ponds with detention times below 30 days.

D.12 Model parameters

From the information presented in previous sections, the values presented in Table D.3 have been adopted to create a model appropriate for water bodies with best management practice design. These systems are assumed to be shallow, have a flat bottom and are generally well mixed. A reasonable assumption is that the hydrodynamic conditions in a best management practice design varies somewhere between fully mixed and diurnally, partially mixed as represented by the shaded zone in Figure D.2.

Table D.3. Summary of model parameters

Variable	Value	Comment	Reference
Hydrodynamics	Well mixed to 1.0 m h^{-1} with diurnal stratification	Water bodies incorporating best practice design are assumed to be relatively shallow, have a flat bottom and be easily mixed by wind. As a result, *in situ* growth rates for a fully mixed system and a partially mixed system with a relatively fast mixing rate, have been adopted. From Figure 2 this approach is considered conservative.	Mixing values from Westwood and Ganf (2004)
Growth rate	0.19 to 0.23 day^{-1}	Adoption of *in situ* growth rate of a common nuisance cyanobacterial species (*Anabaena circinalis*) is considered reasonable given the frequency of *Anabaena* in blooms.	Westwood and Ganf (2004)

Variable	Value	Comment	Reference
Q_{10}	2.9	Adoption of the upper limit of the range of Q_{10} values recorded for various genera including *Anabaena* is considered a conservative assumption.	Reynolds (1984)
Temperature range	15-25°C	Likely temperature ranges of surface waters in Victoria	
Starting concentrations	50 cells/mL	Conservative, or likely upper limit, of background cell concentrations for cyanobacteria in water bodies without chronic bloom problems.	

D.13 Modelling Results

Model results are shown in Figures D.4 and D.5 for partially and well mixed systems respectively. The temperature ranges can be broadly interpreted in Victoria as follows:

15°C Use for upland sites in the Eastern and Western Ranges.

20°C Use for lowland sites south of the Great Dividing Range.

25°C Use for lowland sites north of the Great Dividing Range.

The values represent summer water temperatures. Local water body temperature will clearly vary from site to site and within different years. Where local water temperature data are available they should be used to guide the selection of the critical detention time.

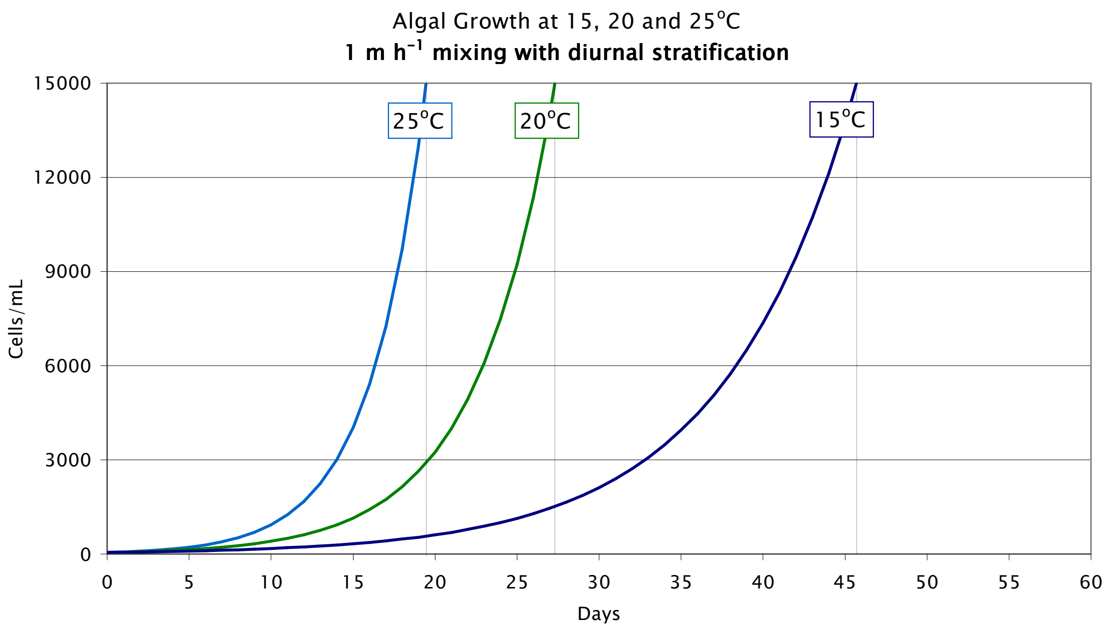

Figure D.4. Growth curves illustrating modelled times for cyanobacterial populations to reach bloom proportions under different temperature conditions and 1 m h^{-1} mixing conditions with diurnal stratification. Based on growth rates of *A. circinalis* measured *in situ* (Westwood and Ganf, 2004) adjusted for temperature, Q_{10} 2.9, and assuming 50 cells/mL starting concentrations.

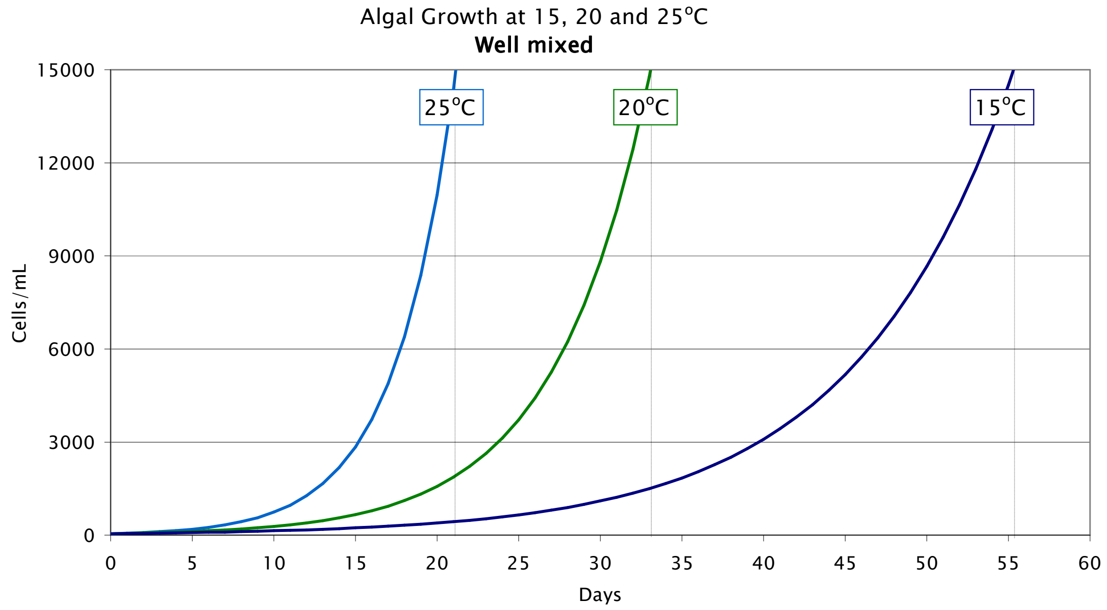

Figure D.5. Growth curves illustrating modelled times for cyanobacterial populations to reach bloom proportions under different temperature conditions and well mixed conditions. Based on growth rates of *A. circinalis* measured *in situ* (Westwood and Ganf, 2004) adjusted for temperature, Q_{10} 2.9, and assuming 50 cells/mL starting concentrations.

Target detention times for the modelled temperature ranges are summarized in Table D.4 for both partially and well mixed systems. It is likely that the hydrodynamic state of best practice design water bodies would move between the proposed mixing conditions.

Table D.4. Modelled times for cyanobacterial populations to reach bloom proportions under different temperature conditions.

Variables	Partially Mixed			Fully Mixed		
Temperature (°C)	15	20	25	15	20	25
Time (days)	46	27	19	55	33	21

The modeling approach taken is considered to be reasonably conservative. For example it adopts:

- Non-limiting conditions for nutrient and light availability
- Growth rates for a known nuisance species (*Anabaena circinalis*)
- Summer temperature values (the main risk period)
- High starting population concentrations (50 cells/mL)

As a result, a probabilistic approach to the use of detention time criteria is recommended. A 20% exceedance is suggested as an acceptable risk to compensate for the occurrence of all other risk factors being favorable for algal growth. The 20% exceedance of a specific detention time objective does not indicate that a bloom will occur; just that detention time (for a given temperature range) is long enough for exponential growth to achieve a bloom alert level of 15,000 cells/mL if all other risk factors were favourable. The 20% exceedance value is an interim value chosen as a relatively conservative estimate of the general variation in ecological factors in the Australian environment.

D.14 Recommended design criteria

The following guideline detention times are recommended. For water bodies with summer water temperatures in the following ranges, the 20%tile detention times should not exceed:

- 50 days (15°C)
- 30 days (20°C)
- 20 days (25°C)

These values are broadly consistent with literature detention time values considered to be protective against the risk of cyanobacterial blooms (Reynolds 2003, Wagner-Lotkowska *et al* 2004) and consistent with current industry experience.

D.15 Acknowledgements

Thank you to Professor Barry Hart, Water Studies Centre, Monash University, for providing a helpful review of the first draft of this technical note and also to Melbourne Water for supporting this work.

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Appendix E Design Flows - t_C

The travel time of the overland flow path (t_c)can be estimated using either the Bransby Williams formula for time of concentration or by the overland kinematic wave equation as presented in Australian Rainfall and Runoff (2003).

Each method has advantages and disadvantages. The Kinematic Wave equation is the most accurate method of calculating t_c and is generally suited to most catchments. As the equation requires the designer to solve for t and $I^{0.4}$ simultaneously, an iterative approach must be undertaken (or use a previously prepared relationship table for $I^{0.4}$ for the study area). The Bransby Williams formula is well suited to situations where no actual relationships for t_c have been calculated based on observed data, and it does not require an iterative process to reach a solution making it attractive to designers new to these theories or in areas where little catchment response data exists.

It should be noted, however, that where a system is being designed to incorporate detention/retention for downstream flood control t_c should be replaced with $T_{c\text{-critical}}$, ie the time of concentration for the critical point of the total downstream catchment (the point at which unacceptable flooding is most likely to occur).

Kinematic wave equation	Bransby Williams formula for t_c
$t = \dfrac{6.94(L \cdot n^*)^{0.6}}{I^{0.4} \cdot S^{0.3}}$	$t_c = \dfrac{91 \times L}{A^{0.1} \times S_e^{\ 0.2}}$
Where: **t** is the overland travel time (minutes) **L** is the overland flow path length (m) **N*** is the surface roughness (concrete or asphalt ~ 0.013) **I** is the design rainfall intensity (mm/hr) **S** is the slope	Where: **t_c** is the time of concentration (minutes) **L** is the main stream length measured to the catchment divide (km) **A** is the catchment area (Ha) **S_e** is the grade of the main stream (m/km)

EquationE.1 and Equation E.2

Appendix F Construction phase management

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F.1. Introduction

Accelerated erosion during construction works poses a significant threat to receiving water ecosystems and natural sediment balances across Tasmania. It has long been understood and widely published those construction activities failing to utilise sound soil and water management practices result in massive increases to sediment loads in stormwater runoff.

Crucial to the long-term success of any WSUD development is effective management of soil and water throughout the construction and establishment phases. Sediment and erosion control during development and construction activities is necessary in all construction works involving ground disturbance.

In the WSUD development, long term stormwater treatment WSUD elements may be utilised for construction-phase sediment control with care and planning. However, construction activities may also jeopardise the longevity and effectiveness of WSUD elements. For example, ponds could be used as sediment control basins during construction prior to vegetating, bordering etc. Conversely, high construction related sediment loads entering a bioretention system established too early would cause blocking of the filtration media rendering it ineffective from day one.

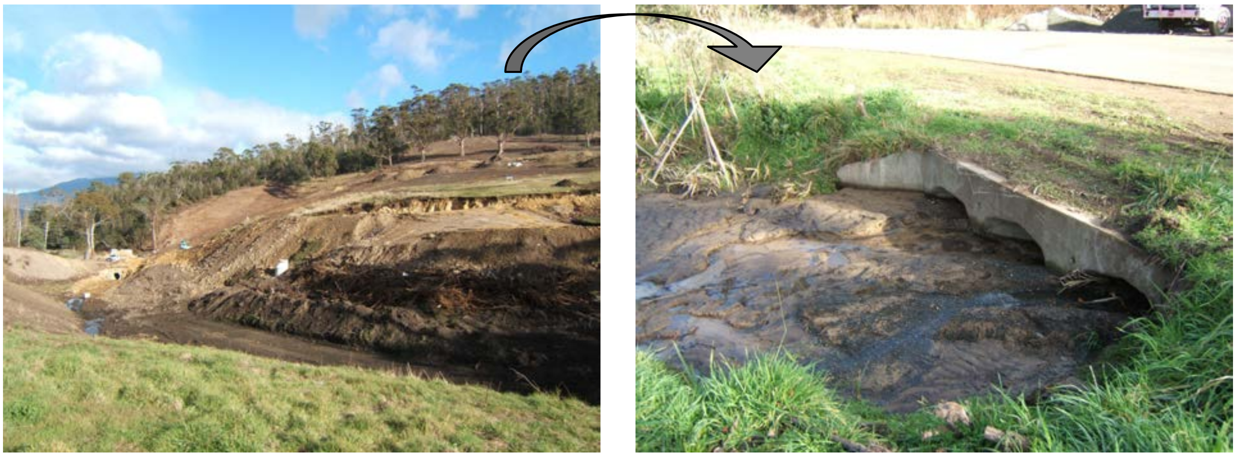


Figure F-1. Unmanaged runoff from this construction site has resulted in blocked stormwater infrastructure and heavy siltation in the nearby creek.

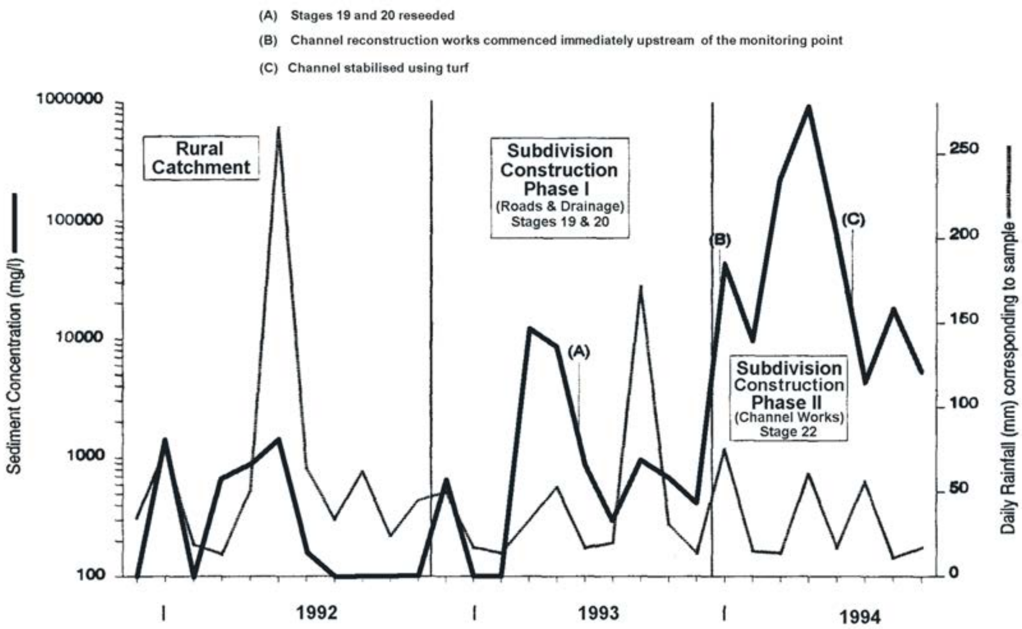


Figure F-2. Sediment concentrations and daily rainfall at a development [source: Goldrich & Armstrong, 1994; cited in Landcom, 2004]

F.2. Core principles of sediment and erosion control

F.2.1. Timing

- Minimise the duration and extent of disturbed soil surfaces
- Reinstate surfaces as soon as works in the immediate area are complete

F.2.2. Materials

- Topsoil should be scraped from an area prior to earthworks and stockpiled so that it may be later used for more effective reinstatement and re-establishment of vegetative cover

F.2.3. Flow management

- Ensure no upslope runoff enters the site
- All stockpiled loose materials (e.g. soil, road base, etc) should be bunded around the circumference of the mound so that no runoff from upslope reaches the pile and material entrained by direct rainfall is not carried away from the pile
- Always consider flow paths from overtopping control measures and utilise natural depressions for runoff detention
- Always construct and maintain a control device (e.g. silt fence) at the point where runoff will leave the site

F.2.4. Sediment capture

- Some sediment control technique will always be required where runoff leaves the site (usually lowest point)
- Distributed or staged techniques around the site are more effective than one control structure at the end

F.2.5. Maintenance

- All management measures should be inspected at the commencement and completion of works each day
- Curb and gutter controls (e.g. 'filter socks') should be cleaned every day

F.3. Planning

Early planning is crucial to successful and cost-effective management of soil and water throughout the construction and establishment phase of a WSUD development. WSUD elements may be utilised in the construction phase for sediment and erosion control, however, partially constructed WSUD elements may also require protection to ensure their long-term viability.

Partially constructed ponds, wetlands and large bioretention basins may be used as sedimentation ponds during the construction of a WSUD development. This reduces excavation and earthworks costs for the development by reducing the need for additional sedimentation basin construction and should be sited in an appropriate location within the development, as a centralised WSUD feature would usually be located at the downstream point of the development site. This technique will require that the wetland/pond/bioretention is the first part of the site to be (partially) constructed and the last stage of the development to be completed.

The level of planning required for sediment and erosion control during the construction phase of a project varies with the size of development and the duration of works. Small maintenance works on WSUD systems may only involve a visual on-site assessment and the installation of temporary sediment control measures such as sandbags/filter socks and/or silt fences. Large-scale projects, such as sub-divisions, will require a full sediment and erosion control plan (or soil and water management plan) to be prepared prior to commencement of works.

The State Policy on Water Quality Management 1997 stated that 'codes of practice' should be developed for the control of construction-related erosion. Soil and Water Management on Building and Construction Sites (2009) is a code of practice with current best practice sediment and erosion control measures that should be used. The Soil and Water Management on Building and Construction Sites (2009) states that soil and water management plans (SWMPs) should be prepared, and submitted for approval prior to any works, for any development covering greater than two hundred and fifty square meters. A range of other 'triggers' necessitating a SWMP are discussed, including: where a high pollution risk to receiving waters exists and where works are to be performed over an extended duration.

F.3.1. Soil & water management plans (SWMPs)

The Soil and Water Management on Building and Construction Sites (2009) states that a soil and water management plan should include the following information:

- Date and author.
- North point and scale.
- Property boundaries.
- General soil description.
- Location and amount of ground disturbance.
- Initial and final contours, location of watercourses, surface drainage and existing stormwater infrastructure.
- Stormwater discharge point, if proposed.
- Location of all proposed temporary drainage control measures.
- Construction details (e.g. building or subdivision layout).
- Location of vegetation to be retained and removed.
- Location of stabilised site access.
- Location of soil, sand or other material stockpiles.
- Location and details of all proposed erosion control measures.
- Location and details of all proposed sediment control measures.
- A statement of who is responsible for establishing and maintaining all erosion and sediment control measures.
- The installation sequence of the different sediment and erosion controls.
- The maintenance program of the sediment and erosion controls.
- The revegetation and rehabilitation program.

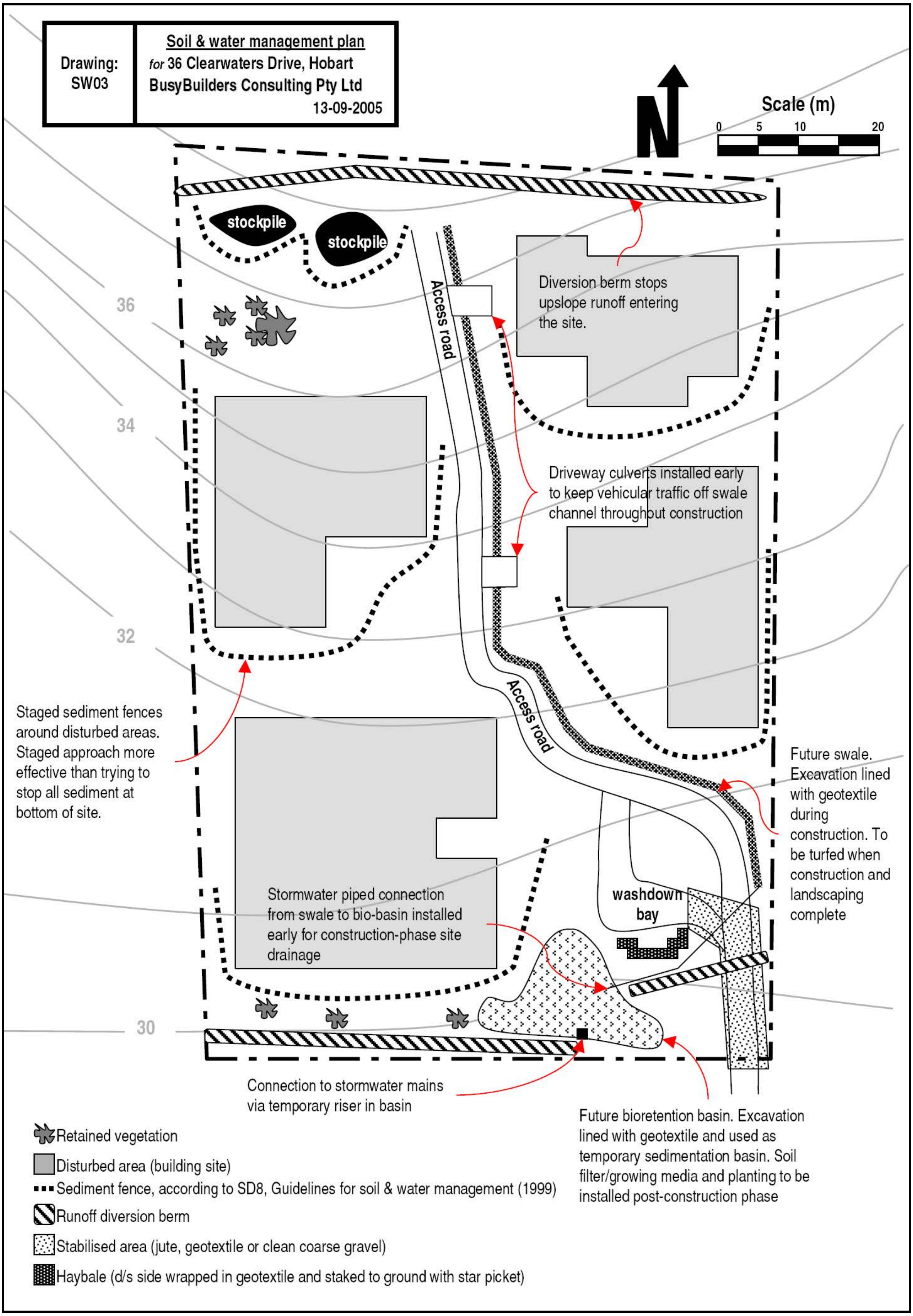


Figure F-3. Soil and water management plan for a small WSUD sub-division

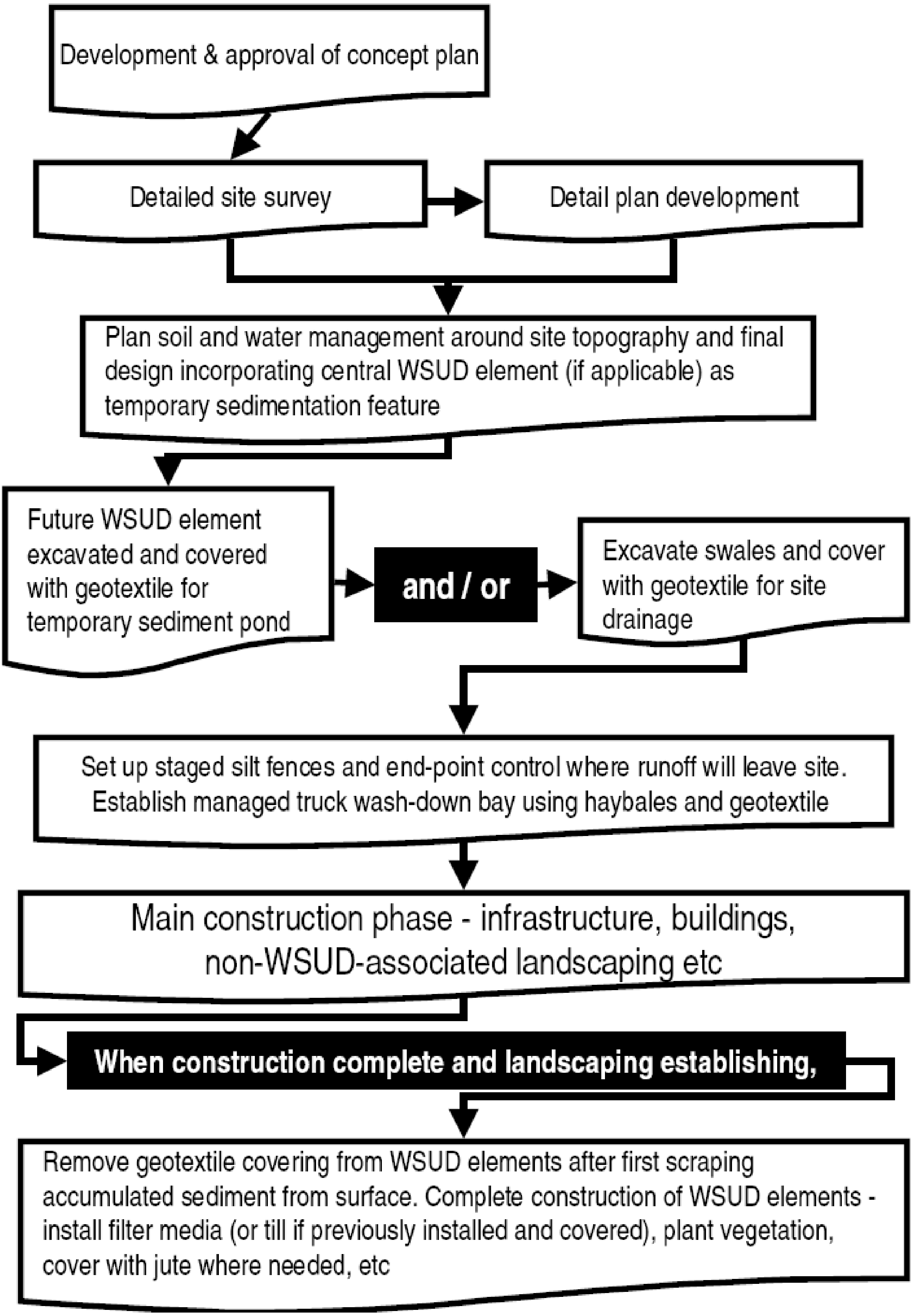


Figure F-4. Possible work-flow for sediment and erosion control at a WSUD sub-division

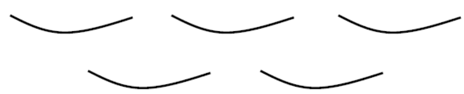
F.4. Occupational health and safety

Implementation of sediment and erosion control techniques around a construction site involves a number of issues that need to be managed within the sites occupational health and safety strategy.

Each site should be assessed for possible risks prior to establishing controls and again when controls are in place to ensure all dangers have been identified, assessed and managed.

Following are some issues identified with common techniques, however, all sites and techniques should be assessed on a site-by-site basis by a suitably qualified individual.

Table F-1. Examples of sediment and erosion control O, H & S risks

Technique	Risk	Management
Sediment fencing	Injury from tripping/falling on star pickets	Star pickets should be capped with high visibility caps immediately following installation to improve visibility and reduce the likelyhood of an incident occuring. If an incident occurs, the capping provides some protection against injury.
	Tripping on fence	Ensure fence is at least 500mm high to increase visibilty. Also use bright coloured geotextile or mark with reflective tape at regular intervals. Set up silt fences in staged arc formations (see below). This provides superior runoff control and provides ‘walk-through’ points. 
Sedimentation basins	Drowning	Sedimentation basins should be surrounded with high visibility temporary fencing to prevent accidental falling in, particularly if works continue in dusk/evenings. Pool standard fencing is also required if site is accessible by public. A flotation rescue device should also be stored nearby. Deep, saturated sediments can have a ‘quick-sand’ effect and make getting out of a sedimentation basin extremely difficult particularly if batter slopes are steep.
	Pests	Long-duration sites with permanent

		sedimentation basins should monitor the water body for mosquito breeding and treat appropriately where required.
Geotextile coverings	Lifting-related injury	Where geotextiles have been used to protect elements from sedimentation, accumulated material should be scraped from the fabric prior to lifting it. General lifting O, H & S principles should be applied wherever manual lifting is required.

NOTE: The risks detailed above are only some possible increased risks associated with sediment and erosion control activities at a construction site. Any construction activity requires a specific site assessment of risk and a strategy for manageing those identified risks.

F.5. References / further reading

Derwent Estuary Program (2009). Soil and Water Management on Building and Construction Sites, Derwent Estuary Program, Hobart

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Workplace Standards Tasmania
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