Water Sensitive Urban Design

Technical Design Guidelines for South East Queensland

Version 1 June 2006

Contents

Chapter 1	**Introduction**
Chapter 2	**Swales (incorporating Buffer Strips)**
Chapter 3	**Bioretention Swales**
Chapter 4	**Sediment Basins**
Chapter 5	**Bioretention Basins**
Chapter 6	**Constructed Stormwater Wetlands**
Chapter 7	**Infiltration Measures**
Chapter 8	**Sand Filters**
Chapter 9	**Aquifer Storage and Recovery**
Appendix A	**Plant Selection for WSUD Systems**

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The guidelines are based on Brisbane City Council's *Water Sensitive Urban Design Engineering Guidelines: Stormwater*. The Brisbane guidelines were developed from a similar document by Melbourne Water. Providing the Partnership with access to these documents represents a substantial contribution from both of these organisations. Additional material has been kindly provided by the *Water Sensitive Urban Design in the Sydney Region* project, from their *Technical Guidelines for Western Sydney*.

Australian Government

Chapter 1 Introduction

1.1 Introducing Water Sensitive Urban Design 1-2

1.2 Scope of These Guidelines 1-2

1.3 Structure of the Guidelines 1-4

1.4 The Policy and Planning Context for WSUD in SEQ 1-5

1.5 Design Objectives for Water Management 1-6

1.6 How to Use These Guidelines 1-7

1.7 Selection of Appropriate Water Management Measures 1-8

1.8 Adopted Climatic Regions for SEQ 1-10

1.9 Safety and Risk Management 1-12

1.1 Introducing Water Sensitive Urban Design

Scientific studies of the waterway catchments of South East Queensland (SEQ) have shown treated sewage and urban stormwater to be key contributors to reduced water quality and reduced waterway health in local waterways and Moreton Bay. The objective of traditional urban development practices was to move these discharge streams to receiving waters as efficiently as possible, providing minimal opportunity for treatment and reuse. With ongoing population growth in the region, a continuation of this traditional approach will result in further, and perhaps irreversible, degradation of the region's waterways.

Water Sensitive Urban Design (WSUD) is an internationally recognised concept that offers an alternative to traditional development practices. WSUD is an holistic approach to the planning and design of urban development that aims to minimise negative impacts on the natural water cycle and protect the health of aquatic ecosystems. It promotes the integration of stormwater, water supply and sewage management at the development scale.

WSUD represents a fundamental change in the way urban development is conceived, planned, designed and built. Rather than using traditional approaches to impose a single form of urban development across all locations, WSUD considers ways in which urban infrastructure and the built form can be integrated with a site's natural features. In addition, WSUD seeks to optimise the use of water as a resource.

The key principles of WSUD are to:

- Protect existing natural features and ecological processes.
- Maintain the natural hydrologic behaviour of catchments.
- Protect water quality of surface and ground waters.
- Minimise demand on the reticulated water supply system.
- Minimise sewage discharges to the natural environment.
- Integrate water into the landscape to enhance visual, social, cultural and ecological values.

1.2 Scope of These Guidelines

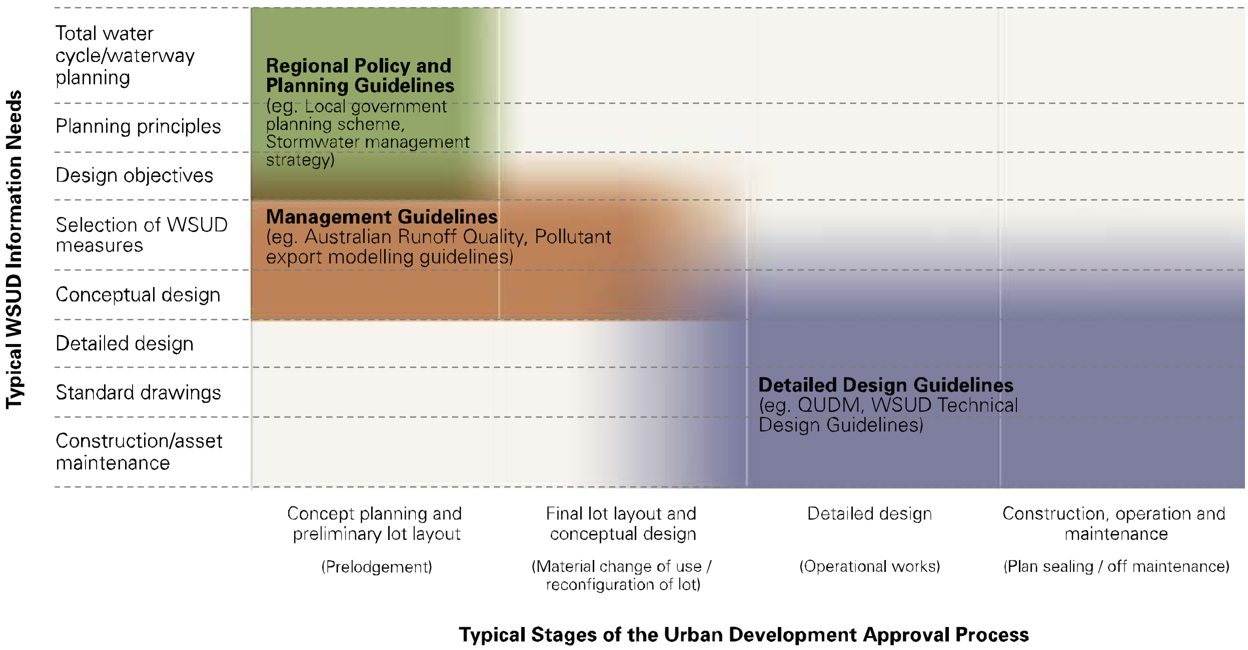
Various tools and guidelines are available to assist in the planning, design and construction of WSUD elements. Figure 1.1 shows different information needs associated with planning and design of WSUD (vertical axis) and stages of the development assessment process (horizontal axis). These guidelines are intended as a detailed design tool, applicable in the mid to latter stages of the urban development process.

These guidelines describe appropriate methods for the detailed design of some common structural stormwater management measures in SEQ. It is not the intention of the guidelines either to advocate or to discourage particular approaches. Hence, exclusion of a particular type of device from the Guidelines does not imply that it can not be used in SEQ.

Management approaches for other elements of the urban water cycle, whilst essential for effective WSUD, are not presently covered in these guidelines. In the future, the Guidelines may be expanded to address a wider suite of management techniques across the full water cycle. However, some of these, particularly demand management measures for water conservation, as well as greywater and blackwater management measures, are currently controlled by state or local government regulation, which provides limited design flexibility.

Knowledge of best practices for the design and construction of stormwater treatment measures is constantly increasing. These guidelines are not intended to limit innovation in design or construction of WSUD elements by restricting alternative approaches to those presented here. Alternative designs should be considered where potential improvements in performance, constructibility or maintenance requirements can be demonstrated. However, the design procedures and recommendations given in these guidelines are based on contemporary best practice, incorporating lessons from local experience, and are regarded as appropriate for the SEQ region.

Figure 1.1 Applicable information tools and resources for different aspects of WSUD implementation



1.3 Structure of the Guidelines

The following eight chapters of these Guidelines each detail the design methodology for a different type of stormwater management measure:

Chapter	Treatment Measure	Description
2	Swales and Buffer Strips	A swale is a shallow trapezoidal channel lined with vegetation. A buffer strip is a vegetated slope. Stormwater flows along a swale, but across a buffer strip. Treatment is provided by infiltration to the soil and by filtration of shallow flow through the vegetation.
3	Bioretention Swales	Bioretention swales include a vegetated infiltration trench within the invert of a swale. Incorporating the infiltration trench enhances removal of both particles and nutrients.
4	Sedimentation Basins	A sedimentation basin is a small pond, about 1 m deep, designed to capture coarse to medium sediment from urban catchments. Treatment is provided primarily through settling of suspended particles.
5	Bioretention Basins	A bioretention basin is a vegetated bed of filter material, such as sand and gravel. The basin is designed to capture stormwater runoff which then drains through the filter media. Pollutants are removed by filtration and by biological uptake of nutrients.
6	Constructed Wetlands	Constructed wetland systems are shallow, vegetated water bodies that use enhanced sedimentation, fine filtration and biological uptake processes to remove pollutants from stormwater.
7	Infiltration Measures	Infiltration measures typically consist of a holding pond or tank designed to promote infiltration of appropriately treated to surrounding soils. The primary function of these devices is runoff volume control rather than pollutant removal.
8	Sand Filters	A sand filter is a sand layer designed to filter fine particulates from stormwater before discharging to a downstream drainage system.
9	Aquifer Storage and Recovery	Aquifer storage and recovery involves enhancing water recharge to underground aquifers through pumping or gravity feed of treated stormwater.

Appendix A, Plant Selection for WSUD Systems, provides advice on the selection of plant species to perform different functional roles within stormwater treatment devices in South East Queensland. The appendix includes lists of recommended plant species.

Each chapter of the Guidelines has a similar generic structure which includes the following elements:

Introduction:	Introduction to the general features of the device, principles of operation and treatment processes.
Design Considerations:	A discussion of important issues that should be considered in the design of various elements of the device.
Design Process:	Step-by-step guide through the details of the recommended design process. Calculation summary sheets are provided to help ensure that key design issues have been addressed.
Landscape Design Notes:	Discussion of landscape design considerations, including illustrations showing possible landscape forms.
Construction and Establishment:	Advice on the construction and establishment of WSUD elements, based on recent industry experience around Australia.
Maintenance Requirements:	Discussion of maintenance requirements for WSUD elements. Maintenance inspection forms are provided for each element to highlight the components of a system that should be routinely checked. These can be used as templates to develop more site-specific maintenance inspection forms.
Checking Tools:	A series of checking tools, comprising design, construction, asset transfer and maintenance checklists, are provided in each chapter, to assist designers and local government officers in checking the integrity of designs, both before and after construction.
Example Engineering Drawings:	Working drawings that detail key elements of the system. These example drawings illustrate the typical level of detail required in the documentation to facilitate successful construction. However, these are not standard drawings and requirements may vary between different local authorities.
Worked Example:	A worked example of the design procedure. The worked example completes a detailed design based on an initial concept design layout and discusses design decisions that are required as well as performing the calculations outlined in the design procedure.
References:	A list of reference documents and information sources.

1.4 The Policy and Planning Context for WSUD in SEQ

There are numerous pieces of planning and environmental protection legislation that have a direct bearing on the regulatory aspects of WSUD in South East Queensland. Due to the wide range of issues encompassed by WSUD, such as environmental protection, stormwater management, water conservation and wastewater management, it does not fit neatly under one Act or regulatory authority.

The Integrated Planning Act (IPA) is the primary planning legislation in Queensland. The Act is focussed on achieving ecological sustainability by using natural resources prudently and minimising environmental impacts. Since these objectives are also fundamental to WSUD, the Act provides strong support for a water sensitive approach to urban development.

IPA is also the legislative basis upon which local governments prepare a planning scheme. The environmental objectives of IPA are reflected in the requirement for every planning scheme to specify Desired Environmental Outcomes (DEOs) which development must achieve. Measures within a planning scheme, such as zones, codes and strategies must work towards the DEOs. WSUD provides an appropriate method for development to comply with some water-related DEOs.

The SEQ Regional Plan (SEQ RP) establishes a range of desired regional outcomes, principles and policies to guide the development of SEQ through to 2026. The SEQ RP provides a strong policy basis for WSUD by recognising that sustainable management of the water cycle is crucial to the ecological health of the region. The SEQ RP adopts Total Water Cycle Management (TWCM) as the underpinning framework for urban water policy and urban water infrastructure development. WSUD is consistent with this approach, since WSUD represents the implementation of TWCM at the development scale. The SEQ RP (in Desired Regional Outcome 11) specifically requires development impacts on the natural water cycle to be minimised by adopting water sensitive design and water quality standards.

The SEQ RP also requires local governments to ensure that regionally consistent and explicit minimum performance standards or 'design objectives' for water management are referred to in their respective Planning Schemes. Design objectives for stormwater management are discussed further in Section 1.5 of this document.

1.5 Design Objectives for Water Management

Design objectives are specific and measurable water management targets, selected to meet desired outcomes, such as a reduction in water usage or protection of downstream environmental values.

For stormwater management, regulation of design objectives is the responsibility of individual local governments in SEQ. The typical form of stormwater design objectives is based on achieving target pollutant concentrations or target reductions in pollutant load. Design objectives for the management of stormwater quantity are also specified in some local government areas.

Experience within Australia and overseas has identified some problematic issues with the application of concentration-based receiving water targets or water quality objectives as discharge criteria for urban stormwater. These issues include selection of a representative median concentration for stormwater flow, which is highly variable. In addition, the substantial increase in runoff volume that typically accompanies urban development can increase pollutant loads to receiving waters (even if concentrations are not increased) and also damage urban streams through increased erosion. For these reasons many authorities across Australia, including South East Queensland are moving towards the use of load-based objectives.

The design approach presented in these guidelines is essentially independent of the design objectives that the device is required to meet. It is assumed that the size and general configuration of the device to meet design objectives has already been determined through a conceptual design process. For small developments, conceptual design may be undertaken by specifying deemed-to-comply solutions based on local government requirements. For larger developments, numerical modelling may be required to demonstrate compliance with design objectives.

Some chapters of these Guidelines include design checking curves, which can be used to provide an order-of-magnitude check on the sizing of the device, determined during conceptual design. These curves reflect a specific set of design objectives, and hence will not be applicable in areas where local authorities specify alternative objectives. The adopted design objectives for the checking curves are to achieve the following reductions in mean annual pollutant load leaving a development site, compared to traditional urban design where stormwater is not treated:

- >= 80% reduction in total suspended solids load
- >= 60% reduction in total phosphorus load
- >= 45% reduction on total nitrogen load
- >= 90% reduction in gross pollutant load.

1.6 How to Use These Guidelines

Table 1.1 shows typical WSUD-related tasks undertaken during each stage of the urban development process, as well as the corresponding progress of the local government approval process. These Guidelines are specifically for use during the **detailed design** and **construction, operation and maintenance** stages of the urban development process.

The Guidelines are not intended to be used for determining the size (typically, plan area and/or volume) of a device to meet design objectives. Throughout the guidelines it is assumed that a conceptual design has been previously completed to determine the size and general arrangement of proposed stormwater treatment devices.

The first step in the detailed design process is to check that the size of the device, determined at the conceptual design stage, is approximately correct. To facilitate this checking process, these guidelines include **design checking curves** that show typical device size (plan area) against the expected pollutant removal performance for total suspended solids (TSS), total nitrogen (TN) and total phosphorus (TP). In using these curves, the following points of caution should be noted:

- The design checking curves have been developed for a specific device configuration. For example, a specific detention time or type of filtration media. The curves should not be used where the proposed conceptual design differs substantially from the device configuration adopted for the checking curves.
- The design checking curves are based on compliance with the load-based objectives described in Section 1.5. These design objectives are not adopted by all local authorities in South East Queensland and hence, the design checking curves may not be appropriate for use in these local government areas.
- The design checking curves have been developed for a single device in isolation, assuming typical stormwater inflows. The curves will not be relevant where devices are used in series, since upstream devices will modify the quality and quantity of inflow to downstream devices.

The guidelines contain many illustrations and photographs of stormwater treatment devices. These are intended as **examples only** and **should not be regarded as acceptable solutions**. Unless specifically indicated, all drawings are not to scale.

Table 1.1 Stages of the Urban Development and Local Government Approval Processes

The Urban Development Process	Typical Tasks	The Local Government Approval Process
Concept planning and preliminary lot layout	■ Site assessment ■ Establish design objectives ■ Device selection and indicative location	Pre-lodgement discussions
Final lot layout and conceptual design	■ Refine device selection and location ■ Size and general arrangement of devices ■ Catchment modelling to demonstrate compliance with design objectives	Material change of use and/or reconfiguration of a lot application
Detailed design	■ Internal configuration of device ■ Design inflow and outflow structures ■ Specify vegetation ■ Develop maintenance plan	Operational works application
Construction, operation and maintenance	■ Implement sediment and erosion control measures ■ Qualitative performance monitoring ■ Implement maintenance plan	Plan sealing, on and off maintenance

WSUD Technical Design Guidelines for SEQ

1.7 Selection of Appropriate Water Management Measures

Not all of the stormwater management measures presented in these Guidelines are suitable for any given site. Appropriate measures should be selected by matching device characteristics to target pollutants and the physical constraints of the site. Figure 1.2 shows the recommended process for planning and designing WSUD measures.

These guidelines are not intended to provide detailed advice on selection of stormwater treatment devices. However, the following tables provide an indication of:

- The scale at which various treatment measures are typically applied (Table 1.2),
- The effectiveness of these treatment measures in removing pollutants, attenuating peak flow rates and reducing runoff volume (Table 1.3), and
- Site conditions that may affect the suitability of different treatment measures (Table 1.4).

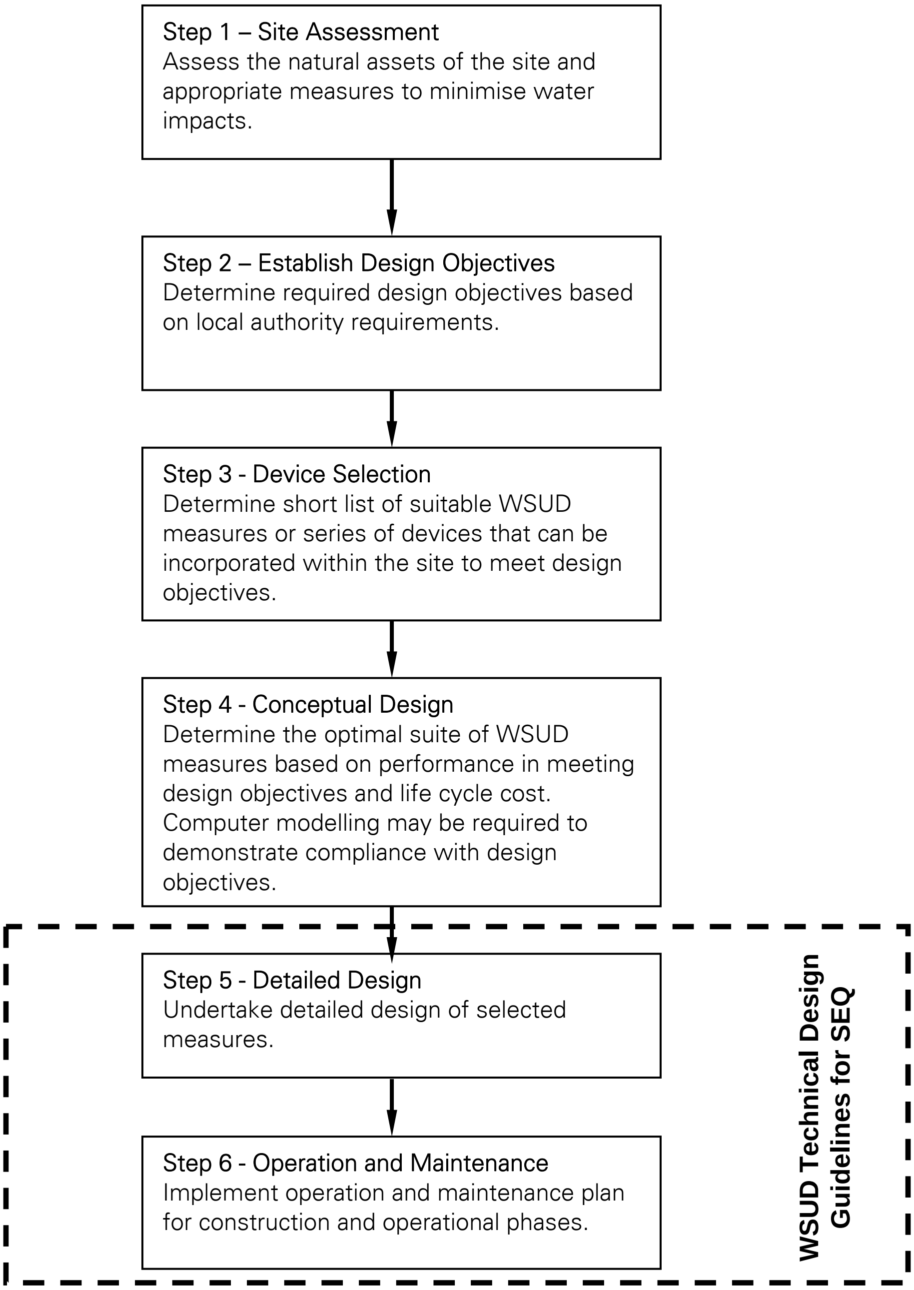


Figure 1.2 Example WSUD Planning Process

Table 1.2 Scale of WSUD Application in Urban Catchments

WSUD Measure	Allotment Scale	Street Scale	Precinct or Regional Scale
Swales and buffer strips		✓	
Bioretention Swales		✓	✓
Sedimentation basins			✓
Bioretention basins	✓	✓	✓
Constructed wetlands		✓	✓
Infiltration measures	✓	✓	
Sand filters	✓	✓	
Aquifer storage and recovery			✓

Table 1. 3 Effectiveness of WSUD Measures for Runoff Quality and Quantity Management

WSUD Measure	Water Quality Treatment	Peak Flow Attenuation *	Reduction in Runoff Volume *
Swales and buffer strips	M	L	L
Bioretention Swales	H	M	L
Sedimentation basins	M	M	L
Bioretention basins	H	M	L
Constructed wetlands	H	H	L
Infiltration measures	H	H	H
Sand filters	M	L	L
Aquifer storage and recovery	H	H	H

H – High; M – Medium; L – Low   * Frequent events only

Table 1. 4 Site Constraints for WSUD Measures

WSUD Measure	Steep site	Shallow bedrock	Acid Sulfate Soils	Low permeability soil (eg. Clay)	High permeability soil (eg. sand)	High water table	High sediment input	Land availability
Swales and buffer strips	C	D	D	✓	✓	D	D	C
Bioretention Swales	C	C	C	✓	✓	C	D	C
Sedimentation basins	C	✓	✓	✓	✓	D	✓	C
Bioretention basins	C	D	D	✓	✓	C	C	C
Constructed wetlands	C	D	C	✓	D	D	D	C
Infiltration measures	C	C	C	C	✓	C	C	C
Sand filters	D	✓	✓	✓	✓	D	C	✓
Aquifer storage and recovery	C	C	C	C	✓	C	C	C

C – Constraint may preclude use; D – Constraint may be overcome through appropriate design;
✓ - Generally not a constraint

1.8 Adopted Climatic Regions for SEQ

Whilst the seasonal pattern of rainfall in SEQ is relatively consistent (wet summers and dry winters), there is substantial variability in average rainfall across the region, and even within some local government areas. For this reason, it is important that representative local climatic conditions are used in conceptual design.

The design checking curves presented in these guidelines have been developed for four sub-regional zones to represent at least some of the observed regional variability. Typical climatic characteristics for each of the four zones are given in Table 1.5. The approximate extent of each zone is shown in Figure 1.3, however, the indicated boundaries between zones should be regarded as indicative only. The South Coast zone contains the highest variability in mean annual rainfall.

Table 1.5 Adopted Climatic Zones for South East Queensland

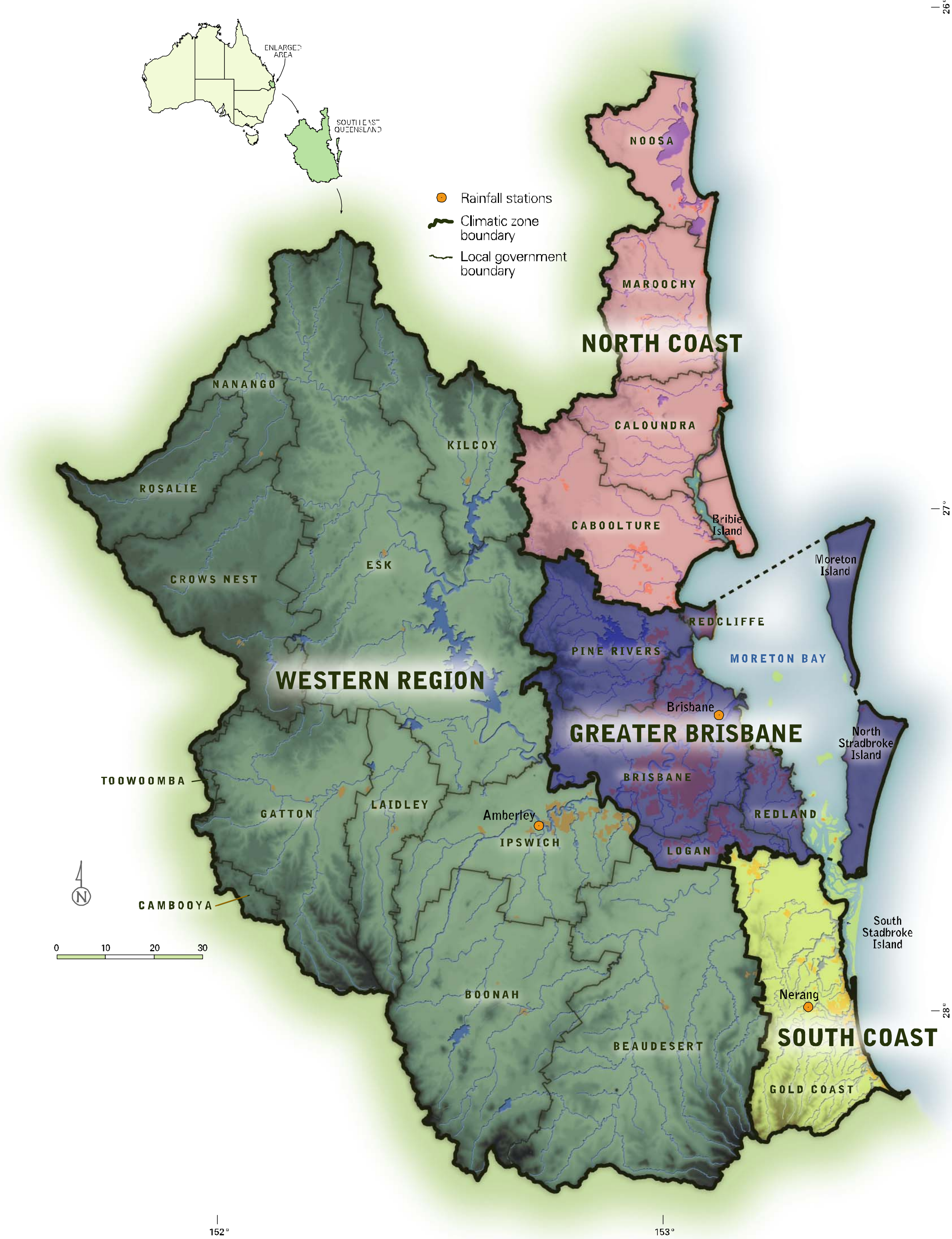
Zone		Description	No. Rain Days per Year	Mean Annual Rainfall (mm)
1	Greater Brisbane	Redland north to Redcliffe and west to Samford	100-200	1000-1250
2	North Coast	Caloundra north to Noosa	120-150	1550-1700
3	Western Region	Amberley west to Toowoomba and Beaudesert north to Ipswich	90-100	800-900
4	South Coast	Coolangatta north to Redland and west to Gold Coast Hinterland	120-140	1300-1700

Table 1.6 shows the adopted climate stations and simulation periods used to represent climatic characteristics for each climatic zone. The adopted simulation periods were selected to represent close-to-average conditions and also provide a period of good quality data. Note however, that these adopted periods may not correspond to local government requirements for simulation periods to be used in conceptual design. Where mean annual rainfall at the location of interest varies by more than about 20% from the reference climate station, the design checking curves should not be relied upon without an independent check based on climatic data that is more locally relevant.

Table 1.6 Adopted Climatic Details for Development of Design Curves

Zone		Adopted Climate Station		Adopted Simulation Period	Mean Annual Rainfall for Simulation Period (mm)	Mean Annual PET for Simulation Period (mm)
		Name	No.			
1	Greater Brisbane	Brisbane	40223	1980-1990	1175	1539
2	North Coast	Nambour	40282	1989-1998	1527	1631
3	Western Region	Amberley	40004	1990-1999	762	1513
4	South Coast	Nerang	40160	1971-1979	1596	1487

Figure 1.3 Extent of Sub-Regional Climatic Zones for Development of Design Checking Curves



1.9 Safety and Risk Management

WSUD aims to protect the environmental assets of a site and enhance livability through greater integration of built and natural features. This approach may introduce some risks to the urban environment that are greater than or different to those encountered in traditional land development practice. The more obvious of these risks relate to the presence of open water bodies and the introduction of street-scape elements that may alter lines of sight or other aspects of traffic safety.

Whilst these Guidelines make occasional comment on various aspects of safety, they are not intended to provide comprehensive advice on appropriate risk management strategies. Designers are responsible for providing an appropriate level of public safety in their designs and for ensuring that risk management procedures, in accordance with relevant standards and guidelines, are followed. Further information on risk management for water-related urban infrastructure is provided in the Queensland Urban Drainage Manual.

Chapter 2 Swales (incorporating Buffer Strips)

2.1 Introduction **2-2**

2.2 Design Considerations for Swales **2-3**
2.2.1 Landscape Design 2-3
2.2.2 Hydraulic Design 2-3
2.2.3 Vegetation Types 2-4
2.2.4 Driveway Crossings 2-4
2.2.5 Traffic Controls 2-5
2.2.6 Roof Water Discharge 2-6
2.2.7 Services 2-6

2.3 Swale Design Process **2-7**
2.3.1 Step 1: Confirm Treatment Performance of Concept Design 2-8
2.3.2 Step 2: Determine Design Flows 2-10
2.3.3 Step 3: Dimension the Swale with Consideration of Site Constraints 2-10
2.3.4 Step 4: Determine Design of Inflow Systems 2-12
2.3.5 Step 5: Verify Design 2-14
2.3.6 Step 6: Size Overflow Pits (Field Inlet Pits) 2-14
2.3.7 Step 7: Make Allowances to Preclude Traffic on Measures 2-15
2.3.8 Step 8: Specify Plant Species and Planting Densities 2-15
2.3.9 Step 9: Consider Maintenance Requirements 2-15
2.3.10 Design Calculation Summary 2-15

2.4 Landscape Design Notes **2-17**
2.4.1 Introduction 2-17
2.4.2 Objectives 2-17
2.4.3 Context and Site Analysis 2-17
2.4.4 Streetscape Swales and Buffer Strips 2-18
2.4.5 Appropriate Plant Selection 2-23
2.4.6 Safety 2-24

2.5 Construction and Establishment **2-24**
2.5.1 Staged Construction and Establishment Approach 2-24
2.5.2 Staged Construction and Establishment Method 2-25
2.5.3 Horticultural Topsoils for Swales (and Buffer Strips) 2-26
2.5.4 Sourcing Swale Vegetation 2-26
2.5.5 Vegetation Establishment 2-27

2.6 Maintenance Requirements **2-27**

2.7 Checking Tools **2-29**
2.7.1 Design Assessment Checklist 2-29
2.7.2 Construction Checklist 2-29
2.7.3 Operation and Maintenance Inspection Form 2-29
2.7.4 Asset Transfer Checklist 2-29

2.8 Engineering Drawings and Standards **2-34**

2.9 Swale Worked Example **2-34**
2.9.1 Step 1: Confirm Treatment Performance of Concept Design 2-37
2.9.2 Step 2: Determine Design Flows 2-37
2.9.3 Step 3: Configuring the Swale 2-40
2.9.4 Step 4 Design Inflow Systems 2-42
2.9.5 Step 5: Verification Checks 2-42
2.9.6 Step 6: Size Overflow Pits 2-42
2.9.7 Step 7: Traffic Control 2-43
2.9.8 Step 8: Vegetation specification 2-43
2.9.9 Calculation summary 2-43

2.10 References **2-45**

2.1 Introduction

Vegetated swales are used to convey stormwater in lieu of, or with, underground pipe drainage systems, and to provide removal of coarse and medium sediments. They are commonly combined with buffer strips and bioretention systems (refer Chapter 3 - Bioretention Swales). Swales utilise overland flow and mild slopes to convey water slowly downstream. They provide a means of disconnecting impervious areas from downstream waterways, assisting in protecting waterways from damage by frequent storm events, by reducing flow velocity compared with piped systems.

The interaction between stormwater flow and vegetation within swale systems facilitates pollutant settlement and retention. Even swales with relatively low vegetation height (such as mown grass) can achieve significant sediment deposition rates provided flows are well distributed across the full width of the swale and the longitudinal grade of the swale is kept low enough (typically less than 4 % grade) to maintain slower flow conditions.

Swales alone cannot provide sufficient treatment to meet current stormwater treatment/ water quality objectives, but can provide an important pretreatment function for other WSUD measures in a treatment train enabling water quality objectives to be met. Swales are particularly good at coarse sediment removal as a pretreatment for tertiary treatment systems such as wetlands and bioretention basins.

A typical sketch of a swale at-grade crossing is shown in Figure 2-1.

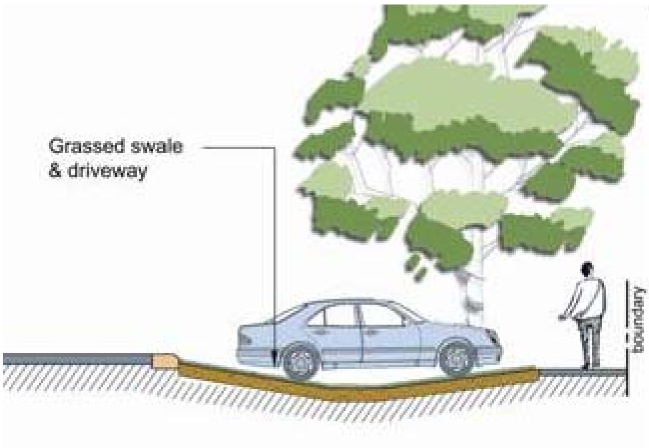


Figure 2-1: Typical Arrangement of a Swale "at-grade" Driveway Crossing

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 depth of flow through vegetation and rely upon well distributed sheet flow. Vegetation tends to slow velocities and coarse sediments are retained. With their requirement for uniformly distributed flow, buffer strips are suited to treatment of road runoff in situations where road runoff is discharged via flush kerbs or through regular kerb 'cut-outs'. In these situations, buffer strips can form part of a roadside swale system, that is, the swale batter that receives the distributed inflows from the adjoining road pavement. The coverage of buffer strips in this chapter is limited to their application as part of a roadside swale system only. The reader is referred to *Australian Runoff Quality* (Engineers Australia 2006) for additional discussion on buffer strip design and for worked examples.

2.2 Design Considerations for Swales

2.2.1 Landscape Design

Swales may be located within parkland areas, easements, carparks or along roadway corridors within footpaths or centre medians. Landscape design of swales and buffer strips along the road edge can assist in defining the boundary of road or street corridors as well as enhancing landscape character. It is important that the landscape design of swales and buffers addresses stormwater quality objectives whilst also incorporating landscape functions. As such, it is important that swales and buffers are carefully designed to integrate with the surrounding landscape character. Further discussion on landscape design considerations is provided in Section 2.4.

2.2.2 Hydraulic Design

Typically, swales are applicable for smaller scale contributing catchments up to 1-2 ha as larger than this, flow depths and velocities are such that the water quality improvement function of the swale, and it's long-term function may be compromised. For water quality improvement, swales need only focus on ensuring frequent storm flows (typically up to the 3 month ARI (Average Recurrence Interval) flow) are conveyed within the swale profile. In most cases, however, a swale will also be required to provide a flow conveyance function as part of a minor drainage and/or major drainage system. In particular, swales located within road reserves must also allow for safe use of adjoining roadway, footpaths and bike paths by providing sufficient conveyance capacity to satisfy current engineering infrastructure design requirements (as defined by the relevant local authority's development guidelines). In some cases, flows will encroach onto the road surface to acceptable levels. It may also be necessary to augment the capacity of the swale with underground pipe drainage to satisfy the road drainage criteria. This can be achieved by locating overflow pits (field inlet pits) along the invert of the swale that discharge into an underlying pipe drainage system. Careful attention should be given to the design of overflow pits to ensure issues of public safety (particularly when raised grates are being used) and aesthetic amenity are taken into account.

The longitudinal slope of a swale is another important hydraulic design consideration. Swales generally operate best with longitudinal slopes of between 1 % and 4 %. Slopes milder than this can become waterlogged and have stagnant ponding, however, the use of subsoil drains (in accordance with local government standard drawings) beneath the invert of the swale can alleviate this problem by providing a pathway for drainage of any small depressions that may form along the swale. For longitudinal slopes steeper than 4 %, check banks (e.g. small rock walls) along the invert of the swale, or equivalent measures, can help to distribute flows evenly across the swales, as well as reduce velocities and potential for scour. Check dams are typically low level (e.g. 100 mm) rock weirs that are constructed across the base of a swale. It is also important to protect the vegetation immediately downstream of check dams. Rock pitching can be used to avoid erosion.

A rule of thumb for locating check dams is for the crest of a downstream check dam to be at 4 % grade from 100 mm below the toe of an upstream check dam (refer Figure 2-2). The impact of check dams on the hydraulic capacity of the swale must be assessed as part of the design process.

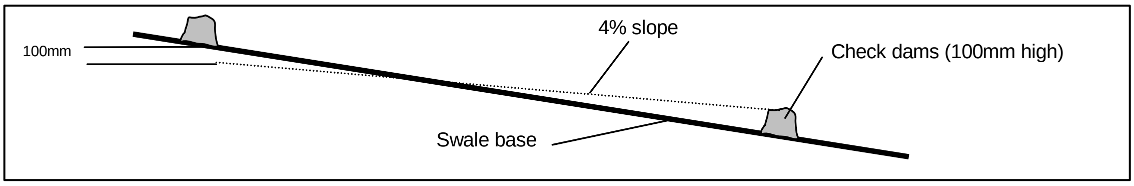


Figure 2-2: Location of Check Dams in Swales

It is important to ensure velocities within swales are kept low (preferably less than 0.5 m/s for minor flood flows and not more than 2.0 m/s for major flood flows) to avoid scouring of collected pollutants and vegetation. When located within road reserves, swales can be subjected to velocities associated with major flood flows (50-100 year ARI) being conveyed along the road corridor. Therefore, appropriate checks need to be undertaken on the resultant velocities within the swale to ensure the maximum velocity within the swale does not exceed 2.0 m/s. Similar checks should also be undertaken to assess depth x velocity within the swale, at crossings and adjacent to pedestrian and bicycle pathways to ensure public safety criteria are satisfied. These are:

- depth x velocity < 0.6 m^2/s for low risk locations and 0.4 m^2/s for high risk locations as defined in QUDM, note that this may change in accordance with local government guidelines
- maximum flow depth on driveway crossings = 0.3 m.

2.2.3 Vegetation Types

Swales can use a variety of vegetation types including turf, sedges and tufted grasses. Vegetation is required to cover the whole width of the swale, be capable of withstanding design flows and be of sufficient density to prevent preferred flow paths and scour of deposited sediments.

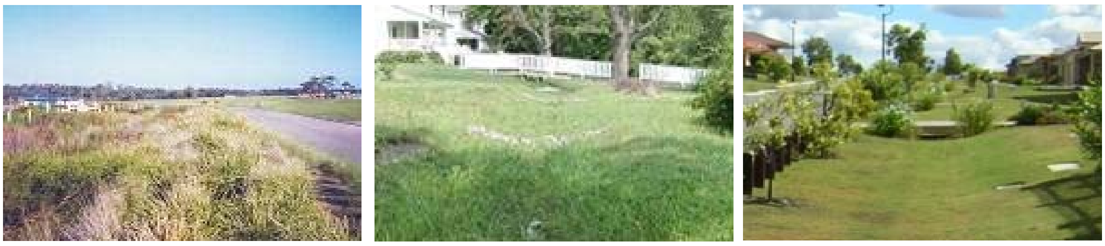


Plate 2-1: Swale systems: heavily vegetated (left), use of check dams (centre), grass swale with elevated crossings (right)

Turf swales are commonly used in residential areas and can appear as a typical road footpath. Turf swales should be mown and well maintained in order for the swale to operate effectively over the long term. Denser vegetated swales can offer improved sediment retention by slowing flows more and providing vegetation enhanced sedimentation for deeper flows. However, densely vegetated swales have higher hydraulic roughness and therefore require a larger area and/ or more frequent use of swale field inlet pits to convey flows compared to turf swales. Densely vegetated swales can become features of the urban landscape and once established, require minimal maintenance and are hardy enough to withstand large flows.

Section 2.4 of this chapter and Appendix A provide more specific guidance on the selection of appropriate vegetation for swales and buffers.

2.2.4 Driveway Crossings

A key consideration when designing swales along roadways is the requirement for provision of driveway crossings (or crossovers). Driveway crossings can be 'at-grade' or 'elevated'. 'At-grade' crossings follow the profile of the swale (e.g. like a ford), while 'elevated' crossings are raised above the invert of the swale (e.g. like a bridge deck or culvert).

Crossings constructed 'at-grade' reduce the maximum allowable swale batter slopes to approximately 1 in 9 to ensure vehicles can traverse the crossing without bottoming out. This means the swale will have a shallow profile thus reducing its flow conveyance capacity. 'At-grade' crossings are typically cheaper to construct than elevated crossings, however they need to be constructed at the same time as the swale to avoid damaging the swale. This imposes a fixed driveway location on each allotment, which can potentially constrain future house layouts. 'At-grade' crossings are best suited to developments where the spacing between crossings is typically more than 15 m. Local government standard drawings may provide guidance on appropriate driveway construction.

Plate 2-2: At-grade (left) under construction with trees yet to be established, pre-constructed 'at-grade' (centre) and elevated driveway crossings to allow vehicle access across swales

'Elevated' crossings are not appropriate in all street applications; however, where appropriate, they can be designed as streetscape features. They also provide an opportunity for locating check dams (to distribute flows) or to provide temporary ponding above a bioretention system (refer Chapter 3 – Bioretention Swales). A major limitation with 'elevated' crossings can be their high life cycle costs compared to 'at-grade' crossings (particularly in dense urban developments) due to the need for on-going maintenance. Safety concerns with traffic movement adjacent to 'elevated' crossings and the potential for blockages of small culvert systems beneath the crossing are other possible limitations. These limitations can be overcome by careful design through the use of spanning crossings rather than using small culverts and through the use of durable decking materials in place of treated timber.

2.2.5 Traffic Controls

Another design consideration is keeping traffic and building materials off swales (particularly during the building phase of a development). If swales are used for parking then the topsoil will be compacted and the swale vegetation may be damaged beyond its ability to regenerate naturally. In addition, vehicles driving on swales can cause ruts along the swale that can create preferential flow paths that will diminish the swale's water quality treatment performance as well as creating depressions that can retain water and potentially become mosquito breeding sites.

Plate 2-3: Swale incorporated into road reserve

To prevent vehicles driving on swales and inadvertent placement of building materials, it is necessary to consider appropriate traffic control solutions as part of the swale design. These can include planting the swale with dense vegetation that will discourage the movement of vehicles onto the swale or, if dense vegetation cannot be used, providing physical barriers such as kerb and channel (with breaks to allow distributed water entry to the swale) or bollards and/ or street tree planting.

Kerb and channel should be used at all corners, intersections, cul-de-sac heads and at traffic calming devices to ensure correct driving path is taken. For all of these applications, the kerb and channel is to extend 5 m beyond tangent points. The transition from barrier or lay back type kerb to flush kerbs and vice versa is to be done in a way that avoids creation of low points that cause ponding onto the road pavement.

Where road edge guide posts or 'bollards' are used, consideration should be given to intermixing mature tree plantings with the bollards to break the visual monotony created by a continuous row of bollards. Bollards should comply with relevant local government specifications.

2.2.6 Roof Water Discharge

Roof runoff can contain a range of stormwater pollutants including nitrogen washed from the atmosphere during rainfall events. Rainfall is consistently the major source of nitrogen in urban stormwater runoff (Duncan 1995) and inorganic nitrogen concentrations in rainfall often exceed the threshold level for algal blooms (Weibel *et al*. 1966). Roof water should therefore be discharged onto the surface of the swale for subsequent conveyance and treatment by the swale (and downstream treatment measures) before being discharged to receiving aquatic environments. Depending on the depth of the roof water drainage system and the finished levels of the swale, this may require the use of a small surcharge pit located within the invert of the swale to allow the roof water to surcharge to the swale. Any residual water in the surcharge pit can be discharged to the underlying subsoil drainage by providing perforations in the base and sides of the surcharge pit. If a surcharge pit is used, an inspection chamber along the roof water drainage line is to be provided within the property boundary. Surcharge pits are discussed further in Section 2.3.4.3.

Roof water should only be directly connected to an underground pipe drainage system if an appropriate level of stormwater treatment is provided along (or at the outfall of) the pipe drainage system.

2.2.7 Services

Swales located within standard road reserves are to have services located within the services corridors in accordance with local government requirements. Sewers located beneath swales are to be fully welded polyethylene pipes with rodding points. Care should be taken to ensure the service conduits do not compromise the performance of the swale. Consideration will also need to be given to access to services for ongoing maintenance without the need to regularly disrupt or replace the swale.

2.3 Swale Design Process

The design process for swales involves in the first instance designing the swale to meet flow conveyance requirements and then ensuring the swale has the necessary design features to optimise its stormwater quality treatment performance.

The key design steps are:

1. Confirm treatment performance of concept design

2. Determine design flows

3. Dimension the swale with consideration of site contraints
a. Swale width and side slopes
b. Maximum swale length (i.e. length between overflow pits)

4. Determine design of inflow systems

5. Verify design
a. Scour velocity checks
b. Safety checks - depth x velocity; maximum depth over crossings
c. Confirm treatment performance

6. Size overflow pits (field inlet pits)

7. Make allowances to preclude traffic

8. Specify plant species and planting densities

9. Consider maintenance requirements, including development of a written maintenance plan

Each of these design steps is discussed in the following sections. A worked example illustrating application of the design process on a case study site is presented in Section 2.9.

2.3.1 Step 1: Confirm Treatment Performance of Concept Design

Before commencing detailed design, the designer should first undertake a preliminary check to confirm the swale outlined on the concept design is adequate to deliver the level of stormwater quality improvement inferred within the concept design documentation. The swale treatment performance curves shown in Figure 2-3 to Figure 2-5 can be used to undertake this verification check.

The curves in Figure 2-3 to Figure 2-5 were derived using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC), assuming the swale is a stand alone system (i.e. not part of a treatment train). The curves show the total suspended solid (TSS), total phosphorus (TP) and total nitrogen (TN) removal performance for a typical swale design, being:

- top width 4.5 m
- base width 1 m
- side slopes 1 in 9
- vegetation height 50 mm (however for vegetation of between 50-250 mm the curves are still valid).

The curves in Figure 2-3 to Figure 2-5 are generally applicable to swale applications within residential, industrial and commercial land uses. Curves are provided for four rainfall station locations selected as being broadly representative of the spatial and temporal climatic variation across South East Queensland.

Where local rainfall data are available, or if the configuration of the swale concept design is significantly different to that described above, then a stormwater quality model such as MUSIC should be used in preference to the curves in Figure 2-3 to Figure 2-5. The detailed designer should also use the stormwater quality model to verify swale concept designs that are part of a "treatment train".

It should be noted that swales should form part of the stormwater 'treatment train' as they will not achieve contemporary load-based objectives on their own. Therefore, other stormwater quality best management practices should be incorporated into the surrounding catchment to augment the stormwater treatment performance of any proposed swale system.

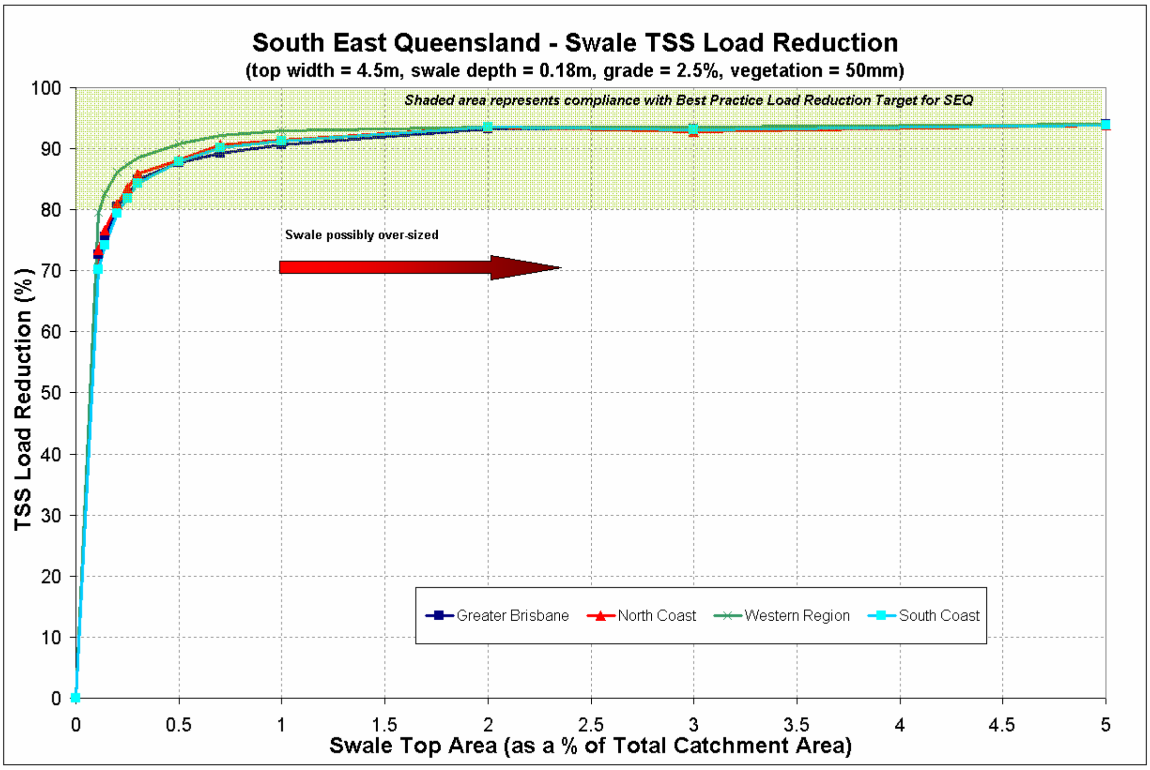


Figure 2-3: Swale TSS Removal Performance

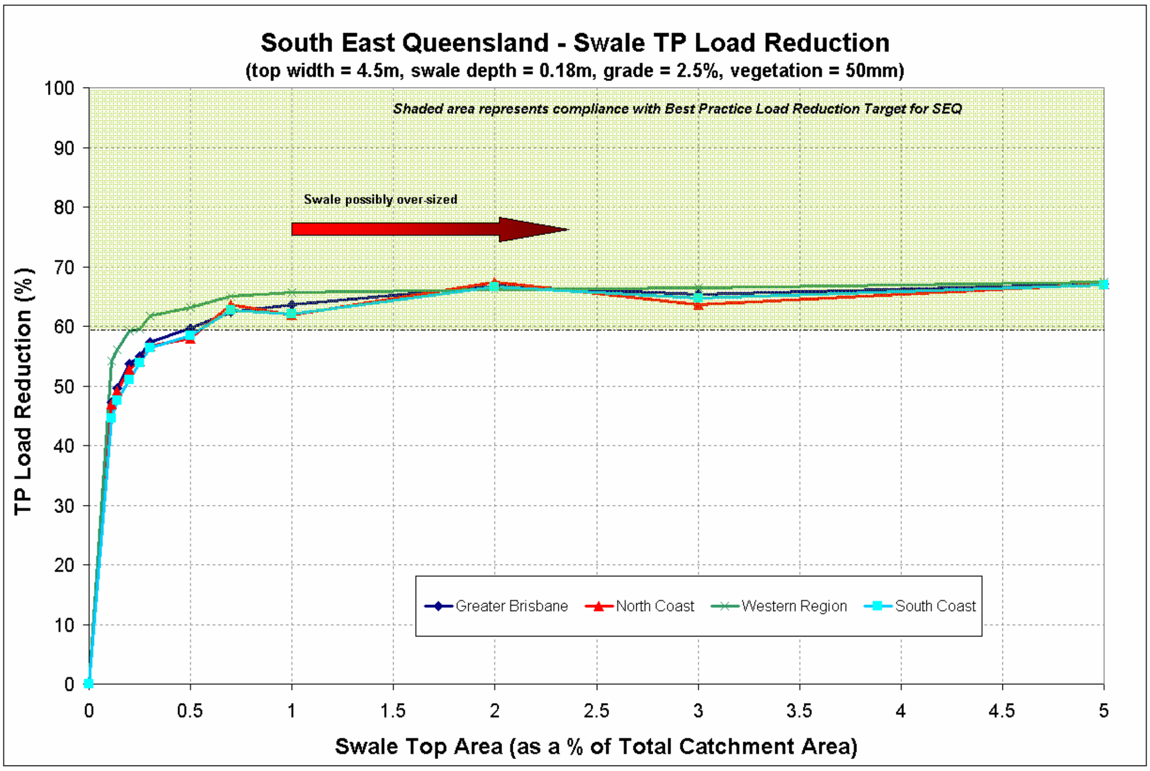


Figure 2-4: Swale TP Removal Performance

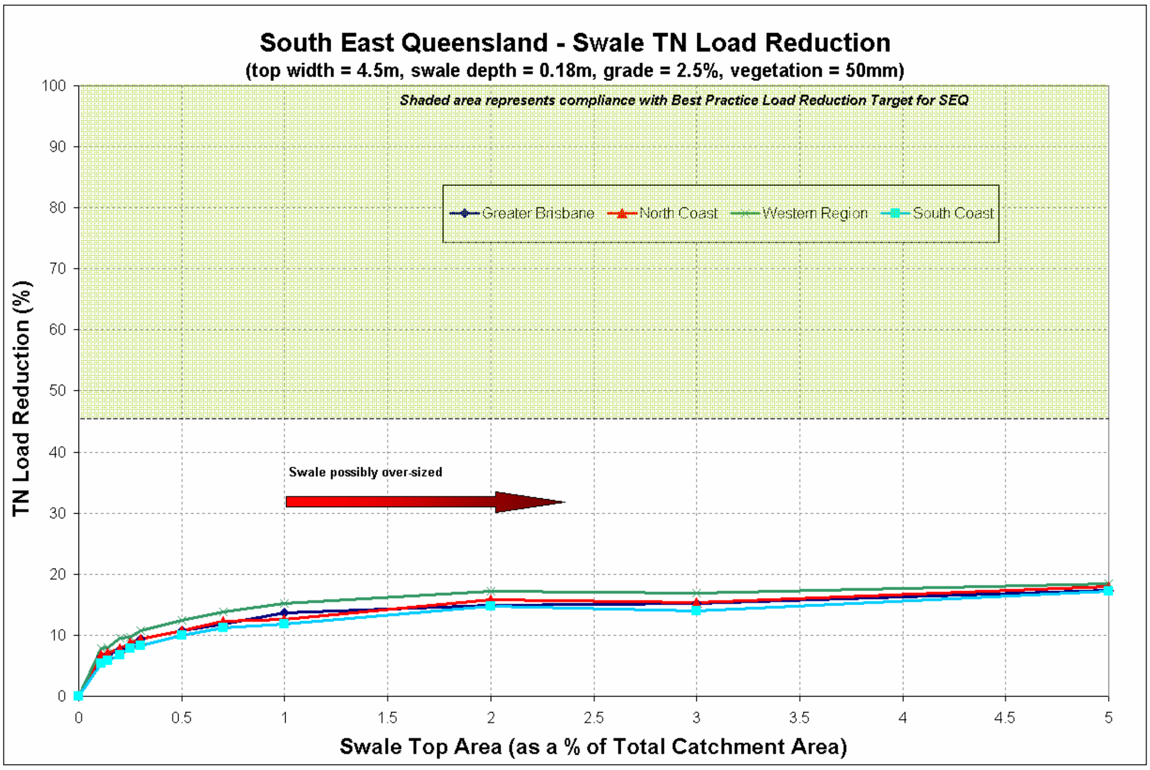


Figure 2-5: Swale TN Removal Performance

2.3.2 Step 2: Determine Design Flows

Two design flows are required to be estimated for the design of a swale, particularly where they are designed within a road reserve. These are to size the swale for conveyance of flows rather than treatment:

- minor flood flow (2-10 year ARI) to allow minor floods to be safely conveyed
- major flood flow (50-100 year ARI) to check flow velocities, velocity depth criteria, conveyance within road reserve, and freeboard to adjoining property.

Queensland Urban Drainage Manual (QUDM) identifies the Rational Method as the procedure most commonly used to estimate peak flows from small catchments in Queensland. Catchment areas delivering flow to swales are typically small, therefore the Rational Method is recognized as an appropriate method to use in the determination of peak design flows.

2.3.3 Step 3: Dimension the Swale with Consideration of Site Constraints

Factors to consider are:

- Maximum contributing catchment area (<1-2ha)
- allowable width given the proposed road reserve and/ or urban layout
- how flows will be delivered into a swale (e.g. cover requirements for pipes or kerb details)
- vegetation height
- longitudinal slope
- maximum side slopes and base width
- provision of crossings
- requirements of QUDM and/or relevant local government requirements.

Depending on which of the above characteristics are fixed, other variables may be adjusted to derive the optimal swale dimensions for the given site conditions. The following sections outline some considerations in relation to configuring a swale.

2.3.3.1 Swale Width and Side Slopes

The maximum width of swale is usually determined from an urban layout and at the concept design stage. Where the swale width is not constrained by an urban layout (e.g. when located within a large open space area), then the width of the swale may be selected based on consideration of landscape objectives, maximum side slopes for ease of maintenance and public safety, hydraulic capacity required to convey the desired design flow, and treatment performance requirements. The maximum swale width needs to be identified early in the design process as it dictates the remaining steps in the swale design process.

Selection of an appropriate side slope for swales located in parks, easements or median strips is heavily dependant on site constraints, and swale side slopes are typically between 1 in 10 and 1 in 4.

For swales located adjacent to roads, side slopes will typically be dictated by the driveway crossing. Where there are no driveway crossings then the maximum swale side slopes will be established from ease of maintenance and public safety considerations. Where 'elevated' crossings are used, swale side slopes would typically be between 1 in 6 and 1 in 4. 'Elevated' crossings will require provision for drainage under the crossings with a culvert or similar. Where 'at grade' crossings are used, swale side slopes are typically 1 in 9. The selection of crossing type should be made in consultation with urban and landscape designers. Local government design requirements or standard drawings for driveway construction should be consulted.

2.3.3.2 Maximum Length of a Swale

Provided the water quality function of the swale is met, the maximum length of a swale is the distance along a swale before an overflow pit (field inlet pit) is required to drain the swale to an underlying pipe drainage system.

The maximum length of a swale located within parkland areas and easements is calculated as the distance along the swale to the point where the flow in the swale from the contributing catchment (for the specific design flood frequency) exceeds the bank full capacity of the swale. For example, if the swale is to convey the minor flood flow without overflowing, then the maximum swale length would be determined as the distance along the swale to the point where the minor flood flow from the contributing catchment is equivalent to the bank full flow capacity of the swale (bank full flow capacity is determined using Manning's equation as discussed below).

The maximum length of a swale located along a roadway is calculated as the distance along the swale to the point where flow on the adjoining road pavement (or road reserve) no longer complies with local government road design standards (for both the minor and major flood flows) as defined by the local authority's development guidelines and/or QUDM.

2.3.3.3 Swale Capacity – Manning's Equation and Selection of Manning's *n*

Manning's equation is used to calculate the flow capacity of a swale. This allows the flow rate and flood levels to be determined for variations in swale dimensions, vegetation type and longitudinal grade. Manning's equation is given by:

$$Q = \frac{A \cdot R^{2/3} \cdot S^{1/2}}{n} \qquad \text{Equation 2.1}$$

Where: Q = flow in swale (m^3/s)

A = cross section area (m^2)

R = hydraulic radius (m)

S = channel slope (m/m)

n = roughness factor (Manning's 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, recommended values are between 0.15 and 0.3 for flow depths shallower than the vegetation height (preferable for treatment) and significantly lower for flows with depth greater than the vegetation (e.g. 0.03 – 0.05 at more than twice the vegetation depth i.e. 50-100 year ARI). It is considered reasonable for Manning's *n* to have a maximum at the vegetation height and then to sharply reduce as depths increase. Figure 2-6 shows a plot of Manning's *n* versus flow depth for a grass swale with longitudinal grade of 5 %. 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 2-6) 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.

Further discussion on selecting an appropriate Manning's *n* for a swale is provided in Appendix E of the *MUSIC User Guide* (CRCCH 2005).

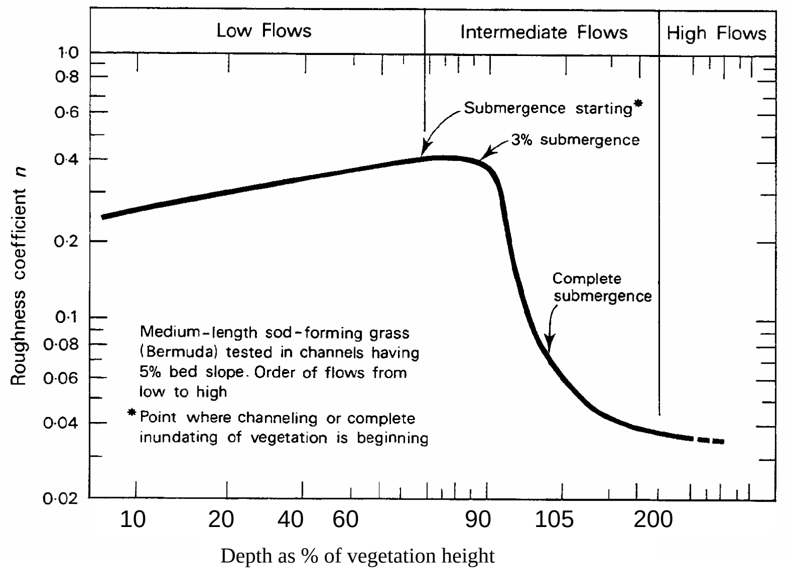


Figure 2-6: Impact of Flow Depth on Hydraulic Roughness (adapted from Barling & Moore (1993))

2.3.4 Step 4: Determine Design of Inflow Systems

Inflows to swales can be via distributed runoff (e.g. from flush kerbs along a road) or point outlets such as pipe culverts. Combinations of these two inflow pathways can also be used.

2.3.4.1 Distributed Inflow

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 kept as shallow sheet flow, which maximises contact with the swale vegetation on the batter receiving the distributed inflows. This swale batter is often referred to as a buffer. The function of the buffer is to ensure there is dense vegetation growth, flow depths are shallow (below the vegetation height) and erosion is avoided. The buffer provides good pretreatment (i.e. significant coarse sediment removal) prior to flows being conveyed along the swale.

Distributed inflows 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.

Plate 2-4: Kerb arrangements to promote distributed flow into swales

2.3.4.2 Buffer Requirements

No specific design rules exist for designing buffer systems, however there are several design guides that are to be applied to ensure buffers operate to improve water quality and provide a pretreatment role. Key design parameters of buffer systems are:

- providing distributed flows onto a buffer (potentially spreading stormwater flows to achieve this)
- avoiding rilling or channelled flows
- maintaining flow depths less than vegetation heights (this may require flow spreaders, or check dams)
- minimising the slope of buffer, best if slopes can be kept below 5 %, however buffers can still perform well with slopes up to 20 % provided flows are well distributed. The steeper the buffer the more likely flow spreaders will be required to avoid rill erosion.

Maintenance of buffers is required to remove accumulated sediment and debris therefore access is important. Sediments will accumulate mostly immediately downstream of the pavement surface and then progressively further downstream as sediment builds up.

It is important to ensure coarse sediments accumulate off the road surface at the start of the buffer. Figure 2-7 (left) shows sediment accumulating on a street surface where the vegetation is the same level or slightly higher than the road. To avoid this accumulation, a flush kerb with an arris should be used that sets the top of the vegetation 60 mm below edge of pavement. This requires the finished topsoil surface of the swale (i.e. before turf is placed) to be approximately 100 mm below the edge of pavement level. This allows sediments to accumulate off any trafficable surface.

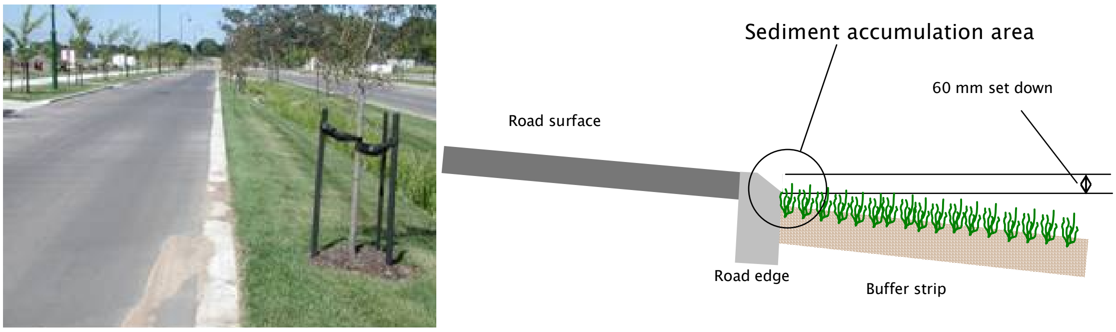


Figure 2-7: Flush kerb without set-down, showing sediment accumulation on road (left) and flush kerb with 60 mm set-down to allow sediment to flow into the vegetated area (right).

2.3.4.3 Concentrated Inflow

Concentrated inflows to a swale can be in the form of a concentrated overland flow or a discharge from a pipe drainage system (e.g. allotment drainage line). For all concentrated inflows, energy dissipation at the inflow location is an important consideration to minimise any erosion potential. This can usually be achieved with rock benching and/ or dense vegetation.

The most common constraint on pipe systems discharging to swales is bringing the pipe flows to the surface of a swale. In situations where the swale geometry does not permit the pipe to achieve 'free' discharge to the surface of the swale, a 'surcharge' pit may need to be used. 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 under-drains 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.

Figure 2-8 shows an example of a typical surcharge pit discharging into a swale. It is noted that surcharge pits are generally not considered good practice (due to additional maintenance issues and mosquito breeding potential) and should therefore be avoided where possible.

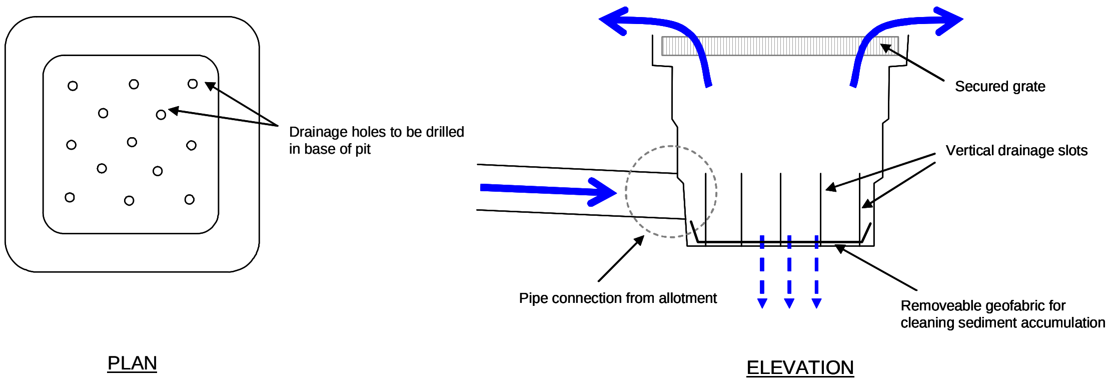


Figure 2-8: Example of Surcharge Pit for Discharging Concentrated Runoff into a Swale

Surcharge pits are most frequently used when allotment runoff is required to cross a road into a swale on the opposite side of the road or for allotment and roof runoff discharging into shallow profile swales. Where allotment runoff needs to cross under a road to discharge into a swale it is preferable to combine the runoff from more than one allotment to reduce the number of crossings required under the road pavement.

2.3.5 Step 5: Verify Design

2.3.5.1 Vegetation Scour Velocity Check

Potential scour velocities are checked by applying Manning's equation to the swale design to ensure the following criteria are met:

- less than 0.5 m/s for minor flood (2 to 10 year ARI) discharge
- less than 2.0 m/s and typically less than 1.0 m/s for major flood (50 to 100 year ARI) discharge.

2.3.5.2 Velocity and Depth Check – Safety

As swales are generally accessible by the public it is important to check that depth x velocity within the swale, at crossings and adjacent to pedestrian and bicycle pathways satisfies the following public safety criteria:

- depth x velocity of < 0.4 m^2/s is not exceeded for all flows up to the major design event, as defined in relevant local government guidelines and/or QUDM
- maximum depth of flow over 'at-grade' crossings = 0.3 m (DPI, IMEA & BCC 1992.)

2.3.5.3 Confirm Treatment Performance

If the previous two checks are satisfactory then the swale design is adequate from a conveyance function perspective and it is now necessary to reconfirm the treatment performance of the swale by reference back to the information presented in Section 2.3.1.

2.3.6 Step 6: Size Overflow Pits (Field Inlet Pits)

To size a swale field inlet pit, two checks should be made to test 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 openings required in the grate cover (assuming drowned outlet conditions). The larger of the two pit configurations should be adopted (as per Section 5.10 QUDM). In addition a blockage factor is to be used, that assumes the field inlet is 50 % blocked.

For free overfall conditions (weir equation):

$$Q_{weir} = B \cdot C_w \cdot L \cdot h^{3/2}$$ Equation 2.2

Where Q_{weir} = flow over weir (pit) (m^3/s)

B = blockage factor (0.5)

C_w = weir coefficient (1.66)

L = length of weir (m)

h = depth of water above weir crest (m)

Once the length of weir is calculated, a standard sized pit can be selected with a perimeter at least the same length of the required weir length.

For drowned outlet conditions (orifice equation):

$$Q_{orifice} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h}$$ Equation 2.3

Where $Q_{orifice}$ = flow into drowned pit (m^3/s)

B = blockage factor (0.5)

C_d = discharge coefficient (0.6)

A = total area of orifice (openings) (m^2)

g = 9.80665 m/s^2

h = depth of water above centre of orifice (m)

When designing grated field inlet pits reference should be made to the procedure described in QDUM Section 5.10.4 (DPI, IMEA & BCC 1992) and the relevant local council's development guidelines.

2.3.7 Step 7: Make Allowances to Preclude Traffic on Measures

Refer to Section 2.2.5 for discussion on traffic control options.

2.3.8 Step 8: Specify Plant Species and Planting Densities

Refer to Section 2.4 and Appendix A for advice on selecting suitable plant species for swales in South East Queensland. Consultation with landscape architects is recommended when selecting vegetation to ensure the treatment system compliments the landscape of the area.

2.3.9 Step 9: Consider Maintenance Requirements

Consider how maintenance is to be performed on the swale (e.g. how and where is access available, where is litter likely to collect etc.). A specific maintenance plan and schedule should be developed for the swale, either as part of a maintenance plan for the whole treatment train, or for each individual asset. Guidance on maintenance plans is provided in Section 2.6.

2.3.10 Design Calculation Summary

The following design calculation table can be used to summarise the design data and calculation results from the design process.

SWALES – DESIGN CALCULATION SUMMARY SHEET

	Calculation Task		CALCULATION SUMMARY	
			Outcome	Check
	Catchment Characteristics			
		Catchment Area	ha	
		Catchment Land Use (i.e. residential, Commercial etc.)		
		Catchment Slope	%	
	Conceptual Design			
		Swale Top Width	m	
		Swale Length	m	
		Swale Location (road reserve/ park/other)		
		Road Reserve Width	m	
1	Confirm Treatment Performance of Concept Design			
		Swale Area	m^2	
		TSS Removal	%	
		TP Removal	%	
		TN Removal	%	
2	Determine Design Flows			
	Time of concentration – refer to local Council's Development Guidelines/ QUDM		minutes	
	Identify Rainfall intensities			
		Minor Storm (I_2– $I_{10\text{ year ARI}}$)	mm/hr	
		Major Storm ($I_{50\text{-}100\text{ year ARI}}$)	mm/hr	
	Design Runoff Coefficient			
		Minor Storm (C_2 – $C_{10\text{ year ARI}}$)		
		Major Storm ($C_{50\text{-}100\text{ year ARI}}$)		
	Peak Design Flows			
		Minor Storm (2 - 10 year ARI)	m^3/s	
		Major Storm (50-100 year ARI)	m^3/s	
3	Dimension the Swale			
	Swale Width and Side Slopes			
		Base Width	m	
		Side Slopes – 1 in		
		Longitudinal Slope	%	
		Vegetation Height	mm	
	Maximum Length of Swale			
		Manning's *n*		
		Swale Capacity		
		Maximum Length of Swale		
4	Design Inflow Systems			
		Swale Kerb Type		
		60 mm set down to Buffer/ Swale Vegetation	Yes/ No	
		Adequate Erosion and Scour Protection (where required)		
5	Verification Checks			
		Velocity for 2-10 year ARI flow (< 0.25 - 0.5 m/s)	m/s	
		Velocity for 50-100 year ARI flow (< 2 m/s)	m/s	
		Velocity x Depth for 50-100 year ARI (< 0.4 m^2/s)	m^2/s	
		Depth of Flow over Driveway Crossing for 50-100 year ARI (< 0.3 m)	m	
		Treatment Performance consistent with Step 1		
6	Size Overflow Pits (Field Inlet Pits)			
		System to convey minor floods (2-10 year ARI)	L x W	

2.3.10.1 Typical Design Parameters

The Table 2-1 provides typical values for a number of key swale design parameters.

Table 2-1: Typical Design Parameters

Design Parameter	Typical Values
Swale longitudinal slope	1 % to 4 %
Swale side slope (for areas not requiring access, e.g. parks, easements, median strips)	1 in 4 to 1 in 10
Swale side slope for trafficability (for footpaths with 'at-grade' crossings)	Maximum 1 in 9
Swale side slope (elevated driveway crossings)	1 in 4 to 1 in 10
Manning's *n* (with flow depth less than vegetation height) (Refer Figure 2-6)	0.15 to 0.3
Manning's n (with flow depth greater than vegetation height)	0.03 to 0.05
Maximum velocity to prevent scour in minor event (e.g. Q_2)	0.25 - 0.5 m/s
Maximum velocity for $Q_{50\text{-}100}$	1.0 - 2.0 m/s

2.4 Landscape Design Notes

2.4.1 Introduction

The design and installation of swales as part of the water sensitive urban design strategy is as much a landscape based solution as it is an engineering solution. Swales can be successfully integrated into a landscape such that both the functional stormwater objectives and landscape aesthetics and amenity are achieved.

2.4.2 Objectives

Landscape design of swales and buffer strips require the following four key objectives to meet WSUD strategies:

- Integrated planning and design of swale and buffer strips within the built and landscape environments
- Ensure surface treatments for swales and buffer strips address the stormwater quality objectives whilst enhancing the overall natural landscape
- Allow for Crime Prevention through Environmental Design (CPTED) principals to be incorporated into swale and buffer strip design and siting.
- Create landscape amenity opportunities that enhances the community and environmental needs such as shade, habitat creation, screening, view framing and visual aesthetics

2.4.3 Context and Site Analysis

Comprehensive site analysis should inform the landscape design as well as road layouts, civil works and maintenance requirements. Existing site factors such as roads, driveways, buildings, landforms, soils, plants, microclimates, services and views should be considered. In the absence of recent and relevant local government guidelines, refer to *Water Sensitive Urban Design in the Sydney Region: 'Practice Note 2 – Site Planning'* (LHCCREMS 2002) for further guidance.

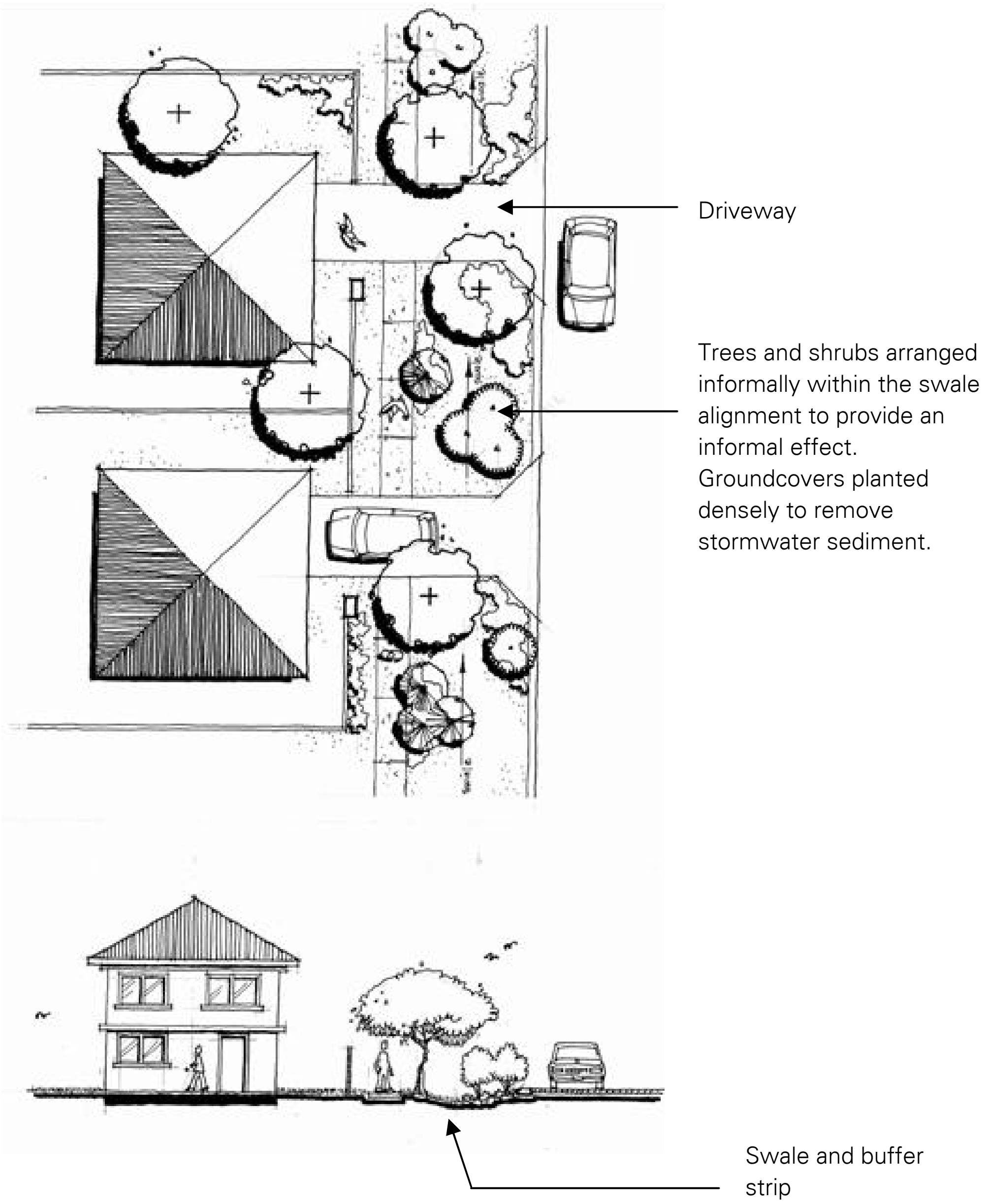
When designing for swales as part of the WSUD strategy, the overall concept layout needs to consider possible road profiles and cross-sections, building and lot layout, possible open space and recreational parks and existing natural landforms. Often things like slope and soil type will also determine which swale type and swale location will be the most effective.

Careful site analysis and integrated design with engineers, landscape architects and urban designers will ensure swales meet functional and aesthetic outcomes. A balanced approach to alignments between roads, footpaths and lot boundaries will be required early in the concept design of new developments to ensure swales are effective in both stormwater quality objectives and built environment arrangements. This is similar to concept planning for parks and open space where a balance is required between useable recreation space and WSUD requirements.

2.4.4 Streetscape Swales and Buffer Strips

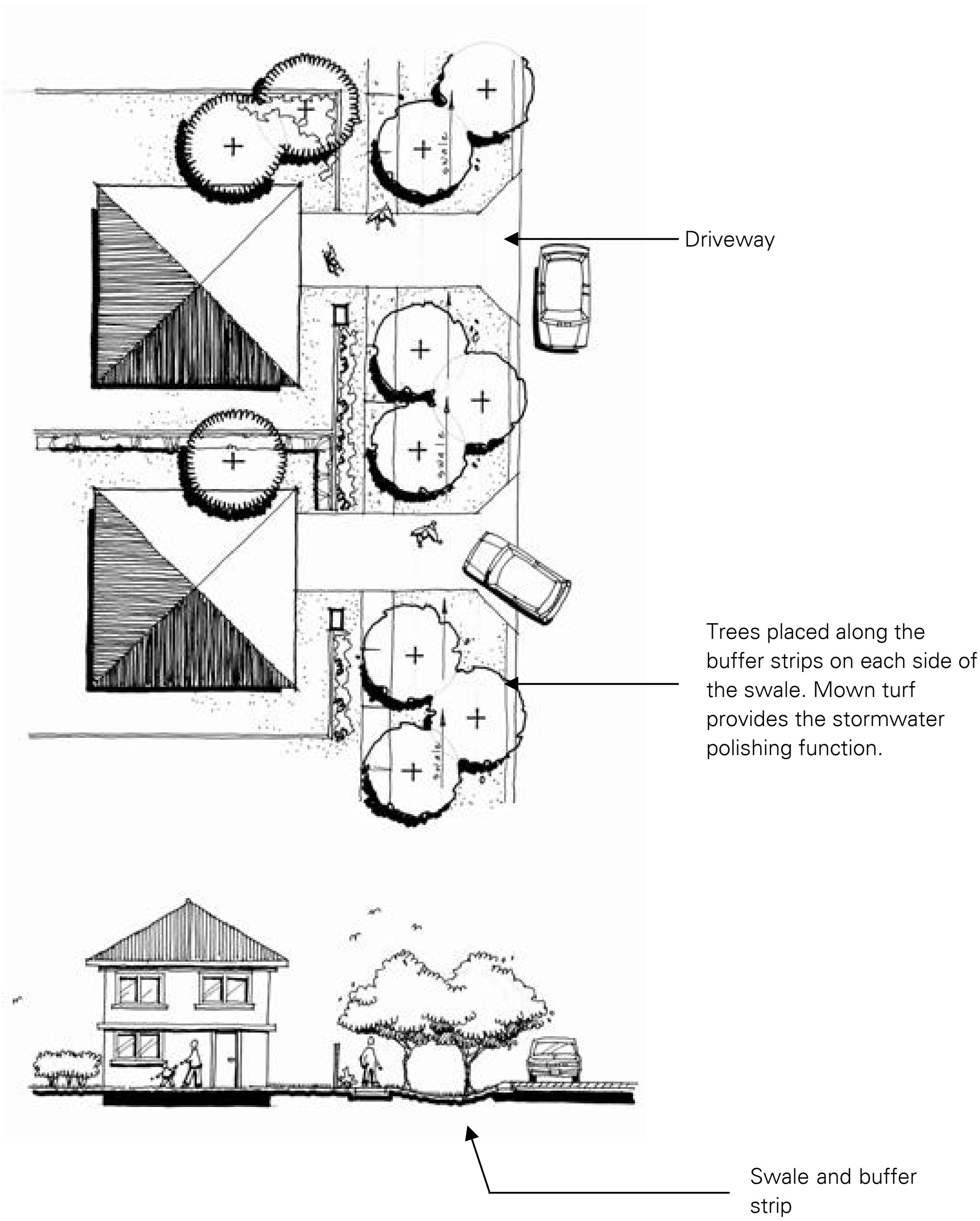
2.4.4.1 Residential Streets

When using swales in road spaces it is important to understand how the swale landscape can be used to define the visual road space. Creative landscape treatments may be possible given that the swale and buffer strip system will typically be a minimum of 4 m in width. Design responses may range from informal 'natural' planting layouts to regimented avenues of trees along each external and internal edge of the swale/ buffer system. Figure 2-9 and Figure 2-10 illustrate potential planting layouts.



Note: Landscape design is subject to local Council Development and CPTED Guidelines, site line safety requirements and standard service allocations detailed in this document.

Figure 2-9: Possible 'natural' planting layout for residential swales



Note: Landscape design is subject to local Council Development Guidelines and the CPTED, site line safety requirements and standard service allocations detailed in this document.

Figure 2-10: Possible Avenue Planting for Residential Swales

Swales can be incorporated into a typical streetscape landscape using either a central splitter median or using one or both sides of the road verge. Generally, the central median swale will provide a greater landscaped amenity, allowing planting and shade trees to enhance the streetscape more effectively, whilst verges remain constraint free. This swale configuration is however confined to roads requiring larger corridors for increased traffic.

In smaller minor roads, one side of the road can have a swale landscape to capture stormwater runoff from road pavements and house lots. To enhance the visual road space, creative landscape treatments to driveway cross-overs, general planting and invert treatments should be used. It is important in this swale arrangement that services and footpaths that are standard for road verges, have been planned and located to avoid clashes of function. Designs should obtain advice and approval from the relevant local government for placement of swales and services.

Swale surface treatments are generally divided into a turfed or a vegetated (planting) finish to the invert. When detailing a turf swale, consideration should be given to the impact of mowing on batters and the generally damp invert. This can be minimised by using different turf species that require less maintenance and respond to wet environments.

Vegetated swales can provide a relatively maintenance free finish if the planting and invert treatment are designed well. Key considerations when detailing are type and size of inorganic mulch, density and types of plantings, locations of trees and shrubs, type of garden (mowing) edges to turf areas that allows unimpeded movement of stormwater flow and overall alignment of swale invert within the streetscape. Placement of trees and shrubs should not impede the maintenance and mowing of the swale.

Figure 2-11 and Figure 2-12 illustrate the potential different treatments based on typical minor road configurations.

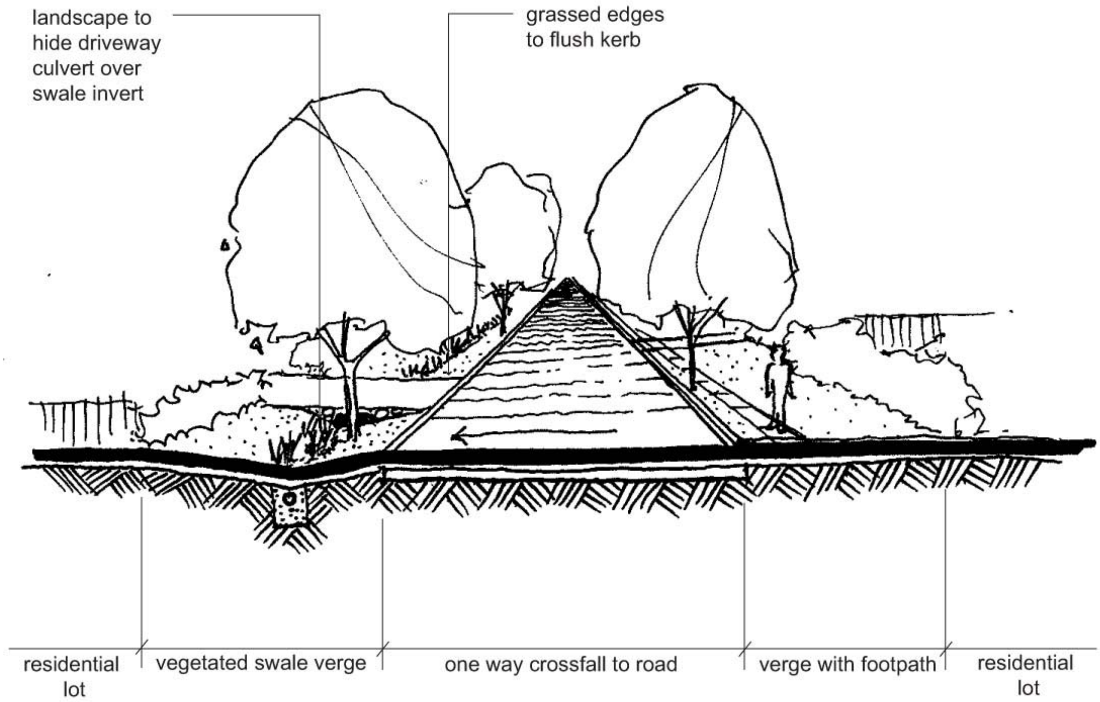


Figure 2-11: Landscape treatment of vegetated swale on single side of road

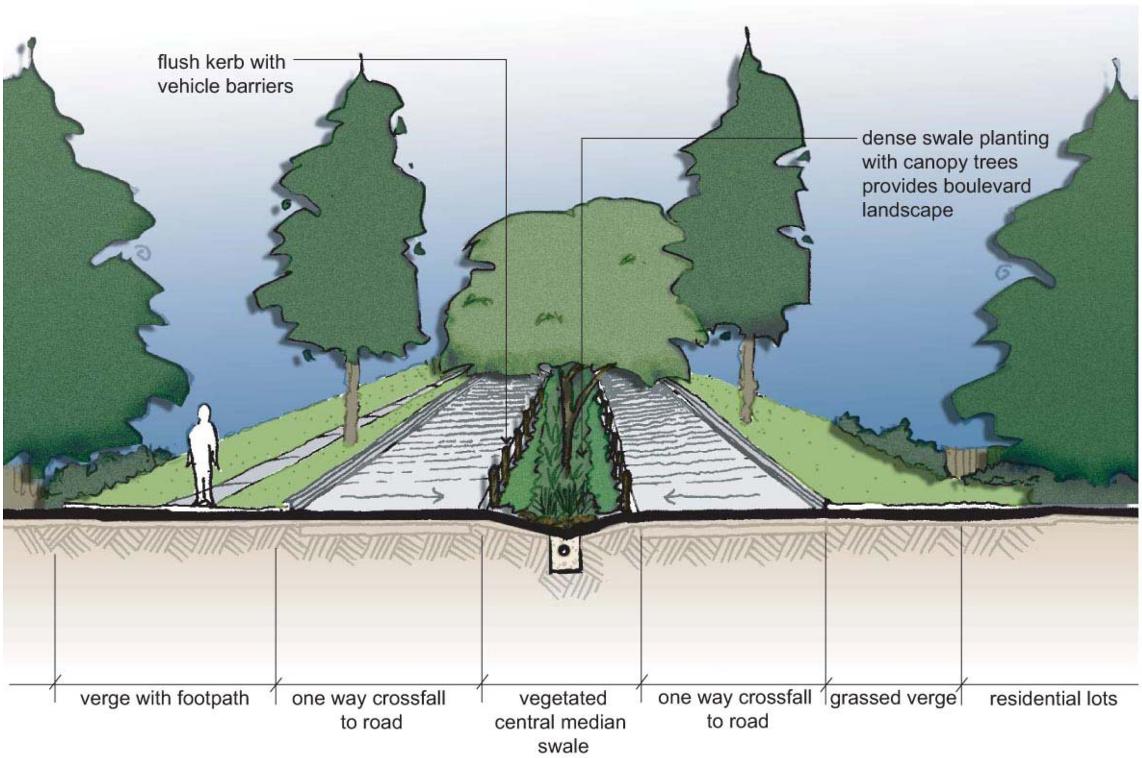
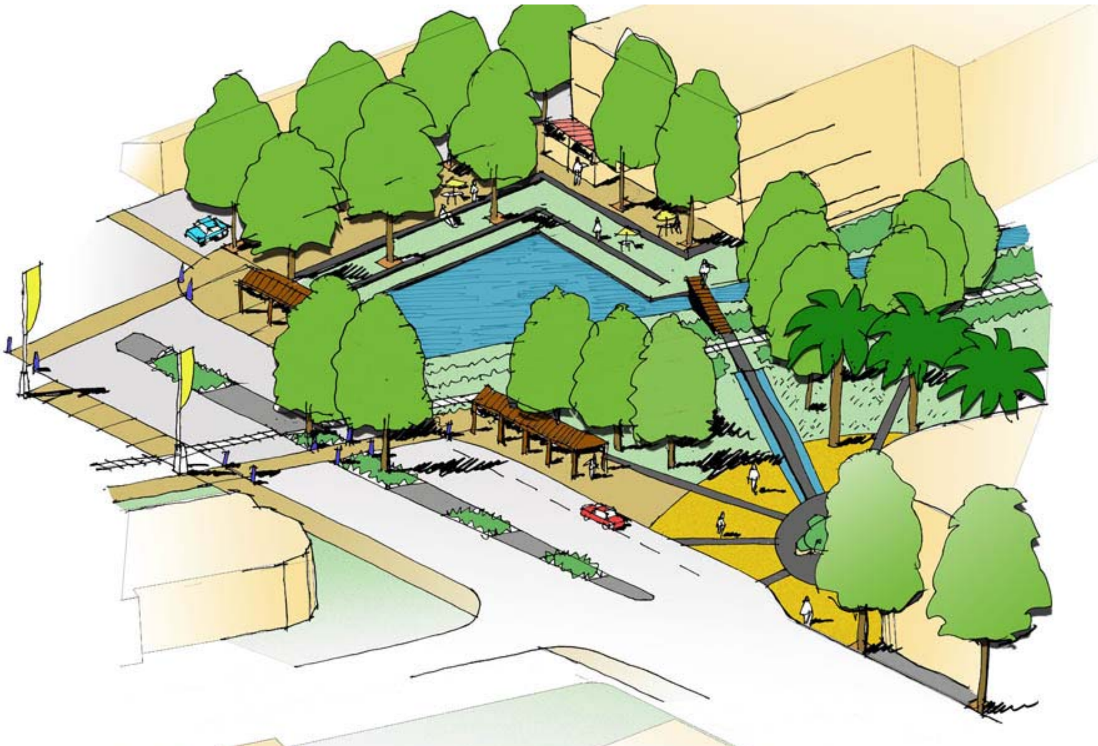


Figure 2-12: Landscape treatment of a vegetated swale in central median

2.4.4.2 Civic and Urban Spaces

With increasing population growth, functional urban design is required to create more robust spaces that meet current environmental and social needs. Often constrained by existing infrastructure, landscape treatments of swales can have a dual role of providing functional stormwater quality objectives whilst creating landscapes that enhance the communities' perception of water sensitive urban design.

Within civic and other highly urbanized spaces, use of hard useable edges to swales and planting strategies can be used to create an aesthetic landscape that meets recreational uses and promotes water sensitive urban design. This is illustrated in Figure 2-13.



West Creek Corridor Design Development,Toowoomba City Council

Figure 2-13: Typical urban treatment to swales

2.4.4.3 Open Space Swales (and buffer strips)

Design and siting of parks/open space swales allows for greater flexibility in sectional profile, treatments and alignments. It is important however for careful landscape planning, to ensure that spaces and function of particular recreational uses (either passive or active) are not encumbered by stormwater management devices including swales.

Swales and buffer strips can form convenient edges to pathway networks, frame recreational areas, create habitat adjacent to existing waterways/vegetation and provide landscape interest. Important issues to consider as part of the open space landscape design is maintenance access and CPTED principles which are further discussed in following sections.

2.4.5 Appropriate Plant Selection

Planting for swale/ buffer strip systems may consist of up to four vegetation types:

- groundcovers for sediment removal and erosion protection (required element)
- shrubbery for screening, glare reduction, character, and other values
- street trees for shading, character and other landscape values
- existing vegetation.

Where the landscape design includes canopy layers, more shade tolerant species should be selected for the groundcover layer. Trees and shrubs should also be managed so that the groundcover layer is not out-competed. If this does occur, replacement planting and possible thinning of the upper vegetation layers may be required to ensure the pollutant removal capacity of the groundcover is maintained.

2.4.5.1 Trees

Trees for swale systems to streets should conform to the relevant local council's landscape guidelines.

Open space swale planting of trees should take into account existing vegetation species, soil types, be able to grow under conditions associated with periodic inundation and allow for open canopies to promote groundcover growth. While Appendix A provides guidance on plant species selection, it is not intended as an exhaustive list and designers should ensure that the proposed planting schedule is suitable for the specific site.

2.4.5.2 Shrubs

Shrubs provide an important role in allowing for visual screening and borders, and should compliment the design and siting of the swale and buffer strip. Some species are outlined in Appendix A that are useful in urban and residential landscapes, however it should be noted that these lists are guides only. Other species and cultivars may be appropriate given the surrounding natural and/ or built environment of the swale.

While Appendix A provides guidance on plant species selection, it is not intended as an exhaustive list and designers should ensure that the proposed planting schedule is suitable for the specific site. Reference to the local government's landscape strategy or plant selection guideliens may provide guidance on choosing suitable shrub and tree species.

2.4.5.3 Groundcovers

Groundcovers provide the main functional component in meeting the stormwater objectives for removing sediment, aiding nutrient uptake and pollutant removal capacity. In selecting appropriate groundcover species the following considerations need to be addressed:

- density of planting
- species tolerance to high or low flows
- leaf surface density
- use of local endemic species.

Appendix A provides guidance on selecting suitable plant (including turf) species and cultivars that remove sediment and deliver the desired stormwater quality objectives. A table of recommended species (Table A.1) is also provided. In general, vegetation should possess:

- a high leaf surface density within the design treatment depth to aid efficient stormwater treatment
- a uniform distribution of vegetative material to prevent stormwater flows from meandering between plants.

2.4.5.4 Existing Vegetation

Existing vegetation, such as remnant native trees, within the swale/ buffer strip alignment may be nominated for retention. In this case, the swale will need to be diverted or piped to avoid the vegetation's critical root zone (equivalent to 0.5 m beyond the vegetation's drip line).

2.4.6 Safety

Swales and buffer strips within streetscapes and parks need to be generally consistent with public safety requirements for new developments. These include reasonable batter profiles for edges, providing adequate barriers to median swales for vehicle/pedestrian safety and safe vertical heights from driveways to intersecting swale inverts.

2.4.6.1 Crime Prevention Through Environmental Design (CPTED)

Landscape design of swales and buffer strips need to accommodate the standard principles of informal surveillance, reducing concealment areas by providing open visible areas as required. Regular clear sight lines between local roads and footpaths/properties, which can be facilitated by vegetation lower than 1 metre or clear trunked trees above 1.6 metres. Refer to the local authority's CPTED guideline where available.

2.4.6.2 Traffic Sightlines

Where landscaping for swales and buffer strips in road verges and medians are located in critical sightline corridors as required for traffic visibility, the standard rules apply to vegetation heights. Refer to *Road Landscape Manual* (DMR 1997) for guidance.

2.5 Construction and Establishment

This section provides general advice for the construction and establishment of swales and key issues to be considered to ensure their successful establishment and operation. Some of the issues raised have been discussed in other sections of this chapter and are reiterated here to emphasise their importance based on observations from construction projects around Australia.

2.5.1 Staged Construction and Establishment Approach

It is important to note that swale systems, like most WSUD elements that employ vegetation based treatment processes, require approximately two growing seasons (i.e. two years) before the vegetation in the systems has reached its design condition (i.e. height and density). In the context of a large development site and associated construction and building works, delivering swales and establishing vegetation can be a challenging task. Swales require a construction and establishment approach to ensure the system establishes in accordance with its design intent. The following sections outline a recommended staged construction and establishment methodology for swales (from Leinster, 2006).

2.5.1.1 Construction and Establishment Challenges

There exist a number of challenges that must be appropriately considered to ensure successful construction and establishment of swales. These challenges are best described in the context of the typical phases in the development of a Greenfield or Infill development, namely the *Subdivision* Construction Phase and the Building Phase (see Figure 2-14).

- *Subdivision* Construction - Involves the civil works required to create the landforms associated with a development and install the related services (roads, water, sewerage, power etc.) followed by the landscape works to create the softscape, streetscape and parkscape features. The risks to successful construction and establishment of swales during this phase of work are generally related to the construction activities which can generate large sediment loads in runoff which can smother vegetation and construction traffic and other works can result in damage to the swales. Importantly, all works undertaken during *Subdivision* Construction are normally 'controlled' through the principle contractor and site manager. This means the risks described above can be readily managed through appropriate guidance and supervision.
- Building Phase - Once the *Subdivision* Construction works are complete and the development plans are sealed then the Building Phase can commence (i.e. construction of the houses or built form). This phase of development is effectively 'uncontrolled' due to the number of building contractors and sub-contractors present on any given allotment. For this reason the Allotment Building Phase represents the greatest risk to the successful establishment of swales.

2.5.2 Staged Construction and Establishment Method

To overcome the challenges associated within delivering swales a Staged Construction and Establishment Method should be adopted (see Figure 2-14):

- Stage 1: Functional Installation – Construction of the functional elements of the swale at the end of *Subdivision* Construction (i.e. during landscape works) and the installation of temporary protective measures. For example, temporary protection of swales can been achieved by using a temporary arrangement of a suitable geofabric covered with shallow topsoil (e.g. 50 mm) and instant turf (laid perpendicular to flow path).
- Stage 2: Sediment and Erosion Control – During the Building Phase the temporary protective measures preserve the functional infrastructure of the swales against damage whilst also allowing for flow conveyance to sediment control devices throughout the building phase to protect downstream aquatic ecosystems.
- Stage 3: Operational Establishment – At the completion of the Building Phase, the temporary measures protecting the functional elements of the swales can be removed along with all accumulated sediment and the system re-profiled and planted in accordance with the design and planting schedule.

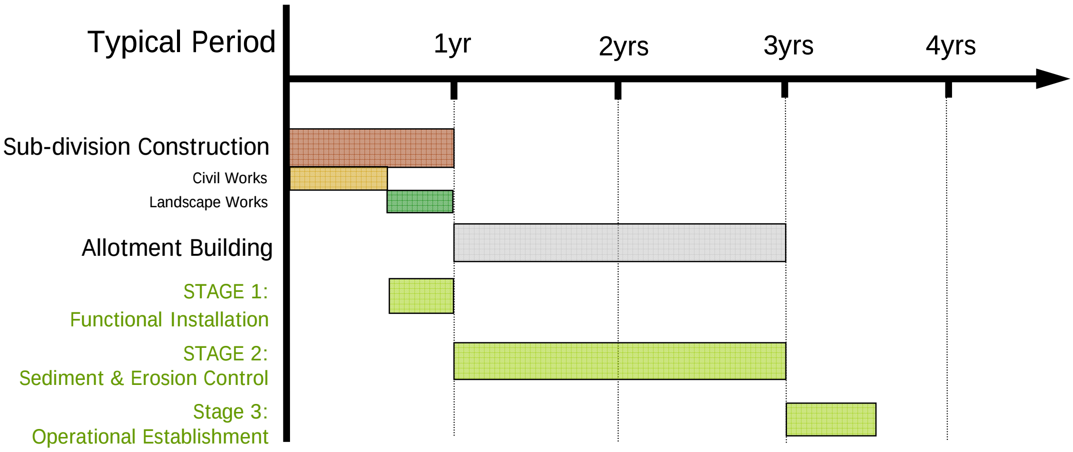


Figure 2-14: Staged Construction and Establishment Method

2.5.2.1 Functional Installation

Protection of the swale during the building phase is important as uncontrolled building site runoff can cause excessive sedimentation and introduce weeds and litter to the swale. As a result, reprofiling and replanting of the swale may be required following the building phase. To avoid this, it is recommended that a staged implementation approach be employed by using, in lieu of the final swale planting, a temporary arrangement of a suitable geofabric covered with shallow topsoil (e.g. 50 mm) and instant turf (laid perpendicular to flow path). This will allow the swale to function as a temporary erosion and sediment control facility throughout the building phase. At the completion of the building phase these temporary measures should be removed with all accumulated sediment and the swale reprofiled (if necessary) and planted in accordance with the proposed swale design. It may be possible to reuse the instant turf as part of the final planting if this is consistent with the proposed landscape design. The local Council may not accept assets that are not performing to design specification (e.g. blocked with construction sediment).

Ensure traffic and deliveries do not access swales during construction. Traffic can compact the soil and cause preferential flow paths, while 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 of wastes.

2.5.2.2 Sediment and Erosion Control

The temporary protective layers should be left in place through the allotment building phase to ensure sediment laden waters do not smother the swale vegetation. Silt fences should be placed around the boundary of the swale to exclude silt and act as a barrier to restrict vehicular and other access.

In addition to regular maintenance (outlined in Section 2.6) 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.

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 road pavement surface. Generally a set down from the kerb of 60 mm to the top of vegetation (if turf) is adequate. Therefore, total set down to the base soil is approximately 100 mm (with turf on top of base soil).

2.5.2.3 Operational Establishment

At the completion of the Allotment Building Phase the temporary measures (i.e. geofabric and turf) are removed with all accumulated sediment and the swale re-profiled and planted in accordance with the proposed landscape design. Establishment of the vegetation to design condition can require more than two growing seasons, depending on the vegetation types, during which regular watering and removal of weeds will be required.

2.5.3 Horticultural Topsoils for Swales (and Buffer Strips)

Soil management for plants should aim to optimise nutrient and soil-water delivery to the plants' root hairs. During the swale construction process, topsoil is to be stripped and stockpiled for possible reuse as a plant growth medium. The quality of the local topsoil should be tested to determine the soils suitability for reuse as a plant growth medium. In situ soils are likely to have changed from its pre-European native state due to prior land uses such as farming and industry. Remediation may be necessary to improve the soils capacity to support plant growth and to suit the intended plant species. Soils applied must also be free from significant weed seed banks as labour intensive weeding can incur large costs in the initial plant establishment phase. On some sites, topsoils may be non-existent and material will need to be imported.

Imported soil must not contain Fire Ants. A visual assessment of the soils is required and any machinery should be free of clumped soil. Soils must not be brought in from Fire Ant restricted areas.

The installation of horticultural soils should follow environmental best practices and include:

- preparation of soil survey reports including maps and test results at the design phase
- stripping and stockpiling of existing site topsoils prior to commencement of civil works
- deep ripping of subsoils using a non-inversion plough
- reapplication of stockpiled topsoils and, if necessary, remedial works to suit the intended plant species
- addition where necessary, of imported topsoils (certified to AS 4419-2003).

The following minimum topsoil depths are required:

- 150 mm for turf species
- 300 mm for groundcovers and small shrubs
- 450 mm for large shrubs
- 600 mm for trees.

2.5.4 Sourcing Swale Vegetation

Notifying nurseries early for contract growing is essential to ensure the specified species are available in the required numbers and of adequate maturity in time for swale planting. When this is not done and the planting specification is compromised, because of sourcing difficulties, poor vegetation establishment and increased initial maintenance costs may occur.

The species listed in Table A.1 (Appendix A) are generally available commercially from local native plant nurseries. Availability is, however, dependent upon many factors including demand, season and seed availability. To ensure the planting specification can be accommodated, the minimum recommended lead time for ordering is 3-6 months. This generally allows adequate time for plants to be grown to the required size. The following sizes are recommended as the minimum:

- Viro Tubes 50 mm wide x 85 mm deep
- 50 mm Tubes 50 mm wide x 75 mm deep
- Native Tubes 50 mm wide x 125 mm deep

2.5.5 Vegetation Establishment

To ensure successful plant establishment the following measures are recommended in addition to regular general maintenance as outlined in the Section 2.6.

2.5.5.1 Timing for Planting

October and November are considered the most ideal time to plant vegetation in treatment elements. This allows for adequate establishment/ root growth before the heavy summer rainfall period but also allows the plants to go through a growth period soon after planting resulting in quicker establishment. Planting late in the year also avoids the dry winter months, reducing maintenance costs associated with watering. Construction planning and phasing should endeavour to correspond with suitable planting months wherever possible. In some circumstances it may be appropriate to leave temporary planting in place (if this is used to protect the swale during the building phase, e.g. turf over geofabric), and then remove this at a suitable time to allow the final swale planting to occur at the preferred time of year.

2.5.5.2 Weed Control

Conventional surface mulching of swale systems with organic material like tanbark, 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. To combat weed invasion and reduce costly maintenance requirements for weed removal, high planting density rates should be adopted. A suitable biodegradable erosion control matting or a heavy application of seedless hydro-mulch can also be applied to swale batters (where appropriate) for short term erosion and weed control.

2.5.5.3 Watering

Regular watering of swale vegetation is essential for successful establishment and healthy growth. The frequency of watering to achieve successful plant establishment is dependent upon rainfall, maturity of planting stock and the water holding capacity of the soil. However, the following watering program is generally adequate but should be adjusted (increased) to suit the site conditions:

- Week 1-2 3 visits/ week
- Week 3-6 2 visits/ week
- Week 7-12 1 visit/ week

After this initial three month period, watering may still be required, particularly during the first winter (dry period). Watering requirements to sustain healthy vegetation should be determined during ongoing maintenance site visits.

2.6 Maintenance Requirements

Swale treatment relies upon good vegetation establishment and therefore ensuring adequate vegetation growth is the key maintenance objective. In addition, they have a flood conveyance role that needs to be maintained to ensure adequate flood protection for local properties.

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 may impact on plant growth, particularly in developing catchments with an inadequate level of erosion and sediment control.

The potential for rilling and erosion along a swale needs to be carefully monitored, particularly during establishment stages of the system. Other components of the system that will require careful consideration are the inlet points (if the system does not have distributed inflows) and surcharge pits. The inlets can be prone to scour and build up of litter and occasional litter removal and potential replanting may be required.

Swale field inlet pits also require routine inspections to ensure structural integrity and that they are free of blockages with debris.

Typical maintenance of swale elements will involve:

- Routine inspection of the swale profile to identify any areas of obvious increased sediment deposition, scouring of the swale invert from storm flows, rill erosion of the swale batters from lateral inflows or damage to the swale profile from vehicles.
- Routine inspection of inlet points (if the swale does not have distributed inflows), surcharge pits and field inlet pits to identify any areas of scour, litter build up and blockages.
- Removal of sediment where it is impeding the conveyance of the swale and/ or smothering the swale vegetation and if necessary reprofiling of the swale and revegetating to original design specification.
- Repairing damage to the swale profile resulting from scour, rill erosion or vehicle damage.
- Clearing of blockages to inlet or outlets.
- Regular watering/ irrigation of vegetation until plants are established and actively growing (see Section 2.5.5.3).
- Mowing of turf or slashing of vegetation (if required) to preserve the optimal design height for the vegetation.
- Removal and management of invasive weeds (see Section 2.5.5.2).
- Removal of plants that have died (from any cause) and replacement with plants of equivalent size and species as detailed in the plant schedule.
- Pruning to remove dead or diseased vegetation material and to stimulate new growth.
- Litter and debris removal.
- Vegetation pest monitoring and control.

Inspections are also recommended following large storm events to check for scour. All maintenance activities must be specified in a maintenance plan (and associated maintenance inspection forms) to be developed as part of the design procedure. Maintenance personnel and asset managers will use this plan to ensure the swales continue to function as designed. Maintenance plans and forms must address the following:

- inspection frequency
- maintenance frequency
- data collection/ storage requirements (i.e. during inspections)
- detailed cleanout procedures (main element of the plans) including:
  - equipment needs
  - maintenance techniques
  - occupational health and safety
  - public safety
  - environmental management considerations
  - disposal requirements (of material removed)
  - access issues
  - stakeholder notification requirements
  - data collection requirements (if any)
- design details

An example operation and maintenance inspection form is provided in the checking tools provided in Section 2.7.

2.7 Checking Tools

This section provides checking aids for designers and Council development assessment officers. Section 2.5also provides general advice for the construction and establishment of swales and key issues to be considered to ensure their successful establishment and operation based on observations from construction projects around Australia.

The following checking tools are provided:

- Design Assessment Checklist
- Construction Inspection Checklist (during and post)
- Operation and Maintenance Inspection Form
- Asset Transfer Checklist (following 'on-maintenance' period).

2.7.1 Design Assessment Checklist

The checklist on page 2-32 presents the key design features that are to be reviewed when assessing a design of a swale. These considerations include configuration, safety, maintenance and operational issues that need to be addressed during the design phase. If an item receives a 'N' when reviewing the design, referral is made back to the design procedure to determine the impact of the omission or error. In addition to the checklist, a proposed design is to have all necessary permits for installation. Local authority development assessment officers will require that all relevant permits be in place prior to accepting the design.

2.7.2 Construction Checklist

The checklist on page 2-33 presents the key items to be reviewed when inspecting the swale during and at the completion of construction. The checklist is to be used by Construction Site Supervisors and local authority compliance inspectors to ensure all the elements of the swale have been constructed in accordance with the design. If an item receives an 'N' in Satisfactory criteria then appropriate actions must be specified and delivered to rectify the construction issue before final inspection sign-off is given.

2.7.3 Operation and Maintenance Inspection Form

The form on page 2-34 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 1 to 6 months depending on the size and complexity of the swale system, and the stage of development (i.e. inspections should be more frequent during building phase).

2.7.4 Asset Transfer Checklist

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. For details on asset transfer to specific to each local government area, contact the relevant local authority. The table on page 2-35 provides an indicative asset transfer checklist.

SWALE DESIGN ASSESSMENT CHECKLIST			
Asset I.D.		DA No.:	
Swale Location:			
Hydraulics:	Minor Flood (m^3/s):	Major Flood (m^3/s):	
Area:	Catchment Area (ha):	Swale Area (m^2):	

TREATMENT	Y	N
Treatment performance verified?		
INFLOW SYSTEMS	**Y**	**N**
Inlet flows appropriately distributed?		
Swale/ buffer vegetation set down of at least 60 mm below kerb invert incorporated?		
Energy dissipation (rock protection) provided at inlet points to the swale?		
SWALE CONFIGURATION/ CONVEYANCE	**Y**	**N**
Longitudinal slope of invert >1% and <4%?		
Manning's n selected appropriate for proposed vegetation type?		
Overall flow conveyance system sufficient for design flood event?		
Maximum flood conveyance width is compliant with QUDM?		
Overflow pits provided where flow capacity exceeded?		
Velocities within swale cells will not cause scour?		
Maximum ponding depth and velocity will not impact on public safety ($V \times d < 0.4$ m/s)?		
Maintenance access provided to invert of conveyance channel?		
LANDSCAPE	**Y**	**N**
Plant species selected can tolerate periodic inundation and design velocities?		
Planting design conforms with acceptable sight line and safety requirements?		
Street trees conform to, Section 3.4.31 of the Land Development Guidelines?		
Top soils are a minimum depth of 300mm for plants and 100 mm for turf?		
Existing trees in good condition are investigated for retention?		
Swale and buffer strip landscape design integrates with surrounding natural and/ or built environment?		
OTHER NOTES		

SWALE CONSTRUCTION INSPECTION CHECKLIST

Asset I.D.:		Inspected by:	
Site:		Date:	
		Time:	
Constructed By:		Weather:	
		Contact during visit:	

Items Inspected	Checked		Satisfactory		Items Inspected	Checked		Satisfactory	
	Y	N	Y	N		Y	N	Y	N
DURING CONSTRUCTION & ESTABLISHMENT									
A. FUNCTIONAL INSTALLATION					Structural Components				
Preliminary Works					13. Location and levels of pits as designed				
1. Erosion/ sediment control plan adopted					14. Safety protection provided				
2. Traffic control measures					15. Location of check dams as designed				
3. Location same as plans					16. Swale crossings located/ built as designed				
4. Site protection from existing flows					17. Pipe joints/ connections as designed				
5. Critical root zones (0.5 m beyond drip line) of nominated trees are protected					18. Concrete and reinforcement as designed				
					19. Inlets appropriately installed				
Earthworks					20. Inlet erosion protection installed				
6. Existing topsoil is stockpiled for reuse					21. Set down to correct level for flush kerbs				
7. Level bed of swale					B. EROSION AND SEDIMENT CONTROL				
8. Batter slopes as plans					22. Silt fences and traffic control in place				
9. Longitudinal slope in design range					23. Stabilisation immediately following earthworks				
10. Provision of sub-soil drainage for mild slopes (<1%)					C. OPERATIONAL ESTABLISHMENT				
11. Compaction process as designed					Vegetation				
12. Appropriate topsoil on swale					24. Test and ameliorate topsoil, if required				
					25. Planting as designed (species/ densities)				
					26. Weed removal and watering as required				
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:
Inspection officer signature:

SWALE (AND BUFFER) MAINTENANCE CHECKLIST

Asset I.D.:			
Inspection Frequency:	1 to 6 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 (e.g. 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?			
Soil additives or amendments required?			
Pruning and/ or removal of dead or diseased vegetation required?			
COMMENTS			

ASSET TRANSFER CHECKLIST		
Asset Description:		
Asset I.D.:		
Asset Location:		
Construction by:		
'On-maintenance' Period:		
TREATMENT	Y	N
System appears to be working as designed visually?		
No obvious signs of under-performance?		
MAINTENANCE	Y	N
Maintenance plans and indicative maintenance costs provided for each asset?		
Vegetation establishment period completed (2 years)?		
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 (e.g. drawings, survey, models) provided?		
Asset listed on asset register or database?		
COMMENTS		

2.8 Engineering Drawings and Standards

The relevant local authority should be consulted to source standard drawings applicable to swales. These drawings may provide example swale dimensions for a number of different road reserve configurations.

If no standard drawings exist for the local government area, Brisbane City Council standard drawings applicable to swales (UMS 151-154, UMS 157 and UMS 158) may be used as reference standards for swale design. BCC Standard drawings are available online at <http://www.brisbane.qld.gov.au/BCC:STANDARD:1084547806:pc=PC_1389>.

2.9 Swale Worked Example

As part of a residential development in the Gold Coast region, runoff from allotments and street surfaces is to be treated in vegetated swale systems where practical. This worked example describes the detailed design of two different swale systems located in the road reserve of a local road network within the residential estate. The conceptual configuration of the swale is presented in Figure 2-15. The layout of the catchment and swales (Swale 1 is 75 m long and Swale 2 is 35 m long) is presented in

Figure 2-16.

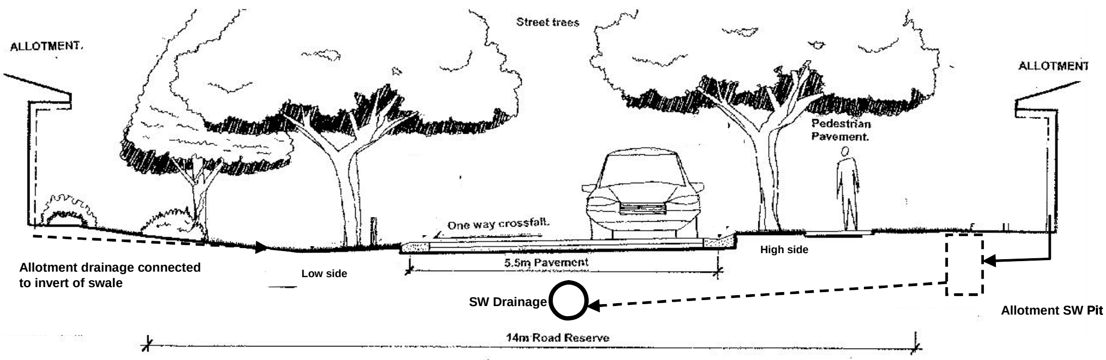


Figure 2-15: Conceptual Configuration of a Swale

Plate 2-5: Typical sections for Swale 1 (left) and Swale 2 (right)(Note that Swale 1 also requires provision for traffic control and Swale 2 requires provision for pedestrians)

Figure 2-16: Typical Layout for Swale Worked Example

A stormwater management concept design for the development recommended these systems (Swale 1 and Swale 2) as part of a treatment train. The site comprises a network of residential allotments served by a 15 m local road reserve within which the swale systems are to be located. The streets will have a one way crossfall (to the high side) with flush kerbs to allow for distributed flows into the swale systems across a buffer zone. The intention is for turf swale systems in most cases, however, the development is located within a bushland setting so the project landscape architect has indicated that densely planted native tufted vegetation (e.g. sedges to 300 mm) should be investigated for Swale 2.

The contributing catchment area to Swale 1 includes 35 m deep allotments on one side and the road reserve. In this case runoff from allotments on the opposite side of the road is to be treated using on site treatment facilities and will discharge via allotment drainage to the piped stormwater drainage system under the road pavement. Access to the allotments draining to the swale will be via an at-grade crossover requiring maximum batter slope for the swale of 1 in 9 (11 %).

Swale 2 is to accept runoff from a small road reserve catchment plus a relatively large inter-allotment catchment which discharges just upstream of a driveway cross over. Flows will enter the swale and be conveyed under a raised driveway crossover via a conventional stormwater culvert and back into the swale system with appropriate erosion control.

For both systems (Swale 1 and Swale 2), the road reserve comprises a 7.5 m wide road pavement surface, 3.5 m footpath reserve on the opposite to the swale and a 4.0 m swale and services easement.

Minor and major flood events are to be conveyed within the swale/ road corridor in accordance with local Council development guidelines (i.e. some inundation of the road is allowable). The top width of both swales is fixed (at 4.0 m) and there will be a maximum catchment area the swale can accommodate, beyond which an underground pipe may be required to augment the conveyance capacity of the swale and road system.

Design Objectives

The design criteria for the swale systems are to:

- Convey all flows associated with minor (10 year, as defined by Council's guidelines) and major (100 year, as defined by Council's guidelines) storm events within the swale/ road system.
- Ensure flow velocities do not result in scour.
- Ensure public safety, in particular vehicle and pedestrian safety.
- Promote sedimentation of coarse particles through the edge of the swale by providing for an even flow distribution and areas for sediment accumulation.
- Provide traffic management measures that will preclude traffic damage (or parking) within the buffer or swale (e.g. bollards or parking bays).
- Integration of the swale and buffer strip landscape design with the surrounding natural and/ or built environment.
- Provision of driveway access to lots given side slope limits.

Site Characteristics

Swale 1:

Catchment area	1,987 m^2	(lots)
	638 m^2	(roads and concrete footpath)
	255 m^2	(swale and services easement)
	2,880 m^2	(total catchment)
Landuse/ surface type	residential lots, roads/ concrete footpaths, swale and service easement	
Overland flow slope:	total main flow path length = 80 m	
	slope = 2 %	
Soil type:	clay	
Fraction impervious:	lots $f_i = 0.7$	
	roads/ footpath $f_i = 1.00$	
	swale/ service easement $f_i = 0.10$	

Swale 2:

Catchment area	1,325 m^2	(lots)
	500 m^2	(roads and concrete footpath)
	200 m^2	(swale and services easement)
	2,025 m^2	(total)
Landuse/ surface type	residential lots, roads/ concrete footpaths, swale and service easement	
Overland flow slope:	total main flow path length = 30 m	
	slope = 2 %	
Soil type:	clay	
Fraction impervious:	lots $f_i = 0.7$	
	roads/ footpath $f_i = 1.00$	
	swale/ service easement $f_i = 0.10$	

2.9.1 Step 1: Confirm Treatment Performance of Concept Design

The earlier conceptual design of the stormwater treatment system required of this project included appropriate modelling using MUSIC to ensure that stormwater discharges from the site comply with the relevant water quality objectives (WQOs). It is noted that subsequent additional treatment elements will be required (e.g. wetlands, bioretention systems) in order to enable such WQO compliance.

Using the curves in Figure 2-3 to Figure 2-5, the swale configuration can be expected to achieve load reductions of 90%, 60% and 15% of TSS, TP and TN respectively. The swales are approximately 10% of the contributing catchment areas, well above the required size noted in the curves, however the swale design is also responding to hydraulic capacity, landscape outcomes and access requirements (i.e. 'at-grade' driveway crossings) within the development.

2.9.2 Step 2: Determine Design Flows

With a small catchment, the Rational Method is recommended to estimate peak flow rates. The development constitutes high density residential development (> 20 dwellings/ha), as defined by the local council's development guidelines. Therefore the minor system design event is the 10 year ARI (in this Council area). The steps in determining peak flow rates for the minor and major design events using the Rational Method is outlined in the calculations below.

2.9.2.1 Major and Minor Design Flows

Time of concentration (t_c)

Approach:

The time of concentration is estimated assuming overland flow across the allotments and along the swale.

From procedures documented in QUDM and the local council's development guidelines, the overland sheet flow component should be limited to 50 m in length and determined using the Kinematic Wave Equation:

$$t = 6.94\,(L.n^*)^{0.6}/I^{0.4}\,S^{0.3}$$

Where
- t = overland sheet flow travel time (mins)
- L = overland sheet flow path length (m)
- n^* = surface roughness/retardance coefficient
- I = rainfall intensity (mm/hr)
- S = slope of surface (m/m)

In urban areas, QUDM notes that sheet flow will typically be between 20 to 50 m, after which the flow will become concentrated against fences, gardens or walls or intercepted by minor channel or piped drainage (from QUDM). Therefore when calculating remaining overland flow travel times, it is recommended that stream velocities in Table 5.05.4 of QUDM be used.

<u>Swale 1</u>

Assuming: Predominant slope = 2%

Overland sheet flow component = 50 m

Overland channel flow component = 30 m

Flow path is predominately lawn, with a typical n* = 0.25 (QUDM)

10 year ARI:

$t_{sheet\ flow}$ $= 6.94\ (50 \times 0.25)^{0.6}/(141.54^{0.4} \times 0.02^{0.3})$

= 14 mins

Iterations will need to be repeated until $t_{sheet\ flow}$ matches 10 year ARI rainfall intensity on the IFD chart for that duration, as shown in the above calculation. Note that IFD data will need to be determined in line with the relevant local council's guidelines.

$t_{channel\ flow}$ = (30m / 0.7m/s)/ 60s/min

= 1 min

t_c $= t_{sheet\ flow} + t_{channel\ flow}$

= 15 mins

100 year ARI:

$t_{sheet\ flow}$ $= 6.94\ (50 \times 0.25)^{0.6}/(209.06^{0.4} \times 0.02^{0.3})$

= 12 mins

Iterations will need to be repeated until $t_{sheet\ flow}$ matches the 100 year ARI rainfall intensity on the IFD chart for that duration, as shown in the above calculation.

$t_{channel\ flow}$ = (30m / 0.7m/s)/ 60s/min

= 1 min

t_c $= t_{sheet\ flow} + t_{channel\ flow}$

= 13 mins

<u>Swale 2</u>

Assuming: Predominant slope = 2%

Overland sheet flow = 30m

Flow path is predominately lawn, with a typical n* = 0.25 (QUDM)

10 year ARI:

t_c $= 6.94\ (30 \times 0.25)^{0.6}/(164.1^{0.4} \times 0.02^{0.3})$

= 10 mins

Repeat iterations until $t_{sheet\ flow}$ matches the 10 year ARI rainfall intensity on IFD chart for that duration.

10 year ARI:

t_c $= 6.94\ (30 \times 0.25)^{0.6}/(246.7^{0.4} \times 0.02^{0.3})$

= 8 mins

Repeat iterations until $t_{sheet\ flow}$ matches the 100 year ARI rainfall intensity on IFD chart for that duration.

Design rainfall intensities

Table 2-2: Summary of Design Rainfall Intensities

Swale	10 year t_c	10 year Rainfall Intensity	100 year t_c	100 year Rainfall Intensity
Swale 1	15 mins	137.1 mm/hr	13 mins	146.4 mm/hr
Swale 2	10 mins	164.1 mm/hr	8 mins	246.7 mm/hr

Design runoff coefficient

Apply the rational formula method outlined in QUDM using runoff coefficients as specified by the local Council's development guidelines.

Assuming the Development Category is High Density (classified as "Res B" within Council's planning scheme), with a slope of 2%, C_{10} = 0.85

Hence using QUDM table 5.04.3

C_{10} = 1.00 x 0.85 = 0.85

C_{100} = 1.20 x 0.85 = 1.02 = 1.00

Peak design flows

As it is a small catchment apply the Rational Method.

Q = CIA/360

Swale 1:

Q_{10} = 0.00278 x 0.85 x 137.1 x 0.288 = 0.093 m^3/s

Q_{100} = 0.00278 x 1.00 x 146.4 x 0.288 = 0.117 m^3/s

Swale 2:

Q_{10} = 0.00278 x 0.85 x 164.1 x 0.2025 = 0.079 m^3/s

Q_{100} = 0.00278 x 1.00 x 246.7 x 0.2025 = 0.139 m^3/s

2.9.3 Step 3: Configuring the Swale

2.9.3.1 Swale Width and Side Slopes

To facilitate at-grade driveway crossings the following cross section is proposed:

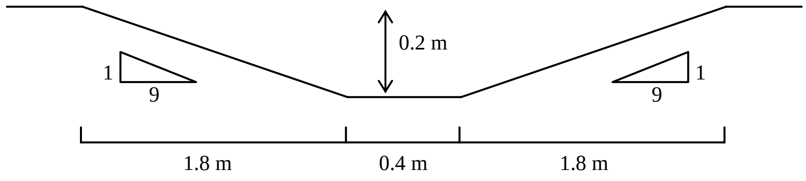


Figure 2-17: Swale Width and Side Slopes cross section

2.9.3.2 Maximum Length of Swale

To determine the maximum length of both swales (i.e. the maximum length before an overflow pit (field entry pit) is required, the 'bank full' capacity of the swale is estimated to establish how much (if any) of the minor flood and major flood flow may need to be conveyed by the road. 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 in Swale 2.

A range of Manning's *n* values are selected for different flow depths appropriate for grass. It is firstly assumed that at bank full capacity, the flow height will be well above the vegetation height in the swale and therefore Manning's *n* will be quite low (refer to Figure 2-6). A figure of 0.04 is adopted (flow depth will need to be checked to ensure it is above the vegetation).

- adopt slope 2 % (stated longitudinal slope)
- Manning's $n = 0.04$ (at 0.2 m depth)
- side slopes 1(v):9(h).

Using Equation 2.1: $$Q = \frac{A \quad R^{2/3} \quad S^{1/2}}{n}$$

$Q_{cap} = 0.357$ m^3/s >> Q_{10} (0.093 m^3/s and 0.079 m^3/s for Swale 1 and 2 respectively)

The nominated swale (determined from the landscape design) has sufficient capacity to convey the required peak 10 year ARI flow (Q_{10}) without any requirement for flow on the adjacent road or for an additional piped drainage system. The capacity of the swale (Q_{cap} = 0.357 m^3/s) is also sufficient to convey the entire peak 100 year ARI flow (Q_{100}) of 0.117 m^3/s (swale 1) and 0.139 m^3/s (swale 2) without requiring flow to be conveyed on the adjacent road and footpath. Therefore, the maximum permissible length of swale for both Swale 1 and Swale 2 is clearly much longer than the 'actual' length of each swale (i.e. Swale 1 = 75 m and Swale 2 = 35 m) and as such no overflow pits are required and no checks are required to confirm compliance with the road drainage standards for minor and major floods as defined in the local council's development guidelines.

To confirm the Manning's *n* assumptions used in the above calculations, 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 shown in Table 2-3.

Table 2-3: Manning's *n* and Flow Capacity Variation with Flow Depth – Turf

Flow Depth (m)	Manning's *n*	Flow Rate (m^3/s)
0.025	0.3	0.001
0.05	0.1	0.006
0.1	0.05	0.056
0.11	0.05	0.069
0.12	0.05	0.085
0.13	0.05	0.102
0.14	0.05	0.121
0.15	0.04	0.179
0.2	0.04	0.356

From the table of Manning's equation output (Table 2-3), it can be seen that the boundary layer effect created by the turf significantly decreases between a flow depth of 0.025 m and 0.1 m with Manning's *n* decreasing from 0.3 to 0.05. This is due to the weight of the water flowing over the grass causing it to 'yield over' creating a 'smoother' surface with less resistance to flow. Once the water depth has reached three times the vegetation height (0.15 m), the Manning's n roughness coefficient has been further reduced to 0.04. The use of Manning's n = 0.04 for the calculation of the 'bank full' capacity of the swale is validated by Table 2-3 which also shows the 10 year ARI peak flows in Swale 1 and Swale 2 would have a flow depth of approximately 0.12m, and the 100 year ARI peak flows in both Swale 1 and 2 would have a flow depth of approximately 0.14 m.

The flow depths of both the minor (0.12 m) and major (0.14 m) event flows are less than the depth of the swale (0.2m), indicating that all flow is contained within the swales. Usually the swale should be sized so that in a major event the road accommodates some of the flow in line with flow width depth requirements outlined in the local Council's development guidelines. However these dimensions are used to facilitate at grade driveway crossings (refer to Figure 2-17).

Based on this result, the maximum permissible length of swale is also much longer than the 'actual' length of Swale 2 (i.e. 35 m) and as such no overflow pits are required except for at the downstream end of the swale to facilitate discharge to the trunk underground pipe drainage system (see Section 2.3.6 for design of overflow pits).

For the purposes of this worked example, the capacity of Swale 2 is also estimated when using 300 mm 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.

Table 2-4 below presents the adopted Manning's *n* values and the corresponding flow capacity of the swale for different flow depths using 300mm high vegetation (sedges).

Table 2-4: Manning's *n* and Flow Capacity Variation with Flow Depth – Sedges

Flow Depth (m)	Manning's *n*	Flow Rate (m^3/s)
0.025	0.3	0.001
0.05	0.3	0.002
0.1	0.3	0.009
0.11	0.3	0.012
0.13	0.3	0.017
0.14	0.3	0.020
0.15	0.3	0.024
0.18	0.3	0.037
0.2	0.3	0.048

Table 2-4 shows that the current dimension of Swale 2 is not capable of conveying the 10 year ARI (Q_{10}) discharge for the higher vegetation. In this case, if the designer wishes to use sedges in the swale, additional hydraulic calculations will be required to determine the maximum length of swale to ensure that the swale and adjacent roadway can convey the 10 and 100 year ARI events, in accordance with the requirements of the local council's development guidelines.

Regardless of the above, this worked example continues using the grass option for Swale 2.

2.9.4 Step 4 Design Inflow Systems

There are two ways for flows to reach the swale, either directly from the road surface or from allotments via an underground 100 mm pipe.

Direct runoff from the road enters the swale via a buffer (the grass edge of the swale). The pavement surface is set 60 mm higher than the top of the swale batter (i.e. the top of the vegetation) and has a taper that will allow sediments to accumulate in the first section of the buffer, off the road pavement surface.

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.

2.9.5 Step 5: Verification Checks

2.9.5.1 Vegetation scour velocity checks

Velocity checks are performed to ensure vegetation is protected from erosion at high flow rates. 10 year and 100 year ARI flow velocities are checked and need to be kept below 0.5 m/s and 2.0 m/s respectively.

Velocities are estimated using Manning's equation. Velocities are checked at the downstream end of each swale:

Swale 1 and Swale 2:

Flow depth, $d_{10\text{-}year} = 0.12$ m

Velocity, $V_{10\ year} = 0.48$ m/s < 0.5 m/s therefore OK

$d_{100\ year} = 0.14$ m

$V_{100\text{-}year} = 0.52$ m/s < 2.0 m/s therefore OK

2.9.5.2 Velocity and Depth Checks - Safety

Given both Swale 1 and 2 can convey their respective 100 year ARI design flows the maximum velocity-depth product will therefore be at the most downstream end of each swale.

At the bottom of both Swale 1 and Swale 2:

$V = 0.52$ m/s, $d = 0.14$ m; therefore $V \times d = 0.073 < 0.4$, therefore OK.

Swale 1 is the only swale that will have flows passing over driveway crossings and the maximum depth of flow in Swale 1 for the 100 year ARI event is only 0.14 m (refer to Table 2-3) which is much less than the maximum allowable 0.3 m.

2.9.5.3 Confirm Treatment Performance

As there has been no requirement to alter the swale geometry established for Swales 1 and 2 in Step 3, the same treatment performance identified in Step 1 still applies. Where modifications to the swale geometry occur during the previous design steps, a check of the new configuration with procedures identified in Step 1 is required to ensure treatment performance is adequate.

2.9.6 Step 6: Size Overflow Pits

As determined in Step 3, both Swale 1 and Swale 2 have sufficient capacity to convey up to the 1 year ARI event from their respective catchments and as such do not require the use of overflow pits. However, the case study requires an overflow pit to be provided at the downstream end of both Swale 1 and 2 to facilitate discharge to the trunk underground pipe drainage system.

The trunk minor drainage system is a 10 year ARI system and therefore the overflow pits at the downstream end of Swales 1 and 2 need to be sized to discharge the peak 10 year ARI flow from each swale. The calculations to size the overflow pits are presented below:

Swale 1:

Q_{10} = 0.093 m^3/s

Check for free overflow using Equation 2.2:

$$Q_{weir} = B \cdot C_w \cdot L \cdot H^{3/2}$$

Q = 0.093 = 0.5 x 1.66 x L x 0.2 $^{3/2}$

Therefore L = 1.2 m (Therefore, a pit dimension of 0.3 m x 0.3 m would be sufficient. However, the minimum pit dimensions to be used in the local Council authority should be checked).

Check for drowned outlet conditions using Equation 2.3:

$$Q_{orifice} = B\ C_d\ A\sqrt{2\ g\ h}$$

Q = 0.093 = 0.5 x 0.6 x A x √3.924

Therefore A = 0.156 m^2 or 0.39 m x 0.39 m (as with free overflow conditions the minimum pit dimension for use in the local council authority should be checked).

Drowned outlet conditions are the 'control' and therefore the selected overflow pit dimension is 390 mm x 390 mm with a grate cover.

Swale 2:

Q_{10}= 0.079 m^3/s

Check for free overflow using Equation 2.2:

$$Q_{weir} = B \quad C_w \quad L \quad h^{3/2}$$

Q = 0.079 = 0.5 x 1.66 x L x 0.2 $^{3/2}$

Therefore L = 1.1 m (a pit dimension of 0.3 m x 0.3 m would be sufficient. However, the minimum pit dimensions for use in the local Council authority should be checked).

Check for drowned outlet conditions using Equation 2.3:

$$Q_{orifice} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h}$$

Q = 0.079 = 0.5 x 0.6 x A x √3.924

Therefore A = 0.133 m^2 or 0.36 m x 0.36 m (as with free overflow conditions the minimum pit dimensions for use in the local Council authority should be checked).

Drowned outlet conditions are the 'control' and therefore the selected overflow pit dimension is 360 mm x 360 mm with a grate cover that complies with local council regulations.

Plate 2-6: Traffic bollards mixed with street trees to protect swale from vehicles

2.9.7 Step 7: Traffic Control

Traffic control in the worked example is achieved by using traffic bollards mixed with street trees.

2.9.8 Step 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 landscape designer will select the actual species.

2.9.9 Calculation summary

The following table shows the results of the design calculations for Swale 1 only. The same calculation summary layout may be used for Swale 2.

SWALES – DESIGN CALCULATION SUMMARY SHEET

	Calculation Task		CALCULATION SUMMARY		
			Outcome		Check
	Catchment Characteristics (Swale 1)				
		Catchment Area	0.384	ha	✓
		Catchment Land Use (i.e. residential, Commercial etc.)	Res B		
		Catchment Slope	2	%	
	Conceptual Design				
		Swale Top Width	4	m	✓
		Swale Length	75	m	
		Swale Location (road reserve/ park/other)	Road res		
		Road Reserve Width	15	m	
1	**Confirm Treatment Performance of Concept Design**				
		Swale Area	300	m^2	✓
		TSS Removal	88	%	
		TP Removal	62	%	
		TN Removal	15	%	
2	**Determine Design Flows**				
	Time of concentration				
	Swale 1				
		2-10 year ARI	15	minutes	✓
		50-100 year ARI	13	minutes	
	Swale 2				
		2-10 year ARI	10	minutes	✓
		50-100 year ARI	8	minutes	
	Identify Rainfall intensities				
	Swale 1				
		$I_{2\text{-}10 \text{ year ARI}}$	137.1	mm/hr	✓
		$I_{50\text{-}100 \text{ year ARI}}$	146.4	mm/hr	
	Swale 2				
		$I_{2\text{-}10 \text{ year ARI}}$	164.1	mm/hr	✓
		$I_{50\text{-}100 \text{ year ARI}}$	246.7	mm/hr	
	Design Runoff Coefficient				
		$C_{2\text{-}10 \text{ year ARI}}$	0.85		✓
		$C_{50\text{-}100 \text{ year ARI}}$	1.00		
	Peak Design Flows				
		2-10 year ARI	0.093	m^3/s	✓
		50-100 year ARI	0.139	m^3/s	
3	**Dimension the Swale**				
	Swale Width and Side Slopes				
		Base Width	0.4	m	✓
		Side Slopes – 1 in	9		
		Longitudinal Slope	2	%	
		Vegetation Height	50	mm	
	Maximum Length of Swale				
		Manning's n	0.04		✓
		Swale Capacity	0.357		
		Maximum Length of Swale	<80		
4	**Design Inflow Systems**				
		Swale Kerb Type	Flush		✓
		60 mm set down to Buffer/ Swale Vegetation	Yes	Yes/ No	
		Adequate Erosion and Scour Protection (where required)	N/A		
5	**Verification Checks (refer to local Council Development Guidelines)**				
		Velocity for 2-10 year ARI flow (< 0.5 m/s)	0.48	m/s	✓
		Velocity for 50-100 year ARI flow (< 2 m/s)	0.52	m/s	
		Velocity x Depth for 50-100 year ARI (< 0.4 m^2/s)	0.07	m^2/s	
		Depth of Flow over Driveway Crossing for 50-100 year ARI (< 0.3 m)	0.14	m	
		Treatment Performance consistent with Step 1	Yes		
6	**Size Overflow Pits (Field Inlet Pits)**				
		System to convey minor floods – Swale 1	390 x 390	L x W	✓
		System to convey minor floods – Swale 2	360 x 360	L x W	

2.10 References

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[1] At the time of preparation of these guidelines, QUDM was under review and a significantly revised edition is expected to be released in 2006. These guidelines refer to and use calculations specified in the existing QUDM document, however the revised version of QUDM should be used as the appropriate reference document. It should be noted by users of this guideline that the structure and content of QUDM will change, and as such, the references to calculations and/or specific sections of QUDM may no longer be correct. Users of this guideline should utilise and adopt the relevant sections and/or calculations of the revised QUDM guideline.

Chapter 3 Bioretention Swales

3.1 Introduction 3-2

3.2 Design Considerations for Bioretention Swales 3-3

3.2.1 Landscape Design 3-3
3.2.2 Hydraulic Design 3-3
3.2.3 Ex-filtration to In-situ Soils 3-3
3.2.4 Vegetation Types 3-4
3.2.5 Bioretention Filter Media 3-4
3.2.6 Traffic Controls 3-5
3.2.7 Roof Water Discharge 3-6
3.2.8 Services 3-6

3.3 Bioretention Swale Design Process 3-7

3.3.1 Step 1: Confirm Treatment Performance of Concept Design 3-8
3.3.2 Step 2: Determine Design Flows for the Swale Component 3-10
3.3.3 Step 3: Dimension the Swale Component with Consideration to Site Constraints 3-10
3.3.4 Step 4: Design Inflow Systems to Swale and Bioretention Components 3-12
3.3.5 Step 5: Design Bioretention Component 3-15
3.3.6 Step 6: Verify Design 3-18
3.3.7 Step 7: Size Overflow Pit (Field Inlet Pits) 3-18
3.3.8 Step 8: Make Allowances to Preclude Traffic on Swales 3-19
3.3.9 Step 9: Specify Plant Species and Planting Densities 3-19
3.3.10 Step 10: Consider Maintenance Requirements 3-19
3.3.11 Design Calculation Summary 3-19
3.3.12 Typical Design Parameters 3-21

3.4 Landscape Design Notes 3-21

3.4.1 Objectives 3-21
3.4.2 Context and Site Analysis 3-21
3.4.3 Streetscape Bioretention Systems 3-22
3.4.4 Civic and Urban Spaces 3-26
3.4.5 Open Space Bioretention Swales 3-27
3.4.6 Appropriate Plant Species 3-27
3.4.7 Safety Issues 3-28

3.5 Construction and Establishment 3-28

3.5.1 Staged Construction and Establishment Method 3-28
3.5.2 Construction Tolerances 3-30
3.5.3 Sourcing Bioretention Vegetation 3-31
3.5.4 Vegetation Establishment 3-31

3.6 Maintenance Requirements 3-31

3.7 Checking Tools 3-33

3.7.1 Design Assessment Checklist 3-33
3.7.2 Construction Checklist 3-33
3.7.3 Operation and Maintenance Inspection Form 3-33
3.7.4 Asset Transfer Checklist 3-33

3.8 Engineering Drawings 3-38

3.9 Bioretention Swale Worked Example 3-38

3.9.1 Step 1: Confirm Treatment Performance of Concept Design 3-40
3.9.2 Step 2: Estimating Design Flows for Swale Component 3-40
3.9.3 Step 3: Dimension the Swale Component 3-42
3.9.4 Step 4: Design Inflow Systems to Swale and Bioretention Components 3-42
3.9.5 Step 5: Design Bioretention Component 3-42
3.9.6 Step 6: Verification Checks 3-45
3.9.7 Step 7: Overflow Pit Design 3-45
3.9.8 Step 8: Allowances to Preclude Traffic on Swales 3-46
3.9.9 Step 9: Vegetation Specification 3-46
3.9.10 Step 10: Maintenance Plan 3-46
3.9.11 Calculation summary 3-46

3.10 References 3-48

3.1 Introduction

Bioretention swales provide both stormwater treatment and conveyance functions, combining a bioretention system installed in the base of a swale that is designed to convey stormwater as part of a minor and/ or major drainage system. The swale component (refer also to Chapter 2 - Swales) provides pre-treatment of stormwater to remove coarse to medium sediments while the bioretention system removes finer particulates and associated contaminants. Bioretention swales provide flow retardation for frequent storm events and are particularly efficient at removing nutrients.

The bioretention swale treatment process operates by filtering stormwater runoff through surface vegetation associated with the swale and then percolating the runoff through a prescribed filter media, forming the bioretention component which provides treatment through fine filtration, extended detention treatment and some biological uptake.

Bioretention swales also act to disconnect impervious areas from downstream waterways and provide protection to natural receiving waterways from frequent storm events by reducing flow velocities compared with piped systems. The bioretention component is typically located at the downstream end of the overlying swale 'cell' (i.e. immediately upstream of the swale overflow pit(s) as shown on Figure 3-1 or can be provided as a continuous "trench" along the full length of a swale).

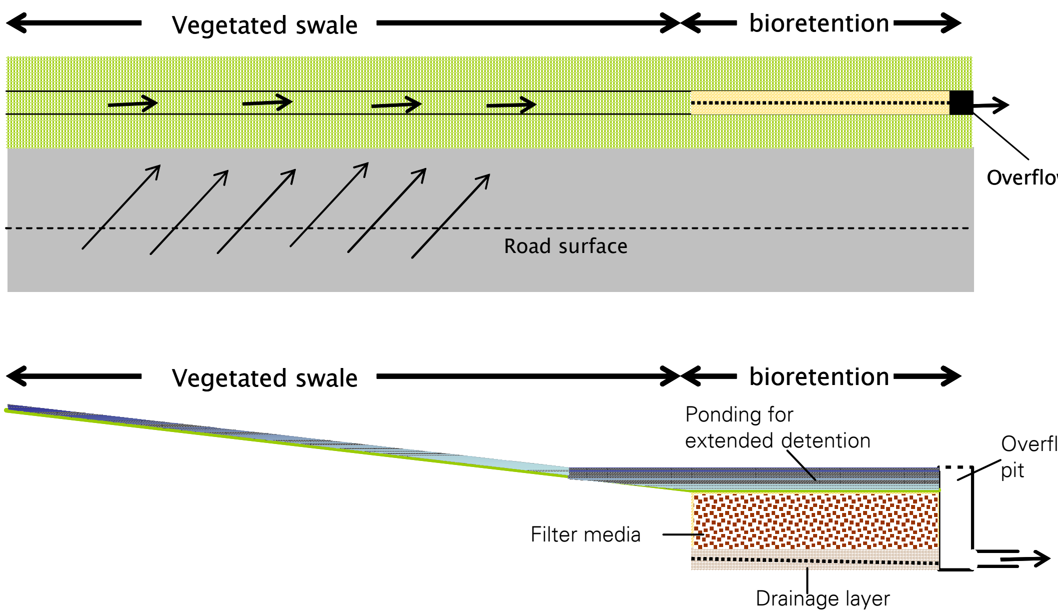


Figure 3-1 Bioretention Swale Conceptual Layout

The choice of bioretention location within the overlying swale will depend on a number of factors, including area available for the bioretention filter media and the maximum batter slopes for the overlying swale. Typically, when used as a continuous trench along the full length of a swale, the desirable maximum longitudinal grade of the swale is 4 %. For other applications, the desirable grade of the bioretention zone is either horizontal or as close as possible to encourage uniform distribution of stormwater flows over the full surface area of bioretention filter media and allowing temporary storage of flows for treatment before bypass occurs.

Bioretention swales are not intended to be 'infiltration' systems in that the intent is typically not to have the stormwater exfiltrate from the bioretention filter media to the surrounding in-situ soils. Rather, the typical design intent is to recover the percolated stormwater runoff at the base of the filter media, within perforated under-drains, for subsequent discharge to receiving waterways or for storage for potential reuse. In some circumstances however, where the in-situ soils allow and there is a particular design intention to recharge local groundwater, it may be desirable to permit the percolated stormwater runoff to infiltrate from the base of the filter media to the underlying in-situ soils.

3.2 Design Considerations for Bioretention Swales

This section outlines some of the key design considerations for bioretention swales that the detailed designer should be familiar with before applying the design procedure presented later in this chapter. Standard design considerations for the swale component of bioretention swales are discussed in detail in Chapter 2 (Swales) and are not reproduced here. However, other swale design considerations that relate specifically to the interactions between the swale and bioretention components are presented in the following sections together with design considerations relating specifically to the bioretention component.

3.2.1 Landscape Design

Bioretention swales may be located within parkland areas, easements, carparks or along roadway corridors within footpaths (i.e. road verges) or centre medians. Landscape design of bioretention swales along the road edge can assist in defining the boundary of road or street corridors as well as providing landscape character and amenity. It is therefore important that the landscape design of bioretention swales addresses stormwater quality objectives whilst also being sensitive to these other important landscape functions.

3.2.2 Hydraulic Design

A key hydraulic design consideration for bioretention swales is the delivery of stormwater runoff from the swale onto the surface of a bioretention filter media. Flow must not scour the bioretention surface and needs to be uniformly distributed over the full surface area of the filter media. In steeper areas, check dams may be required along the swale to reduce flow velocities discharged onto the bioretention filter media.

It is important to ensure that velocities in the bioretention swale from both minor (2-10 year ARI) and major (50-100 year ARI) runoff events are kept sufficiently low (preferably below 0.5 m/s and not more than 2.0 m/s for major flood) to avoid scouring. This can be achieved by ensuring the slope and hydraulic roughness of the overlying swale reduce flow velocities by creating shallow temporary ponding (i.e. extended detention) over the surface of the bioretention filter media via the use of a check dam and raised field inlet pits. This may also increase the overall volume of stormwater runoff that can be treated by the bioretention filter media.

3.2.3 Ex-filtration to In-situ Soils

Bioretention swales can be designed to either preclude or promote ex-filtration of treated stormwater to the surrounding in-situ soils depending on the overall stormwater management objectives established for the given project. When considering ex-filtration to surrounding soils, the designer must consider site terrain, hydraulic conductivity of the in-situ soil, soil salinity, groundwater and building setback. Further guidance in this regard is provided in Chapter 7 Infiltration Measures.

Where the concept design specifically aims to preclude ex-filtration of treated stormwater runoff it is necessary to consider if the bioretention swale needs to be provided with an impermeable liner. The amount of water lost from bioretention trenches to surrounding in-situ soils is largely dependant on the characteristics of the local soils and the saturated hydraulic conductivity of the bioretention filter media (see Section 3.2.5). Typically, if the selected saturated hydraulic conductivity of the filter media is one to two orders of magnitude (i.e. 10 to 100 times) greater than that of the native surrounding soil profile, then the preferred flow path for stormwater runoff will be vertically through the bioretention filter media and into the perforated under-drains at the base of the filter media. As such, there will be little if any ex-filtration to the native surrounding soils. However, if the selected saturated hydraulic conductivity of the bioretention filter media is less than 10 times that of the native surrounding soils, it may be necessary to provide an impermeable liner. Flexible membranes or a concrete casting are commonly used to prevent excessive ex-filtration. This is particularly applicable for surrounding soils that are very sensitive to any ex-filtration (e.g. sodic soils and reactive clays in close proximity to significant structures such as roads).

The greatest pathway of ex-filtration is through the base of a bioretention trench, as gravity and the difference in hydraulic conductivity between the filter media and the surrounding native soil would typically act to minimise ex-filtration through the walls of the trench. If lining is required, it is likely that only the base and the sides of the *drainage layer* (refer Section 3.2.5) will need to be lined.

Where ex-filtration of treated stormwater to the surrounding in-situ soils is promoted by the bioretention swale concept design, it is necessary to ensure the saturated hydraulic conductivity of the in-situ soils is at least equivalent to that of the bioretention filter media, thus ensuring no impedance of the desired rate of flow through the bioretention filter media. Depending on the saturated hydraulic conductivity of the in-situ soils it may be necessary to provide an impermeable liner to the sides of the bioretention filter media to prevent horizontal ex-filtration and subsequent short-circuiting of the treatment provided by the filter media. Bioretention trenches promoting ex-filtration do not require perforated under-drains at the base of the filter media or a drainage layer.

A subsurface pipe is often used to prevent water intrusion into a road sub-base. This practice is to continue as a precautionary measure to collect any water seepage from bioretention swales located along roadways.

3.2.4 Vegetation Types

Bioretention swales can use a variety of vegetation types including turf (swale component only), sedges and tufted grasses. Vegetation is required to cover the whole width of the swale and bioretention filter media surface, be capable of withstanding design flows and be of sufficient density to prevent preferred flow paths and scour of deposited sediments.

Grassed (turf) bioretention swales can be used in residential areas where a continuous bioretention trench approach is used. However, grassed bioretention swales need to be mown to protect the conveyance capacity of the swale component and therefore repeated mowing of the grass over a continuous bioretention trench can result in long term compaction of the filter media and reduce its treatment performance. The preferred vegetation for the bioretention component of bioretention swales is therefore sedges and tufted grasses (with potential occasional tree plantings) that do not require mowing.

The denser and taller the vegetation planted in the bioretention filter media, the better the treatment provided, especially during extended detention. Taller vegetation has better interaction with temporarily stored stormwater during ponding, which results in enhanced sedimentation of suspended sediments and associated pollutants. The vegetation that grows in the bioretention filter media also acts to continuously break up the surface of the media through plant root growth and wind induced agitation, which prevents surface clogging. Vegetation also provides a substrate for biofilm growth in the upper layer of the filter media which facilitates biological transformation of pollutants (particularly nitrogen).

Dense vegetation planted along the swale component can also offer improved sediment retention by reducing flow velocity and providing vegetation enhanced sedimentation for deeper flows. However, densely vegetated swales have higher hydraulic roughness and therefore require a larger area and/ or more frequent use of swale field inlet pits to convey flows compared to grass swales. Densely vegetated bioretention swales can become features of an urban landscape and once established, require minimal maintenance and are hardy enough to withstand large flows. Appendix A (Plant Selection for WSUD Systems) provides more specific guidance on the selection of appropriate vegetation for bioretention swales.

3.2.5 Bioretention Filter Media

Selection of an appropriate bioretention filter media is a key design step involving consideration of three inter-related factors:

- Saturated hydraulic conductivity required to optimise the treatment performance of the bioretention component given site constraints on available filter media area.
- Depth of extended detention provided above the filter media.
- Suitability as a growing media to support vegetation growth (i.e. retaining sufficient soil moisture and organic content).

The high rainfall intensities experienced in South East Queensland (SEQ) relative to the southern capital cities means bioretention treatment areas tend to be larger in SEQ in order to achieve the same level of stormwater treatment. The area available for bioretention swales in an urban layout is often constrained by factors such as the available area within the footpaths of standard road reserves. Selecting bioretention filter media for bioretention swale applications in SEQ will often require careful consideration of saturated hydraulic conductivity and extended detention depth to ensure the desired minimum volume of stormwater runoff receives treatment. This must also be balanced with the requirement to also ensure the saturated hydraulic conductivity does not become too high such that it can no longer sustain healthy vegetation growth. The maximum saturated hydraulic conductivity should not exceed 500 mm/hr (and preferably be between 50 - 200 mm/hr) in order to sustain vegetation growth.

The concept design stage will have established the optimal combination of filter media saturated hydraulic conductivity and extended detention depth using a continuous simulation modelling approach (i.e. MUSIC). Any adjustment of either of these two design parameters during the detailed design stage will require the continuous simulation modelling to be re-run to assess the impact on the overall treatment performance of the bioretention basin.

As shown in Figure 3-2, a bioretention filter media can consist of three layers. In addition to the filter media required for stormwater treatment, a drainage layer is also required to convey treated water from the base of the filter media into perforated under-drains (if the design intent is to recover the treated stormwater). The drainage layer surrounds perforated under-drains and can be either coarse sand (1 mm) or fine gravel (2-5 mm). If fine gravel is used, it is advisable to install a transition layer between the filter media and the drainage layer comprising of a clean sand (1mm) to prevent migration of the base filter media into the drainage layer and into the perforated under-drains.

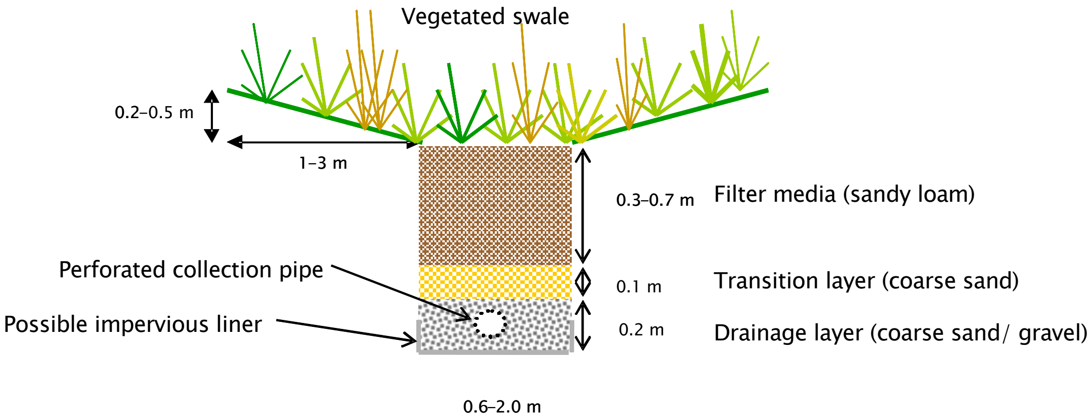


Figure 3-2: Typical Section of a Bioretention Swale

3.2.6 Traffic Controls

Another design consideration is keeping traffic and building material deliveries off swales, particularly during the building phase of a development. If bioretention swales are used for parking, then the surface will be compacted and vegetation damaged beyond its ability to regenerate naturally. Compacting the surface of a bioretention swale will reduce the infiltration into the filter media and lead to early bypass and reduced treatment. Vehicles driving on swales can cause ruts that can create preferential flow paths that diminish the water quality treatment performance as well as creating depressions that can retain water and potentially become mosquito breeding sites.

A staged construction and establishment method (Section 3.5) affords protection to the sub-surface elements of a bioretention swale from heavily sediment ladened runoff during the subdivision construction and allotment building phases. However, to prevent vehicles driving on bioretention swales and inadvertent placement of building materials, it is necessary to consider appropriate traffic control solutions as part of the system design. These can include temporary fencing of the swale during the subdivision construction and allotment building phases with signage erected to alert builders and constractors of the purpose and function of the swales. Management of traffic onto the swales after completion of the allotment building phase can be achieved in a number of ways such as planting the interface to the road carriageway with dense vegetation that will discourage the movement of vehicles onto the swale or, if dense vegetation cannot be used, by providing physical barriers such as kerb and channel (with breaks to allow distributed water entry to the swale) or bollards and/ or street tree planting.

Kerb and channel should be used at all corners, intersections, cul-de-sac heads and at traffic calming devices to ensure correct driving path is taken. For all of these applications, the kerb and channel is to extend 5 m beyond tangent points. The transition from barrier or lay back type kerb to flush kerbs and vice versa is to be done in a way that avoids creation of low points that cause ponding onto the road pavement.

Where bollards/road edge guide posts are used, consideration should be given to intermixing mature tree plantings with the bollards to break the visual monotony created by a continuous row of bollards. Bollards and any landscaping (soft or hard) must comply with the relevant local authority guidelines.

3.2.7 Roof Water Discharge

Roof runoff can contain a range of stormwater pollutants including nitrogen washed from the atmosphere during rainfall events. Rainfall is consistently the major source of nitrogen in urban stormwater runoff (Duncan 1995) and inorganic nitrogen concentrations in rainfall often exceed the threshold level for algal blooms (Weibel et al. 1966). Roof water should be discharged onto the surface of the swale for subsequent conveyance and treatment by the swale (and downstream treatment measures) before being discharged to receiving aquatic environments. Depending on the depth of the roof water drainage system and the finished levels of the bioretention swale, this may require the use of a small surcharge pit located within the invert of the swale to allow the roof water to surcharge to the swale. Any residual water left in the surcharge pit can be discharged to the underlying subsoil drainage by providing perforations in the base and sides of the surcharge pit. If a surcharge pit is used then an inspection chamber along the roof water drainage line is to be provided within the property boundary. Surcharge pits are discussed further in Section 3.3.4.3.

Roof water should only be directly connected to an underground pipe drainage system if an appropriate level of stormwater treatment is provided along (or at the outfall of) the pipe drainage system.

3.2.8 Services

Bioretention swales located within footpaths (i.e. road verges) must consider the standard location for services within the verge and ensure access for maintenance of services. Typically it is acceptable to have water and sewer services located beneath the batters of the swale with any sewers located beneath bioretention swales to be fully welded polyethylene pipes with rodding points.

3.3 Bioretention Swale Design Process

To create bioretention swales, separate calculations are performed to design the swale and the bioretention system, with iterations to ensure appropriate criteria are met in each section. The calculations and decisions required to design the swale component are presented in detail in Chapter 2 (Swales) and are reproduced in this chapter. This is to allow designers and Council development assessment officers to consult with this chapter only for designing and checking bioretention swale designs. The key design steps are:

1. Confirm treatment performance of concept design

↓

2. Determine design flows for swale component

↓

3. Dimension the swale component with consideration to site contraints
a. Swale width and side slopes
b. Maximum swale length (i.e. length between overflow pits)

↓

4. Design inflow systems to swale and bioretention components

↓

5. Design bioretention component
a. Determine bioretention filter media saturated hydraulic conductivity and extended detention
b. Prescribe soil media characteristics (filter, transition and drainage layers)
c. Underdrain design and capacity check
d. Check requirement for impermeable lining

↓

6. Verify design
a. Scour velocity checks
b. Safety checks
c. Confirm treatment performance

↓

7. Size overflow pits (field inlet pits)

↓

8. Make allowances to preclude traffic

↓

9. Specify plant species and planting densities

↓

10. Consider maintenance requirements, including written maintenance plan

Each of these design steps is discussed below, followed by a worked example illustrating application of the design process on a case study site.

3.3.1 Step 1: Confirm Treatment Performance of Concept Design

Before commencing detailed design, the designer should first undertake a preliminary check to confirm the bioretention swale treatment area from the concept design is adequate to deliver the required level of stormwater quality improvement. This design process assumes a conceptual design has been undertaken. The treatment performance curves shown in Figure 3-3 to Figure 3-5 reflect the treatment performance of the bioretention component only and will be conservative as they preclude the sediment and nutrient removal performance of the overlying swale component. Notwithstanding this, the performance of the swale component for nitrogen removal is typically only minor and thus the sizing of the bioretention component will typically be driven by achieving compliance with best practice load reduction targets for Total Nitrogen. Therefore, by using the performance curves below the detailed designer can be confident that the combined performance of the swale and bioretention components of a bioretention swale will be similar to that shown in the curves for total Nitrogen and will exceed that shown for Total Suspended Sediment and total Phosphorus.

These curves are intended to provide an indication only of appropriate sizing and do not substitute the need for a thorough conceptual design process.

The curves in Figure 3-3 to Figure 3-5 were derived using the *Model for Urban Stormwater Improvement Conceptualisation* (MUSIC), assuming the bioretention trench is a stand alone system (i.e. not part of a treatment train). The curves show the total suspended solid (TSS), total phosphorus (TP) and total nitrogen (TN) removal performance for a typical bioretention basin design, being:

- Filter media saturated hydraulic conductivity (k) = 200mm/hr
- Filter Media average particle size = 0.5mm
- Filter Media Depth = 0.6m
- Extended Detention Depth = 0.2m

The curves in Figure 3-3 to Figure 3-5 are generally applicable to bioretention swale applications within residential, industrial and commercial land uses. Curves are provided for four rainfall station locations selected as being broadly representative of the spatial and temporal climatic variation across South East Queensland. The shaded area on each of the curves indicates where the bioretention swale performance meets the Best Practice Pollutant Load Reduction Targets for South East Queensland.

If the characteristics of the bioretention component of the bioretention swale concept design are significantly different to that described above, then the curves in Figure 3-3 to Figure 3-5 may not provide an accurate indication of treatment performance. In these cases, the detailed designer should use MUSIC to verify the performance of the bioretention swale. (if not already undertaken as part of concept design process).

The curves in Figure 3-3 to Figure 3-5 provide the detailed designer with a useful visual guide to illustrate the sensitivity of bioretention treatment performance to the ratio of bioretention treatment area and contributing catchment area. The curves allow the detailed designer to make a rapid assessment as to whether the bioretention trench component size falls within the "optimal size range" or if it is potentially under or over-sized. In particular, bioretention treatment area will typically need to be between 1% to 2% of the contributing catchment area to meet current best practice load reduction targets for TSS, TP and TN. Bioretention swales will likely be closer to 1% due to the pre-treatment function provided by the overlying swale component whilst bioretention basins (Chapter 5) will typically be closer to 2% except where flows entering the bioretention swales are afforded pre-treatment by means of swales or other measures such a sedimentation basin (Chapter 4).

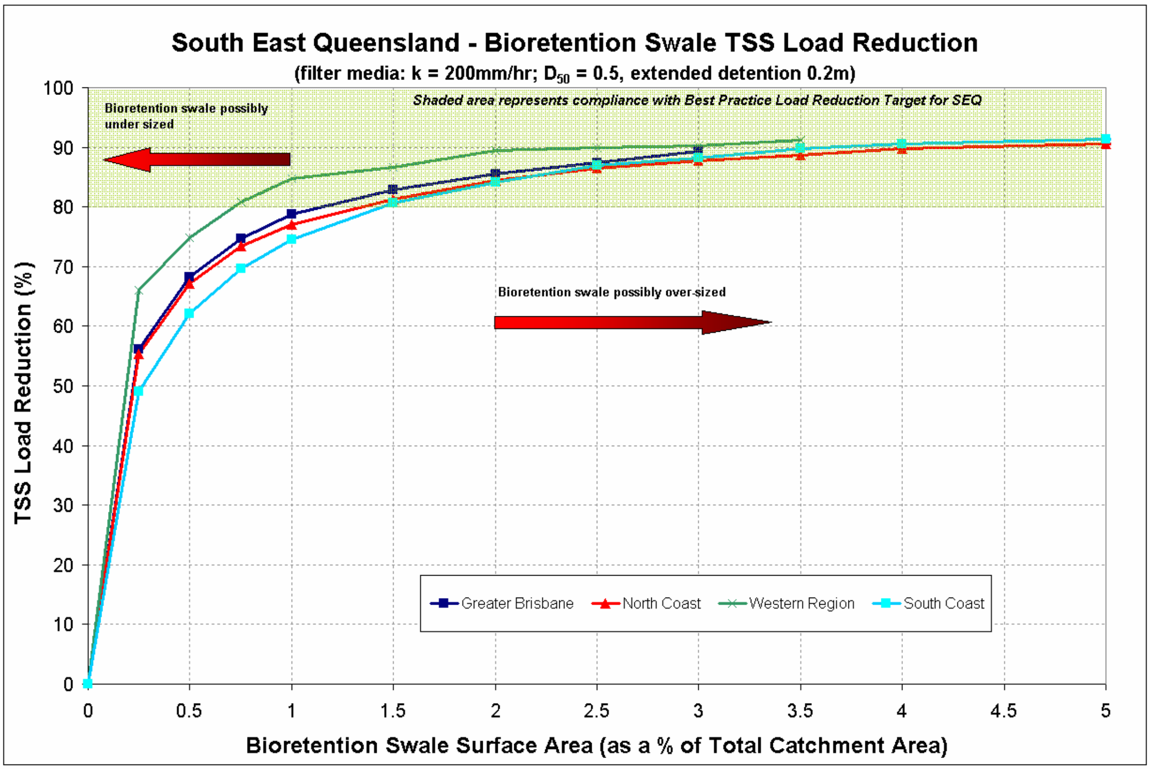


Figure 3-3: Bioretention Swale TSS Removal Performance (based on Bioretention Basin Performance)

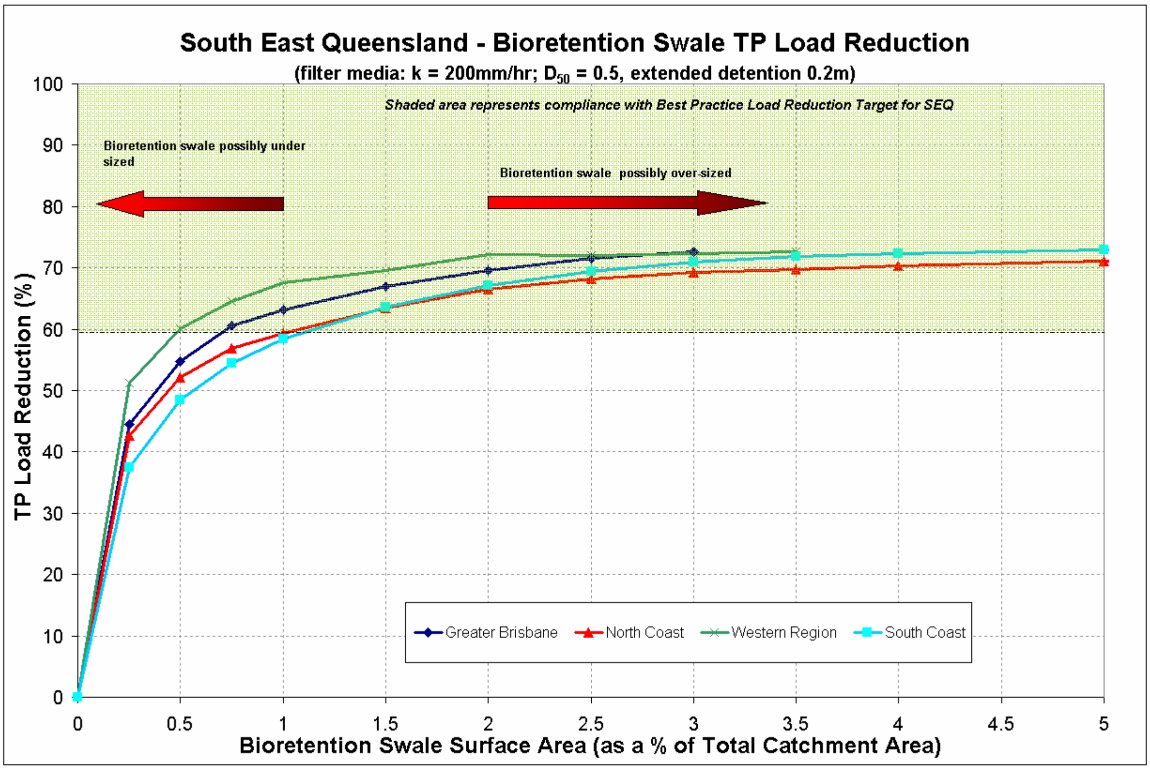


Figure 3-4: Bioretention Swale TP Removal Performance (based on Bioretention Basin Performance

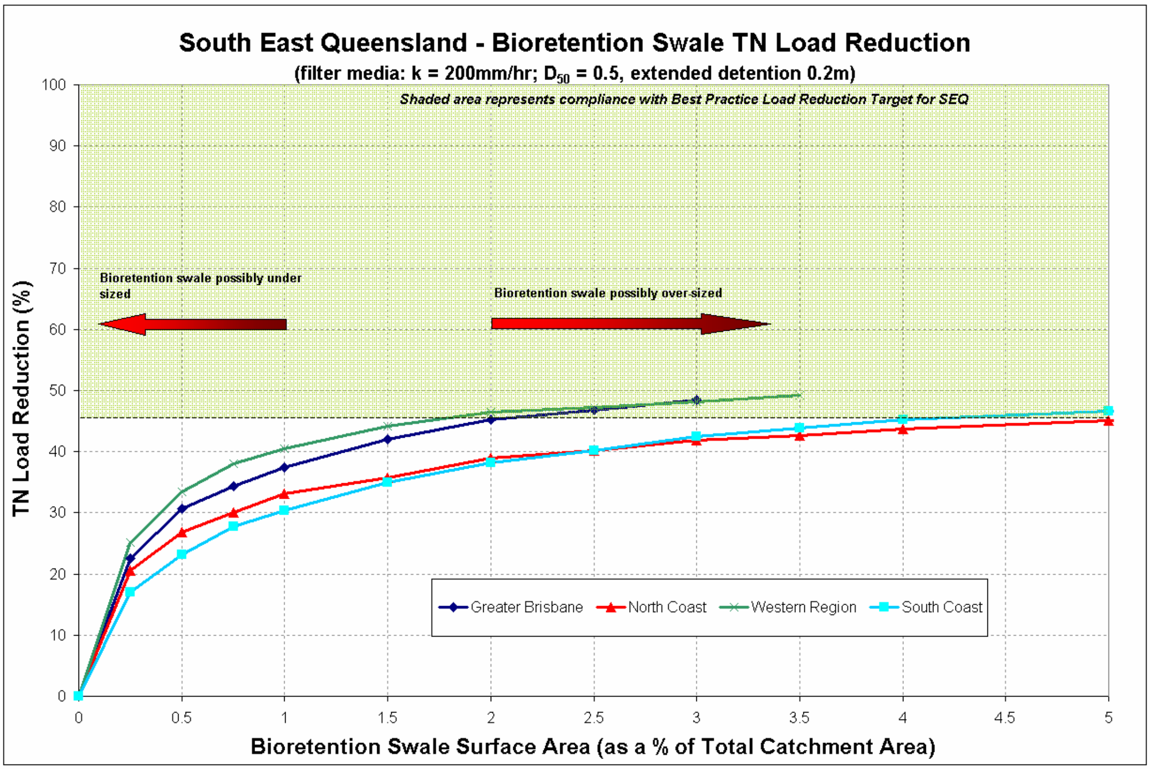


Figure 3-5: Bioretention Swale TN Removal Performance (based on Bioretention Basin Performance)

3.3.2 Step 2: Determine Design Flows for the Swale Component

3.3.2.1 Design Flows

Two design flows are required for the design of a swale:

- minor flood flow (2 to 10 year ARI), to allow the minor storm to be safely conveyed within the swale
- major flood flow (50 to 100 year ARI) to check flow velocities, velocity depth criteria, conveyance within road reserve, and freeboard to adjoining property.

3.3.2.2 Design Flow Estimation

A range of hydrologic methods can be applied to estimate design flows. As the typical catchment area should be relatively small (>2 ha) the Rational Method design procedure is considered to be a suitable method for estimating design peak flows.

3.3.3 Step 3: Dimension the Swale Component with Consideration to Site Constraints

Factors to consider are:

- allowable width given the proposed road reserve and/ or urban layout
- how flows are delivered into a swale (e.g. cover requirements for pipes or kerb details)
- vegetation height
- longitudinal slope
- maximum side slopes and base width
- provision of crossings (elevated or at grade)
- requirements of QUDM (DPI, IMEA & BCC, 1992).

Depending on which of the above factors are fixed, the other variables can be adjusted to derive the optimal swale dimensions for the given site conditions. The following sections outline some considerations in relation to dimensioning a swale.

3.3.3.1 Swale Width and Side Slopes

The maximum width of swale is usually determined from an urban layout and at the concept design stage. And should be undertaken in accordance with relevant local authority guidelines or standards. Brisbane City council's Standard Drawing UMS 151 presents examples of swale profiles that can be provided within typical residential road reserves and can be used as a reference for swale design in lieu of any local equivalent. Where the swale width is not constrained by an urban layout (e.g. when located within a large parkland area) then the width of the swale can be selected based on consideration of landscape objectives, maximum side slopes for ease of maintenance and public safety, hydraulic capacity required to convey the desired design flow, and treatment performance requirements. The maximum swale width needs to be identified early in the design process as it dictates the remaining steps in the swale design process. Selection of appropriate side slopes for swales in parks, easements or median strips is heavily dependant on site constraints, and swale side slopes are typically between 1 in 10 and 1 in 4.

For swales located adjacent to roads, the types of driveway crossing used will typically dictate batter slopes. Where there are no driveway crossings, the maximum swale side slopes will be established from ease of maintenance and public safety considerations. Generally 'at-grade' crossings, are preferred which require the swale to have 1:9 side slopes with a nominal 0.5 m flat base to provide sufficient transitions to allow for traffic movement across the crossing. Flatter swale side slopes can be adopted but this will reduce the depth of the swale and its conveyance capacity. Where 'elevated' crossings are used, swale side slopes would typically be between 1 in 6 and 1 in 4. 'Elevated' crossings will require provision for drainage under the crossings with a culvert or similar. The selection of crossing type should be made in consultation with urban and landscape designers.

3.3.3.2 Maximum Length of a Swale

The maximum length of a swale is the distance along a swale before an overflow pit (or field inlet pit) is required to drain the swale to an underlying pipe drainage system.

The maximum length of a swale located within parkland areas and easements is calculated as the distance along the swale to the point where the flow in the swale from the contributing catchment (for the specific design flood frequency) exceeds the bank full capacity of the swale. For example, if the swale is to convey the minor flood flow (2-10 year ARI) without overflowing, then the maximum swale length would be determined as the distance along the swale to the point where the 2-10 year ARI flow from the contributing catchment is equivalent to the bank full flow capacity of the swale (bank full flow capacity is determined using Manning's equation as discussed section 3.3.3.3).

The maximum length of a swale located along a roadway is calculated as the distance along the swale to the point where flow on the adjoining road pavement (or road reserve) no longer complies with the local standards for road drainage (for both the minor and major flood flows) or in lieu of any specific standrds then in compliance with the relevant design standards presented in QUDM.

3.3.3.3 Swale Capacity – Manning's Equation and Selection of Manning's *n*

To calculate the flow capacity of a swale, use Manning's equation. This allows the flow rate and flood levels to be determined for variations in swale dimensions, vegetation type and longitudinal grade.

$$Q = \frac{A \cdot R^{2/3} \cdot S^{1/2}}{n}$$ Equation 3.1

Where A = cross section area of swale (m^2)

R = hydraulic radius (m)

S = channel slope (m/m)

n = roughness factor (Manning's n)

Manning's *n* is a critical variable in Manning's equation relating to roughness of the channel. It varies with flow depth, channel dimensions and vegetation type. For constructed swale systems, 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 for flows with greater depth than the vegetation (e.g. 0.03 for flow depth more than twice the vegetation height). It is considered reasonable for Manning's *n* to have a maximum at the vegetation height and then to sharply reduce as depths increase.

Figure 3-6 shows a plot of Manning's *n* versus flow depth for a grass swale with longitudinal grade of 5 %. 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 3-6) 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.

Further discussion on selecting an appropriate Manning's *n* for a swale is provided in Appendix E of the *MUSIC User Guide* (CRCCH 2005).

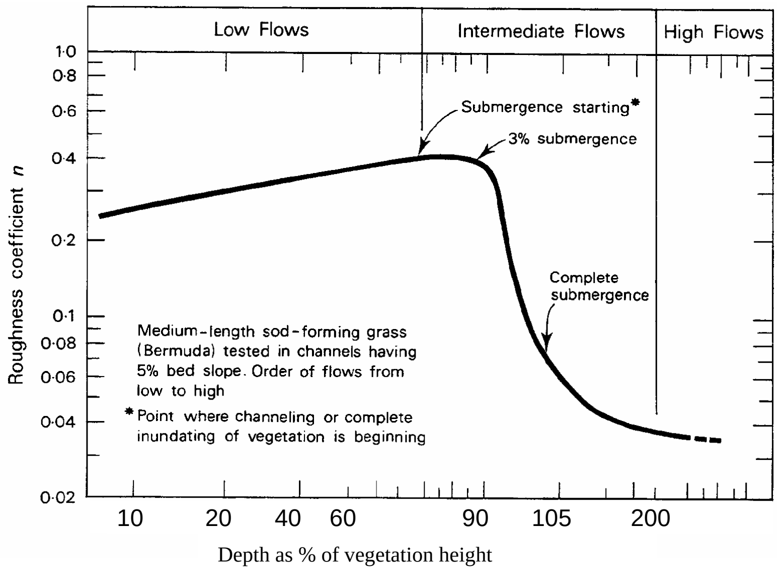


Figure 3-6: Impact of Flow Depth on Hydraulic Roughness (adapted from Barling and Moore (1993))

3.3.4 Step 4: Design Inflow Systems to Swale and Bioretention Components

Inflows to bioretention swales can be via distributed runoff (e.g. from flush kerbs on a road) or point outlets such as pipe outfalls. Combinations of these inflow pathways can also be used.

3.3.4.1 Distributed Inflow

An advantage of flows entering a bioretention swale system in a distributed manner (i.e. entering perpendicular to the direction of the swale) is that flow depths are kept as shallow sheet flow, which maximises contact with the swale and bioretention vegetation, particularly on the batter receiving the distributed inflows. This swale and bioretention batter is often referred to as a buffer (see Figure 3-7). The requirement of the buffer is to ensure there is dense vegetation growth, flow depths are shallow (below the vegetation height) and erosion is avoided. The buffer provides good pretreatment (i.e. significant coarse sediment removal) prior to flows being conveyed along the swale.

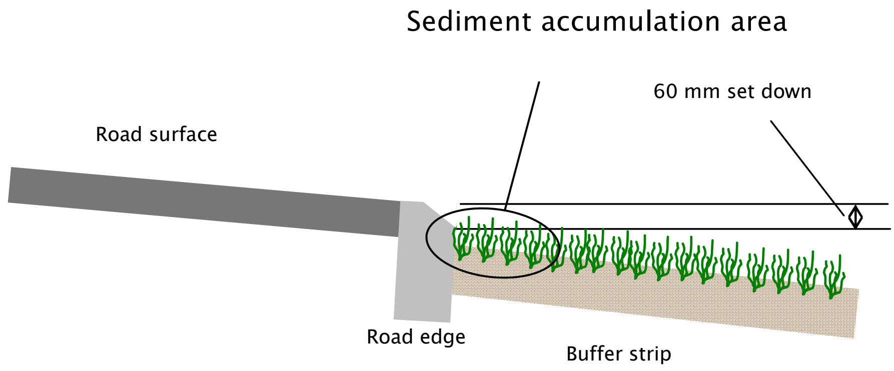


Figure 3-7: Flush Kerb with 60 mm Setdown to allow Sediment to Flow into Vegetated Area

Distributed inflows 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 (Plate 3-1).

Plate 3-1: Kerb Arrangements with Breaks to Distribute Inflows on to Bioretention Swales and Prevent Vehicle Access

No specific design rules exist for designing buffer systems, however there are several design guides that are to be applied to ensure buffers operate to improve water quality and provide a pre-treatment role. Key design parameters of buffer systems are:

- providing distributed flows into a buffer (potentially spreading stormwater flows to achieve this)
- avoiding rilling or channelised flows
- maintaining flow heights lower than vegetation heights (this may require flow spreaders, or check dams)
- minimising the slope of buffer, best if slopes can be kept below 5 %, however buffers can still perform well with slopes up to 20 % provided flows are well distributed. The steeper the buffer the more likely flow spreaders will be required to avoid rill erosion.

Maintenance of buffers is required to remove accumulated sediment and debris therefore access is important. Most sediments will accumulate immediately downstream of the pavement surface and then progressively further downstream as sediment builds up.

It is important to ensure coarse sediments accumulate off the road surface at the start of the buffer. Plate 3-2 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 must be used that sets the top of the vegetation 60 mm (refer Figure 3.7), which requires the top of the ground surface (before turf is placed) to be approximately 100 mm below the road surface. This allows sediments to accumulate off any trafficable surface.

Plate 3-2: Flush Kerb without Setdown, showing Sediment Accumulation on Road

3.3.4.2 Concentrated Inflow

Concentrated inflows to a bioretention swale can be in the form of a concentrated overland flow or a discharge from a piped drainage system (e.g. allotment drainage line). For all concentrated inflows, energy dissipation at the inflow location is an important consideration to minimise any erosion potential. This can usually be achieved with rock benching and/ or dense vegetation.

The most common constraint on pipe systems discharging to bioretention swales is bringing the pipe flows to the surface of a swale. In situations where the swale geometry does not allow the pipe to achieve 'free' discharge to the surface of the swale, a 'surcharge' pit may need to be used. 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 under-drains 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. It is noted that surcharge pits are generally not considered good practice (due to additional maintenance issues and mosquito breeding potential) and should therefore be avoided where possible.

Surcharge pit systems are most frequently used when allotment runoff is required to cross a road into a swale on the opposite side of the road or for allotment runoff discharging into shallow profile swales. Where allotment runoff needs to cross under a road to discharge to a swale, it is preferable to combine the runoff from more than one allotment to reduce the number of crossings required under the road pavement. Figure 3-8 illustrates a typical surcharge pit discharging into a swale.

Another important form of concentrated inflow in a bioretention swale is the discharge from the swale component into the bioretention component, particularly where the bioretention component is located at the downstream end of the overlying swale and receives flows concentrated within the swale. Depending on the grade, its top width and batter slopes, the resultant flow velocities at the transition from the swale to the bioretention filter media may require the use of energy dissipation to prevent scour of the filter media. For most cases, this can be achieved by placing several large rocks in the flow path to reduce velocities and spread flows. Energy dissipaters located within footpaths must be designed to ensure pedestrian safety.

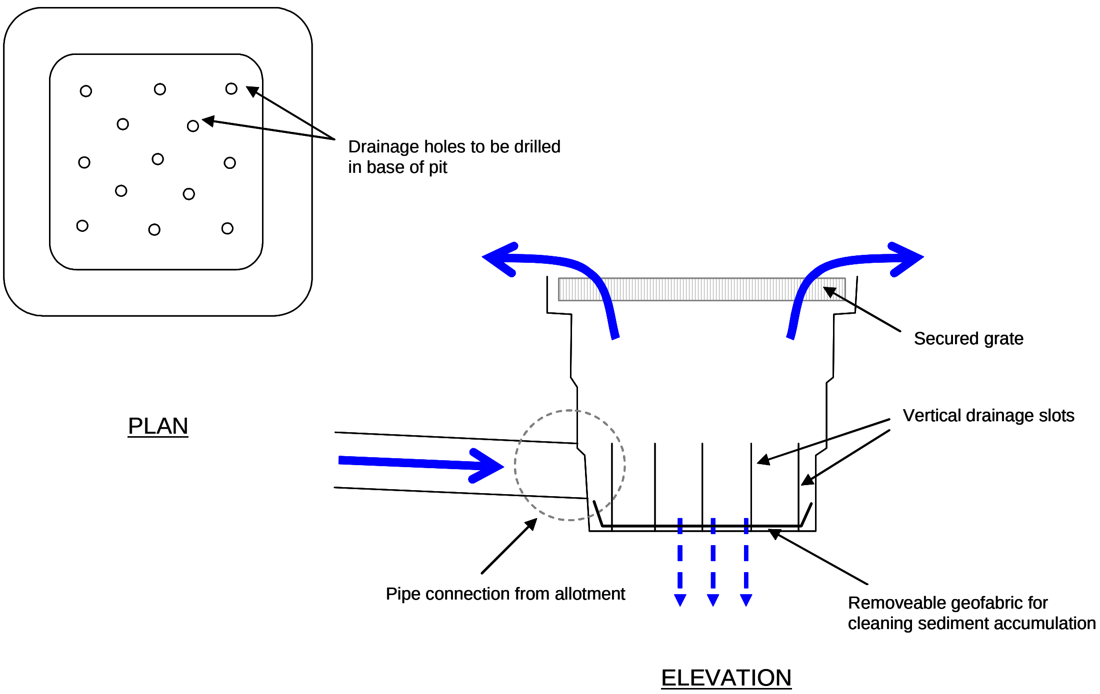


Figure 3-8: Example of Surcharge Pit for Discharging Allotment Runoff into a Swale

3.3.5 Step 5: Design Bioretention Component

3.3.5.1 Select Filter Media Saturated Hydraulic Conductivity and Extended Detention

Where design Steps 2 and 3 (Section 3.3.2 and 3.3.3) reveal that the swale geometry derived during the concept design stage does not comply with the relevant local road drainage design standards or the standards established in QUDM for minor flood and major flood flows on adjoining road pavements and minimum freeboard requirements to adjoining properties, it is necessary to revise the swale geometry. As such, an alternative dimension for the surface area of the bioretention component may result and this may require further MUSIC modelling to determine the 'new' optimal combination of filter media saturated hydraulic conductivity and extended detention depth to maximise the water quality treatment function of the bioretention component.

3.3.5.2 Specify the Bioretention Filter Media Characteristics

At least two (and possibly three) types of media are required in the bioretention component of bioretention swales (refer Figure 3-2 in Section 3.2.5).

Filter Media

The filter media layer provides the majority of the pollutant treatment function, through fine filtration and also by supporting vegetation. The vegetation enhances filtration, keeps the filter media porous, provides substrate for biofilm formation and provides some uptake of nutrients and other stormwater pollutants. As a minimum, the filter media is required to have sufficient depth to support vegetation. Typical depths are between 600-1000 mm with a minimum depth of 300mm accepted in depth constrained situations. It is important to note that if deep rooted plants such as trees are to be planted in bioretention swales, the filter media must have a minimum depth of 800 mm to avoid roots interfering with the perforated under-drain system.

The saturated hydraulic conductivity of the filter media is established by optimising the treatment performance of the bioretention system given site constraints of the particular site (using a continuous simulation model such as MUSIC). Saturated hydraulic conductivity should remain between 50-200 mm/hr (saturated hydraulic conductivity of greater than 500 mm/hr would not be accepted by most Councils). Once the saturated hydraulic conductivity of the filter media has been determined for a particular bioretention swale, the following process can then be applied to derive a suitable filter media soil to match the design saturated hydraulic conductivity:

- Identify available sources of a suitable base soil (i.e. topsoil) capable of supporting vegetation growth such as a sandy loam or sandy clay loam. In-situ topsoil should be considered first before importing a suitable base soil. Any base soil found to contain high levels of salt (see last bullet point), extremely low levels of organic carbon (< 5%), or other extremes considered retardant to plant growth and denitrification should be rejected. The base soil must also be structurally sound and not prone to structural collapse as this can result in a significant reduction in saturated hydraulic conductivity. The risk of structural collapse can be reduced by ensuring the soil has a well graded particle size distribution with a combined clay and silt fraction of < 12%.
- Using laboratory analysis, determine the saturated hydraulic conductivity of the base soil using standard testing procedures (AS 4419-2003 Appendix H Soil Permeability). A minimum of five samples of the base soil should be tested. Any occurrence of structural collapse during laboratory testing must be noted and an alternative base soil sourced.
- To amend the base soil to achieve the desired design saturated hydraulic conductivity either mix in a loose non-angular sand (to increase saturated hydraulic conductivity) or conversely 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. The sand or clay content (by weight) that achieves the desired saturated hydraulic conductivity should then be adopted on-site. A minimum of five samples of the selected base soil and sand (or clay) content mix must be tested in the laboratory to ensure saturated hydraulic conductivity is consistent across all samples. If the average saturated hydraulic

conductivity of the final filter media mix is within ± 20% of the design saturated hydraulic conductivity then the filter media can be adopted and installed in the bioretention system. Otherwise, further amendment of the filter media must occur through the addition of sand (or clay) and retested until the design saturated hydraulic conductivity is achieved.

- The base soil must have sufficient organic content to establish vegetation on the surface of the bioretention system. If the proportion of base soil in the final mix is less than 50%, it may be necessary to add organic material. This should not result in more than 10% organic content (measured in accordance with AS 1289.4.1.1-1997) and should not alter the saturated hydraulic conductivity of the final filter media mix.
- The pH of the final filter media is to be amended (if required) to between 6 and 7. If the filter media mix is being prepared off-site, this amendment should be undertaken before delivery to the site.
- The salt content of the final filter media (as measured by EC1:5) must be less than 0.63 dS/m for low clay content soils like sandy loam. (EC1:5 is the electrical conductivity of a 1:5 soil/ water suspension).

Imported soils must not contain Fire Ants. Visual assessment is required and any machinery should be free of clumped dirt. Soils must not be brought in from Fire Ant restricted areas. For further information on Fire Ant restrictions, contact the Department of Primary Industries and Fisheries.

Drainage Layer (if required)

The drainage layer is used to convey treated flows from the base of the filter media layer into the perforated under-drainage system. The composition of the drainage layer is to be considered in conjunction with the selection and design of the perforated under-drainage system (refer to Section 3.3.5.6) as the slot sizes in the perforated pipes may determine the minimum drainage layer particle size to avoid washout of the drainage layer into the perforated pipe system. Coarser material (e.g. fine gravel) is to be used for the drainage layer if the slot sizes in the perforated pipes are too large for use of a sand based drainage layer. Otherwise, sand is the preferred drainage layer media as it is likely to avoid having to provide a transition layer between the filter media and the drainage layer. The drainage layer is to be a minimum of 200 mm thick.

Ensure drainage media is washed prior to placement in bioretention system to remove any fines. Drainage media must also be free from Fire Ants and visually checked to confirm this. Drainage media must not be imported from a Fire Ant restricted area.

Transition Layer (if required)

The particle size difference between the filter media and the underlying drainage layer should be not more than one order of magnitude to avoid the filter media being washed through the voids of the drainage layer. Therefore, if fine gravels are used for the drainage layer (which will be at least two orders of magnitude coarser than the likely average particle size of the filter media), then a transition layer is recommended to prevent the filter media from washing into the perforated pipes. If a transition layer is required then the material must be sand/ coarse sand material. An example particle size distribution (% passing) is provided below (typical specification only):

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

The transition layer is recommended to be 100 mm thick.

The addition of a transition layer increases the overall depth of the bioretention system and may be an important consideration for some sites where total depth of the bioretention system may be constrained. In such cases, the use of a sand drainage layer and/ or perforated pipes with smaller slot sized may need to be considered (Section 3.3.5.6).

3.3.5.3 Under-drain Design and Capacity Checks

The maximum spacing of the perforated pipes in wide bioretention trenches is 1.5 m (centre to centre) so that the distance water needs to travel (horizontally) through the drainage layer does not hinder drainage of the filtration media.

By installing parallel pipes, the capacity of the perforated pipe under-drain system can be increased. The recommended maximum size for the perforated pipes 100 mm to minimise the required 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 is also a consideration when specifying the drainage layer media.

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

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

The maximum infiltration rate represents the maximum rate of flow through the bioretention filter media and is calculated by applying Darcy's equation (Equation 3.2) as follows:

$$Q_{max} = K_{sat} \cdot L \cdot W_{base} \cdot \frac{h_{max} + d}{d}$$ Equation 3.2

Where
Q_{max} = maximum infiltration rate (m^3/s)

K_{sat} = hydraulic conductivity of the soil filter (m/s)

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

L = length of the bioretention zone (m)

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

d = depth of filter media (m)

The capacity of the perforated under-drains need to be greater than the maximum infiltration rate to ensure the filter media drains freely and the pipe(s) do not become the hydraulic 'control' in the bioretention system (i.e. to ensure the filter media sets the travel time for flows percolating through the bioretention system rather than the perforated under-drainage system).

To ensure the perforated under-drainage system has sufficient capacity to collect and convey the maximum infiltration rate, it is necessary to determine the capacity for flows to enter the under-drainage system via perforations in the pipes. To do this, orifice flow can be assumed and the 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 the maximum driving head (being the depth of the filtration media if no extended detention is provided or if extended detention is provided in the design then to the top of extended detention). It is conservative but reasonable to use a blockage factor to account for partial blockage of the perforations by the drainage layer media. A 50 % blockage of the perforation is recommended.

$$Q_{perf} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h}$$ Equation 3.3

Where
Q_{perf} = flow through perforations (m^3/s)

B = blockage factor (0.5)

C_d = orifice discharge coefficient (assume 0.61 for sharp edge orifice)

A = total area of the orifice (m^2)

g = gravity (9.79 m/s^2)

h = head above the perforated pipe (m)

If the capacity of the drainage system is unable to collect the maximum infiltration rate then additional under-drains will be required.

After confirming the capacity of the under-drainage system to collect the maximum infiltration rate is it then necessary to confirm the conveyance capacity of the underdrainage system is sufficient to convey the collected runoff. To do this, Manning's equation (Equation 3.1) can be used (which assumes pipe full flow (in place of channel flow) but not under pressure). The Manning's roughness used will be dependent on the type of pipe used.

The under-drains should be extended vertically to the surface of the bioretention system to allow inspection and maintenance when required. The vertical section of the under-drain should be unperforated and capped to avoid short circuiting of flows directly to the drain.

3.3.5.4 Check Requirement for Impermeable Lining

The saturated hydraulic conductivity of the natural soil profile surrounding the bioretention system should be tested together with depth to groundwater, chemical composition and proximity to structures and other infrastructure. This is to establish if an impermeable liner is required at the base (only for systems designed to preclude ex-filtration to in-situ soils) and/or sides of the bioretention basin (refer also to discussion in Section 3.2.3). If the saturated hydraulic conductivity of the filter media in the bioretention system is more than one order of magnitude (10 times) greater than that of the surrounding in-situ soil profile, no impermeable lining is required.

3.3.6 Step 6: Verify Design

3.3.6.1 Vegetation Scour Velocity Check

Potential scour velocities are checked by applying Manning's equation (Equation 3.1) to the bioretention swale design to ensure the following criteria are met:

- less than 0.5 m/s for minor flood (2-10 year ARI) discharge
- less than 2.0 m/s for major flood (50-100 year ARI) discharge.

3.3.6.2 Velocity and Depth Check – Safety

As bioretention swales are generally accessible by the public, it is important to check that depth x velocity within the bioretention swale, at any crossings and adjacent pedestrian and bicycle pathways, satisfies the following public safety criteria:

- depth x velocity < 0.6.m^2/s for low risk locations and 0.4 m^2/s for high risk locations as defined in QUDM
- maximum depth of flow over crossing = 0.3 m.

3.3.6.3 Confirm Treatment Performance

If the previous two checks are satisfactory then the bioretention swale design is satisfactory from a conveyance function perspective and it is now necessary to confirm the treatment performance of the bioretention swale by reference to the performance information presented in Section 3.3.1.

3.3.7 Step 7: Size Overflow Pit (Field Inlet Pits)

In a bioretention swale system, an overflow pit can be provided flush with the invert of the swale and/ or bioretention system filter media (i.e. when no extended detention is provided in the design) or it can be provided with the pit crest raised above the level of the bioretention filter media to establish the design extended detention depth (if extended detention is provided for in the design).

Grated pits are typically used and the allowable head for discharges into the pits is the difference in level between the pit crest and the maximum permissible water level to satisfy the local council's minimum freeboard requirements. Depending on the location of the bioretention swale, the design flow to be used to size the overflow pit could be the maximum capacity of the swale, the minor flood flow (2-10 year ARI) or the major flood flow (50-100 year ARI). There should be a minimum of 100 mm head over the overflow pit crest to facilitate discharge of the design flow into the overflow pit.

To size an overflow pit, two checks should be made to test for either drowned or free flowing conditions. A broad crested weir equation can be used to determine the length of weir required (assuming free overflowing conditions) and an orifice equation used to estimate the area between openings required in the grate cover (assuming drowned outlet conditions). The larger of the two pit configurations should be adopted (as per Section 5.10 QUDM). In addition, a blockage factor is to be used, that assumes the grate is 50 % blocked.

For free overfall conditions (weir equation):

$$Q_{weir} = B \cdot C_w \cdot L \cdot h^{3/2}$$ Equation 3.4

Where
Q_{weir} = Flow into pit (weir) under free overfall conditions (m^3/s)
B = Blockage factor (= 0.5)
C_w = Weir coefficient (= 1.66)
L = Length of weir (perimeter of pit) (m)
h = Flow depth above the weir (pit) (m)

Once the length of weir is calculated, a standard sized pit can be selected with a perimeter at least the same length of the required weir length.

For drowned outlet conditions (orifice equation):

$$Q_{orifice} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h}$$ Equation 3.5

Where
B, *g* and *h* have the same meaning as in Equation 3.4
$Q_{orifice}$ = flow rate into pit under drowned conditions (m^3/s)
C_d = discharge coefficient (drowned conditions = 0.6)
A = area of orifice (perforations in inlet grate) (m^2)

When designing grated field inlet pits, reference is also to be made to the procedure described in QUDM Section 5.10.4 (DPI, IMEA & BCC 1992). Refer to relevant local authority guidelines or standards for grate types for inlet pits. In the absence of local guidelines designers can refer to Brisbane City Council's Standard Drawings UMS 157 and UMS 337 which provide examples of grate types for overflow pits located in bioretention systems.

3.3.8 Step 8: Make Allowances to Preclude Traffic on Swales

Refer to Section 3.2.6 for discussion on traffic control options.

3.3.9 Step 9: Specify Plant Species and Planting Densities

Refer to Sections 3.4 and Appendix A for advice on selecting suitable plant species for bioretention swales in South East Queensland. Consultation with landscape architects is recommended when selecting vegetation to ensure the treatment system compliments the landscape design of the area.

3.3.10 Step 10: Consider Maintenance Requirements

Consider how maintenance is to be performed on the bioretention swale (e.g. how and where is access available, where is litter likely to collect etc.). A specific maintenance plan and schedule should be developed for the bioretention swale in accordance with Section 3.6.

3.3.11 Design Calculation Summary

The following design calculation table can be used to summarise the design data and calculation results from the design process.

BIORETENTION SWALES DESIGN CALCULATION SUMMARY

	Calculation Task	CALCULATION SUMMARY Outcome		Check
	Catchment Characteristics			
	Catchment Area		ha	
	Catchment Land Use (i.e. residential, Commercial etc.)			
	Conceptual Design			
	Bioretention area		m^2	
	Filter media saturated hydraulic conductivity		mm/hr	
	Extended detention depth		mm	
1	**Confirm Treatment Performance of Concept Design**			
	Bioretention area to achieve water quality objectives		m^2	
	TSS Removal		%	
	TP Removal		%	
	TN Removal		%	
2	**Estimate Design Flows for Swale Compnent**			
	Time of concentration – QUDM or relevant local government guideline		minutes	
	Identify Rainfall intensities			
	$I_{2\text{-}10 \text{ year ARI}}$		mm/hr	
	$I_{50\text{-}100 \text{ year ARI}}$		mm/hr	
	Design Runoff Coefficient			
	$C_{2\text{-}10 \text{ year ARI}}$			
	$C_{50\text{-}100 \text{ year ARI}}$			
	Peak Design Flows			
	2-10 year ARI		m^3/s	
	50-100 year ARI		m^3/s	
3	**Dimension the Swale Component**			
	Swale Width and Side Slopes			
	Base Width		m	
	Side Slopes – 1 in			
	Longitudinal Slope		%	
	Vegetation Height		mm	
	Maximum Length of Swale			
	Manning's *n*			
	Swale Capacity			
	Maximum Length of Swale			
4	**Design Inflow Systems to Swale & Bioretention Components**			
	Swale Kerb Type			
	Adequate Erosion and Scour Protection (where required)			
5	**Design Bioretention Component**			
	Filter media hydraulic conductivity		mm/hr	
	Extended detention depth		mm	
	Filter media depth		mm	
	Drainage layer media (sand or fine screenings)			
	Drainage layer depth		mm	
	Transition layer (sand) required			
	Transition layer depth		mm	
	Under-drain Design and Capacity Checks			
	Flow capacity of filter media (maximum infiltration rate)		m^3/s	
	Perforations inflow check			
	Pipe diameter		mm	
	Number of pipes			
	Capacity of perforations		m^3/s	
	CHECK PERFORATION CAPACITY > FILTER MEDIA CAPACITY			
	Perforated pipe capacity			
	Pipe capacity		m^3/s	
	CHECK PIPE CAPACITY > FILTER MEDIA CAPACITY			
	Check requirement for impermeable lining			
	Soil hydraulic conductivity		mm/hr	
	Filter media hydraulic conductivity		mm/hr	
	MORE THAN 10 TIMES HIGHER THAN IN-SITU SOILS?			
5	**Verification Checks**			
	Velocity for 2-10 year ARI flow (< 0.5 m/s)		m/s	
	Velocity for 50-100 year ARI flow (< 2 m/s)		m/s	
	Velocity x Depth for 50-100 year ARI (< 0.4 m^2/s)		m^2/s	
	Treatment Performance consistent with Step 1			
6	**Overflow Pit Design**			
	System to convey minor floods		L x W	

3.3.12 Typical Design Parameters

Table 3-1 shows typical values for a number of key bioretention swale design parameters.

Table 3-1: Typical Design Parameters for Bioretention Swales

Design Parameter	Typical Values
Swale longitudinal slope	1% to 4 %
Swale side slope for trafficability (with 'at grade' crossover)	Maximum 1 in 9
Swale side slope (with elevated driveway crossover)	1 in 4 to 1 in 10
Manning's *n* (with flow depth lower than vegetation height)	0.15 to 0.3
Manning's *n* (with flow depth greater than vegetation height)	0.03 to 0.05
Maximum velocity for scour in minor event (e.g. 2-10 yr ARI)	0.5 m/s
Maximum velocity for 50-100 yr ARI	2.0 m/s
Perforated pipe size	100 mm (maximum)
Drainage layer average material diameter (typically fine gravel or coarse sand)	1-5 mm diameter
Transition layer average material diameter typically sand to coarse sand	0.7 – 1.0 mm diameter

3.4 Landscape Design Notes

Bioretention swales are a combined solution that involves integrating a swale (Chapter 2) with the filtration function of a bioretention basin/trench (Chapter 5). These can involve an extended detention treatment and some biological uptake through the planted bioretention component. The landscaping for both the swale and bioretention parts are essentially similar to the treatments for the stand alone components however consideration of the interface landscape between the vegetated swale and bioretention is important.

3.4.1 Objectives

Landscape design for bioretention swales has four key objectives:

- Ensure surface treatments and planting designs address stormwater quality objectives by incorporating appropriate plant species for stormwater treatment (biologically active root zone) whilst enhancing the overall natural landscape.
- Integrated planning and design of bioretention swales within the built and landscape environments.
- Incorporating Crime Prevention Through Environmental Design (CPTED) principles and road, driveway and footpath visibility safety standards.
- Create landscape amenity opportunities that enhance community and environmental needs, such as visual aesthetics, shade, screening, view framing, and way finding.

3.4.2 Context and Site Analysis

When designing for bioretention swales as part of a WSUD strategy, the overall concept layout needs to consider:

- possible road profiles and cross-sections
- building and lot layout
- possible open space and recreational parks
- existing natural landforms.

Slope and soil type will also determine if swales are appropriate to the site and which swale type and swale location will be the most effective.

Careful site analysis and integrated design with engineers, landscape architects and urban designers will ensure the bioretention swales meet functional and aesthetic outcomes. A balanced approach to alignments between roads, footpaths and lot boundaries will be required early in the concept design of new developments to ensure swales are effective in both stormwater quality objectives and built environment arrangements. This is similar to concept planning for parks and open space where a balance is required between useable recreation space and WSUD requirements.

Comprehensive site analysis should inform the landscape design as well as road layouts, civil works and maintenance requirements. Existing site factors such as roads, driveways, buildings, landforms, soils, plants, microclimates, services and views should be considered. A useful reference at the time of writing these guidelines is *Water Sensitive Urban Design in the Sydney Region: 'Practice Note 2 – Site Planning'* (LHCCREMS 2002).

3.4.3 Streetscape Bioretention Systems

When using bioretention swales in road reserves it is important to understand how the swale landscape can be used to define the visual road space. Creative landscape treatments may be possible given that the bioretention swale element will typically be a minimum of 4 m in width. Design responses may range from informal 'natural' planting layouts to regimented avenues of trees along each external and internal edge of the bioretention swale element. Bioretention swales can be incorporated into a typical streetscape landscape using either a central splitter median or using one side of the road reserve.

Bioretention swale surface treatments are generally a vegetated swale that integrates into a densely planted bioretention component. The use of turf for the bioretention parts of the system is discouraged as mowing and public use of these areas will compact the upper filter media and limit the amount of filtration.

Vegetated bioretention swales can provide a relatively maintenance free finish if the planting and invert treatment are designed well. Key considerations when detailing are density and types of plantings, locations of trees and shrubs, type of garden (mowing) edges to turf areas that allows unimpeded movement of stormwater flow and overall alignment of swale invert within the streetscape.

3.4.3.1 Centre Median

Generally, the central median swale will provide a greater landscaped amenity, allowing planting and shade trees to enhance the streetscape more effectively, whilst verges remain constraint free. This swale configuration is however confined to roads requiring larger corridors for increased traffic. This can be seen in Figure 3-9 and Figure 3-10.

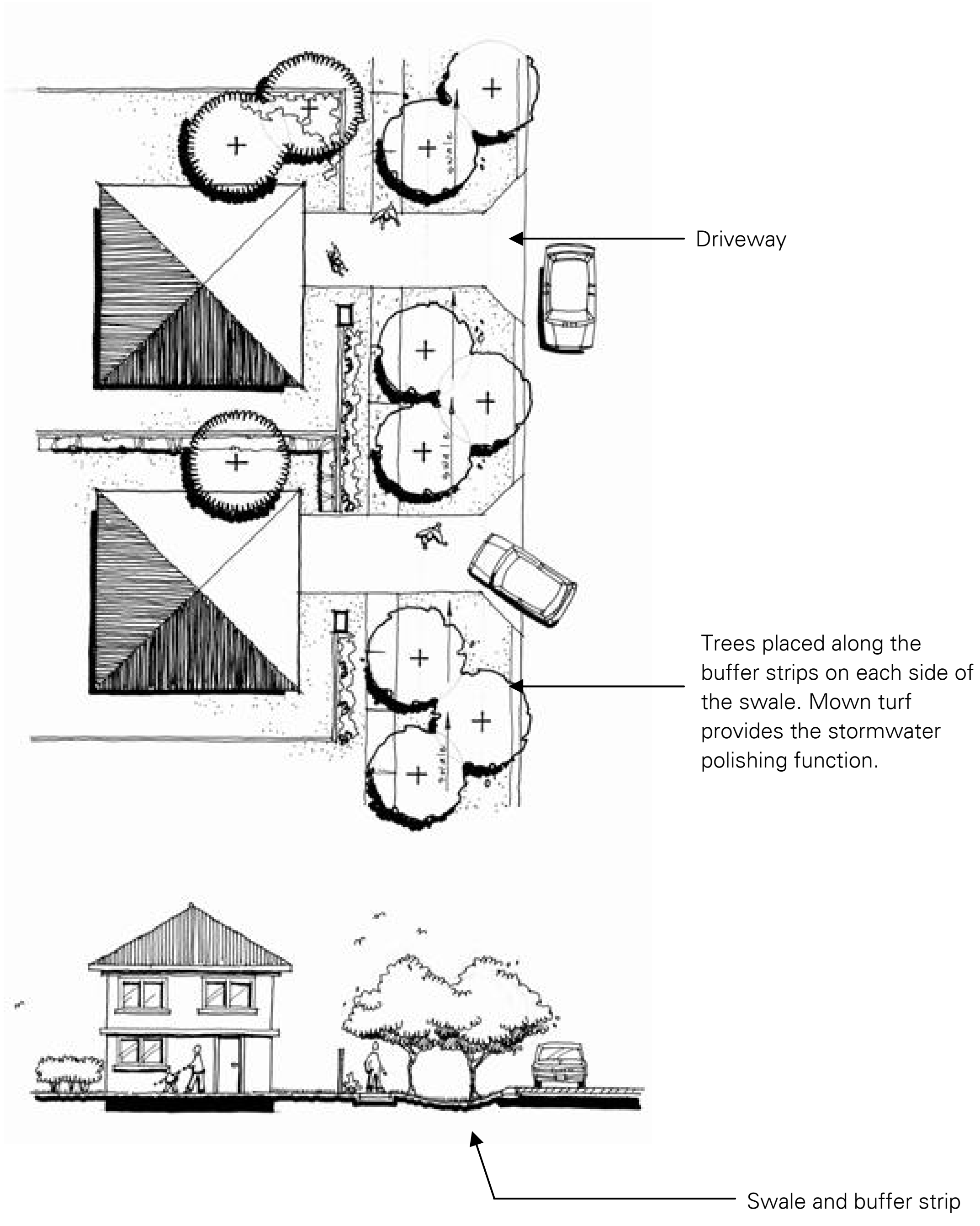


Figure 3-9:Possible Avenue Planting for Residential Swales

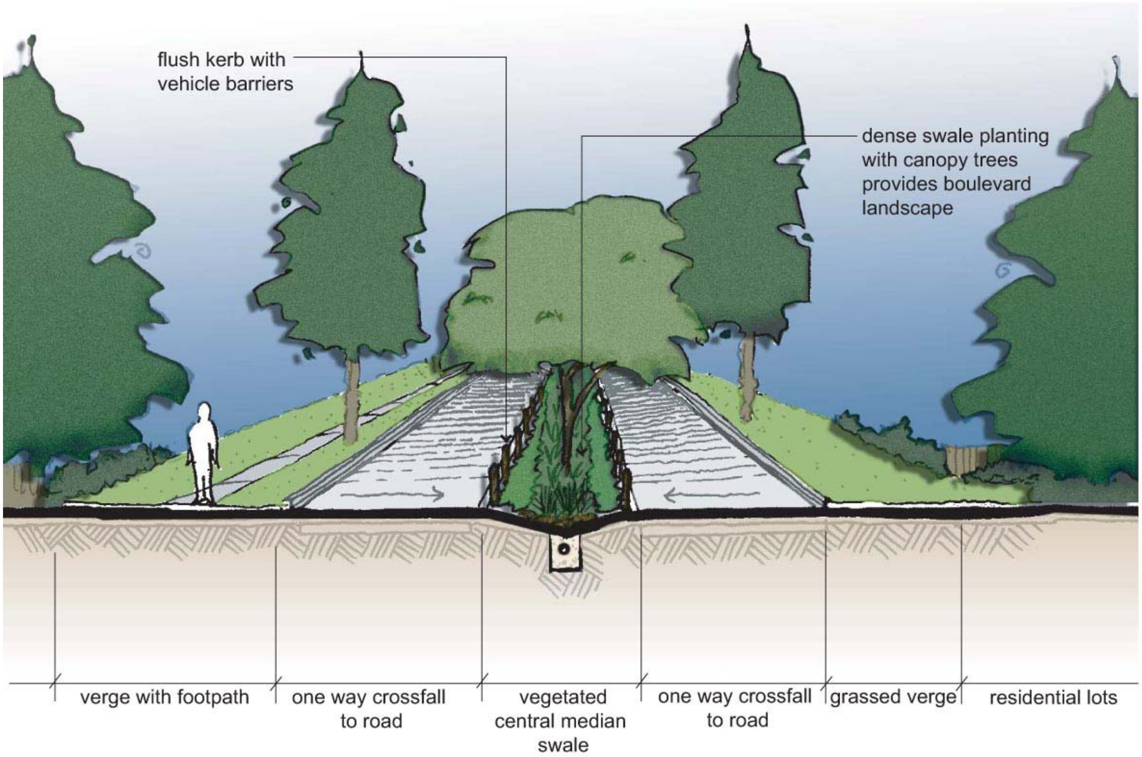


Figure 3-10: Landscape treatment of a vegetated swale in centre median

Plate 3-3: Median Strip Bioretention applications

3.4.3.2 Side of Road

In smaller minor roads, one side of the road can have a swale landscape to capture stormwater runoff from road pavements and house lots. To enhance the visual road space, creative landscape treatments to driveway cross-overs, general planting and invert treatments should be used. It is important in this swale arrangement that services and footpaths that are standard for road verges, have been planned and located to avoid clashes of function. This can be seen in Figure 3-11 and Figure 3-12.

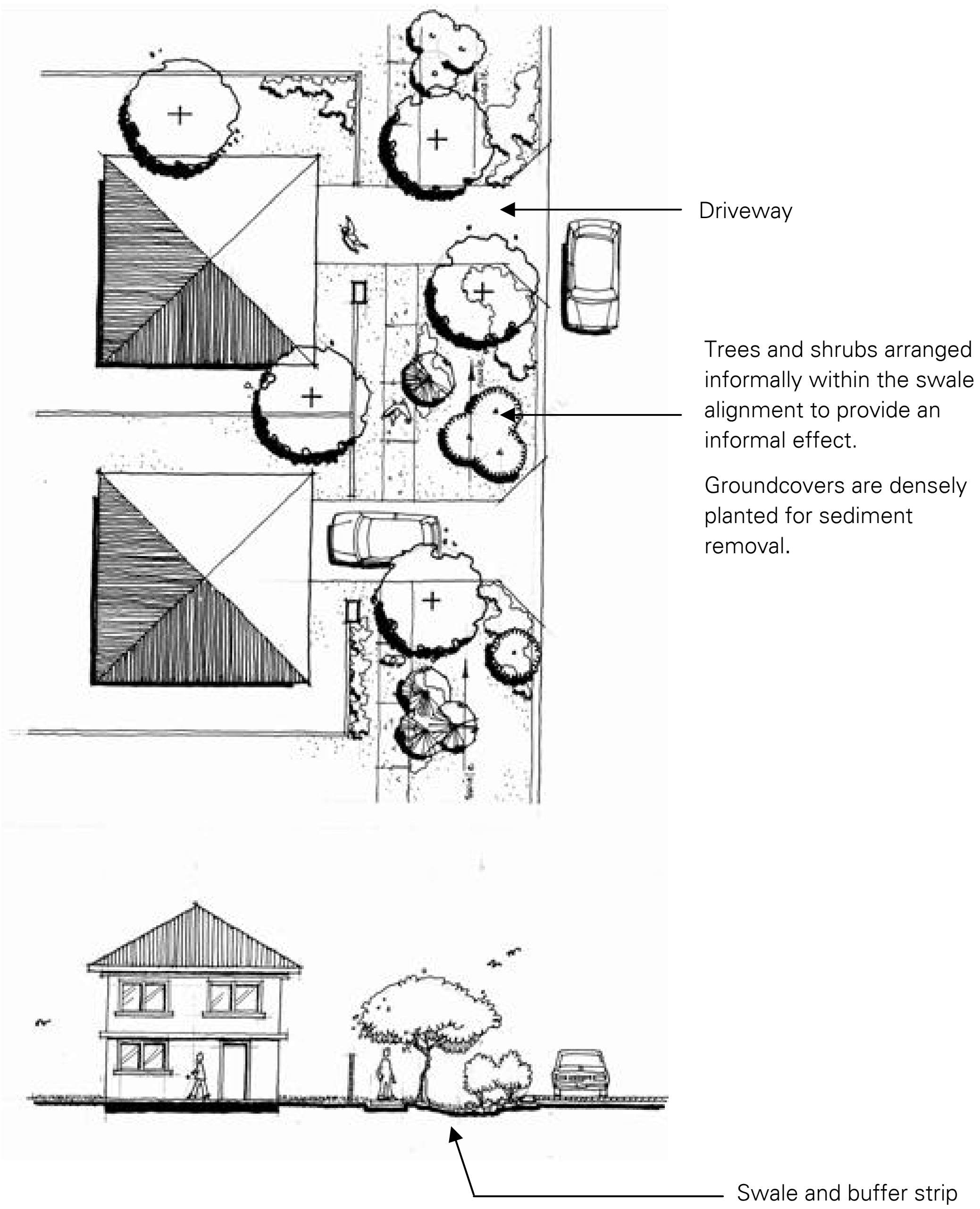
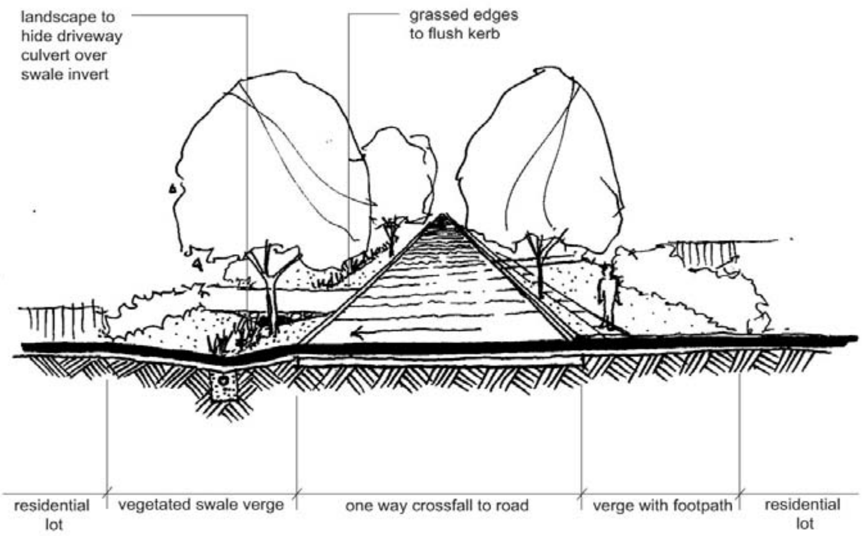


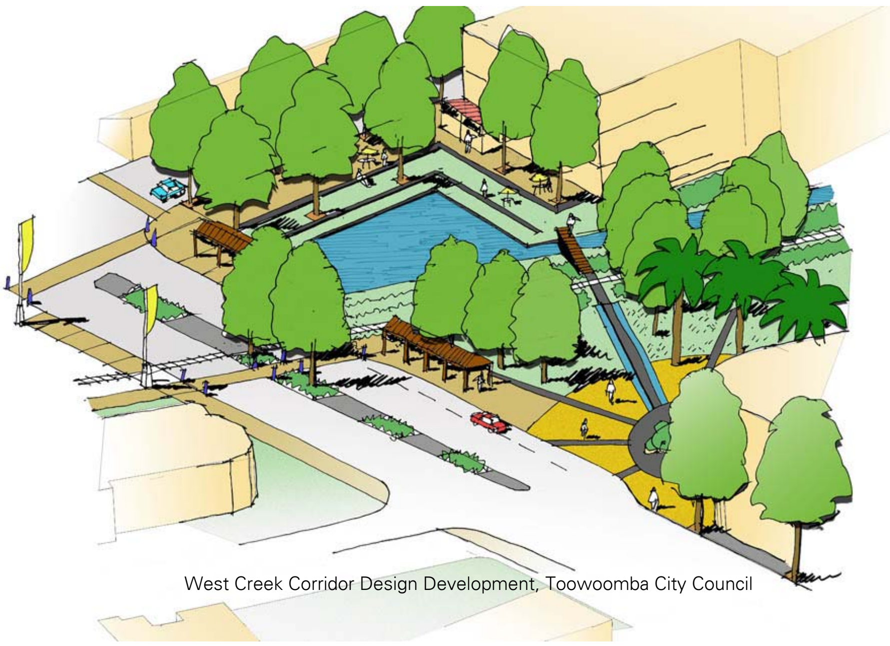
Figure 3-11: Possible 'Natural' Planting Layout for Residential Swales

Figure 3-12: Landscape treatment of vegetated swale on single side of road



3.4.4 Civic and Urban Spaces

With the increasing population growth in SEQ, gentrification of urban areas is required to create more robust spaces that meet current environmental and social needs. Often constrained by existing infrastructure, landscape treatments of swales can have a dual role of providing functional stormwater quality objectives whilst creating landscapes that enhance the communities' perception of water sensitive design.



By creating hard useable edges to swales and using complimentary planting strategies, civic spaces can provide an aesthetic landscape that meets recreational uses and promotes water sensitive design to the community. Refer to Figure 3-13 for an illustrative example.

Figure 3-13: Swale treatment in Civic Space

3.4.5 Open Space Bioretention Swales

Design and siting of parks/open space swales allows for greater flexibility in sectional profile, treatments and alignments. It is important however for careful landscape planning, to ensure that spaces for particular recreational uses are not encumbered by stormwater management devices including swales.

Bioretention strips can form convenient edges to pathway networks, frame recreational areas, create habitat adjacent to existing waterways/vegetation and provide landscape interest. Important issues to consider as part of the open space landscape design is maintenance access and CPTED principles which are further discussed in following sections.

3.4.6 Appropriate Plant Species

Planting for bioretention swale elements may consist of up to four vegetation types:

- groundcovers for stormwater treatment and erosion protection (required element)
- shrubbery for screening, glare reduction, character, and other values
- street trees for shading, character and other landscape values
- existing vegetation.

It is important to note that deep rooted plants such as trees are to be planted towards the top of the swale bank rather than near the bioretention trench, to avoid roots interfering with the underdrain system.

Where the landscape design includes canopy layers, more shade tolerant species should be selected for the groundcover layer. Trees and shrubbery should be managed so that the groundcover layer is not out-competed. If this does occur, replacement planting and possible thinning of the upper vegetation layers may be required.

3.4.6.1 Groundcovers

Groundcover vegetation is an essential functional component of bioretention swales. Appendix A provides guidance on selecting suitable plant (including turf) species and cultivars that meet the functional requirements of bioretention swales to deliver the desired stormwater quality objectives. Other species may be considered to aid in providing a visually aesthetic landscape. A table of recommended species is provided in Appendix A. Generally species selection should aim to ensure:

- a high leaf surface density within the design treatment depth to aid efficient stormwater treatment
- a dense and uniform distribution of vegetation to prevent stormwater flows from meandering between plants and to create a uniform root zone within the bioretention filter media.

3.4.6.2 Shrubs

Shrubs provide an important role in allowing for visual screening, providing interest and should compliment the design and siting of the bioretention swale. Some species are outlined in Appendix A that are useful in urban and residential landscapes, however it should be noted that these lists are guides only. Other species and cultivars may be appropriate given the surrounding natural and/ or built environment of the bioretention swale. Designers should ensure that the proposed planting schedule is suitable for the specific site. Local authorities may also provide guidance on choosing suitable shrub and tree species.

3.4.6.3 Street Trees

Trees for systems located on roadsides should conform with the local authority's relevant policy and landscape design guidelines. Also refer to Appendix A for further guidance on tree species selection.

It is important when considering planting trees within the bioretention swale system that deep rooting species are planted to the top of the bioretention zone batter to reduce roots impacting upon the filter media. If planting trees in the bioretention zone is important to the overall landscape design then creating a deeper filter media zone (min of 800mm) that further separates invasive roots from the lower drainage system is important.

3.4.6.4 Existing Vegetation

Existing vegetation, such as remnant native trees, within the bioretention swale alignment may be nominated for retention. In this case, the swale will need to be diverted or piped to avoid the vegetation's critical root zone (equivalent to 0.5 m beyond the vegetation's drip line).

3.4.7 Safety Issues

Bioretention swales within streetscapes and parks need to be generally consistent with public safety requirements for new developments. These include reasonable batter profiles for edges, providing adequate barriers to median swales for vehicle/pedestrian safety and safe vertical heights from driveways to intersecting swale inverts.

3.4.7.1 Crime Prevention Through Environmental Design (CPTED)

Landscape design of bioretention swales need to accommodate the standard principles of informal surveillance, exclusion of places of concealment and open visible areas. Regular clear sightlines should be provided between the roadway and footpaths/ property. Where planting may create places of concealment or hinder informal surveillance, groundcovers and shrubs should not generally exceed 1 m in height.

3.4.7.2 Traffic Sightlines

The standard rules of sightline geometry apply – planting designs should allow for visibility at pedestrian crossings, intersections, rest areas, medians, driveways and roundabouts. Refer to the *Road Landscape Manual* (DMR 1997) for further guidance.

3.5 Construction and Establishment

This section provides general advice for the construction and establishment of bioretention swales and key issues to be considered to ensure their successful establishment and operation. Some of the issues raised have been discussed in other sections of this chapter and are reiterated here to emphasise their importance based on observations from construction projects around Australia.

3.5.1 Staged Construction and Establishment Method

It is important to note that bioretention swale systems, like most WSUD elements that employ soil and vegetation based treatment processes, require approximately two growing seasons (i.e. two years) before the vegetation in the systems has reached its design condition (i.e. height and density). In the context of a large development site and associated construction and building works, delivering bioretention swales and establishing vegetation can be a challenging task. Therefore, bioretention swales require a careful construction and establishment approach to ensure the basin establishes in accordance with its design intent. The following sections outline a recommended staged construction and establishment methodology for bioretention swales (Leinster, 2006).

3.5.1.1 Construction and Establishment Challenges

There exist a number of challenges that must be appropriately considered to ensure successful construction and establishment of bioretention swales. These challenges are best described in the context of the typical phases in the development of a Greenfield or Infill development, namely the Subdivision Construction Phase and the Building Phase (see Figure 3-14).

- <u>Subdivision Construction</u> - Involves the civil works required to create the landforms associated with a development and install the related services (roads, water, sewerage, power etc.) followed by the landscape works to create the softscape, streetscape and parkscape features. The risks to successful construction and establishment of the WSUD systems during this phase of work have generally related to the following:
  - Construction activities which can generate large sediment loads in runoff which can smother vegetation and clog bioretention filter media
  - Construction traffic and other works can result in damage to the bioretention swales.

Importantly, all works undertaken during Subdivision Construction are normally 'controlled' through the principle contractor and site manager. This means the risks described above can be readily managed through appropriate guidance and supervision.

- Building Phase - Once the Subdivision Construction works are complete and the development plans are sealed then the Building Phase can commence (i.e. construction of the houses or built form). This phase of development is effectively 'uncontrolled' due to the number of building contractors and sub-contractors present on any given allotment. For this reason the Allotment Building Phase represents the greatest risk to the successful establishment of bioretention swales.

Plate 3-4: Example of Building Phase

3.5.1.2 Staged Construction and Establishment Method

To overcome the challenges associated within delivering bioretention swales a Staged Construction and Establishment Method should be adopted (see Figure 3-14):

- Stage 1: Functional Installation - Construction of the functional elements of the bioretention basin at the end of Subdivision Construction (i.e. during landscape works) and the installation of temporary protective measures. For example, temporary protection of bioretention swales can been achieved by using a temporary arrangement of a suitable geofabric covered with shallow topsoil (e.g. 25mm) and instant turf, in lieu of the final basin planting.
- Stage 2: Sediment and Erosion Control – During the Building Phase the temporary protective measures preserve the functional infrastructure of the bioretention swales against damage whilst also providing a temporary erosion and sediment control facility throughout the building phase to protect downstream aquatic ecosystems.
- Stage 3: Operational Establishment - At the completion of the Building Phase, the temporary measures protecting the functional elements of the bioretention swales can be removed along with all accumulated sediment and the system planted in accordance with the design planting schedule.

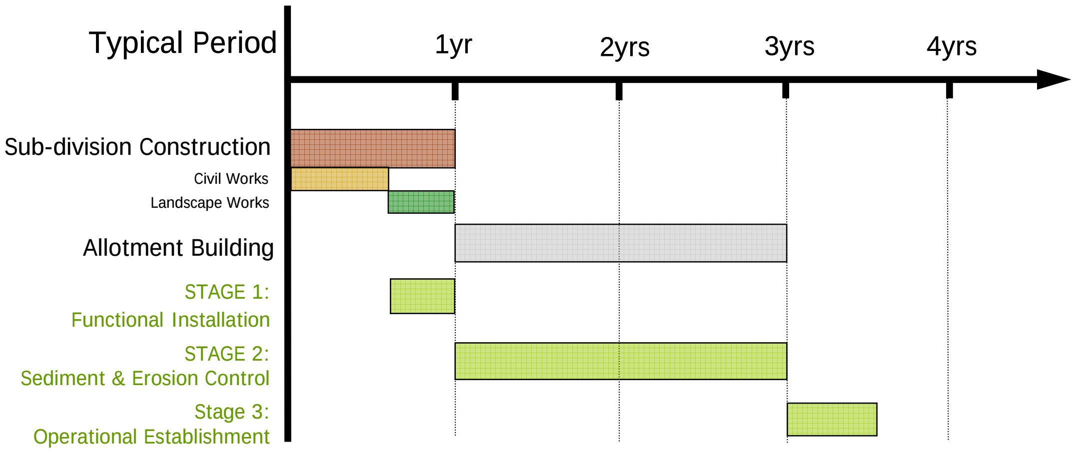


Figure 3-14: Staged Construction and Establishment Method

3.5.1.3 Functional Installation

Functional installation of bioretention swales occurs at the end of Subdivision Construction as part of landscape works and involves:

- Bulking out and trimming
- Installation of the outlet structures
- Placement of liner and installation of drainage layer (i.e. under-drains and drainage layer)
- Placement of filter media
- Placement of a temporary protective layer - Covering the surface of filtration media with geofabric and placement of 25mm topsoil and turf over geofabric. This temporary geofabric and turf layer will protect the bioretention basin during construction (Subdivision and Building Phases) ensuring sediment/litter laden waters do not enter the filter media causing clogging.
- Place silt fences around the boundary of the bioretention swale to exclude silt and restrict access.

Plate 3-5: Bioretention Swale Functional Installation

3.5.1.4 Sediment and Erosion Control

The temporary protective layers are left in place through the allotment building phase to ensure sediment laden waters do not clog the filtration media and allotment building traffic does not enter the bioretention swale. Importantly the configuration of the bioretention swale and the turf vegetation allow the system to function effectively as a shallow sedimentation basin reducing the load of coarse sediment discharging to the receiving environment. Using this approach WSUD systems can operate effectively to protect downstream ecosystems immediately after construction.

Plate 3-6: Bioretention Swale Sediment & Erosion Control

3.5.1.5 Operational Establishment

At the completion of the Allotment Building Phase the temporary measures (i.e. geofabric and turf) are removed with all accumulated sediment and the bioretention swale re-profiled and planted in accordance with the proposed landscape design. Establishment of the vegetation to design condition can require more than two growing seasons, depending on the vegetation types, during which regular watering and removal of weeds will be required.

3.5.2 Construction Tolerances

It is important to emphasise the significance of tolerances in the construction of bioretention swales (e.g. profiling of swale and bioretention trench base and surface grades). Ensuring the base of the filtration trench and surface of the bioretention filter media is free from localised depressions resulting from construction is particularly important to achieve even distribution of stormwater flows across the surface and to prevent localised ponding on the surface, which may cause mosquito problems. In addition, to enable the perforated sub-surface drainage pipes to drain freely, the base of the trench should be sloped towards the outlet pit (min 0.5% longitudinal grade). Generally an earthworks tolerance of plus or minus 50 mm is considered acceptable.

3.5.3 Sourcing Bioretention Vegetation

Notifying nurseries early for contract growing is essential to ensure the specified species are available in the required numbers and of adequate maturity in time for bioretention swale planting. When this is not done and the planting specification is compromised, poor vegetation establishment and increased initial maintenance costs may occur. The species listed in Appendix A are generally available commercially from local native plant nurseries. Availability is, however, dependent upon many factors including demand, season and seed availability. To ensure planting specification can be accommodated, the minimum recommended lead time for ordering is 3-6 months. This usually allows enough time for plants to be grown to the required size. The following pot sizes are recommended as the minimum:

- Viro Tubes 50 mm wide x 85 mm deep
- 50 mm Tubes 50 mm wide x 75 mm deep
- Native Tubes 50 mm wide x 125 mm deep

3.5.4 Vegetation Establishment

The following weed control measures and watering schedule are recommended to ensure successful plant establishment. Regular general maintenance as outlined in Section 3.6 will also be required.

3.5.4.1 Weed Control

Conventional surface mulching of bioretention swales with organic material like tanbark, should not be undertaken. Most organic mulch floats and runoff typically causes this material to be washed away with the risk of blockage of drains occurring. Weed management will need to be done manually until such time that the design vegetation is established with sufficient density to effectively prevent weed propogation.

3.5.4.2 Watering

Regular watering of bioretention swale vegetation is essential for successful establishment and healthy growth. The frequency of watering to achieve successful plant establishment is dependent upon rainfall, maturity of planting stock and the water holding capacity of the soil. The following watering program is generally adequate but should be adjusted (increased) to suit the site conditions:

- Week 1-2 3 visits/ week
- Week 3-6 2 visits/ week
- Week 7-12 1 visit/ week

After this initial three month period, watering may still be required, particularly during the first winter (dry period). Watering requirements to sustain healthy vegetation should be determined during ongoing maintenance site visits.

3.6 Maintenance Requirements

Bioretention swales have a flood conveyance role that needs to be maintained to ensure adequate flood protection for local properties. 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 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 an inadequate level of erosion and sediment control.

The potential for rilling and erosion down the swale component of the system needs to be carefully monitored during establishment stages of the system. Other components of the system that will require careful consideration are the inlet points (if the system does not have distributed inflows) and surcharge pits, as these inlets can be prone to scour and the build up of litter and sediment. Bioretention swale field inlet pits also require routine inspections to ensure structural integrity and that they are free of blockages with debris. Debris removal is an ongoing maintenance requirement. Debris can block inlets or outlets and can be unsightly, particularly in high visibility areas. Inspection and removal of debris should be done regularly.

Typical maintenance of bioretention swale elements will involve:

- Routine inspection of the swale profile to identify any areas of obvious increased sediment deposition, scouring of the swale invert from storm flows, rill erosion of the swale batters from lateral inflows, damage to the swale profile from vehicles and clogging of the bioretention trench (evident by a 'boggy' swale invert).
- Routine inspection of inlet points (if the swale does not have distributed inflows), surcharge pits and field inlet pits to identify any areas of scour, litter build up and blockages.
- Removal of sediment where it is impeding the conveyance of the swale and/ or smothering the swale vegetation, and if necessary, reprofiling of the swale and revegetating to original design specification.
- Repairing any damage to the swale profile resulting from scour, rill erosion or vehicle damage.
- Tilling of the bioretention trench surface if there is evidence of clogging.
- Clearing of blockages to inlet or outlets.
- Regular watering/ irrigation of vegetation until plants are established and actively growing (see section 3.5.4).
- Mowing of turf or slashing of vegetation (if required) to preserve the optimal design height for the vegetation.
- Removal and management of invasive weeds.
- Removal of plants that have died and replacement with plants of equivalent size and species as detailed in the plant schedule.
- Pruning to remove dead or diseased vegetation material and to stimulate new growth.
- Litter and debris removal.
- Vegetation pest monitoring and control.

Resetting (i.e. complete reconstruction) of bioretention elements will be required if the available flow area of the overlying swale is reduced by 25 % (due to accumulation of sediment) or if the bioretention trench fails to drain adequately after tilling of the surface. Inspections are also recommended following large storm events to check for scour.

All maintenance activities must be specified in a maintenance plan (and associated maintenance inspection forms) to be developed as part of the design procedure. Maintenance personnel and asset managers will use this plan to ensure the bioretention swales continue to function as designed. The maintenance plans and forms must address the following:

- inspection frequency
- maintenance frequency
- data collection/ storage requirements (i.e. during inspections)
- detailed cleanout procedures (main element of the plans) including:
  - equipment needs
  - maintenance techniques
  - occupational health and safety
  - public safety
  - environmental management considerations
  - disposal requirements (of material removed)
  - access issues
  - stakeholder notification requirements
  - data collection requirements (if any)
- design details

An example operation and maintenance inspection form is included in the checking tools provided in Section 3.7.

3.7 Checking Tools

The following sections provide a number of checking aids for designers and Council development assessment officers. In addition, advice on construction techniques and lessons learnt from building bioretention swale systems are provided. Checklists are provided for:

- Design Assessment
- Construction (during and post)
- Operation and Maintenance Inspections
- Asset Transfer (following defects period).

3.7.1 Design Assessment Checklist

The checklist on page 3-37 presents the key design features to be reviewed when assessing design of a bioretention swale. These considerations include configuration, safety, maintenance and operational issues that need to be addressed during the design phase. Where an item results in an 'N' when reviewing the design, referral is to be made back to the design procedure to determine the impact of the omission or error.

In addition to the checklist, a proposed design is to have all necessary permits for its installations. Council development assessment officers need to 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.

3.7.2 Construction Checklist

The checklist on page 3-38 presents the key items to be reviewed when inspecting the bioretention swale during and at the completion of construction. The checklist is to be used by construction site supervisors and compliance inspectors to ensure all the elements of the bioretention basin have been constructed in accordance with the design. If an item receives an 'N' in Satisfactory criteria then appropriate actions must be specified and delivered to rectify the construction issue before final inspection sign-off is given.

3.7.3 Operation and Maintenance Inspection Form

The form on page 3-39 is to be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

3.7.4 Asset Transfer Checklist

Land ownership and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design is to clearly identify the ultimate asset owner and who is responsible for its maintenance. Local authorities will use the asset transfer checklist on page 3-40 when the asset is to be transferred to the local authority.

BIORETENTION SWALE DESIGN ASSESSMENT CHECKLIST

Asset I.D.				
Bioretention Location:				
Hydraulics:	Minor Flood (m^2/s):	Major Flood (m^2/s):		
Area:	Catchment Area (ha):	Bioretention Area (m^2):		
TREATMENT			Y	N
Treatment performance verified from curves?				
SWALE COMPONENT			Y	N
Longitudinal slope of invert >1% and <4%?				
Manning's '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 requirements?				
Overflow pits provided where flow capacity exceeded?				
Energy dissipation provided at inlet points to the swale?				
Velocities within bioretention cells will not cause scour?				
Set down of at least 60mm below kerb invert to top of vegetation incorporated?				
BIORETENTION COMPONENT			Y	N
Design documents bioretention area and extended detention depth as defined by treatment performance requirements?				
Overflow pit crest set at top of extended detention?				
Maximum ponding depth and velocity will not impact on public safety (v x d <0.4)				
Bioretention media specification includes details of filter media, drainage layer and transition layer (if required)?				
Design saturated hydraulic conductivity included in specification?				
Transition layer provided where drainage layer consists of gravel (rather than coarse sand)?				
Perforated pipe capacity > infiltration capacity of filter media?				
Selected filter media hydraulic conductivity > 10 x hydraulic conductivity of surrounding soil?				
Maximum spacing of collection pipes <1.5m?				
Collection pipes extended to surface to allow inspection and flushing?				
Liner provided if selected filter media hydraulic conductivity > 10x hydraulic conductivity of surrounding soil?				
Maintenance access provided to invert of conveyance channel?				
LANDSCAPE & VEGETATION			Y	N
Plant species selected can tolerate periodic inundation and design velocities?				
Bioretention swale landscape design integrates with surrounding natural and/ or built environment?				
Planting design conforms with acceptable sight line and safety requirements?				
Top soils are a minimum depth of 300 mm for plants and 100 mm for turf?				
Existing trees in good condition are investigated for retention?				
Detailed soil specification included in design?				
COMMENTS				

BIORETENTION SWALE CONSTRUCTION INSPECTION CHECKLIST

Asset I.D.		Inspected by:	
Site:		Date:	
		Time:	
Constructed by:		Weather:	
		Contact during site visit:	

Items inspected	Checked		Satisfactory		Items inspected	Checked		Satisfactory	
	Y	N	Y	N		Y	N	Y	N
DURING CONSTRUCTION & ESTABLISHMENT									
A. FUNCTIONAL INSTALLATION					Structural components				
Preliminary Works					15. Location and configuration of inflow systems as designed				
1. Erosion and sediment control plan adopted					16. Location and levels of overflow pits as designed				
2. Temporary traffic/safety control measures					17. Under-drainage connected to overflow pits as designed				
3. Location same as plans					18. Concrete and reinforcement as designed				
4. Site protection from existing flows					19. Set down to correct level for flush kerbs (streetscape applications only)				
Earthworks and Filter Media					19. Kerb opening width as designed				
5. Bed of swale correct shape and slope									
6. Batter slopes as plans					B. SEDIMENT & EROSION CONTROL (IF REQUIRED)				
7. Dimensions of bioretention area as plans					20. Stabilisation immediately following earthworks and planting of terrestrial landscape around basin				
8. Confirm surrounding soil type with design					21. Silt fences and traffic control in place				
9. Confirm filter media specification in accordance with Step 4					22. Temporary protection layers in place				
9. Provision of liner (if required)									
10. Under-drainage installed as designed					C. OPERATIONAL ESTABLISHMENT				
11. Drainage layer media as designed					23. Temporary protection layers and associated silt removed				
12. Transition layer media as designed (if required)					Vegetation				
14. Extended detention depth as designed					24. Planting as designed (species and densities)				
					25. Weed removal and watering as required				

FINAL INSPECTION									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of banks				
2. Confirm structural element sizes					7. Under-drainage working				
3. Check batter slopes					8. Inflow systems working				
4. Vegetation as designed					9. Maintenance access provided				
5. Bioretention filter media surface flat and free of clogging									

COMMENTS ON INSPECTION

ACTIONS REQUIRED
1.
2.
3.

Inspection officer signature:

BIORETENTION SWALE MAINTENANCE CHECKLIST			
Asset I.D.			
Inspection Frequency:	1 to 6 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?			
Remulching of trees and shrubs required?			
Soil additives or amendments required?			
Pruning and/ or removal of dead or diseased vegetation required?			
Resetting of system required?			
COMMENTS			

BIORETENTION SWALE ASSET TRANSFER CHECKLIST

Asset I.D.:		
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 and indicative maintenance costs provided for each asset?		
Vegetation establishment period completed (as per LGA requirements)?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
ASSET INSPECTED FOR DEFECTS AND/OR MAINTENANCE ISSUES AT TIME OF ASSET TRANSFER		
Sediment accumulation at inflow points?		
Litter within swale?		
Erosion at inlet or other key structures?		
Traffic damage present?		
Evidence of dumping (e.g. building waste)?		
Vegetation condition satisfactory (density, weeds)?		
Watering of vegetation required?		
Replanting required?		
Mowing/slashing required?		
Clogging of drainage points (sediment or debris)?		
Evidence of ponding?		
Damage/vandalism to structures present?		
Surface clogging visible?		
Drainage system inspected?		
COMMENTS/ACTIONS REQUIRED FOR ASSET TRANSFER		
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?		

3.8 Engineering Drawings

The relevant local authority should be consulted to source standard drawings applicable to bioretention swales. These drawings may provide example dimensions for a number of different road reserve configurations. Standard drawings are not intended to be prescriptive drawings which must be adhered to, rather they are intended to provide detailed examples of swales which can be incorporated into commonly used urban subdivision layouts. Designers are encouraged to develop alternative bioretention swale designs to suit site specific conditions.

In the absence of locally specific guidelines, BCC standard drawings applicable to swales and bioretention systems are UMS 151-158. These may also be used as reference standards for swale design. BCC Standard drawings are available online at <http://www.brisbane.qld.gov.au/>.

3.9 Bioretention Swale Worked Example

Modelling using MUSIC was undertaken to develop a stormwater quality treatment system for the concept design stage of a new greenfield residential estate. This worked example describes the detailed design of a vegetated 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 3-15. A photograph of a similar bioretention swale in a median strip is shown in Plate 3-7.

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

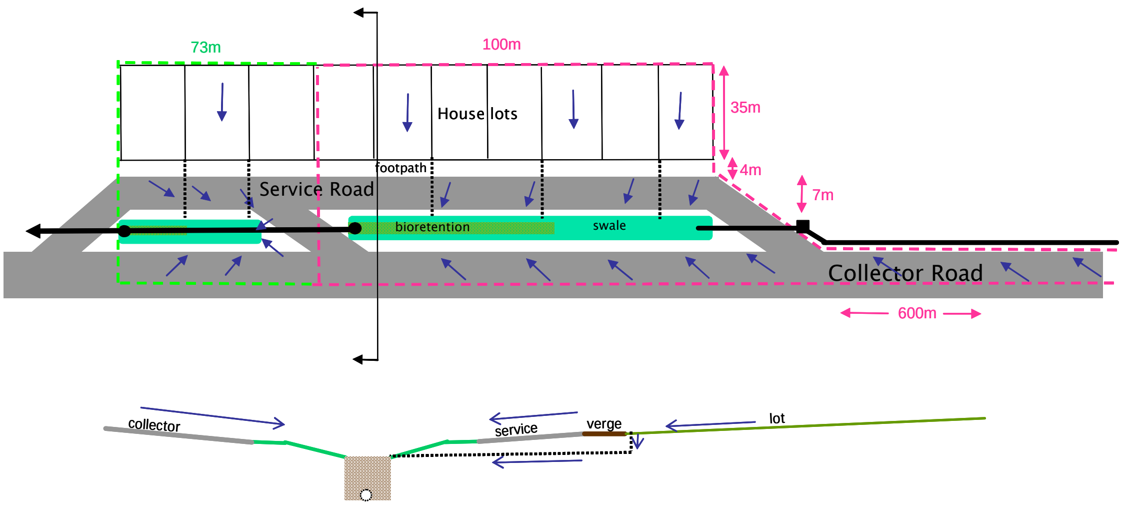


Figure 3-15: Catchment Area Layout and Section for Worked Example

Plate 3-7: Bioretention Swale Located Between a Main Road and Local Road

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 pre-treatment 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 (via inlet pits) at the downstream end of each bioretention cell with a 2 year ARI capacity (the minor storm for the hypothetical worked example).

MUSIC modelling undertaken during the concept design stage found that the area of bioretention to meet the required water quality objectives is approximately 65 m^2 and 25 m^2 for Cell A and B respectively. The filter media saturated hydraulic conductivity derived from the MUSIC modelling was 180 mm/hr based on 200 mm of extended detention and dense plantings of sedges and tufted grasses in the bioretention filter media.

Design Objectives

- Treatment to achieve 75 %, 45 % and 35 % reductions of mean annual loads of TSS, TP and TN respectively, with these reductions having been defined by earlier MUSIC modelling that indicated such standards were required in order for the stormwater treatment train proposed for the site to comply with the relevant local water quality objectives.
- Perforated under-drainage to be designed to ensure that the capacity of the perforated pipes exceeds the saturated hydraulic conductivity of the filter media.
- Design flows up to 2 year ARI range are to be safely conveyed into a piped drainage system with acceptable inundation of the adjacent road.
- The hydraulics for the swale and road system need to be checked to confirm flow capacity for the 2 year and 50 year ARI peak flows, in accordance with the road drainage standards for the local Council.
- Acceptable safety and scouring behaviour for 2 year and 50 year ARI peak flows.
- Integration of the bioretention swale landscape design with the surrounding natural and built environment.

Constraints and Concept Design Criteria

- Depth of the bioretention filter layer shall be a maximum of 600 mm.
- Maximum extended depth allowable is 200 mm.
- Width of median available for siting the system is 6 m.
- The filter media available is a sandy loam top soil stripped from the site and amended by mixing in a loose non-angular sand to achieve the design saturated hydraulic conductivity of 180 mm/hour determined to be the optimal saturated hydraulic conductivity by the MUSIC modelling undertaken at the concept design stage.

Site Characteristics

- Land use: urban, low density residential (greater Brisbane area)
- Overland flow slopes: Cell A and B = 1.3 %
- Soil: Clay
- Fraction impervious, f_i: 0.60 (lots); 0.90 (roads); 0.50 (footpaths); 0.0 (Swale)
- Catchment areas:

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

3.9.1 Step 1: Confirm Treatment Performance of Concept Design

Interpretation of Figure 3-3 to Figure 3-5 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.

- Location is within the Greater Brisbane Region
- 200 mm extended detention
- treatment area to catchment area ratio:
  - Cell A: 65 m^2/ 6710 m^2 = 0.97 %
  - Cell B: 25 m^2/ 2599 m^2 = 0.96 %

From the graphs, the expected pollutant reductions are 77 %, 68 % and 38 % for TSS, TP and TN respectively, and exceed the design requirements of 75 %, 45 % and 35 %.

3.9.2 Step 2: Estimating Design Flows for Swale Component

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

3.9.2.1 Major and Minor Design Flows

Time of concentration (t_c)

Approach:

Cell A and Cell B are effectively separate elements for the purpose of sizing the swales. Therefore, t_c values are estimated separately for each cell.

- Cell A - the t_c calculations include consideration of runoff from the allotments as well as from gutter and pipe flow along the collector road. Comparison 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 the relevant local authority guidelines, the following t_c values are estimated:

- t_c Cell A : 8 mins (5 min inlet time and 3 min pipe flow time (assuming a pipe flow velocity of 3 m/s)
- t_c Cell B: 15 mins (inlet time from QUDM for land with a slope of < 3%)

Design rainfall intensities (from local government QUDM Supplement)

Design ARI	Cell A (8 min t_c)	Cell B (15 min t_c)
2	126 mm/hr	97 mm/hr
50	246 mm/hr	194 mm/hr

Design runoff coefficient

- Fraction impervious

Cell A:	Area (m^2)	f_i	Impervious Area (m^2)
Allotments	3500	0.6	2100
Roads	4900	0.9	4410
Footpath	400	0.5	200
Swale	772.5	0.0	-
TOTAL	9572.5	-	6710

Hence effective f_i = 0.7

Cell B:	Area (m^2)	f_i	Impervious Area (m^2)
Allotments	2555	0.6	1533
Roads	1022	0.9	919.8
Footpath	292	0.5	1467
Swale	330	0.0	-
TOTAL	4199	-	2599

Hence effective f_i = 0.62

- Runoff coefficients, as per QUDM (DPI, IMEA & BCC, 1992)

Design ARI	Cell A	Cell B
2	0.71	0.70
50	0.97	0.95

Peak Design flows

- Rational Method

$$Q = CIA/360 \text{ (m}^3\text{/s)}$$

Design ARI	Cell A (m^3/s)	Cell B (m^3/s)
2	0.24	0.08
50	0.64	0.22

3.9.3 Step 3: Dimension the Swale Component

3.9.3.1 Swale Width and Side Slopes

The swale component of Cell A and B need to be sized such that they can convey the 2 year and 50 year ARI flows with acceptable amount of water encroaching on the road. Manning's equation (Equation 3.2) is used with the following parameters. Note the depth of the swale (and hence the side slopes) was determined by the requirement of discharging allotment runoff onto the surface of the bioretention system. The cover requirements of the allotment drainage pipes as they flow under the service road set the surface of the bioretention system. In this example, a Class 4 pipe is adopted and as such requires 300 mm cover. Allowing for this cover, a 100 mm diameter pipe and 100 mm fall with passage across the service road, the surface level of the bioretention systems must be 0.5 m below the edge of road pavement surface level.

The adopted swale dimensions for both Cell A and Cell B were:

- swale base width of 1 m with 1:5 side slopes, max depth of 0.5 m
- moderate vegetation height 200 mm (assume Manning's n = 0.04 for flows above vegetation height)
- 1.3% slope

3.9.3.2 Maximum Length of Swale

The approach taken is to first determine the maximum length of the swale component of Cell A and then assume this same maximum length also applies to the swale component of Cell B (which has lower flow rates than Cell A).

To determine the maximum length of swale for the swale component of Cell A, it is necessary to calculate the maximum capacity of the swale using Manning's equation (Equation 3.1) and the design parameters presented above. This equates to:

Q_{cap} = 2.17 m^3/s >> 0.64 m^3/s (Q_{50}) and 0.24 m^3/s (Q_2)

Therefore, there is adequate capacity in the swale to convey all flows up to and well in excess of the Q_{50} with no flow required to be conveyed on the adjacent road pavement. This result indicates that the maximum length of swale for the swale component of Cell A (and therefore Cell B) is much longer than the 'actual' length of the swale components of Cell A and B. As such, no additional calculations are required to check flow widths and depths on the adjacent road pavements to confirm compliance with the minor flood and major flood criteria outlined in Section 5.09 of QUDM

Freeboard to adjoining property must also be checked and comply with the relevant local requirements. Given, in this instance, that Q_{50} is contained within the swale, the freeboard requirements are satisfied.

3.9.4 Step 4: Design Inflow Systems to Swale and Bioretention Components

There are two mechanisms for flows to enter the bioretention swale systems Cell A and Cell B. Firstly, underground pipes (either from the upstream road into Cell A or from allotment runoff) and secondly,

Flush kerbs with a 60 mm set down are intended to be used to allow for sediment accumulation off 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.

3.9.5 Step 5: Design Bioretention Component

3.9.5.1 Select Filter Media Saturated Hydraulic Conductivity and Extended Detention

The calculations undertaken for Steps 2 and 3 show that the dimensions of the swale component are sufficient to satisfy flow conveyance criteria and therefore there is no requirement for the bioretention component's saturated hydraulic conductivity or extended detention depth to be altered from what was determined by the MUSIC modelling undertaken at the concept design stage and presented in Section 3.7.1.2.

3.9.5.2 Specify the Bioretention Filter Media Characteristics (Filter, Transition and Drainage Layers)

The specification of the filter media and drainage layers requires consideration of the perforated under-drainage system. In this case, a perforated pipe with a slot width of 1.5 mm has been selected, meaning there is a risk that sand (typically 1 mm diameter and less) could wash into the pipe. Therefore, in this case, three layers are to the used, an amended sandy loam as the filter media (600 mm), a coarse sand transition layer (100 mm) and a fine gravel drainage layer (200 mm).

Filter media specifications

The filter media is to be a sandy loam, formed through the procedure documented in Section 3.3.5.2. The filter media will generally meet the following geotechnical requirements:

- saturated hydraulic conductivity of 180 mm/hr determined from appropriate laboratory testing (see section 3.3.5.2)
- between 5 % and 10 % organic content, measured in accordance with AS 1289.4.1.1-1997
- pH neutral.

Transition layer specifications

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

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

Drainage layer specifications

The drainage layer is to be 5 mm screenings.

3.9.5.3 Under Drainage Design and Capacity Checks

Maximum infiltration rate

The maximum infiltration rate reaching the perforated pipe system at the base of the bioretention filter media is estimated by using the hydraulic conductivity of the media and the head above the pipes and applying Darcy's equation (Equation 3.2).

Saturated hydraulic conductivity = 180 mm/hr

Flow capacity of the infiltration media = (1-ϒ). As k_h – (Engineers Australia 2006)

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

$$Q_{max} = 5 \times 10^{-5} \cdot L \cdot W_{base} \cdot \frac{0.2 + 0.6}{0.6}$$

Where	Q_{max}	= maximum infiltration rate (m^3/s)
	K_{sat}	= hydraulic conductivity of the soil filter (m/s)
	W_{base}	= base width of the ponded cross section above the soil filter (m)
	L	= length of the bioretention zone (m)
	h_{max}	= depth of pondage above the soil filter (m)
	d	= depth of filter media

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

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

Perforations inflow check

Estimate the inlet capacity of sub-surface drainage system to ensure it is not a choke in the system. As a conservative approach, 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 (Equation 3.3) is applied using the following parameters:

Head above pipe (*h*)= 0.95 m [0.6 m (filter depth) + 0.1m (transition) + 0.1 (half drainage layer) + 0.2 m (max. pond level) + 0.05 (half of pipe diameter)]

Assume sub-surface drains with half of all pipes blocked.

Clear Opening	= 2100 mm^2/m, hence blocked openings
	= 1050 mm^2/m (50%)
Slot Width	= 1.5 mm
Slot Length	= 7.5 mm
Number of Rows	= 6
Diameter	= 100 mm

Number of slots per metre = (1050)/(1.5x7.5) = 93.3

Assume orifice flow conditions:

$$Q_{perf} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h}$$

Where C_d = 0.61 (Assume slot width acts as a sharp edged orifice).

Note: Blockage Factor *B* (=0.5) has already been accounted for in the 'Clear Opening' calculation above

Inlet capacity /m of pipe:

$$Q_{perf} = [0.61 \times (0.0015 \times 0.0075)\sqrt{2 \times 9.81 \times 1.05}] \times 93.3$$

$$= 0.0029 \text{ m}^3\text{/s}$$

Inlet capacity/m x total length:

Cell A = 0.0029 x 61 = 0.18 m^3/s > 0.004 L/s (max infiltration rate), hence one pipe has sufficient perforation capacity to pass flows into the perforated pipe.

Cell B = 0.0029 x 22 = 0.06 m^3/s > 0.0014 L/s (max infiltration rate), hence one pipe is sufficient.

Check perforated pipe capacity

Manning's equation is applied to estimate the flow rate in the perforated pipe. A slope of 0.5 % is assumed and a 100 mm perforated pipe (as above) with Manning's *n* of 0.02 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 Manning's Equation:

Q = 0.0024 m^3/s

Therefore, will need two pipes for Cell A (0.004 m^3/s max. infiltration rate) and one pipe for Cell B (0.001 m^3/s max. infiltration rate).

Check drainage layer hydraulic conductivity

Typically, flexible perforated pipes are installed using fine gravel media to surround them. In this worked example, 5 mm 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 as specified in section 3.7.6.2.

3.9.5.4 Impervious Liner Requirement

In this catchment, the surrounding soils are clay to silty clays with a saturated hydraulic conductivity of approximately 3.6 mm/hr. The sandy loam media that is proposed as the filter media has a hydraulic conductivity of 180 mm/hr. Therefore, the conductivity of the filter media is greater than 10 times the conductivity of the surrounding soils and an impervious liner is not required.

3.9.6 Step 6: Verification Checks

3.9.6.1 Vegetation Scour Velocity Check

Potential scour velocities within the swale and on the bioretention surface are checked by applying Manning's equation (Equation 3.1) to the bioretention swale design to ensure the following criteria is met:

- Less than 0.5 m/s for minor flood (2 year ARI) discharge.
- Less than 2.0 m/s for major flood (50 year ARI) discharge.

Using Manning's equation to solve for depth for Q_2 and Q_{50} in Cell A gives the following results. Note, Manning's n used for Q_2 = 0.1 (flow below vegetation height) and for Q_{50} = 0.04 (flow above vegetation height).

Q_2 = 0.24 m^3/s, velocity = 0.36 m/s < 0.5 m/s – therefore OK

Q_{50} = 0.64 m^3/s, velocity = 1.35 m/s < 2.0 m/s – therefore OK

Hence, the swale can satisfactorily convey the peak 2 year and 50 year ARI flood flows with minimal risk of vegetation scour.

3.9.6.2 Safety Velocity Check

Check velocity *(V)* x depth *(d)* product in Cell A during peak 50 year ARI flow for pedestrian safety criteria.

V = 1.42 m/s

d = 0.32 m

$V \times d$ = 1.42 x 0.32 = 0.45 < 0.6 m^2/s (QUDM Supplement (BCC 1994))

Therefore, velocities and depths are OK.

3.9.7 Step 7: Overflow Pit Design

The overflow pits are required to convey 2 year ARI flows safely from the bioretention systems and into an underground pipe network. Grated pits are to be used at the downstream end of each bioretention system.

The sizes of the pits are calculated using a broad crested weir equation (Equation 3.4) with the height above the maximum ponding depth and below the road surface, less freeboard (i.e. 0.76 – (0.2 + 0.15) = 0.41 m).

First check using a broad crested weir equation (refer Section 5.10.4 from QUDM (DPI, IMEA & BCC, 1992) and Equation 3.4):

$$Q_{weir} = B \cdot C_w \cdot L \cdot h^{3/2}$$

Where *B* = Blockage factor (= 0.5)

C_w = Weir coefficient (= 1.66)

L = required length of weir (pit perimeter) (m)

h = Flow depth above the weir (0.41 m)

Solving for *L* gives *L* = 1.1 m of weir length required (equivalent to 300 x 300 mm pit).

Now check for drowned conditions (Equation 3.5):

$$Q_{orifice} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h}$$

with C_d = 0.6 and *h* = 0.41 m we have:

$$0.24 = 0.6 \times A\sqrt{2 \times 9.81 \times 0.41}$$

Gives *A* = 0.14 m^2 (equivalent to 400 x 400 mm pit)

Hence, drowned outlet flow conditions dominate, a minimum pit size of 400 x 400 mm is required for both Cell A and Cell B. The minimum pit size from local government standard is 600 x 600 mm therefore, this is to be adopted for both Cell A and Cell B.

3.9.8 Step 8: Allowances to Preclude Traffic on Swales

Traffic control is achieved by using traffic bollards.

3.9.9 Step 9: Vegetation Specification

To compliment the landscape design of the area a mix of tufted grass and sedges is to be used. For this application, species with the average height of 200 mm have been proposed. The actual species to be planted will be selected by the landscape designer.

3.9.10 Step 10: Maintenance Plan

A maintenance plan for Swales 1 and 2 is to be prepared in accordance with local authority requirements and the recommendation in Section 3.5.

3.9.11 Calculation summary

The sheet below summarises the results of the design calculations.

BIORETENTION SWALES DESIGN CALCULATION SUMMARY

	Calculation Task	CALCULATION SUMMARY Outcome		Check
	Catchment Characteristics			
	Catchment Area	0.671	Ha	✓
	Catchment Land Use (i.e. residential, Commercial etc.)	Res		
	Conceptual Design			
	Bioretention area	65	m^2	✓
	Filter media saturated hydraulic conductivity	180	mm/hr	
	Extended detention depth	200	mm	
1	**Confirm Treatment Performance of Concept Design**			
	Bioretention area to achieve water quality objectives	0.97	m^2	✓
	TSS Removal	0.5	%	
	TP Removal	0.97	%	
	TN Removal		%	
2	**Estimate Design Flows for Swale Compnent**			
	Time of concentration – QUDM or relevant local government guideline	8	minutes	✓
	Identify Rainfall intensities			
	$I_{2\text{-}10\ year\ ARI}$	126	mm/hr	✓
	$I_{50\text{-}100\ year\ ARI}$	246	mm/hr	
	Design Runoff Coefficient			
	$C_{2\text{-}10\ year\ ARI}$	0.71		✓
	$C_{50\text{-}100\ year\ ARI}$	0.97		
	Peak Design Flows			
	2-10 year ARI	0.24	m^3/s	✓
	50-100 year ARI	0.64	m^3/s	
3	**Dimension the Swale Component**			
	Swale Width and Side Slopes			
	Base Width	1	m	✓
	Side Slopes – 1 in	5		
	Longitudinal Slope	1.2	%	
	Vegetation Height	200	mm	
	Maximum Length of Swale			
	Manning's n	0.04		✓
	Swale Capacity	2.17		
	Maximum Length of Swale	Yes		
4	**Design Inflow Systems to Swale & Bioretention Components**			
	Swale Kerb Type	Flush		✓
	Adequate Erosion and Scour Protection (where required)	N/A		
5	**Design Bioretention Component**			
	Filter media hydraulic conductivity	180	mm/hr	✓
	Extended detention depth	200	mm	
	Filter media depth	600	mm	
	Drainage layer media (sand or fine screenings)	Fine screenings		
	Drainage layer depth	200	mm	
	Transition layer (sand) required	Yes		
	Transition layer depth	100	mm	
	Under-drain Design and Capacity Checks			
	Flow capacity of filter media (maximum infiltration rate)	0.004	m^3/s	✓
	Perforations inflow check	Yes		
	Pipe diameter	100	mm	✓
	Number of pipes	2		
	Capacity of perforations	0.15	m^3/s	
	CHECK PERFORATION CAPACITY > FILTER MEDIA CAPACITY	Yes		
	Perforated pipe capacity			
	Pipe capacity	0.0024x2	m^3/s	✓
	CHECK PIPE CAPACITY > FILTER MEDIA CAPACITY	Yes		
	Check requirement for impermeable lining			
	Soil hydraulic conductivity	180	mm/hr	✓
	Filter media hydraulic conductivity	3.6 (clay)	mm/hr	
	MORE THAN 10 TIMES HIGHER THAN IN-SITU SOILS?	Yes		
5	**Verification Checks**			
	Velocity for 2-10 year ARI flow (< 0.5 m/s)	0.35	m/s	✓
	Velocity for 50-100 year ARI flow (< 2 m/s)	1.35	m/s	
	Velocity x Depth for 50-100 year ARI (< 0.4 m^2/s)	0.45	m^2/s	
	Treatment Performance consistent with Step 1	Yes		
6	**Overflow Pit Design**			
	System to convey minor floods	400x400	L x W	✓

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[1] At the time of preparation of these guidelines, QUDM was under review and a significantly revised edition is expected to be released in 2006. These guidelines refer to and use calculations specified in the existing QUDM document, however the revised version of QUDM should be used as the appropriate reference document. It should be noted by users of this guideline that the structure and content of QUDM will change, and as such, the references to calculations and/or specific sections of QUDM may no longer be correct. Users of this guideline should utilise and adopt the relevant sections and/or calculations of the revised QUDM guideline.

Chapter 4 Sediment Basins

4.1 Introduction 4-2

4.2 Design Considerations 4-3
4.2.1 Role in the Treatment Train 4-3
4.2.2 Sizing a Sedimentation Basin 4-4
4.2.3 Sediment Storage 4-4
4.2.4 Outlet Design 4-4
4.2.5 Landscape Design 4-4
4.2.6 Vegetation Specification 4-5
4.2.7 Maintenance 4-5

4.3 Design Process 4-6
4.3.1 Step 1: Determine Design Flows 4-7
4.3.2 Step 2: Confirm Treatment Performance of Concept Design 4-7
4.3.3 Step 3: Confirm Size and Dimension the Sedimentation Basin 4-9
4.3.4 Step 4: Design Inflow Systems 4-13
4.3.5 Step 5: Design Outlet Structure 4-13
4.3.6 Step 6: Specify Vegetation 4-17
4.3.7 Step 7: Consider Maintenance Requirements 4-17
4.3.8 Design Calculation Summary 4-17

4.4 Landscape Design Notes 4-20
4.4.1 Objectives 4-20
4.4.2 Context and Site Analysis 4-20
4.4.3 Specific Landscape Considerations 4-20
4.4.4 Sedimentation Basin Vegetation 4-26
4.4.5 Safety Issues 4-27

4.5 Construction and Establishment Advice 4-28
4.5.1 Staged Construction and Establishment Method 4-28
4.5.2 Construction Tolerances 4-29
4.5.3 Sourcing Sedimentation Basin Vegetation 4-29
4.5.4 Topsoil Specification and Preparation 4-30
4.5.5 Vegetation Establishment 4-30

4.6 Maintenance Requirements 4-32

4.7 Checking Tools 4-33
4.7.1 Design Assessment Checklist 4-33
4.7.2 Construction Checklist 4-33
4.7.3 Operation and Maintenance Inspection Form 4-33
4.7.4 Asset Transfer Checklist 4-33

4.8 Sedimentation Basin Worked Example 4-38
4.8.1 Step 1: Determine Design Flows 4-39
4.8.2 Step 2: Confirm Treatment Performance of Concept Design 4-40
4.8.3 Step 3: Confirm Size and Dimensions of the Sedimentation Basin 4-41
4.8.4 Step 4: Design Inflow Systems 4-42
4.8.5 Step 5: Design Outlet Structures 4-43
4.8.6 Step 6: Vegetation Specification 4-44
4.8.7 Design Calculation Summary 4-44
4.8.8 Worked Example Drawings 4-47

4.9 References 4-49

4.1 Introduction

Reducing sediment loads is an important component of improving stormwater quality. Sedimentation basins can form an integral component of a stormwater treatment train and are specifically employed to remove coarse to medium sized sediments by settling them from the water column. Sedimentation basins can take various forms and can be used as permanent systems integrated into an urban design, or temporary measures to control sediment discharge during construction. This chapter describes the design and construction of permanent sedimentation basins ('wet' basins) that form part of a treatment train (e.g. an inlet zone/ pond to a constructed wetland) for operation in the post construction/building phase. For the design and application of temporary sedimentation ('dry') basins to control sediment discharge during the construction/ building phase, refer to *Sediment Basin Design, Construction and Maintenance Guidelines* (BCC 2001).

Sedimentation basins are stormwater detention systems that promote settling of sediments through the reduction of flow velocities and temporary detention. Key elements include purpose designed inlet and outlet structures, settling pond, and high flow, overflow structures. The storage volume consists of two components: the permanent pool settling zone and the sediment storage zone. Access for maintenance must also be provided. These elements are shown below in Figure 4-1 and Figure 4-2. Key design parameters are selecting a target sediment size, design discharge, basin area and shape, sediment storage volume and outlet structures.

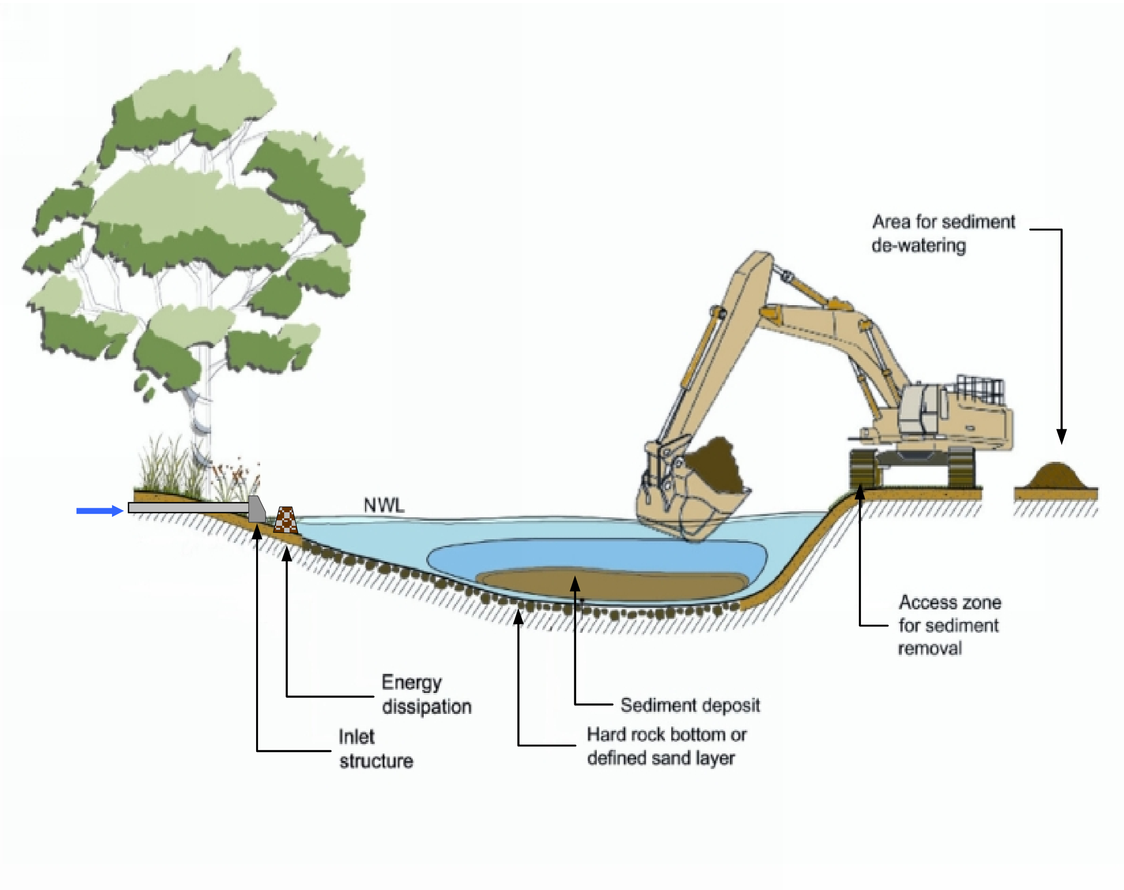


Figure 4-1: Sedimentation Basin Conceptual Layout

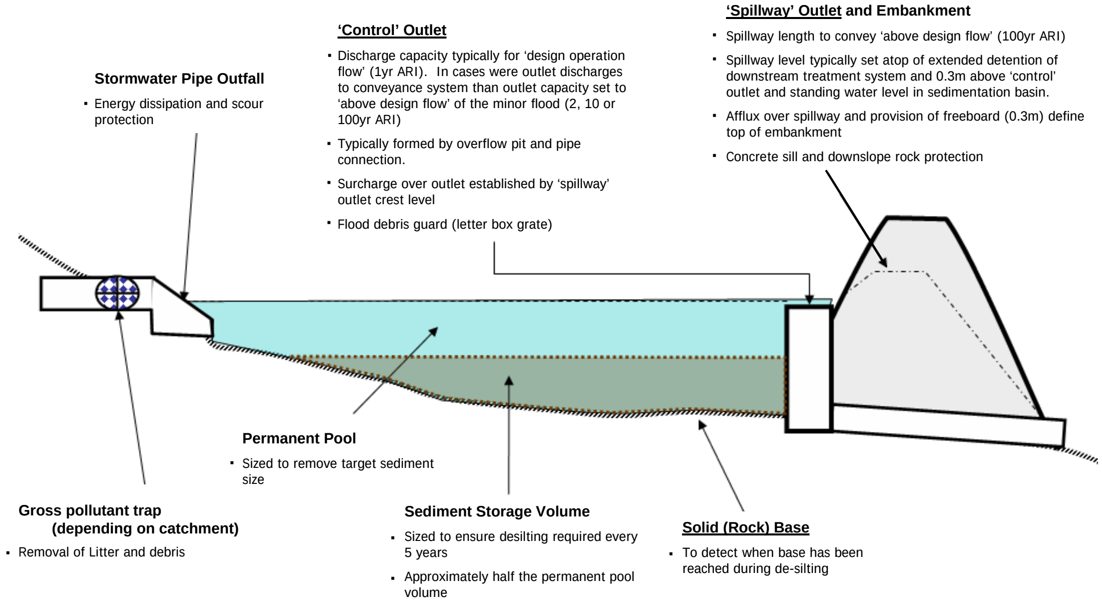


Figure 4-2: Sedimentation Basin Key Elements

4.2 Design Considerations

4.2.1 Role in the Treatment Train

Sedimentation basins have two keys roles when designed as part of a stormwater treatment train. The primary function is as a sedimentation basin to target coarse to medium sized sediment (i.e. 125 µm or larger) prior to waters entering the downstream treatment systems (e.g. macrophyte zone of a constructed wetland or a bioretention basin). This ensures the vegetation in the downstream treatment system is not smothered by coarse sediment and allows downstream treatment systems to target finer particulates, nutrients and other pollutants.

The second function is the control or regulation of flows entering the downstream treatment system during 'design operation' and 'above design' conditions. The outlet structures from the sedimentation basin are designed such that flows up to the 'design operation flow' (typically the 1 year ARI) enter the downstream treatment system, whereas 'above design flows' are bypassed around the downstream treatment system. In providing this function, the sedimentation basin protects the vegetation in the downstream treatment system against scour during high flows. The configuration of outlet structures within sedimentation basins depends on the design flows entering the basin and the type of treatment systems located downstream as described in Section 4.2.4.

Where the sedimentation basin forms part of a treatment train and when available space is constrained, it is important to ensure that the size of the sedimentation basin (i.e. inlet zone of a constructed wetland) is not reduced. This ensures the larger sediments are effectively trapped and prevented from smothering the downstream treatment system. If the site constrains the total area available for the treatment train, the downstream treatment system should be reduced accordingly.

4.2.2 Sizing a Sedimentation Basin

The required size of a sedimentation basin is calculated to match the settling velocity of a target sediment size with a design flow (typically 1 year ARI). Selecting a target sediment size is an important design consideration. As a pretreatment facility, it is recommended that particles of 125 μm or larger be the selected target sediment size because analysis of typical catchment sediment loads suggest that between 50 - 80 % of suspended solids conveyed in urban stormwater are 125 μm or larger. Almost all sediment bed loads are larger than this target sediment size.

Analysis of the characteristics of particulate nutrients and metals indicates that coarse to medium sized sediments (i.e. > 125 μm) have low concentrations of attached pollutants (e.g. nutrients, heavy metals) when compared to finer sediment and colloidal particles. Basins sized to target coarse to medium sized sediment are therefore expected to capture sediment that has low levels of contamination and is unlikely to require special handling and disposal. Removal of particles < 125 μm is best undertaken by treatment measures other than sedimentation basins (e.g. constructed wetlands and bioretention systems). Therefore, while a basin must have adequate size for capturing the target sediment size, they should not be grossly oversized. Conversely, a sedimentation basin that is too small could have limited effectiveness, resulting in sediment smothering of downstream treatment measures.

4.2.3 Sediment Storage

A further consideration in the design of a sedimentation basin is the provision of adequate storage for settled sediment to prevent the need for frequent desilting. A desirable frequency of basin desilting is once every five years (generally triggered when sediment accumulates to half the basin depth). The volume of accumulated sediment can be estimated from regular monitoring of sediment levels with a measuring post and reference against the top water level.

4.2.4 Outlet Design

An outlet structure of a sedimentation basin can be configured in many ways and is generally dependant on the design flow entering the basin and the type of stormwater treatment system or conveyance system downstream of its outlet. For example, a sedimentation basin forming the inlet zone of a constructed wetland (refer Chapter 6 – Constructed Wetlands), would typically include an overflow pit located within the sedimentation basin with one or more pipes connecting the sedimentation basin to an open water zone at the head of the wetland macrophyte zone. A sedimentation basin pretreating stormwater runoff entering a bioretention basin (refer Chapter 5 – Bioretention Basins) would typically use a weir outlet to keep stormwater flows at surface, to enable the flow to discharge onto the surface of the bioretention filter media. Where the sedimentation basin is formed by constructing an embankment across a drainage gully (such as shown on Figure 4-1), it may also be possible to use an overflow pit and pipe outlet and still be able to discharge to the bioretention surface.

In most cases, the outlet design of a sedimentation basin will consist of a 'control' outlet structure and a 'spillway' outlet structure:

- The 'control' outlet can be either an overflow pit/ pipe or weir which delivers flows up to the 'design operation flow' (Section 4.3.1) to the downstream treatment system(s).
- The 'spillway' outlet structure ensures that flows above the 'design operation flow' (Section 4.3.1) are discharged to a bypass channel or conveyance system. The 'spillway' bypass weir level is set above the 'control' outlet structure and typically at the top of the extended detention depth of the downstream treatment system.

Where the sedimentation basin discharges to a conveyance system (e.g. swale or piped system), a 'control' outlet may not be required and one outlet can be designed to allow discharge of all flows including flood flows.

4.2.5 Landscape Design

Sedimentation basins are often located within open space zones areas and can be landscaped to create a focal point for passive recreation. Landscape design treatments to sedimentation basins generally focus on dense littoral vegetation planting to restrict access to the open water zone, and therefore increase public safety, but can also include pathways and information signs. Plant species selection and placement should aim at creating a barrier to restrict public access to the open water zone and integrate with the surrounding landscape (i.e. constructed wetland landscape) and community character as discussed below, as well as providing or enhancing local habitat. Landscape design must also consider access to the sedimentation basin for maintenance (e.g. excavator).

4.2.6 Vegetation Specification

The role of vegetation in sedimentation basin design is to provide scour and erosion protection to the basin batters. In addition, dense planting of the littoral zones will restrict public access to the open water, reducing the potential safety risks posed by water bodies. Terrestrial planting may also be recommended to screen areas and provide a barrier to steeper batters.

Planting of the shallow marsh zone (to a depth of 0.2 m) and ephemeral marsh zone (to 0.2 m above water level) around the perimeter of a sedimentation basin is recommended to bind the bank and reduce erosion at the waters edge. Plant species should be selected based on the water level regime, soil types of the region, and the life histories, physiological and structural characteristics, natural distribution, and community groups of the plants. Appendix A (Plant Selection for WSUD Systems) provides a list of suggested plant species suitable for sedimentation basins. The planting densities recommended in the list should ensure that 70 – 80 % cover is achieved after two growing seasons (2 years).

Only the waters edge and batters of sedimentation basins should be planted and care needs to be taken in species selection to ensure vegetative growth will not spread to cover the deeper water zones. Similarly, floating or submerged macrophytes should be avoided. A sedimentation basin should primarily consist of open water to allow for settling of only the target sediments (e.g. > 125 µm) and to permit periodic sediment removal.

4.2.7 Maintenance

Sedimentation basins are designed with a sediment storage capacity to ensure sediment removal is only required approximately every 5 years (triggered when sediment accumulates to half the basin depth, determine from regular monitoring of sediment depth with a measuring post during maintenance visits). Accessibility for maintenance is an important design consideration. If an excavator is able to reach all parts of the basin from the top of the batter then an access ramp may not be required; however, an access track around the perimeter of the basin will be required and will affect the overall landscape design. If sediment collection requires earthmoving equipment to enter the basin, a stable ramp will be required into the base of the sedimentation basin (maximum slope 1:10).

It is recommended that a sedimentation basin is constructed with a hard (i.e. rock) bottom (with a bearing capacity to support maintenance machinery when access is required within the basin). This serves an important role by allowing excavator operators to detect when they have reached the base of the basin during desilting operations.

Provision to drain the sedimentation basin of water for maintenance must be considered, or alternatively a pump can be used to draw down the basin. Approvals must be obtained to discharge flows downstream or to sewer. Alternatively, a temporary structure (e.g. sand bags) can hold water upstream until maintenance is complete.

4.3 Design Process

The following sections detail the design steps required for sedimentation basins. Key design steps following the site planning and concept development stages are:

1. Determine design flows

↓

2. Confirm treatment performance of concept design

↓

3. Confirm size and dimension of the sedimentation basin

a. Sedimentation basin area
b. Storage volume for sediments
c. Internal batters

↓

4. Design of inflow systems

↓

5. Design outlet structure

a. Design of 'control' outlet – overflow pit/pipe outlet configuration
b. Design of 'control' outlet – weir configuration
c. Design of 'spillway' outlet – weir configuration

↓

6. Specify vegetation

↓

7. Consider maintenance requirements

4.3.1 Step 1: Determine Design Flows

4.3.1.1 Design Discharges

Two design discharges are required to size sedimentation basins and their structures:

- 'Design Operation Flow' (1 year ARI) for sizing the basin area and to size a 'control' outlet structure when discharging directly into a treatment system (e.g. wetland or bioretention system)
- 'Above Design Flow' for design of the 'spillway' outlet structure to allow for bypass of high flows around a downstream treatment system. Defined by either:
  - Minor design flow (2 to 10 year ARI) – required for situations where only the minor drainage system is directed to the sedimentation basin. Refer to relevant local government guidelines for the required design event for the minor design flow.
  - Major flood flow (50 to 100 year ARI) – required for situations where the major drainage system discharges into the sedimentation basin.

Where the sedimentation basin discharges to a conveyance system (e.g. open channel flow or piped drainage system), the 'Design Operation Flow' is only required to size the sedimentation basin, not for outlets from the system.

Sedimentation basins should not be designed to have high flows diverted around them. All flows should be directed through a sedimentation basin such that some level of sedimentation is achieved even during high flow conditions.

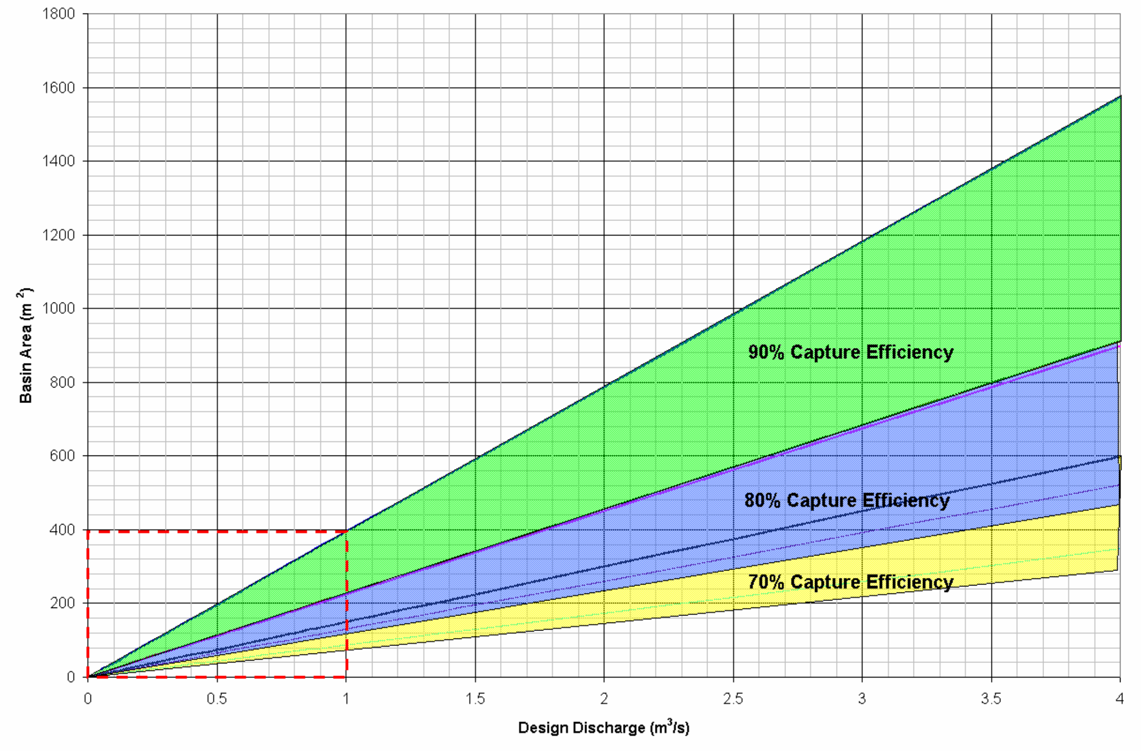
4.3.1.2 Design Flow Estimation

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 the most suitable method. For sediment basins with large catchments (> 50 ha), a runoff routing model should be used to estimate design flows.

4.3.2 Step 2: Confirm Treatment Performance of Concept Design

Figure 4-3 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 4.3.3). The influence of a permanent pool reduces flow velocities in the sedimentation basin and thus increases detention times (and hence removal efficiency). Therefore, a range of values are presented for 70%, 80% and 90% removal (shown as shaded bands), depending on permanent pool depths. A typical 2 m deep permanent pool was used to define the lower limit of the required sedimentation basin thus forming three shaded areas in the figure, with the upper limit being defined using no permanent pool.

The performance of typical designs of sedimentation basins can be expected to fall within the shaded curves shown and they can be used to estimate the size of the proposed sedimentation basin as part of conceptual design and to verify the size derived as part of Step 3. 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. However, sizing of sediment basins should be balanced with practicality and as such, extravagantly large basins should not be designed based primarily on long term storage of sediment. Design guidance for this sediment storage is provided in Section 4.3.3 (Step 3).



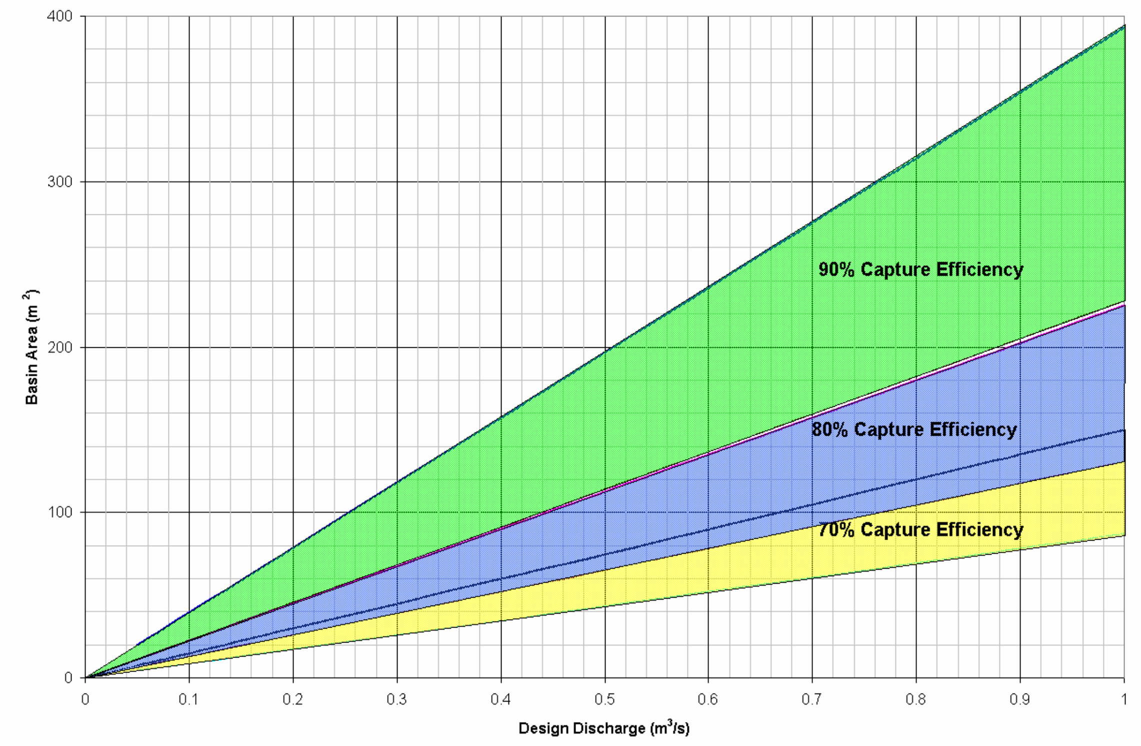


Figure 4-3: Sedimentation Basin Area vs Design Discharges for Varying Capture Efficiencies of 125 µm Sediment

4.3.3 Step 3: Confirm Size and Dimension the Sedimentation Basin

4.3.3.1 Sedimentation Basin Area

The required area (*A*) of a sedimentation basin should be defined through the use of the following expression (modified version of Fair and Geyer (1954)):

$$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)}\right]^{-n}$$ Equation 4.1

Where *R* = fraction of target sediment removed

v_s = settling velocity of target sediment (see Table 4.1)

Q/A = applied flow rate divided by basin surface area ($m^3/s/m^2$)

n = turbulence or short-circuiting parameter

d_e = extended detention depth (m) above permanent pool level

d_p = depth (m) of the permanent pool

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

The concept design stage will generally guide the selection of the fraction of target sediment removed (*R*) and permanent pool depth (d_p) depending on water quality objectives and the nature of local soils/ sediments. Table 4.1 lists the typical settling velocities (v_s) of sediments under 'ideal conditions' (velocity in standing water).

Table 4-1: Settling Velocities (vs) 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

Source: (Maryland Dept. of Environment 1987 in Engineers Australia 2006)

Equation 4.1 is applied with *n* being a turbulence parameter that is related to hydraulic efficiency (λ). Figure 4-4 provides guidance on estimating a hydraulic efficiency (λ) value that is then used to calculate an appropriate *n* value (according to the configuration of the basin). The shape of a basin has a large impact on the effectiveness of the basin to retain sediments. Generally, a length to width ratio of at least 3 to 1 should be achieved. In addition, the location of the inlet and outlet, flow 'spreaders' and internal baffles impact the hydraulic efficiency of the basin for stormwater treatment as the range of values in Figure 4-4 demonstrates. Figure 4-4 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. Sedimentation basins should be designed to have a λ value of not less than 0.5. 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).

Consideration of maintenance access to a basin is also required when developing the shape, 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 dewatering should also be provided, that drains back to the basin. This may impact on the footprint area required for a sedimentation basin system.

A value of n is estimated using the following relationship:

$$\lambda = 1 - 1/n;\ \text{so} \quad n = \frac{1}{1-\lambda} \qquad \text{Equation 4.2}$$

λ is estimated from the configuration of the basin according to Figure 4-4.

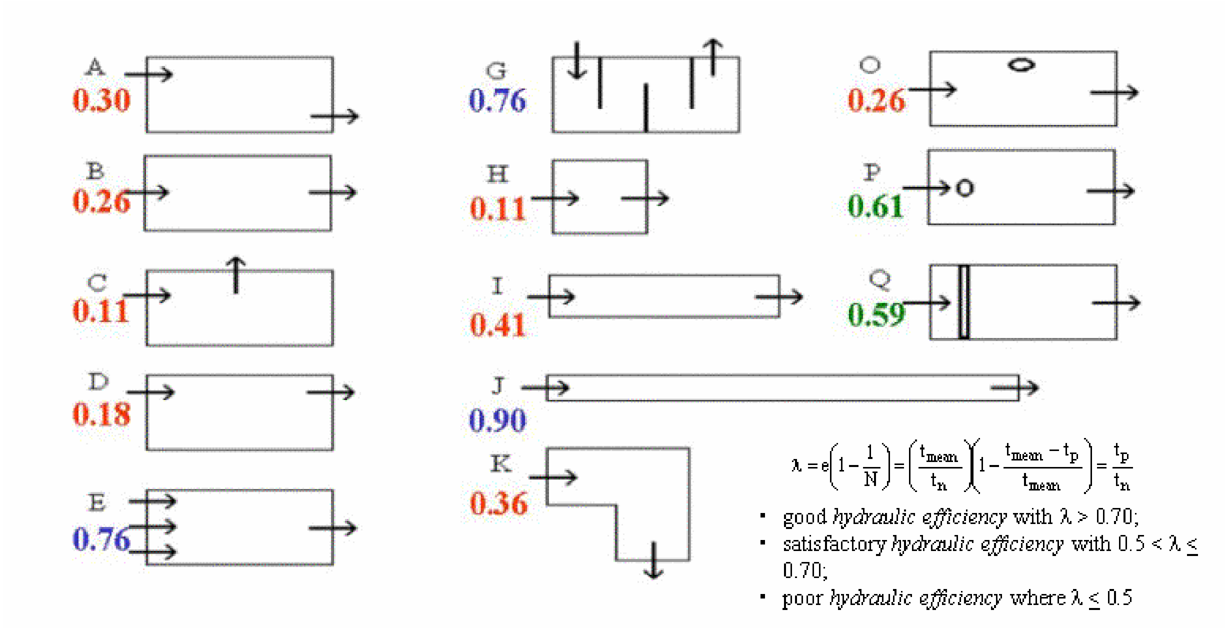


Figure 4-4: Hydraulic Efficiency, λ

Hydraulic efficiency ranges from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment. The o in diagrams O and P represent islands in the waterbody and the double line in diagram Q represents a weir structure to distribute flows evenly (Persson et al. 1999).

Good practice in the design of sedimentation basins is to 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. With the outlet structure being located some distance above the bed of a sedimentation basin, it is also not necessary for sediment particles to settle all the way to the bed of the basin to be effectively retained. It is envisaged that sediments need only settle to an effective depth (d^*) which is less than the depth to the bed of the sedimentation basin. This depth is considered to be approximately 1.0 m below the permanent pool level.

4.3.3.2 Storage Volume for Sediments

A further consideration in the design of a sedimentation basin is the provision of adequate storage for settled sediment to prevent the need for frequent desilting. A desirable frequency of basin desilting is once every five years (triggered when sediment accumulates to half the basin depth). To ensure this storage zone is appropriate the following must be met:

Sedimentation Basin Storage Volume (V_s) > Volume of accumulated sediment over 5 yrs ($V_{s:5yr}$)

The sedimentation basin storage volume (V_s) is defined as the storage available in the bottom half of the sedimentation basin permanent pool depth. V_s can be calculated using a product of the sedimentation basin area (A_s) and half the permanent pool depth (0.5 x d_p) and appropriate consideration of the internal batters (see Internal Batters below).

The volume of accumulated sediments over 5 years ($V_{s:5yr}$) is established by gaining an understanding of the sediment loads entering the sedimentation basin and applying the fraction of target sediment removed (R):

$$V_s = A_c \cdot R \cdot L_o \cdot F_c \qquad \text{Equation 4.3}$$

Where V_s = volume of sediment storage required (m^3)

A_c = contributing catchment area (ha)

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

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

F_c = desired cleanout frequency (years)

A catchment loading rate (L_o) of 1.6 m^3/ha/year for developed catchments can be used to estimate the sediment loads entering the basin.

4.3.3.3 Internal Batters

Batter slopes above and immediately below the water line of a basin should be configured with consideration of public safety and landscape integration. Both hard and soft edge treatments can be applied to compliment the landscape of a surrounding area. Soft edge treatments involve using gentle slopes to the waters edge (e.g. 1:8 to 1:10), extending below the water line for a distance (e.g. 2.4 m) before batter slopes steepen into deeper areas. This is illustrated in Figure 4-5.

Figure 4-6 shows an example of a hard edge treatment with a larger vertical wall and associated handrail for public safety

Plate 4-1: Example of Soft Edge Embankment Planting

In both edge treatments, it is recommended to line the bottom of the basin with rock to prevent vegetation (particularly weed) growth and to guide extraction depths during sediment removal (see Section 4.2.7).

The safety requirements for individual basins will vary from site to site, and it is recommended that developers engage an independent safety audit of each design. The *Sediment Basin Design, Construction and Maintenance Guidelines* (BCC 2001) requires the following:

- For water depths > 150 mm and maximum slope of 5:1 (H:V) or less, no fencing is required.
- For water depths > 150 mm and maximum slope > 5:1 (H:V) fencing is required.

Further guidance on landscape and public safety considerations for designing sediment basins is contained in Section 4.4.

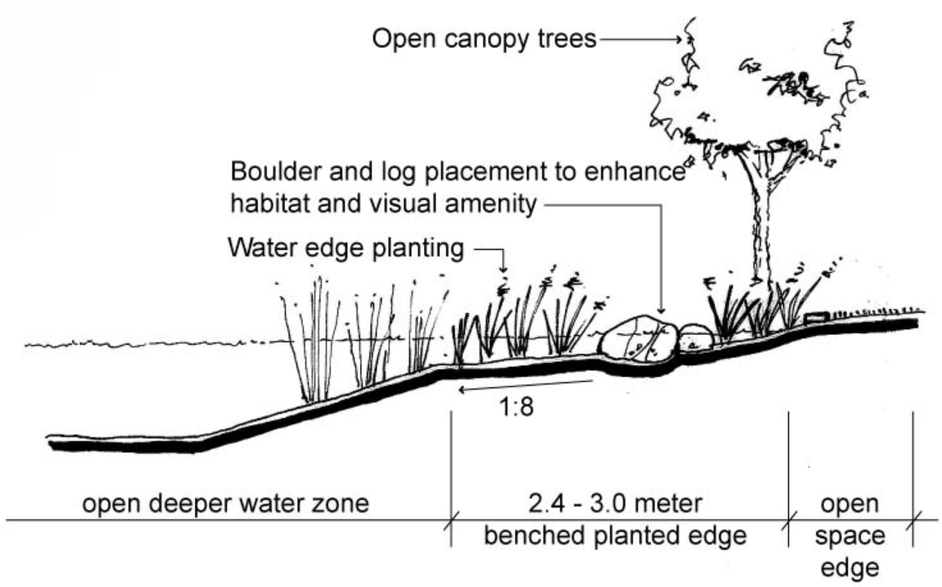


Figure 4-5: Illustration of a Soft Edge Treatment for Open Waterbodies (GBLA 2004)

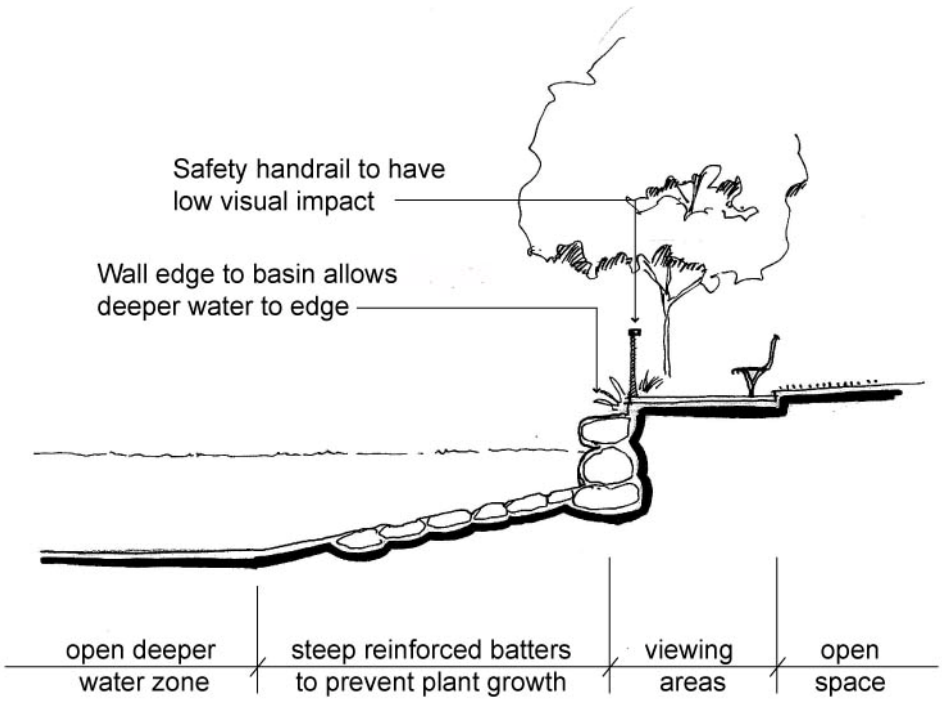


Figure 4-6: Illustration of Hard Edge Treatment for a Sediment Basin

Plate 4-2: Examples of Hard edge Treatment for Open Waterbodies

Additionally, the designs should be verified with the Building Code of Australia for compliance. An alternative to the adoption of a fence is to provide a 2.4 m 'safety bench' that is less than 0.2 m deep below the permanent pool level around the waterbody.

4.3.4 Step 4: Design Inflow Systems

Stormwater conveyed by a pipe or open channel would normally discharge directly into a sedimentation basin as this is often the first element of a stormwater treatment train. It will be necessary to ensure that inflow energy is adequately dissipated to prevent localised scour in the vicinity of a pipe or channel outlet.

Design of inlet structures for adequate scour protection is common hydraulic engineering practice and the reader is referred to standard hydraulic design handbooks for further guidance on design of scour prevention methods and appropriate sizing of energy dissipation structures (e.g. Henderson 1966; Chow 1959).

If conceptual design of the stormwater system identified the need to remove anthropogenic litter (i.e. industrial or commercial situations) then some form of gross pollutant trap (GPT) may be required 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 available for removing gross pollutants and these are discussed in Chapter 7 of *Australian Runoff Quality* (Engineers Australia 2006). The storage capacity of gross pollutant traps should be sized to ensure that maintenance (cleanout) frequency is not greater than once every 3 months.

4.3.5 Step 5: Design Outlet Structure

As outlined in Section 4.2.4, the outlet of a sedimentation basin can be configured in many ways and is generally dependant on the design flow entering the basin and the type of stormwater treatment system or conveyance system downstream of its outlet. In most cases, the outlet design of a sedimentation basin will consist of a 'control' outlet structure and a 'spillway' outlet structure:

- The 'control' outlet can be an overflow pit/ pipe or weir which delivers flows up to the 'design operation flow' (Section 4.3.1) to the downstream treatment systems.
- The 'spillway' outlet structure ensures that flows above the 'design operation flow' (Section 4.3.1) are discharged to a bypass channel or conveyance system.

Where the sedimentation basin discharges to a conveyance system (e.g. bioretention basin or piped system), a 'control' outlet may not be required and hence one outlet ('spillway' outlet) can be designed to allow discharge of all flows including flood flows.

Where the sedimentation basin is formed by constructing an embankment across a drainage gully (such as shown on Figure 4-1) it may also be possible to use an overflow pit and pipe outlet and still be able to discharge to a bioretention surface or wetland macrophyte zone.

4.3.5.1 Design of 'Control' Outlet - Overflow Pit and Pipe Outlet Configuration

For sedimentation basins that discharge directly to a treatment system (i.e. constructed wetland or bioretention basin) and the 'control' outlet structure discharging to the treatment system is an overflow pit and pipe, the following criteria apply:

- Ensure that the crest of the overflow pit is set at the permanent pool level of the sedimentation basin (which is typically a minimum of 0.3 m above the permanent water level of the downstream treatment system).
- The overflow pit is sized to convey the design operational flow (e.g. 1 year ARI). The dimension of an outlet pit is determined by considering two flow conditions: weir and orifice flow (Equations 4.4 and 4.5 below). Generally, the discharge pipe from the sedimentation basin (and downstream water levels) will control the maximum flow rate from the basin; it is therefore less critical if the outlet pit is oversized to allow for blockage.
- Provide protection against blockage by flood debris.

Plate 4-3: Debris screens in Coorparoo, Mill Park (Victoria) and Herston

The following equations apply to the design of 'control' outlet devices:

1 Weir flow condition – when free overfall conditions occur over the pit:

$$P = \frac{Q_{des}}{B \cdot C_w \cdot h^{3/2}} \qquad \text{Equation 4.4}$$

Where	P	=	perimeter of the outlet pit (m)
	B	=	blockage factor (0.5)
	h	=	depth of water above the crest of the outlet pit (m)
	Q_{des}	=	design discharge (m^3/s)
	C_w	=	weir coefficient (1.66)

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

$$A_o = \frac{Q_{des}}{B \cdot C_d \cdot \sqrt{2 \cdot g \cdot h}}$$ Equation 4.5

Where	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 a means of minimising potential blockage of the outlet structure.

The pipe that connects the sedimentation basin to the downstream treatment system (e.g. macrophyte zone of a constructed wetland or bioretention system) must have sufficient capacity to convey a 1 year ARI flow, assuming the downstream treatment system is at the permanent pool level of the sedimentation basin and without resulting in any flow in the bypass system. This ensures the majority of flows have the opportunity to enter the downstream treatment system before the bypass system is engaged. An energy dissipater is usually required at the end of the pipes to reduce velocities and distribute flows into the downstream treatment system.

If the outlet of the connection pipe is submerged, an energy loss equation can be used to estimate the pipe velocity using the following:

$$h = \frac{2 \cdot V^2}{2 \cdot g}$$ Equation 4.6

Where:	h	=	head level driving flow through the pipe (defined as the 'spillway' outlet level minus the normal water level in the downstream treatment system)
	V	=	pipe velocity (m/s)
	g	=	gravity (9.79 m/s^2)

Note: the coefficient of 2 in the equation is a conservative estimate of the sum of entry and exit loss coefficients (K_{in} + K_{out}).

The area of pipe required to convey the 'design operation flow' (1 year ARI) is then calculated by dividing the above 'design operation flow' by the velocity. Alternatively, if the pipe outlet is not fully submerged, the orifice equation should be used (Equation 4.5) to estimate the size of the connection pipe.

An example configuration of a sedimentation basin 'control' overflow pit and pipe outlet to the macrophyte zone of a constructed wetland is provided in Figure 4-7 (over page).

4.3.5.2 Design of 'Control' Outlet – Weir Configuration

The required length of the weir for 'control' outlet operation can be computed using the weir flow equation (Equation 4.4) and the 'design operation flow' (Section 4.3.1), adopting a blockage factor of 1.0 (as weir is unlikely to become blocked by debris).

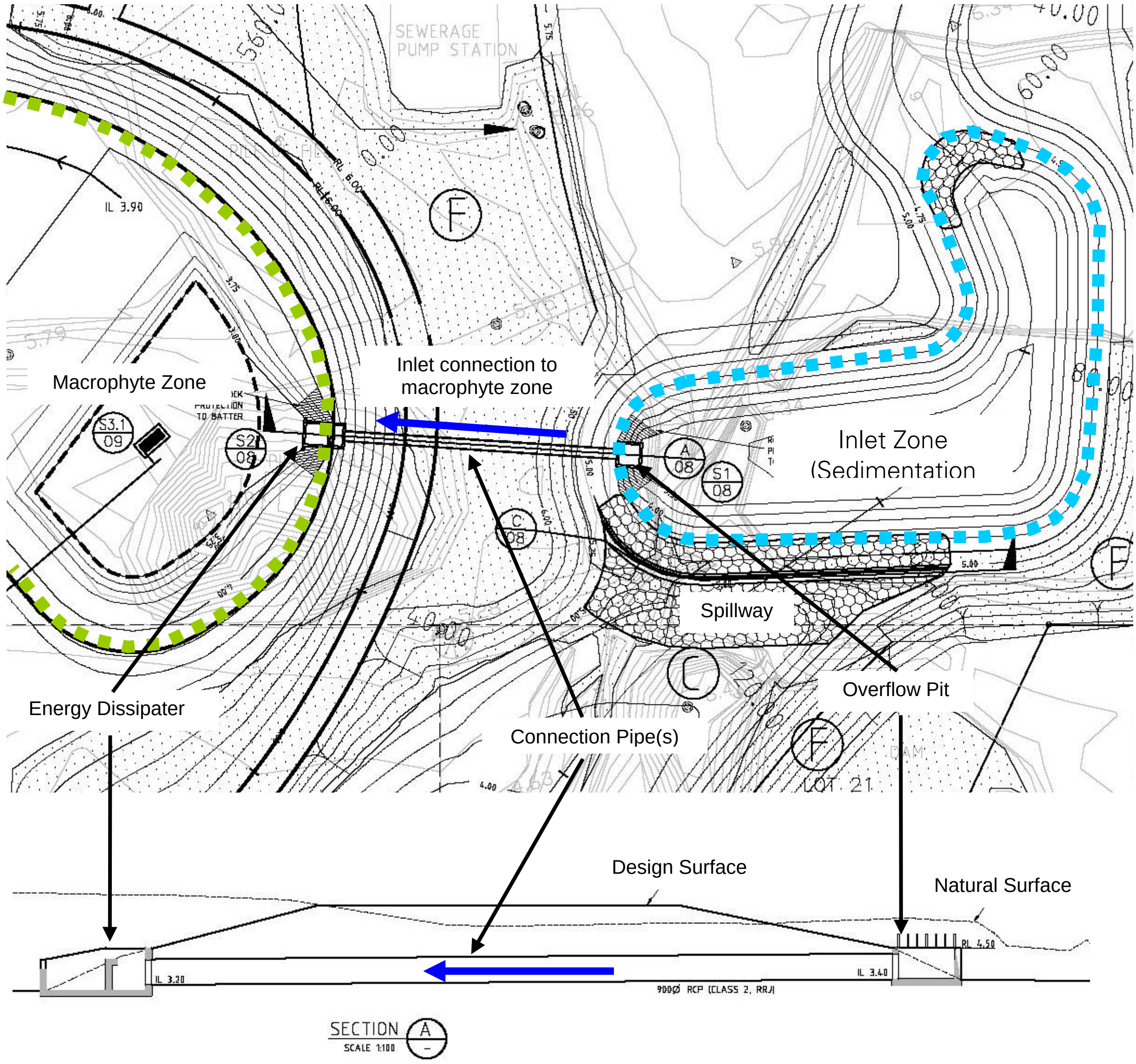


Figure 4-7: Example layout (top) of sedimentation basin 'control' overflow pit and pipe connection to a macrophyte zone and control overflow pit installation (bottom)

4.3.5.3 Design Of 'Spillway' Outlet – Weir Configuration

In most applications the 'spillway' outlet weir will form part of the high flow bypass system, which protects the downstream treatment system from scouring during 'above design' storm flows. Ideally, the 'spillway' outlet weir level should be set at the top of the extended detention level of the downstream treatment system. This ensures that a significant proportion of catchment inflow will bypass the downstream treatment system once the extended detention is filled. The length of the 'spillway' outlet weir is to be sized to safely pass the maximum flow discharged into the downstream treatment system (as defined by the 'above design flow' in Section 4.3.1). The water level above the crest of the bypass weir is 0.3 m below the embankment crest separating the sedimentation basin and the downstream treatment system.

The required length of the 'spillway' outlet weir can be computed using the weir flow equation (Equation 4.4 with blockage factor equal to 1.0) and the 'above design flow' (Section 4.3.1). Plate 4-4 shows examples of 'spillway' weir outlets. The 'spillway' outlet weir should be designed using standard methods to avoid scour and erosion. Typically, a concrete sill is required with rock protection on the downslope sides of the sill.

Plate 4-4: Spillway outlet weir structure of sedimentation basins at the Gold Coast and Coorparoo

4.3.6 Step 6: Specify Vegetation

Refer to Section 4.4 and Appendix A for advice on selecting suitable plant species for planting of the littoral zones around sedimentation basins.

4.3.7 Step 7: Consider Maintenance Requirements

Consider how maintenance is to be performed on the sediment basin (e.g. how and where is access available, where is litter likely to collect etc.). A specific maintenance plan and schedule should be developed for the basin, either as part of a maintenance plan for the whole treatment train, or for each individual asset. Guidance on maintenance plans is provided in Section

4.3.8 Design Calculation Summary

Below is a design calculation summary sheet for the key design elements of a sedimentation basin to aid the design process.

SEDIMENTATION BASIN DESIGN CALCULATION SUMMARY			
	Calculation Task	CALCULATION SUMMARY	
		Outcome	Check
	Catchment Characteristics		
	Residential	Ha	
	Commercial	Ha	
	Roads	Ha	
	Storm event entering inlet pond (minor or major)	yr ARI	
	Conceptual Design		
	Notional permanent pool depth	m	
	Permanent pool level of sedimentation basin	m AHD	
1	**Determine design flows**		
	'Design operation flow' (1 year ARI)	year ARI	
	'Above design flow' (2 to 100 year ARI)	year ARI	
	Time of concentration		
	Refer to relevant Local Government Guidelines and QUDM	minutes	
	Identify rainfall intensities		
	'Design operation flow' - $I_{1 \text{ year ARI}}$	mm/hr	
	'Above design flow'- $I_{2 \text{ year ARI}}$ to $I_{100 \text{ year ARI}}$	mm/hr	
	Design runoff coefficient		
	'Design operation flow' - $C_{1 \text{ year ARI}}$		
	'Above design flow'- $I_{2 \text{ year ARI}}$ to $I_{100 \text{ year ARI}}$		
	Peak design flows		
	'Design operation flow' - 1 year ARI	m^3/s	
	'Above design flow' – 2 to 100 year ARI	m^3/s	
2	**Confirm Treatment Performance of Concept Design**		
	Capture efficiency (of 125 µm sediment)	%	
	Area of sedimentation basin	m^2	
3	**Confirm size and dimension of sedimentation basin**		
	Inlet zone size		
	Area of sedimentation basin	m^2	
	Aspect ratio	L:W	
	Hydraulic efficiency		
	Depth of permanent pool	m	
	Storage volume for sediments		
	Sedimentation basin storage Volume V_s	m^3	
	Volume of accumulated sediment over 5 years ($V_{s:5yr}$)	m^3	
	$V_s > V_{s:5yr}$		
	Sediment cleanout frequency	years	
	Internal batters		
	Edge batter slope	V:H	
	Fence required		
4	**Design inflow systems**		
	Provision of scour protection or energy dissipation		
5	**Design outlet structures**		
	Design of 'control' outlet - overflow pit and pipe outlet configuration		
	Overflow pit crest level	m AHD	
	Overflow pit dimension	L x W	
	Provision of debris trap		
	Connection pipe dimension	mm diam	
	Connection pipe invert level	m AHD	

SEDIMENTATION BASIN DESIGN CALCULATION SUMMARY			
Calculation Task	CALCULATION SUMMARY		
	Outcome		Check
Design of 'control' outlet - weir configuration			
Weir crest level		m AHD	
Weir length		m	
Design of 'spillway' outlet - weir configuration			
Weir crest level		m AHD	
Weir length		m	
Depth above spillway		m	
Freeboard to top of embankment		m	

4.4 Landscape Design Notes

The successful landscape design and integration of sedimentation basins within open space and parkland areas will ensure that visual amenity, environment, habitat, community safety and stormwater quality are all enhanced.

Within a constructed wetland treatment system, sedimentation basins provide a transition between urbanised streams – possibly piped or channelised – that may have limited access, and natural wetland systems within accessible parkland. They are located at the highest point of a constructed wetland and may provide viewing opportunities across the wetland.

Sedimentation basins are a potential place for community education (through signage and other interpretative elements) as they are large and visible (and perhaps part of a larger constructed wetland). In addition, they may be the first place in an urban water catchment where treatment takes place. They therefore make good locations to tell the story of stormwater treatment processes.

Landscape design has a key role in overcoming negative perceptions that permanent water bodies like sedimentation basins have in some communities. In the past this may have been due to legitimate pest and safety concerns that have arisen from poorly designed and/ or managed systems, particularly remnant swamps and lagoons. Additionally, these older systems may have provided poor amenity values to the community due to lack of access or industrial scale treatment infrastructure.

4.4.1 Objectives

Landscape design for sedimentation basins has five key objectives:

- Addressing stormwater quality objectives by applying adequate edge and littoral zone planting to prevent scour and erosion of batters while ensuring an unvegetated open water pool is retained.
- Addressing public safety issues by ensuring the landscape design and edge treatments restrict public access to the open water zone and allow egress where appropriate.
- Ensuring that the overall landscape design of the sedimentation basin integrates with its host natural and/ or built environment and compliments the landscape design of adjacent treatment measures (e.g. constructed wetlands or bioretention basins).
- Incorporating Crime Prevention through Environmental Design (CPTED) principles.
- Providing other landscape values, such as shade, amenity, habitat, character and place making.

4.4.2 Context and Site Analysis

The sedimentation basin can provide a positive landscape environment and needs to be responsive to the site for this to be maximized. Existing features such as slope, vegetation, waterways and soils need to be considered in planning layouts and locations when designing within constrained sites. Other factors like road layout, buildings, driveways and services can also affect layouts. With appropriate landscape design, sedimentation basins can become interesting features in the local community. Their location and function provide opportunities to view large volumes of flowing water during and just after storm events, and to observe wildlife adapted to lagoon-like environments such as cormorants, kingfishers, turtles and eels. Sedimentation basins provide an interface between fast flowing, shallow, energetic water and deep, slow and serene water. This dynamism can be exploited to provide significant place making opportunities.

Comprehensive site analysis should inform the landscape design as well as road layouts, civil works and maintenance requirements. Existing site factors such as roads, driveways, buildings, landforms, soils, plants, microclimates, services and views should be considered. Refer to *Water Sensitive Urban Design in the Sydney Region: 'Practice Note 2 – Site Planning'* (LHCCREMS 2002) for further guidance.

4.4.3 Specific Landscape Considerations

Opportunities are available for creative design solutions to specific elements. Close collaboration between the landscape designer, ecologist, hydraulic designer, civil/ structural engineer and maintenance personnel is essential. In parklands and residential areas, a key aim is to ensure elements are sympathetic to their surroundings and are not overly engineered or industrial in style and appearance whilst achieving their desired functions. Additionally, landscape design to specific elements should aim to create places where local residents and visitors will come to enjoy and regard as an asset.

4.4.3.1 Basin Siting and Shapes

Through integrated landscape design, sedimentation basins can become important features within open space areas. Areas of open water provide passive viewing opportunities for plants and wildlife that have adapted to the urban lagoon landscapes. By siting basins such that inlet structures create dramatic "water features" in a highly visible area during high flows, basins can create invigorating large-scale urban environments (see Figure 4-8). Often the sedimentation basins are part of a broader treatment train and generally form the first part of wetlands. This allows the integrated landscape design of habitat renewal and open water vistas with public and recreational areas. This often can be part of broader community education strategy, for the role of sedimentation basins, through appropriate interpretive signage outlining both the natural habitat and water quality benefits.

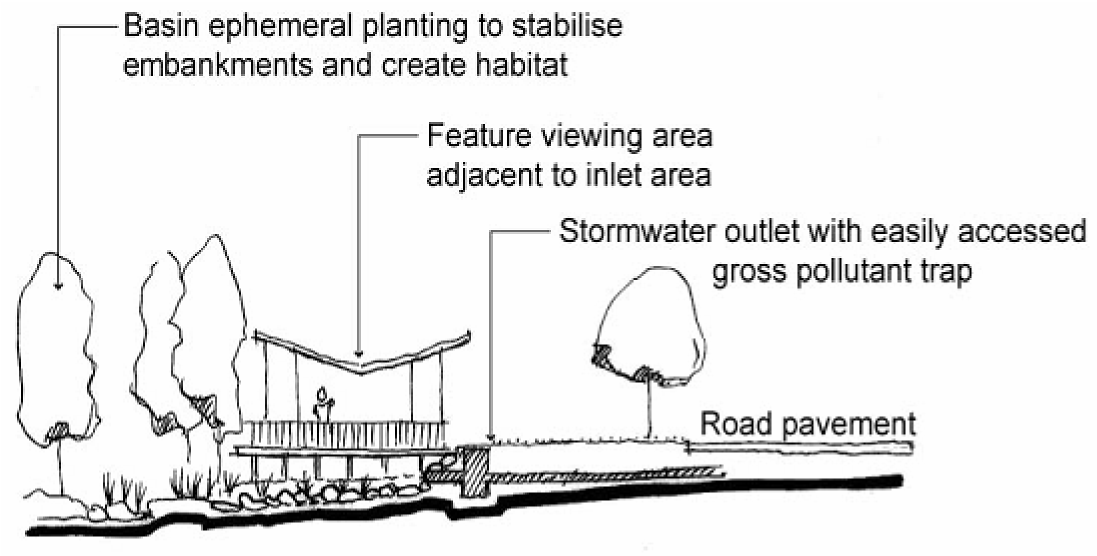


Figure 4-8: Typical Section Through Feature Inlet Structure

Basins shapes can vary widely and need to be primarily responsive to the hydraulic engineers length to width ratio, depths and inlet requirements. The landscape designer has the opportunity to shape the basin to respond to adjacent land uses (i.e. recreational spaces, local landforms and existing features). This often can result in "natural" informal shapes that provide visually aesthetic landscape outcomes. Embankments and batter profiles play an important role in providing an interesting and functional water body.

The length to width ratio of the basin should be determined by the hydraulic designer working within the site constraints (refer to section 4.3.2). Once the overall shape has been determined, one of the first considerations should be if a formal or informal style is required depending on setting. Figure 4-9 illustrates formal and informal options for a given length to width ratio.

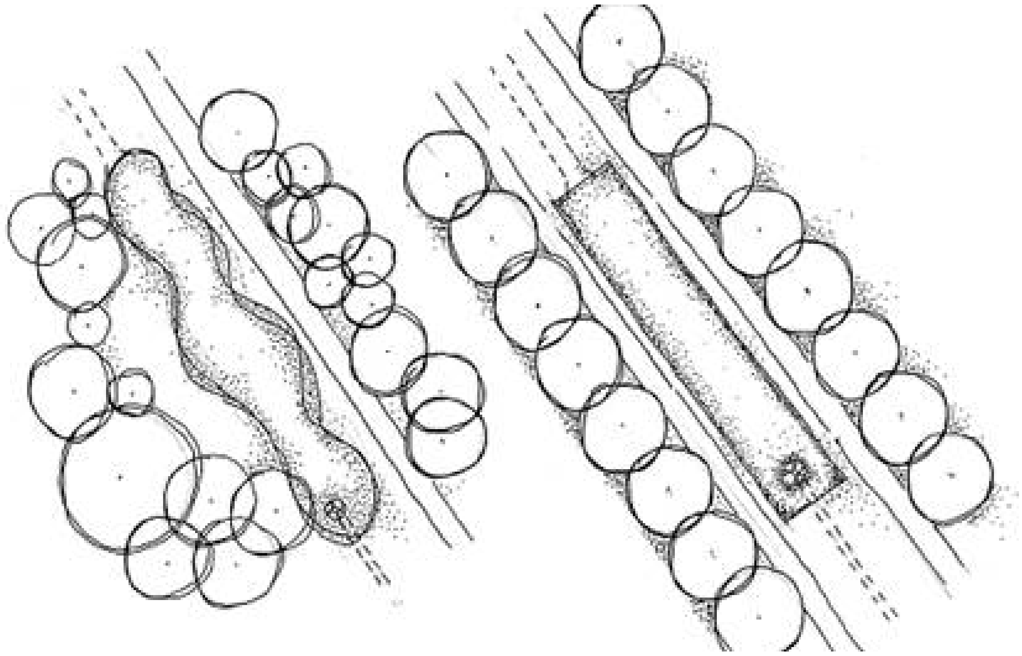


Figure 4-9: Informal and Formal Basin Configuration Given Length to Width Ratio

4.4.3.2 Basin Embankment

Where a natural look is required, the designer should explore opportunities for landform grading to the embankment to create variation in the slope. Geometric planar batters should be avoided. The grading approach also creates a diversity of habitat niches along the slope and can assist in reducing erosion. Figure 4-10 illustrates this technique. It is important that shaping to the slope does not allow areas for mosquitos to breed such as isolated areas of stagnant water. Designing to avoid mosquitos is discussed in detail in Chapter 6 (Section 6.2.8) with respect to constructed wetlands.

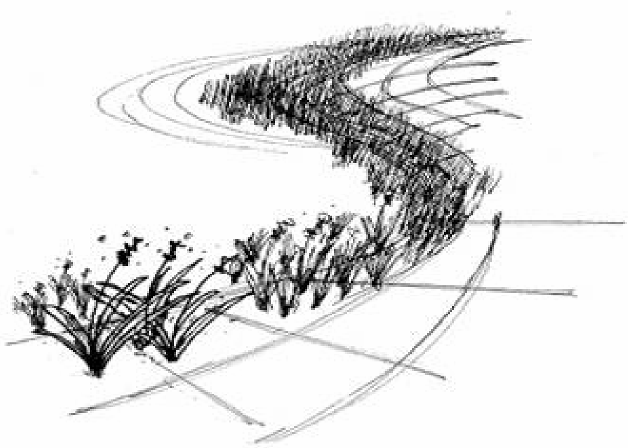


Figure 4-10: Conceptual Landform Grading of Embankment to Waters Edge

4.4.3.3 Edge Treatments and Profiles

In coordination with hydraulic engineers, the landscape design and grading of embankment batters allows a variety of edge treatments and opportunities.

The edge treatments that maximise natural habitats for flora and fauna can be facilitated by slopes flatter than 1:5 and often have benched foreshores with shallow standing water levels. This allows for safe egress from the basin. Rock edges and 'beaches' can also provide interest at key viewing areas and aid in providing further localised habitat and visual interest. This can be seen in Figure 4-5.

For areas where public access is to be restricted, batter slopes can be steeper than 1:5, and require safety fencing to restrict access. This typical treatment can also include a wall to further maximise deep, clear water. This can be seen in Figure 4-6.

Planting of sedimentation basin edges requires analysis of several issues including water depths and variances, soil and basin topsoil types, batter profiles, public access and habitat rehabilitation. For further information refer Section 4.4.4 Appropriate Plant Selection.

4.4.3.4 Basin Inlet

The basin inlet is an important place to experience the confluence of fast flowing water with still water and is a dynamic place within the local landscape. Designers have scope to approach this element in a variety of ways provided the hydraulic design is not compromised. Options to consider include:

- Using salvaged site rocks or patterned and coloured concrete to emphasise the feature and create niche habitats.
- Enhancing the microclimate created by cool running water by adding shade trees.
- Creating places to view running water. Where suitable, this can be achieved with footbridges located above the water. Such structures should be designed appropriately with consideration of life cycle costs (i.e. timber piers should not be used where contact with water occurs). Alternatively, views from the side will provide a different experience. Viewing areas should be located a minimum of 5 m from the open water body to discourage wildlife feeding.

4.4.3.5 Sediment Removal Access

As part of the siting and layout of sedimentation basins, suitable access from an adjacent roadway needs to be provided to periodically remove sediment. The landscape design of these access ramps needs to consider the visual impact created in the landscape and how this can be minimised. Access to the basin floor to remove sediments requires either the installation of a ramp/ ramps, or an access track around the perimeter for smaller basins (refer section 4.2.7). These elements are crucial to the operation of a sedimentation basin, but should be designed sensitively so they do not become visually prominent.

For both ramps and perimeter access tracks, reinforced turfing pavers should be considered as the pavement to create a green surface that blends with the surrounding plantings. Surfaces of concrete or rock should be avoided where possible. Consideration must be given to the size and weight of machinery likely to utilise the access ramp. Reinforced vegetated surfaces should be able to respond to impacts given that desilting of the basin will only be required approximately every 5 years.

Consider incorporating the sediment removal access into other landscape elements. For example, perimeter access tracks could also be used as recreational trails (in this case part of the track width could be paved using reinforced concrete). Investigate if the weir could become part of this access way. Ramps may potentially be integrated with viewing areas.

Trees and shrubs can be employed to screen these elements. The shadow cast by trees also assists in breaking up the form of linear structures so that they blend into formally designed landscapes.

Where gates and fences are required, it is important to use materials and styles that are sensitive to the setting. Products aimed for industrial applications should generally be avoided in parkland spaces, as should products designed for domestic garden situations.

4.4.3.6 Overflow Pit

Grates to overflow pits need to be designed to minimize visual impact on the landscape. The grate above the overflow pit can become an interesting local landmark, particularly if it is sited within the open water surface. Provided that the grate performs its intended function of preventing blockages by debris (refer section 4.3.5) and is structurally sound, there are opportunities for creative design solutions to this component. An important consideration is to prevent local fauna (e.g. ducks) from entering the overflow pit and becoming trapped. Investigate installing 200 mm wide perforated plates (holes to 20 mm) or similar at the base of the grate.

4.4.3.7 Weir Outlets

Weir outlets may be large items that can potentially add character to the design. Grouted rock wall or off-form concrete finishes should be investigated rather than loose dumped rocks, particularly where the weir is visible. Loose rock fill structures create glare, weed and cane toad issues. Alternatives to consider include rock pitched concrete with planting pockets to soften the visual impact of reinforces weirs. Refer to for a typical treatment in Figure 4-11.

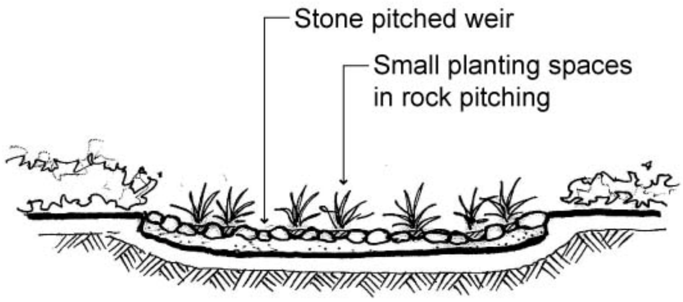


Figure 4-11: Typical weir outlet with Planting Areas

4.4.3.8 Viewing Area

In parkland areas, turfed spaces within barrier fencing offer a simple low maintenance solution. Figure 4-12 provides illustrations. Constructed decks may be appropriate in more urbanised areas. Hardwood timber construction should generally be avoided due to its inherent life-cycle costs.

Viewing areas should be located with a minimum distance of 5 m separating the viewing area from the waterbody, so that wildlife feeding is discouraged.

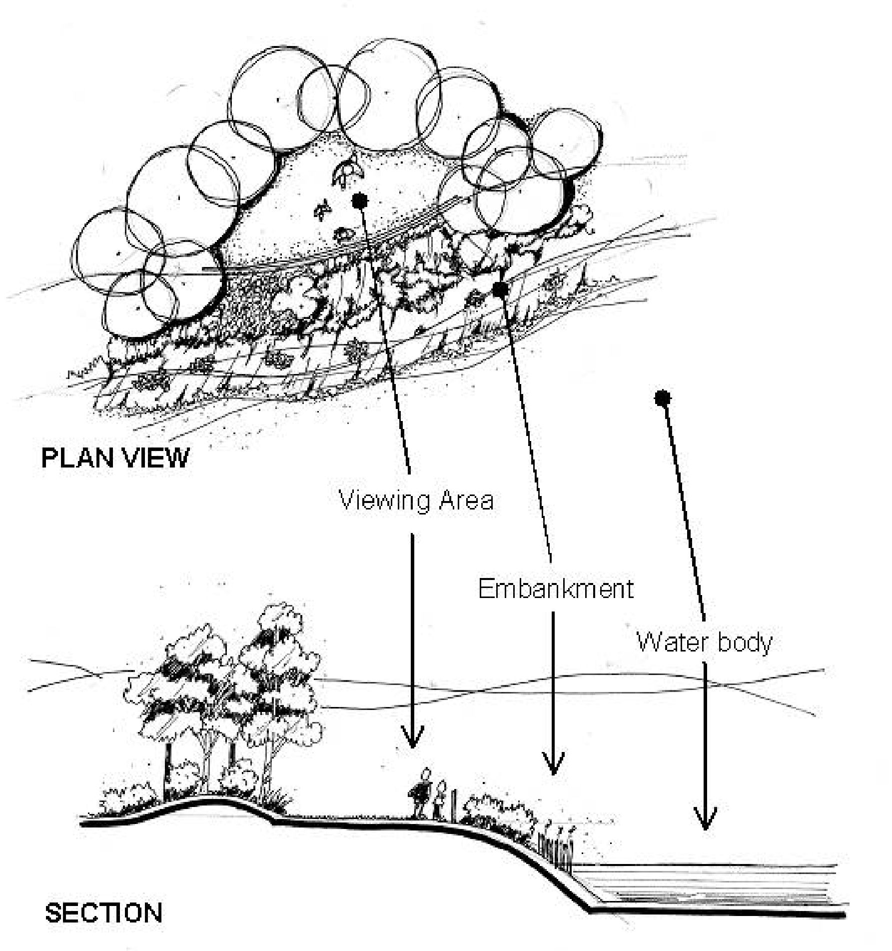


Figure 4-12: Turfed Viewing Areas with Barrier Fencing and Planting

4.4.3.9 Fencing

Where fences are required to sedimentation basin embankment edges, layout and design of fencing is important in creating an overall attractive landscape solution. Fence styles need to respond to functional requirements but also the contextual setting of the sedimentation basin i.e. if it's an urban residential or open space/ parkland area.

If fences are used, consider styles suitable for parkland and urban/ suburban contexts. Products designed for domestic gardens or industrial applications should generally be avoided. Fence types are similar to manufactured pool safety fences to relevant Australian standards. By specifying a black finish, and allowing for a screening garden in front of fences, the visual impact can be greatly reduced. Further safety issues are discussed in Section 5.4).

Plate 4-5: Typical examples of safety fencing to water edge

4.4.3.10 Signage and Interpretation

All signage and artwork proposed for public information must be approved by the relevant local government. Signage is an important part of educating the general public on the positive benefits of WSUD strategies. It can be based on stormwater quality information but also educate on waterways, habitat created, local fauna and flora. The following key issues and considerations need to be part of the signage strategy:

- Signage where possible should be kept simple and easy to interpret. Detailed design plans and system flow charts should be avoided, as these are often difficult to understand. Artistic illustrations may be used to explain processes. Text should be kept to a minimum. Annotated photographs or sketches are a more effective way of explaining processes;
- Signage location should take into account pathway networks, designated feature "people places" and locality to key areas requiring interpretive signage;
- Signage materials need to be low maintenance and durable, resistant to UV and graffiti and be easily installed.

Plate 4-6: Annotated sketches/ photographs are an effective way of explaining treatment process to the public

4.4.3.11 Baffles and Flow Spreaders

Within highly visible parkland and urban settings, investigate the use of interesting forms, patterns and colours that still achieve the desired function. For example, off-form concrete patterning, artwork to downstream side, coloured concrete, or organic shapes could be employed.

4.4.4 Sedimentation Basin Vegetation

Planting for sedimentation basins may consist of up to three vegetation types:

- Marsh zone planting (from 0.2 m below design water level to 0.2 m above).
- Embankment vegetation (greater than 0.2 m above design water level).
- Parkland plants, including existing vegetation, adjacent to the embankment edge.

4.4.4.1 Marsh Zone Planting (from –0.2 m to +0.2 m)

Plant selection for sedimentation basin edges need to respond to both edge profiles, water depths and functionality. This typically is between where seasonal water level changes will occur. Generally the edge planting should aid in stormwater quality improvement and provide aquatic habitat. Appendix A provides guidance on selecting suitable plant species and cultivators that deliver the desired stormwater quality objectives for sedimentation basins. In general, vegetation should provide:

- Scour and erosion protection to the basin embankment
- A buffer between water body and parkland that inhibits access.

4.4.4.2 Embankment (above +0.2m) and Open Space Vegetation

The battered embankment and fringing vegetation to open space or urban areas is important in providing soil stability, screening, habitat, visual amenity and interest. Between the marsh zone and the top of the embankment, trees, shrubs and groundcovers can be selected. Some key consideration when selecting appropriate sedimentation basin embankment planting include:

- Selecting locally endemic groundcovers, particularly for slopes greater than 1 in 3 with erodable soils, with matting or rhizomataceous root systems to assist in binding the soil surface during the establishment phase. Examples include *Imperata cylindrica, Lomandra sp.* and *Cyndocaton sp.*
- Preventing marsh zone plants from being shaded out by planting to ensure an open canopy, minimising tree densities at the waters edge and choosing species such as Melaleuca that allow sunlight to penetrate the tree canopy.
- Allowing excavators and other vehicles access to the water body for sediment removal purposes (refer to Section 4.4.3.4 below for further guidance).
- Locating and selecting species that in key view areas are below 1.0m high and form a dense habitat to discourage public access to the water edge.
- Screening planting that provides interest in form and colour, screens fences where applicable and are locally endemic.

Open space vegetation may be of a similar species and layout to visually integrate the sedimentation basin with its surrounds. Alternatively, vegetation of a contrasting species and/ or layout may be selected to highlight the water body as a feature within the landscape. Turf is an ideal consideration for accessible open space.

A wide range of species is at the designer's disposal depending on the desired scheme. *Growing Native Plants in Brisbane* (BCC 2005 on-line), *Successful Gardening in Warm Climates* (McFarlane 1997) and other contemporary publications give further guidance.

4.4.5 Safety Issues

4.4.5.1 Crime Prevention Through Environmental Design (CPTED)

The standard principles of informal surveillance, exclusion of places of concealment and open visible areas apply to the landscape design of sedimentation basins. Where planting may create places of concealment or hinder informal surveillance, groundcovers and shrubs should not generally exceed 1 m in height. For specific guidance on CPTED requirements the designer should refer to relevant local authority guidelines.

4.4.5.2 Restricting Access to Open Water

Fences or vegetation barriers to restrict access should be incorporated into sediment basin areas, particularly on top of concrete or stone walls where:

- There is risk of serious injury in the event of a fall (over 0.5 m high and too steep to comfortably walk up/ down or the lower surface or has sharp or jagged edges).
- There is a high pedestrian or vehicular exposure (on footpaths, near bikeways, near playing/ sporting fields, near swings and playgrounds etc).

- Water ponds to a depth of greater than 300 mm on a constructed surface of concrete or stone. Natural water features are exempt.
- Water is expected to contain concentrated pollutants.
- Grassed areas requiring mowing abut the asset.

Fences considered appropriate are:

- Pool fences in accordance with Australian Standards (for areas adjacent to playgrounds/ sports fields where a child drowning or infection hazard is present).
- Galvanised tubular handrails (without chain wire) in other areas.
- Dense vegetative hedges.

Dense littoral planting around the sedimentation basin (with the exception of any maintenance access and dewatering areas) will deter public access to the open water and create a barrier to improve public safety. Careful selection of plant species (e.g. tall, dense or spiky species) and planting layouts can improve safety as well as preventing damage to the vegetation by trampling.

Dense vegetation (hedge) at least 2 m wide and 1.2 m high (minimum) may be suitable if vandalism is not a demonstrated concern (this may be shown during the initial 12 month maintenance period). A temporary fence (e.g. 1.2 m high silt fence) will be required until the vegetation has established and becomes a deterrent to pedestrians/ cyclists.

An alternative to the adoption of a barrier/ fence is to provide a 2.4 m 'safety bench' that is less than 0.2 m deep below the permanent pool level around the waterbody. This is discussed in Section 4.3.3 with respect to appropriate batter slopes.

4.5 Construction and Establishment Advice

This section provides general advice for the construction and establishment of sedimentation basins and key issues to be considered to ensure their successful establishment and operation. Some of the issues raised have been discussed in other sections of this chapter and are reiterated here to emphasise their importance based on observations from construction projects around Australia.

4.5.1 Staged Construction and Establishment Method

It is important to note that delivering sedimentation basins can be a challenging task in the context of a large development site and associated construction and building works. Therefore, sedimentation basins require a careful construction and establishment approach to ensure the wetland establishes in accordance with its design intent. The following sections outline a recommended staged construction and establishment methodology for sedimentation basins based on the methods presented in Leinster (2006).

4.5.1.1 Construction and Establishment Challenges

There exist a number of challenges that must be appropriately considered to ensure successful construction and establishment of sedimentation basin. These challenges are best described in the context of the typical phases in the development of a Greenfield or Infill development, namely the Subdivision Construction Phase and the Building Phase (see Figure 4-13).

- <u>Subdivision Construction</u> - Involves the civil works required to create the landforms associated with a development and install the related services (roads, water, sewerage, power etc.) followed by the landscape works to create the softscape, streetscape and parkscape features. The risks to successful construction and establishment of the WSUD systems during this phase of work have generally related to the following:
  - Construction activities which can generate large sediment loads in runoff
  - Construction traffic and other works can result in damage to the sedimentation basins.

  Importantly, all works undertaken during Subdivision Construction are normally 'controlled' through the principle contractor and site manager. This means the risks described above can be readily managed through appropriate guidance and supervision.

- Building Phase - Once the Subdivision Construction works are complete and the development plans are sealed then the Building Phase can commence (i.e. construction of the houses or built form). This phase of development is effectively 'uncontrolled' due to the number of building contractors and sub-contractors present on any given allotment. For this reason the Allotment Building Phase represents the greatest risk to the successful establishment of sedimentation basins.

4.5.1.2 Staged Construction and Establishment Method

To overcome the challenges associated within delivering sedimentation basins a Staged Construction and Establishment Method should be adopted (see Figure 4-13):

- Stage 1: Functional Installation - Construction of the functional elements of the sedimentation basin as part of the Subdivision Construction and allowing the basin to form part of the sediment and erosion control strategy.
- Stage 2: Sediment and Erosion Control – During the Building Phase the sedimentation basin will form part of the sediment and erosion control strategy to protect downstream aquatic ecosystems.
- Stage 3: Operational Establishment - At the completion of the Building Phase, the sedimentation basins can be desilted to establish the design bathymetry and landscaped.

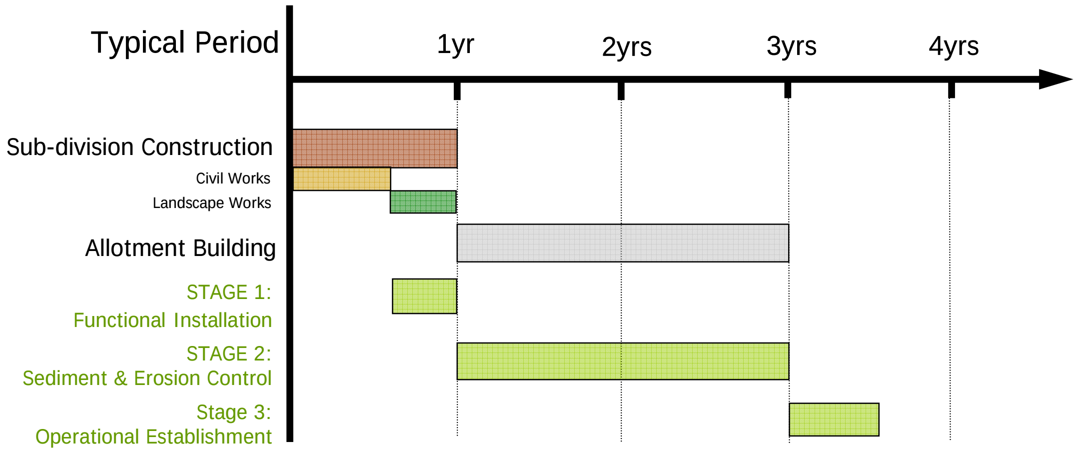


Figure 4-13: Staged Construction and Establishment Method

4.5.2 Construction Tolerances

It is important to emphasise the significance of tolerances in the construction of sedimentation basins. Ensuring the relative levels of the control structures are correct is particularly important to achieve appropriate hydraulic functions. Generally control structure tolerance of plus or minus 5 mm is considered acceptable.

Additionally the bathymetry of the sedimentation basin must ensure appropriate storage is available for accumulated sediment. In this regarding an earthworks tolerance of plus or minus 25 mm is considered acceptable.

4.5.3 Sourcing Sedimentation Basin Vegetation

In the majority of cases, the sedimentation basin will form an inlet pond to a constructed wetland or bioretention basin. In such cases, the landscape and vegetation design of the sedimentation basin will be undertaken in conjunction with the vegetation design of the other treatment measures and hence ordering of plant stock can be combined into one order. The species listed in Table A-2 (Appendix A) are generally available commercially from local native plant nurseries. Availability is, however, dependent upon many factors including demand, season and seed availability. To ensure the planting specification can be accommodated, the minimum recommended lead-time for ordering plants is 3-6 months. This generally allows adequate time for plants to be grown to the required size, so they are completely inundated by water for extended times. The following sizes are recommended as the minimum:

- Viro Tubes 50 mm wide x 85 mm deep
- 50 mm Tubes 50 mm wide x 75 mm deep
- Native Tubes 50 mm wide x 125 mm deep

4.5.4 Topsoil Specification and Preparation

During the sedimentation basin construction process, topsoil is to be stripped and stockpiled for possible reuse as a plant growth medium. It is important to test the quality of the local topsoil, which is likely to have changed from its pre-European native state due to prior land uses such as farming and industry, to determine the soils suitability for reuse as a plant growth medium. Remediation may be necessary to improve the soils capacity to support plant growth and to suit the intended plant species. Soils applied to the littoral zones of sedimentation basins must also be free from significant weed seed banks as labour intensive weeding can incur large costs in the initial plant establishment phase. On some sites, topsoils may be nonexistent and material will need to be imported. It is important that imported soil does not contain Fire Ants. A visual assessment of the soils is required and any machinery should be free of clumped dirt. Soils must not be brought in from Fire Ant restricted areas.

The installation of horticultural soils should follow environmental best practices and include:

- Preparation of soil survey reports including maps and test results at the design phase.
- Stripping and stockpiling of existing site topsoils prior to commencement of civil works.
- Deep ripping of subsoils using a non-inversion plough.
- Reapplication of stockpiled topsoils and, if necessary, remedial works to suit the intended plant species.
- Addition where necessary, of imported topsoils (certified to AS 4419-2003).

The following minimum topsoil depths are required:

- 150 mm for turf species.
- 300 mm for groundcovers and small shrubs.
- 450 mm for large shrubs.
- 600 mm for trees.

4.5.5 Vegetation Establishment

4.5.5.1 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. October and November are considered ideal times to plant vegetation in treatment elements. This allows for adequate establishment/ root growth before the heavy summer rainfall period but also allows the plants to go through a growth period soon after planting, resulting in quicker establishment. Planting late in the year also avoids the dry winter months, reducing maintenance costs associated with watering. Construction planning and phasing should endeavour to correspond with suitable planting months wherever possible. However, as lead times from earthworks to planting can often be long, temporary erosion controls (e.g. use of matting or sterile grasses to stabilise exposed batters) should always be used prior to planting.

4.5.5.2 Water Level Manipulation

To maximise the chances of successful vegetation establishment, the water level of the sedimentation basin is to be manipulated in the early stages of vegetation growth. When first planted, vegetation in the deep marsh zones may be too small to be able to exist in their prescribed water depths (depending on the maturity of the plant stock provided). Macrophytes intended for the deep marsh sections will need to have half of their form above the water level, which may not be possible if initially planted at their intended depth. Similarly, if planted too deep, 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.

4.5.5.3 Weed Control

Weed management in sedimentation basins is important to ensure that weeds do not out compete the species planted for the particular design requirements. This may also include some native species like Phragmites that naturally can appear in constructed wetlands and out-compete other more important planted species.

Conventional surface mulching of the wetland littoral berms with organic material like tanbark is not recommended. Most organic mulch floats and water level fluctuations and runoff typically causes this material to be washed into the wetland with a risk of causing blockages to outlet structures. Mulch can also increase the wetland organic load, potentially increasing nutrient concentrations and the risk of algal blooms. Adopting high planting density rates and if necessary applying a suitable biodegradable erosion control matting to the wetland batters (where appropriate), will help to combat weed invasion and will reduce maintenance requirements for weed removal. If the use of mulch on the littoral zones is preferred, it must be secured in place with appropriate mesh or netting (e.g. jute mesh).

4.5.5.4 Watering

Regular watering of the littoral and ephemeral marsh zone vegetation during the plant establishment phase is essential for successful establishment and healthy growth. The frequency of watering to achieve successful plant establishment is dependent upon rainfall, maturity of planting stock and the water level within the wetland. However, the following watering program is generally adequate but should be adjusted (i.e. increased) as required to suit site conditions:

- Week 1-2 3 visits/ week
- Week 3-6 2 visits/ week
- Week 7-12 1 visit/ week

After this initial three month period, watering may still be required, particularly during the first winter (dry period). Watering requirements to sustain healthy vegetation should be determined during ongoing maintenance site visits.

4.6 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. The outlets from sedimentation basins are to be designed such that access to the outlet does not require a water vessel. Maintenance of the littoral vegetation including watering and weeding is also required, particularly during the plant establishment period (first two years).
- 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 vary significantly. The basins must be cleaned out if more than half full of accumulated sediment.
- Similar to other types of WSUD elements, debris removal is an ongoing maintenance requirement. Debris, if not removed, can block inlets or outlets, and can be unsightly if deposited in a visible location. Inspection and removal of debris should be done regularly and debris removed whenever it is observed on the site.
- Typical maintenance of sedimentation basins will involve:
- Routine inspection of the sedimentation basin to identify depth of sediment accumulation, damage to vegetation, scouring or litter and debris build up (after first 3 significant storm events and then at least every 3 months).
- Routine inspection of inlet and outlet points to identify any areas of scour, litter build up and blockages.
- Removal of litter and debris.
- Removal and management of invasive weeds (both terrestrial and aquatic).
- Periodic (usually every 5 years) draining and desilting, which will require excavation and dewatering of removed sediment (and disposal to an approved location).
- Regular watering of littoral vegetation during plant establishment (refer section 4.4.6).
- Replacement of plants that have died (from any cause) with plants of equivalent size and species as detailed in the planting schedule.

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

All maintenance activities must be specified in a maintenance plan (and associated maintenance inspection forms) to be developed as part of the design process (Step 7). Maintenance personnel and asset managers will use this plan to ensure the sediment basins continue to function as designed.

The maintenance plans and forms must address the following:

- Inspection Frequency
- Maintenance Frequency
- Data Collection/ Storage Requirements (i.e. during inspections)
- Detailed Clean Out Procedures (main element of the plans) including:
  - equipment needs
  - maintenance techniques
  - occupational health and safety
  - public safety
  - environmental management considerations
  - disposal requirements (of material removed)
  - access issues
  - stakeholder notification requirements
  - data collection requirements (if any)
- Design Details.

An approved maintenance plan is required prior to asset transfer to Council.

An example operation and maintenance inspection form is included in the checking tools provided in Section 4.7. These forms must be developed on a site specific basis as the configuration and nature of sediment basins varies significantly.

4.7 Checking Tools

This section provides a number of checking aids for designers and Council development assessment officers. In addition, Section 4.6.5 provides general advice for the construction and establishment of sedimentation basins and key issues to be considered to ensure their successful establishment and operation based on observations from construction projects around Australia.

Checking tools include:

- Design Assessment Checklist.
- Construction Checklist (during and post).
- Operation and Maintenance Inspection Form.
- Asset Transfer Checklist (following 'on-maintenance' period).
- Construction and Establishment Advice.

4.7.1 Design Assessment Checklist

The checklist on page 4-35 presents the key design features that are to be reviewed when assessing a design of a sedimentation basin. These considerations include (but not limited to) configuration, safety, maintenance and operational issues that should be addressed during the design phase. Where an item receives a 'N' from the review process, 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. Development proponents will need to ensure that all relevant permits are in place. These can include permits to clear vegetation, to dredge, create a waterbody, divert flows or disturb habitat.

4.7.2 Construction Checklist

The checklist on page 4-36 presents the key items to be reviewed when inspecting the sediment basin during and at the completion of construction. The checklist is to be used by Construction Site Supervisors and local authority Compliance Inspectors to ensure all the elements of the basin have been constructed in accordance with the design. If an item receives an 'N' in Satisfactory criteria then appropriate actions must be specified and delivered to rectify the construction issue before final inspection sign-off is given.

4.7.3 Operation and Maintenance Inspection Form

The example form on page 4-37 should be developed and 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 1 to 6 months depending on the size and complexity of the system. More detailed site specific maintenance schedules should be developed for major sedimentation basins and include a brief overview of the operation of the system and key aspects to be checked during each inspection.

4.7.4 Asset Transfer Checklist

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. The table on page 4-38 provides an indicative asset transfer checklist.

SEDIMENTATION BASIN DESIGN ASSESSMENT CHECKLIST			
Basin Location:			
Hydraulics:	Design operational flow (m^3/s):	Above design flow (m^3/s):	
Area:	Catchment Area (ha):	Basin Area (ha):	
TREATMENT		Y	N
MUSIC modelling performed?			
BASIN CONFIGURATION		Y	N
Discharge pipe/structure to sedimentation basin sufficient for design flow?			
Scour protection provided at inlet?			
Basin located upstream of treatment system (i.e. macrophyte zone of wetland)?			
Configuration of basin (aspect, depth and flows) allows settling of particles >125 µm?			
Basin capacity sufficient for desilting period >=5 years?			
Maintenance access allowed for into base of sediment basin?			
Public access to basin prevented through dense vegetation or other means?			
Gross pollutant protection measures provided on inlet structures where required?			
Freeboard provided to top of embankment?			
Public safety design considerations included in design and safety audit of publicly accessible areas undertaken?			
Overall shape, form, edge treatment and planting integrate well (visually) with host landscape?			
OUTLET STRUCTURES		Y	N
'Control' outlet structure required?			
'Control' outlet structure sized to convey the design operation flow?			
Designed to prevent clogging of outlet structures (i.e. provision of appropriate grate structures)?			
'Spillway' outlet control (weir) sufficient to convey 'above design flow'?			
'Spillway' outlet has sufficient scour protection?			
Visual impact of outlet structures has been considered?			
COMMENTS			

SEDIMENTATION BASIN CONSTRUCTION INSPECTION CHECKLIST

Site:	Inspected by:
	Date:
	Time:
Constructed by:	Weather:
	Contact during site visit:

Items inspected	Checked		Satisfactory		Items inspected	Checked		Satisfactory	
	Y	N	Y	N		Y	N	Y	N
DURING CONSTRUCTION									
A. FUNCTIIONAL INSTALLATION									
Preliminary works					Structural components (continued)				
1. Erosion and sediment control plan adopted					19. No seepage through banks				
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				
B. Earthworks					23. Provision of maintenance drain				
5. Integrity of banks					24. Collar installed on pipes				
6. Batter slopes as plans					Vegetation				
7. Impermeable (eg. clay) base installed					25. Stabilisation immediately following earthworks				
8. Maintenance access (eg. ramp) installed					26. Weed removal prior to planting				
9. Compaction process as designed					27. Planting as designed (species and densities)				
10. Level of base, banks/ spillway as designed					28. Vegetation layout and densities as designed				
11. Check for groundwater intrusion									
12. Stabilisation with sterile grass					B. EROSION AND SEDIMENT CONTROL				
Structural components					29. Sediment bains to be used during construction				
13. Location and levels of outlet as designed					30. Stabilisation immediately following earthworks and planting of terrestrial landscape around basin				
14. Safety protection provided					31. Silt fences and traffic control in place				
15. Pipe joints and connections as designed									
16. Concrete and reinforcement as designed					C. OPERATIONAL ESTABLISHMENT				
17. Inlets appropriately installed					32. Sediment basin desilted				
18. Inlet energy dissipation installed									

FINAL INSPECTION									
1. Confirm levels of inlets and outlets					8. Check for uneven settling of banks				
2. Confirm structural element sizes					9. Evidence of stagnant water, short circuiting or vegetation scouring				
3. Check batter slopes					10. Evidence of litter or excessive debris				
4. Vegetation plantings as designed					11. Inlet erosion protection working				
5. Erosion protection measures working					12. Maintenance access provided				
6. Maintenance access provided					13. Construction generated sediment removed				
7. Public safety adequate					14. Provision of removed sediment drainage area				

COMMENTS ON INSPECTION

ACTIONS REQUIRED
1.
2.
3.
4.
Inspection officer signature:

SEDIMENTATION BASIN MAINTENANCE CHECKLIST			
Inspection Frequency:	1 to 6 monthly	Date of Visit:	
Location:			
Description:			
Site Visit by:			

INSPECTION ITEMS	Y	N	Action Required (details)
Litter accumulation?			
Sediment accumulation at inflow points?			
Sediment requires removal (record depth, remove if >50%)?			
All structures in satisfactory condition (pits, pipes, ramps etc)?			
Evidence of dumping (building waste, oils etc)?			
Littoral vegetation condition satisfactory (density, weeds etc)?			
Replanting required?			
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)?			
Resetting of system required?			
COMMENTS			

ASSET TRANSFER CHECKLIST		
Asset Description:		
Asset ID:		
Asset Location:		
Construction by:		
'On-maintenance' Period:		
TREATMENT	Y	N
System appears to be working as designed visually?		
No obvious signs of under-performance?		
MAINTENANCE	Y	N
Maintenance plans and indicative maintenance costs provided for each asset?		
Vegetation establishment period completed (2 years?)		
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 (e.g. drawings, survey, models) provided?		
Asset listed on asset register or database?		
COMMENTS		

4.8 Sedimentation Basin Worked Example

A constructed wetland system is proposed to treat runoff from a freeway in the Gold Coast region. A sedimentation basin forms the 'inlet zone' of the wetland system. This worked example focuses on the design of the sedimentation basin component of the system. A photograph of a similar system is shown in Plate 4-7.

Plate 4-7: Example Sedimentation Basin Configuration

The site is triangular in shape with a surface area of approximately 7,000 m^2 as shown in Figure 4-14. Road runoff is conveyed by roadside open channels and conventional stormwater pipes (up to the 100 year ARI event) to a single outfall that discharges to the top apex of the sedimentation basin site as shown in Figure 4-14. Approximately 1.0 km of the freeway, with a total contributing area of 8 ha (90 % impervious), discharges to the sedimentation basin. The site of the sedimentation basin has a fall of approximately 2 m (from 5 m AHD to 3 m AHD) towards a watercourse.

The conceptual design process established the following key design elements to ensure effective operation of the constructed wetland and sedimentation basin:

- Notional permanent pool depth of sedimentation basin of 2 m
- Permanent pool 0.3 m (3.8 m AHD)
- Wetland macrophyte zone extended detention depth of 0.5 m (permanent water level of 3.5 m AHD)
- Sedimentation basin permanent pool level ('control' outlet pit level) 0.3 m above the permanent pool level of the wetland (3.8 m AHD)
- 'Spillway' outlet weir set at the top of extended detention for the wetland and 0.3 m above the sediment basin permanent pool level (4.1 m AHD).

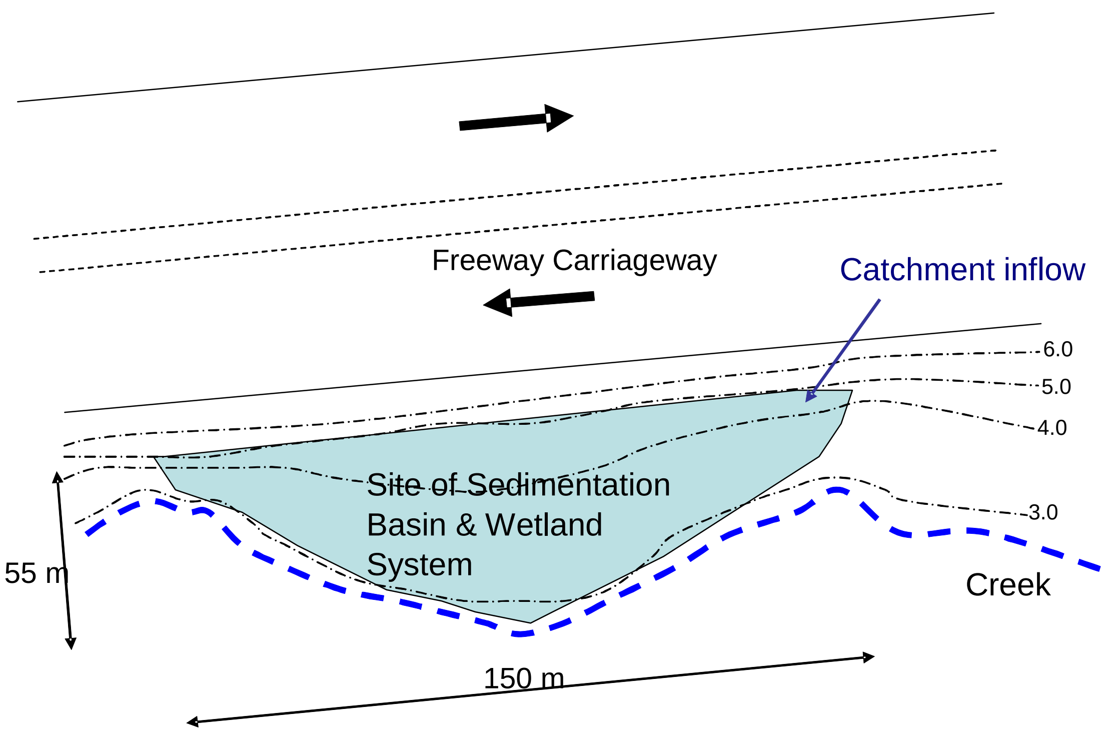


Figure 4-14: Layout of Proposed site for Sedimentation Basin

Design Objectives

As the sedimentation basin forms part of a treatment train (with the wetland macrophyte zone downstream) with the design requirements of the sedimentation basin system to:

- Promote sedimentation of particles larger than 125 µm with a 90 % capture efficiency for flows up to the 'design operation flow' (1 year ARI peak discharge).
- Provide for connection to the downstream wetland macrophyte zone with discharge capacity corresponding to the 'design operation flow' (1 year ARI peak discharge).
- Provide for bypass of the 'above design flow' around the wetland macrophyte zone when the inundation of the macrophyte zone reaches the design maximum extended detention depth.

4.8.1 Step 1: Determine Design Flows

4.8.1.1 Design Operation Flow

As described in Section 4.3.1, the 'design operation flow' is defined as the 1 year ARI and provides a basis for sizing the sedimentation basin area and 'control' outlet structure.

Design flows are established using the Rational Method and the procedures provided in QUDM (DPI et al, 1992). The site has one contributing catchment being 8 ha in area, 1 km long (along the freeway) and drained by roadside open channels and stormwater pipes.

For the purposes of establishing the time of concentration, the flow velocity in the roadside channels and underground pipes is estimated at 1 m/s. Therefore:

Time of concentration (t_c) = 1000 m/1 m/s
= 1000s
= 17 minutes

The coefficient of runoffs were calculated using relevant local government guidelines and Table 5.04.3 of QUDM (DPI *et al* 1992) as follows:

C_{10} = 0.9 (from local government guidelines)

Table 4-2: Runoff Coefficients

	C Runoff		
ARI	1	10	100
QUDM Factor	0.8	1	1.20
C_{ARI}	0.72	0.9	1.08

Rational Method $Q = CIA/360$

Where: C_{10} = 0.9
Catchment area = 8 ha
t_c = 17 mins
I_1 = 78 mm/hr
I_{100} = 179 mm/hr

Design operation flow ($Q_{1\ year\ ARI}$) = 1.25 m^3/s

4.8.1.2 Above Design Flow

The 'above design flow' is used to design the 'spillway' outlet structure which forms part of the high flow bypass around the wetland. In this case, the major flood flow (100 year ARI) enters the sedimentation basin and thus forms the 'above design flow'.

Rational Method $Q = CIA/360$

Where A = 8 ha
C_{100} = 1.08
I_{100} = 179 mm/hr
'Above design flow' ($Q_{100\ year\ ARI}$) = 4.30 m^3/s

4.8.2 Step 2: Confirm Treatment Performance of Concept Design

An initial estimate of the sedimentation basin area can be established using the curves provided in Figure 4-3. Assuming a notional permanent pool depth of 2 m, a sedimentation basin area of approximately 320m^2 is required to capture 90 % of the 125 µm particles for flows up to the design operation flow (1 year ARI = 1.1 m^3/s).

4.8.3 Step 3: Confirm Size and Dimensions of the Sedimentation Basin

4.8.3.1 Sedimentation Basin Area

Confirmation of the sedimentation basin area is provided by using Equation 4.1:

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

Based on the description of the sedimentation basin and wetland provided in Section 4.8.1, the following applies:

d_p = 2.0 m

d^* = 1.0 m

d_e = 0.3 m

V_s = 0.011 m/s for 125 µm particles

R = 0.9 (90% removal target)

Q = design operation flow rate (1 year ARI) = 1.25 m^3/s

An aspect ratio of 1 (W) to 4 (L) is adopted based on the available space (Figure 4-14). Using Figure 4-4 (configuration I), the hydraulic efficiency (λ) is estimated to be approximately 0.4. This value is less than desirable; however, site constraints prevent any other configuration. The turbulence factor (*n*) is computed from Equation 4.2 to be 1.67. Thus:

$\lambda = 0.4$

$n = 1.67$

Inserting the above parameters into Equation 4.1, the required sedimentation basin area to achieve a target sediment (125 µm) capture efficiency of 90 % is 319m^2. With a W to L ratio of 1:4, the notional dimensions of the basin are approximately 8.9 m x 35.8 m.

4.8.3.2 Storage Volume for Sediments

To ensure the storage zone is appropriate the following must be met:

Sedimentation Basin Storage Volume V_s > Volume of accumulated sediment over 5 years ($V_{s:5yr}$)

The sedimentation basin storage volume (V_s) is defined as the storage available in the bottom half of the sedimentation basin permanent pool depth. Considering the internal batters of the basin (Section 4.7.4) are 2:1 (H:V) below the permanent water level, the area of the basin at 1 m depth is 307 m^2 and at 2 m depth is 294 m^2. Therefore, the sedimentation basin storage volume V_s is 300m^3.

The volume of accumulated sediments over 5 years ($V_{s:5yr}$) is established using Equation 4.3 (using a sediment discharge rate (L_o) of 1.6 m^3/ha/yr):

$$V_{s:5yr} = A_c \cdot R \cdot L_o \cdot F_c$$

Given	A_c	=	8 ha
	R	=	90 %
	L_o	=	1.6 m^3/ha/year
	F_c	=	5 years

The total sediment accumulation is estimated to be:

= 8 x 0.9 x 1.6 x 5

= 57.6 m^3

Therefore, $V_s > V_{s:5yr}$

Rearranging Equation 4.3, the required clean out frequency (Fc) is estimated to be:

$$Fc = \frac{300}{1.6 \times 8 \times 0.9}$$

= 26.1 years

4.8.3.3 Internal Batters

Considering the relatively small size of the sedimentation basin (8.9 m width), it is not possible to achieve the notional permanent pond depth of 2 m using the 5:1 (H:V) required for public safety. Therefore 4:1 (H:V) batter is to be adopted for the ground above the permanent pool level and to 0.2 m below permanent pool level and a 2:1 (H:V) internal batter slope for 0.2 m to 2 m below the permanent pool level. The sedimentation basin will be fenced around most of its perimeter to ensure public safety.

The base of the sedimentation basin will be lined with rock to prevent vegetation growth and to guide extraction depths during sediment removal. A summary of the sedimentation basin configuration is as follows:

Open Water Area	= 319 m^2
Width	= 8.9 m
Length	= 35.8 m
Depth of Permanent Pool (d_p)	= 2.0 m

4.8.4 Step 4: Design Inflow Systems

To prevent scour of deposited sediments from piped inflows, rock protection and benching is to be placed at the pipe headwall as shown in Figure 4-15.

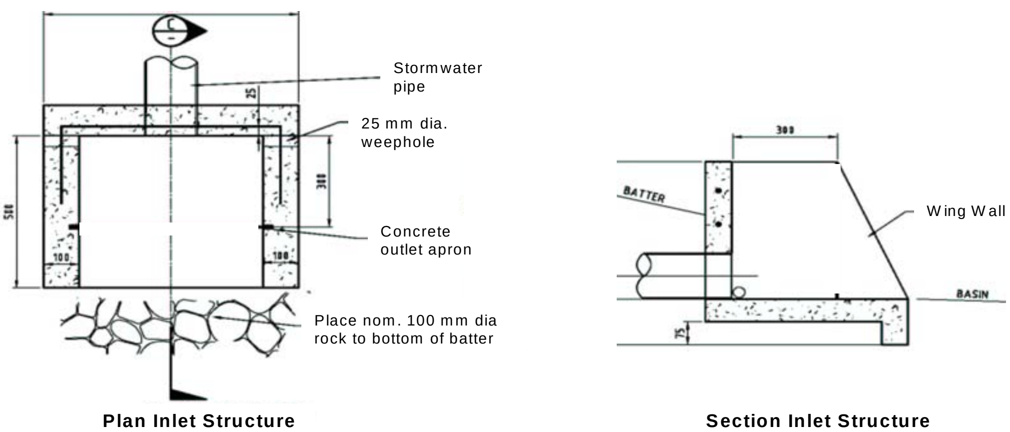


Figure 4-15: Conceptual Inlet Structure with Rock Benching

4.8.5 Step 5: Design Outlet Structures

4.8.5.1 Design of 'Control' Outlet - Overflow Pit and Pipe Outlet Configuration

The 'control' outlet structure is to consist of an outlet pit with the crest of the pit set at the permanent pool level of the sedimentation basin (3.8 m AHD) which is 0.3 m above the permanent water level in the wetland. The overflow pit is sized to convey the design operational flow (1 year ARI).

According to Section 4.3.5, two possible flow conditions need to be checked, i.e. weir flow conditions (with extended detention of 0.3 m) and orifice flow conditions.

Weir Flow Conditions

From Equation 4.4, the required perimeter of the outlet pit to pass 1.25 m^3/s with an afflux of 0.3 m can be calculated assuming 50% blockage:

$$P = \frac{Q_{des}}{B \cdot C_w \cdot h^{3/2}} = \frac{1.25}{0.5 \cdot 1.66 \cdot 0.3^{3/2}} = 9.2\text{m}$$

Orifice Flow Conditions

From Equation 4.5, the required area of the outlet pit can be calculated as follows:

$$A_o = \frac{Q_{des}}{B \cdot C_d \cdot \sqrt{2 \cdot g \cdot h}} = \frac{1.25}{0.5 \cdot 0.6 \cdot \sqrt{2(9.81)(0.3)}} = 1.7\ \text{m}^2$$

In this case, the weir flow condition is limiting. Considering the overflow pit is to convey the 'design operation flow' (1 year ARI) or slightly greater, a 2000 x 3000 mm pit is adopted providing a perimeter of 10 m which is greater than the 9.2 m calculated using the weir flow equation above. The top of the pit is to be fitted with a letter box grate. This will ensure large debris does not enter the 'control' structure while avoiding grate blockage by smaller debris.

The size of the connection pipe (i.e. between the sedimentation basin and wetland macrophyte zone) can be calculated by firstly estimating the velocity in the connection pipe (as the outlet is submerged) using the following (Equation 4.5):

$$h = \frac{2 \cdot V^2}{2 \cdot g}$$

Where: *h* = Head level driving flow through the pipe (defined as the 'Spillway' outlet level minus the normal water level in the downstream treatment system)

V = Pipe velocity (m/s)

g = Gravity (9.79 m/s^2)

Note: the coefficient of 2 in the equation is a conservative estimate of the sum of entry and exit loss coefficients ($K_{in} + K_{out}$).

Hence, $V = (9.79 \times 0.6)^{0.5} = 2.43$ m/s

The area of pipe required to convey the 'design operation flow' (1 year ARI) is then calculated by dividing the above 'design operation flow' by the velocity:

$A = 1.25/2.43 = 0.453\ m^2$

This area is equivalent to a 750 mm reinforced concrete pipe (RCP). The obvert of the pipe is to be set just below the permanent water level in the wetland macrophyte zone (3.5 m AHD) meaning the invert is at 2.7 m AHD.

'Control' Outlet Structure:

Overflow pit = 2000 x 3000 mm with letter box grate set at 3.8 m AHD pipe connection (to wetland) = 750mm RCP at 2.7m AHD

Design of 'Spillway' Outlet - Weir Outlet

The 'above design flow' is to bypass the macrophyte zone of the wetland. This will be provided by a 'spillway' outlet weir designed to convey the 'above design flow' (100 year ARI) set at 0.3 m above the permanent pool of the sedimentation basin.

The length of the 'spillway' outlet weir determines the afflux for the 100 year ARI peak discharge and sets the top of embankment of the sedimentation basin. It is common practice to allow for 0.3 m of freeboard above the afflux level when setting the top of embankment elevation. An afflux of 0.3 m has been adopted in defining the length of the spillway weir. This value was adopted as a trade off between the bank height and the width of the weir. A bank height of 0.9 m (0.3 m afflux and 0.3 m freeboard plus 0.3 m extended detention) above the normal water level was deemed acceptable. The weir length is calculated using the weir flow equation (Equation 4.4) substituting outlet perimeter *P* with weir length *L* and blockage factor *B*=1 (no blockage):

$$L = \frac{Q_{des}}{C_w \cdot h^{3/2}} = \frac{4.30}{1.66 \cdot 0.3^{3/2}} = 15.8 \text{ m}$$

The 'spillway' outlet is located adjacent to the inflow culvert to minimise risk of sediment scour.

<u>'Spillway' Outlet Structure:</u>

Spillway length = 15.8 m set at 0.30 m above permanent pool level (4.1 m AHD)

Top of embankment set at 0.9 m above the permanent pool level (4.7 m AHD)

4.8.6 Step 6: Vegetation Specification

The vegetation specification and recommended planting density for the littoral zone around the sedimentation basin have been adapted from Appendix A and are summarised in Table 4-3.

Table 4-3: Vegetation Specification for Worked Example

Zone	Plant Species	Planting Density (plants/m^2)
Littoral Zone (edge)	*Carex appressa* *Isolepis nodosa*	8 8
Marsh to a depth of 0.25m	*Eleocharis equisetina* *Juncus usitatus*	10 10

Refer to Appendix A for further discussion and guidance on vegetation establishment and maintenance.

4.8.7 Design Calculation Summary

The sheet below summarises the results of the design calculations.

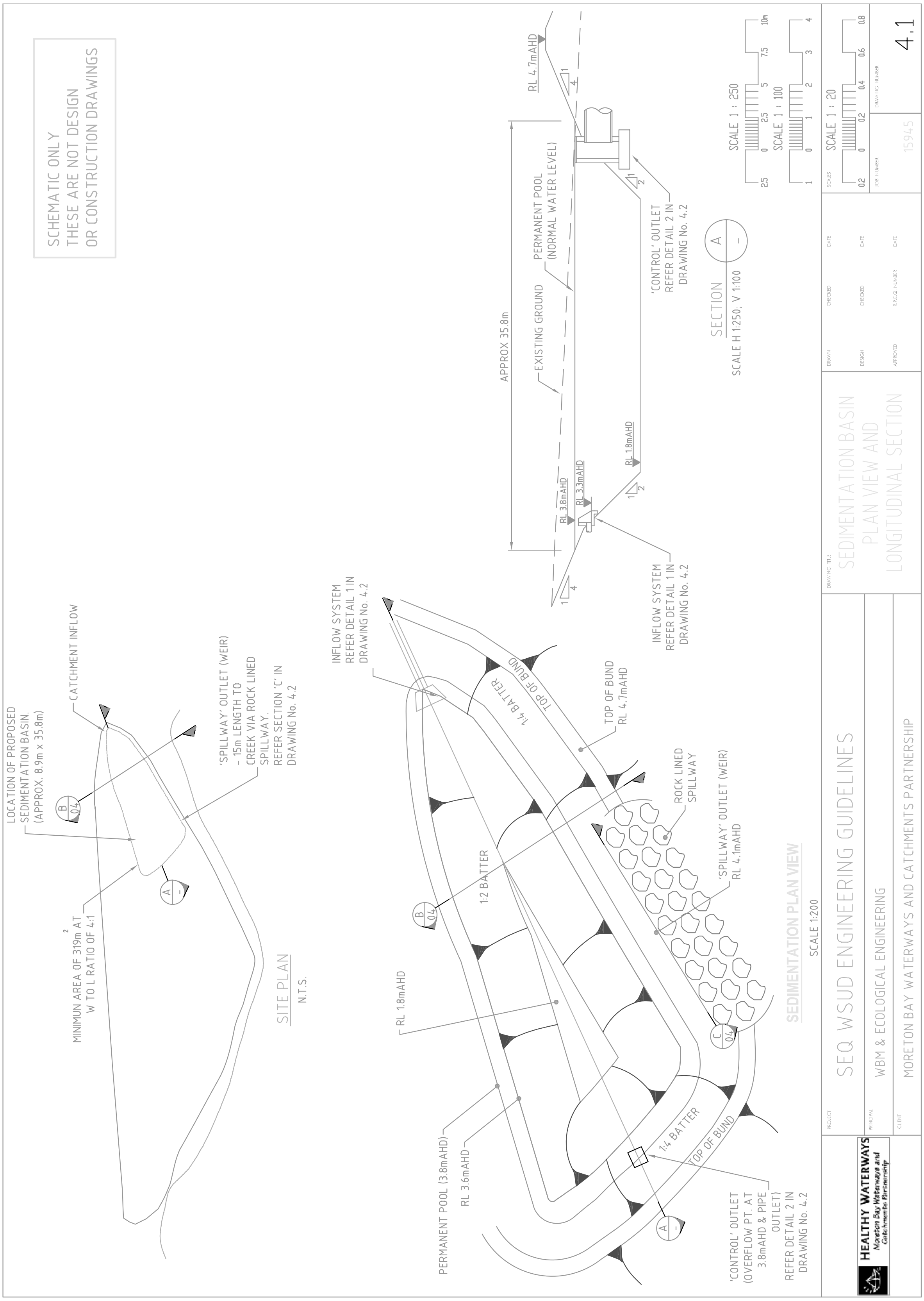
SEDIMENTATION BASIN DESIGN CALCULATION SUMMARY				
	Calculation Task	CALCULATION SUMMARY		
		Outcome		Check
	Catchment Characteristics			
	Residential		Ha	✓
	Commercial		Ha	
	Roads	8	Ha	
	Storm event entering inlet pond (minor or major)	100	yr ARI	
	Conceptual Design			
	Notional permanent pool depth	2	m	✓
	Permanent pool level of sedimentation basin	3.8	m AHD	
1	**Determine design flows**			
	'Design operation flow' (1 year ARI)	1	year ARI	✓
	'Above design flow' (either 2 or 100 year ARI)	100	year ARI	
	Time of concentration			
	Refer to relevant Local Government Guidelines and QUDM	17	minutes	✓
	Identify rainfall intensities			
	'Design operation flow' - $I_{1\ year\ ARI}$	78	mm/hr	
	'Above design flow'- $I_{2\ year\ ARI}$ to $I_{100\ year\ ARI}$	179	mm/hr	
	Design runoff coefficient			
	'Design operation flow' - $C_{1\ year\ ARI}$	0.7		✓
	'Above design flow'- $I_{2\ year\ ARI}$ to $I_{100\ year\ ARI}$	1.08		
	Peak design flows			
	'Design operation flow' - 1 year ARI	1.25	m^3/s	✓
	'Above design flow' – 2 to 100 year ARI	4.09	m^3/s	
2	**Confirm Treatment Performance of Concept Design**			
	Capture efficiency (of 125 µm sediment)	90	%	✓
	Area of sedimentation basin	320	m^2	
3	**Confirm size and dimension of sedimentation basin**			
	Area of sedimentation basin	319	m^2	✓
	Aspect ratio	4:1	L:W	
	Hydraulic efficiency	0.4		
	Depth of permanent pool	2	m	
	Storage volume for sediments			
	Sedimentation basin storage Volume V_s	300	m^3	✓
	Volume of accumulated sediment over 5 years ($V_{s:5yr}$)	58	m^3	
	$V_s > V_{s:5yr}$	Yes		
	Sediment cleanout frequency		years	
	Internal batters			
	Edge batter slope	1:2	V:H	✓
	Fence required	Yes		
4	**Design inflow systems**			
	Provision of scour protection or energy dissipation	Yes		✓
5	**Design outlet structures**			
	Design of 'control' outlet - overflow pit and pipe outlet configuration			
	Overflow pit crest level	3.8	m AHD	✓
	Overflow pit dimension	2000 x 3000	L x W	
	Provision of debris trap	Yes		
	Connection pipe dimension	750	mm diam	✓
	Connection pipe invert level	2.7	m AHD	

SEDIMENTATION BASIN DESIGN CALCULATION SUMMARY			
Calculation Task	CALCULATION SUMMARY		
	Outcome		Check
Design of 'control' outlet - weir configuration			
Weir crest level	N/A	m AHD	✓
Weir length	N/A	m	
Design of 'spillway' outlet - weir configuration			
Weir crest level	4.1	m AHD	
Weir length	15.8	m	✓
Depth above spillway	0.3	m	
Freeboard to top of embankment	0.3	m	

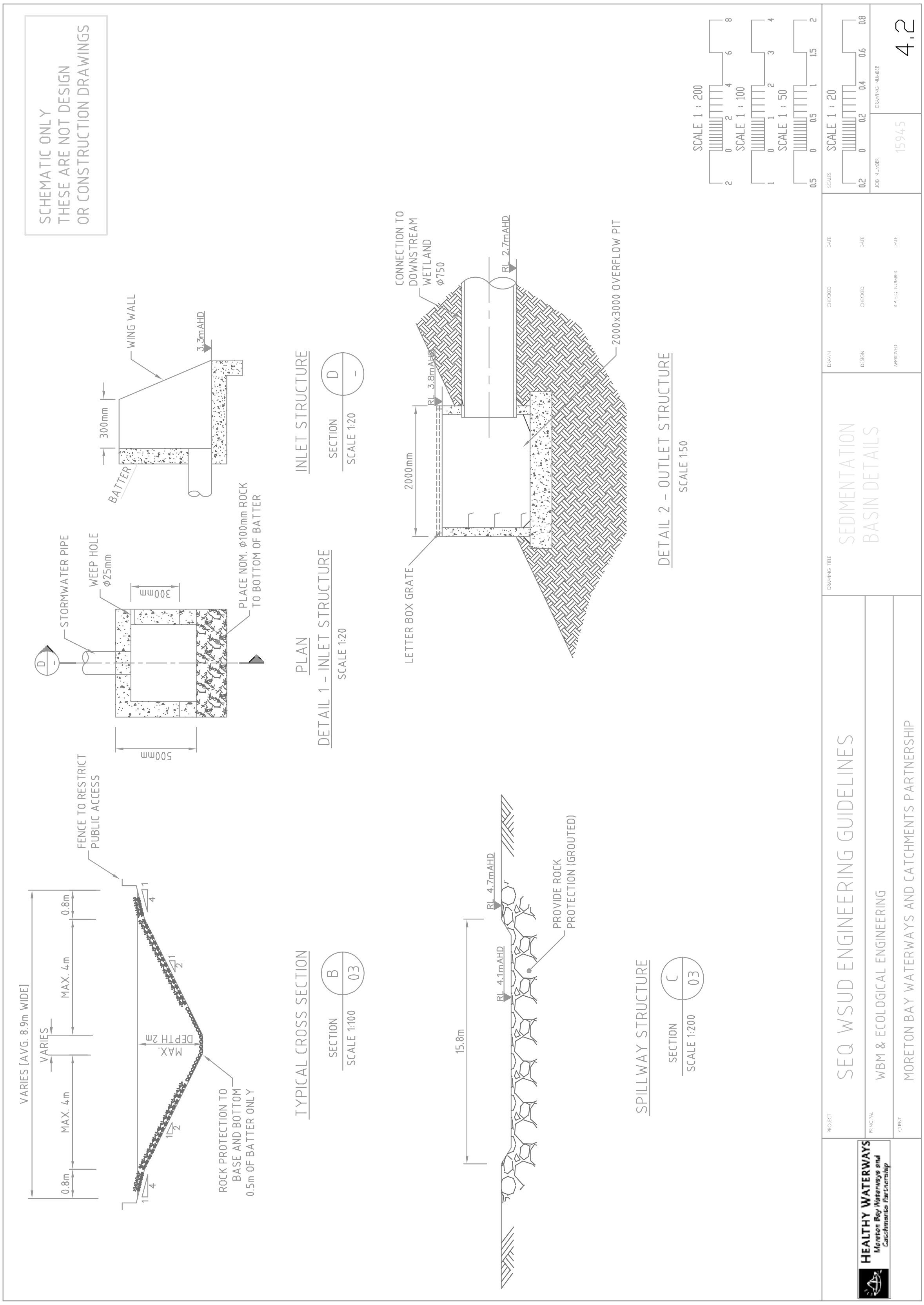
4.8.8 Worked Example Drawings

Drawing 4.1 and 4.2 illustrate the sediment basin worked example layout.

Drawing 4.1 Sedimentation Basin Plan View an Longitudinal Section



Drawing 4.2 Sedimentation Basin Details



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[1] At the time of preparation of these guidelines, QUDM was under review and a significantly revised edition is expected to be released in 2006. These guidelines refer to and use calculations specified in the existing QUDM document, however the revised version of QUDM should be used as the appropriate reference document. It should be noted by users of this guideline that the structure and content of QUDM will change, and as such, the references to calculations and/or specific sections of QUDM may no longer be correct. Users of this guideline should utilise and adopt the relevant sections and/or calculations of the revised QUDM guideline.

Chapter 5 Bioretention Basins

5.1 Introduction 5-2

5.2 Design Considerations 5-3
5.2.1 Landscape Design 5-3
5.2.2 Hydraulic Design 5-3
5.2.3 Ex-filtration to In-Situ Soils 5-3
5.2.4 Vegetation Types 5-4
5.2.5 Bioretention Filter Media 5-4
5.2.6 Traffic Controls 5-5
5.2.7 Services 5-6

5.3 Design Process 5-6
5.3.1 Step 1: Check Treatment Performance of Concept Design 5-7
5.3.2 Step 2: Determine Design Flows 5-9
5.3.3 Step 3: Design Inflow Systems 5-9
5.3.4 Step 4: Specify the Bioretention Filter Media Characteristics 5-12
5.3.5 Step 5: Design Under-Drain and Undertake Capacity Checks(if required) 5-14
5.1.1 Step 6: Check Requirement for Impermeable Lining 5-15
5.1.2 Step 7: Size Overflow Pit 5-16
5.3.6 Step 8: Specify Vegetation 5-17
5.3.7 Step 9: Undertake Verification Checks 5-17
5.3.8 Design Calculation Summary 5-17

5.4 Landscape Design Notes 5-20
5.4.1 Objectives 5-20
5.4.2 Bioretention Basin Vegetation 5-20
5.4.3 Other specific Landscape considerations 5-21
5.4.4 Safety Issues 5-21

5.5 Construction and Establishment 5-22
5.5.1 Staged Construction and Establishment Method 5-22
5.5.2 Construction Tolerances 5-24
5.5.3 Sourcing Bioretention Vegetation 5-24
5.5.4 Vegetation Establishment 5-25

5.6 Maintenance Requirements 5-25

5.7 Checking Tools 5-26
5.7.1 Design Assessment Checklist 5-26
5.7.2 Construction Checklist 5-27
5.7.3 Operation and Maintenance Inspection Form 5-27
5.7.4 Asset Transfer Checklist 5-27

5.8 Example Engineering Drawings 5-32

5.9 Bioretention Basin Worked Example 5-32
5.9.1 Step 1: Confirm Treatment Performance of Concept Design 5-34
5.9.2 Step 2: Determine Design Flows 5-34
5.9.3 Step 3: Design Inflow Systems 5-34
5.9.4 Step 4: Specify the Bioretention Media Characteristics 5-37
5.9.5 Step 5: Under-drain Design and Capacity Checks 5-38
5.9.6 Step 6: Check Requirement for Impermeable Lining 5-39
5.9.7 Step 7: Size Overflow Pit 5-39
5.9.8 Step 9: Vegetation Specification 5-39
5.9.9 Step 8: Verification Checks 5-39
5.9.10 Design Calculation Summary 5-39
5.9.11 Worked Example Drawing 5-42

5.10 References and Additional Information 5-44

5.1 Introduction

Bioretention basins are vegetated areas where runoff is filtered through a filter media layer (e.g. sandy loam) as it percolates downwards. It is then collected via perforated under-drains and flows to downstream waterways or to storages for reuse. Bioretention basins often use temporary ponding above the filter media surface to increase the volume of runoff treated through the filter media. They treat stormwater in the same way as bioretention swales; however, 'above design' flows are conveyed through overflow pits or bypass paths rather than over the filter media. This has the advantage of protecting the filter media surface from high velocities that can dislodge collected pollutants or scour vegetation.

Bioretention basins operate by filtering stormwater runoff through densely planted surface vegetation and then percolating runoff through a prescribed filter media. During percolation, pollutants are retained through fine filtration, adsorption and some biological uptake. The vegetation in a bioretention system is a vital functional element of the system providing a substrate for biofilm growth within the upper layer of the filter media. Vegetation facilitates the transport of oxygen to the soil and enhances soil microbial communities which enhance biological transformation of pollutants.

Bioretention basins are generally not intended to be 'infiltration' systems that discharge from the filter media to surrounding in-situ soils. Rather, the typical design intent is to recover stormwater at the base of the filter media in perforated under-drains and discharge to receiving waterways or to storages for potential reuse. In some circumstances however, where the in-situ soils allow and there is a particular design intention to recharge local groundwater, it may be desirable to allow stormwater to infiltrate from the base of a filter media to underlying in-situ soils.

Bioretention basins can be installed at various scales, for example, as landscape planter boxes, in streetscapes integrated with traffic calming measures, in suburban parks and in retarding basins. In larger applications, it is considered good practice to have pretreatment measures (e.g. swales) upstream of the basin to reduce the maintenance frequency of the bioretention basin. Figure 5-1 shows examples of a basin integrated into a local streetscape and into a car park. Figure 5-1 also illustrates the key elements of bioretention basins, namely surface vegetation, extended detention, filter media, drainage layer and overflow pit.

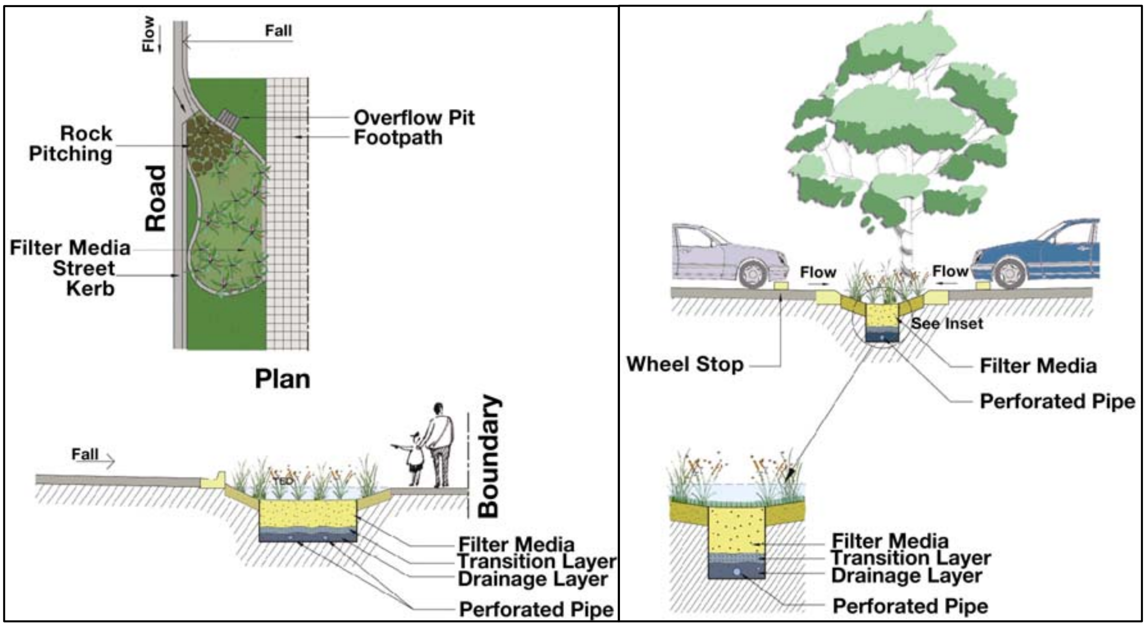


Figure 5-1: Bioretention basin integrated into a local streetscape (left) and a car park (right). (TED = top of extended detention)

5.2 Design Considerations

This section outlines some of the key design considerations for bioretention basins that the detailed designer should be familiar with before applying the design procedure presented later in this chapter.

5.2.1 Landscape Design

Bioretention basins are predominantly located within public areas, such as open space or within streets, that provide a primary setting for people to experience their local community and environment. It is therefore necessary for bioretention basins to be given an appropriate level of landscape design consideration to compliment the surrounding landscape character. The landscape design of bioretention basins must address stormwater quality objectives whilst also being sensitive to other important landscape objectives such as road visibility, public safety and community character and habitat.

Plate 5-1: Raised Overflow Pit Surrounded by Bioretention Vegetation

5.2.2 Hydraulic Design

The correct hydraulic design of bioretention basins is essential to ensure effective stormwater treatment performance, minimize damage by storm flows, and to protect the hydraulic integrity and function of associated minor and major drainage systems. The following aspects are of key importance:

- The finished surface of the bioretention filter media must be horizontal (i.e. flat) to ensure full engagement of the filter media by stormwater flows and to prevent concentration of stormwater flows within depressions and ruts resulting in potential scour and damage to the filter media.
- Temporary ponding (i.e. extended detention) of up to 0.3 m depth over the surface of the bioretention filter media created through the use of raised field inlet pits (overflow pits) can assist in managing flow velocities over the surface of the filter media as well as increase the overall volume of stormwater runoff that can be treated by the bioretention filter media.
- Where possible, the overflow pit or bypass channel should be located near the inflow zone (refer to Figure 5-1(left)) to prevent high flows passing over the surface of the filter media. If this is not possible, then velocities during the minor (2-10 year ARI) and major (50-100 year ARI) floods should be maintained sufficiently low (preferably below values of 0.5 m/s and not more than 1.5 m/s for major flood) to avoid scouring of the filter media and vegetation.
- Where the field inlets in a bioretention system is required to convey the minor storm flow (i.e. is part of the minor drainage system), the inlet must be designed to avoid blockage, flow conveyance and public safety issues.
- For streetscape applications, the design of the inflow to the bioretention basin must ensure the kerb and channel flow requirements are preserved as specified in the Queensland Urban Drainage Manual (QUDM) (DPI, IMEA & BCC 1992)

5.2.3 Ex-filtration to In-Situ Soils

Bioretention basins can be designed to either preclude or promote ex-filtration of treated stormwater to the surrounding in-situ soils depending on the overall stormwater management objectives established for the given project. When considering ex-filtration to surrounding soils, the designer must consider site terrain, hydraulic conductivity of the in-situ soil, soil salinity, groundwater and building setback. Further guidance in this regard is provided in Chapter 7 Infiltration Measures.

Where the concept design specifically aims to preclude ex-filtration of treated stormwater runoff it is necessary to consider if the bioretention basin needs to be provided with an impermeable liner. The amount of water lost from bioretention basins to surrounding in-situ soils is largely dependant on the characteristics of the local soils and the saturated hydraulic conductivity of the bioretention filter media (see Section 5.2.5). Typically, if the selected saturated hydraulic conductivity of the filter media is one to two orders of magnitude (i.e. 10 to 100 times) greater than that of the native surrounding soil profile, then the preferred flow path for stormwater runoff will be vertically through the bioretention filter media and into the perforated under-drains at the base of the filter media. As such, there will be little if any ex-filtration to the native surrounding soils. However, if the selected saturated hydraulic conductivity of the bioretention filter media is less than 10 times that of the native surrounding soils, it may be necessary to provide an impermeable liner. Flexible membranes or a concrete casting are commonly used to prevent excessive ex-filtration. This is particularly applicable for surrounding soils that are very sensitive to any ex-filtration (e.g. sodic soils, shallow groundwater or close proximity to significant structures such as roads).

The greatest pathway of ex-filtration is through the base of a bioretention basin, as gravity and the difference in hydraulic conductivity between the filter media and the surrounding native soil would typically act to minimise ex-filtration through the walls of the trench. If lining is required, it is likely that only the base and the sides of the *drainage layer* (refer Section 5.2.5) will need to be lined.

Where ex-filtration of treated stormwater to the surrounding in-situ soils is promoted by the bioretention basin concept design it is necessary to ensure the saturated hydraulic conductivity of the in-situ soils is at least equivalent to that of the bioretention filter media, thus ensuring no impedance of the desired rate of flow through the bioretention filter media. Depending on the saturated hydraulic conductivity of the in-situ soils it may be necessary to provide an impermeable liner to the sides of the bioretention filter media to prevent horizontal ex-filtration and subsequent short-circuiting of the treatment provided by the filter media. Bioretention basins promoting ex-filtration do not require perforated under-drains at the base of the filter media or a drainage layer (refer to Section 5.2.5).

5.2.4 Vegetation Types

Vegetation is required to cover the whole bioretention filter media surface, be capable of withstanding minor and major design flows, and be of sufficient density to prevent preferred flow paths, scour and re-suspension of deposited sediments. Additionally, vegetation that grows in the bioretention filter media functions to continuously break up the surface of the filter media through root growth and wind induced agitation, which prevents surface clogging. The vegetation also provides a substrate for biofilm growth within the upper layer of the filter media, which facilitates biological transformation of pollutants (particularly nitrogen).

Plate 5-2: Established Vegetation

Ground cover vegetation (e.g. sedges and tufted grasses) is an essential component of bioretention basin function. Generally, the greater the density and height of vegetation planted in bioretention filter media, the better the treatment provided especially when extended detention is provided for in the design. This occurs when stormwater is temporarily stored and the contact between stormwater and vegetation results in enhanced sedimentation of suspended sediments and adsorption of associated pollutants.

Appendix A provides more specific guidance on the selection of appropriate vegetation for bioretention basins. It should be noted that turf is not considered to be suitable vegetation for bioretention basins. The stem and root structure of turf is not suitably robust and rapid growing to ensure the surface of the bioretention filter media is continuously broken up to prevent clogging.

5.2.5 Bioretention Filter Media

Selection of an appropriate bioretention filter media is a key design step that involves consideration of the following three inter-related factors:

- Saturated hydraulic conductivity required to optimise the treatment performance of the bioretention basin given site constraints and available filter media area.
- Depth of extended detention provided above the filter media.
- Suitability as a growing media to support vegetation (i.e. retains sufficient soil moisture and organic content).

The high rainfall intensities experienced in SEQ relative to the southern capital cities means bioretention treatment areas tend to be larger in SEQ to achieve the same level of stormwater treatment. However, the area available for providing bioretention basins within the urban layout will often be constrained by the same factors defining available treatment area as apply in the southern capital cities. Consequently, bioretention filter media in SEQ is often required to have higher saturated hydraulic conductivity and extended detention depths. However, it is important to ensure the saturated hydraulic conductivity does not become too high so it can no longer retain enough moisture to sustain vegetation growth. The maximum saturated hydraulic conductivity should not exceed 500 mm/hr (and preferably be less than 200 mm/hr).

The concept design stage will have established the optimal combination of filter media saturated hydraulic conductivity and extended detention depth using a continuous simulation modeling approach (i.e. MUSIC). Any adjustment of either of these two design parameters during the detailed design stage will require the continuous simulation modeling to be re-run to assess the impact on the overall treatment performance of the bioretention basin.

As shown in Figure 5-2, bioretention media can consist of three layers. In addition to the filter media required for stormwater treatment, a drainage layer is also required to convey treated water from the base of the filter media into the perforated under-drains. The drainage layer surrounds the perforated under-drains and can be either coarse sand (1 mm) or fine gravel (2-5 mm). If fine gravel is used, a transition layer of sand must also be installed to prevent migration of the filter media into the drainage layer and subsequently into the perforated under-drains.

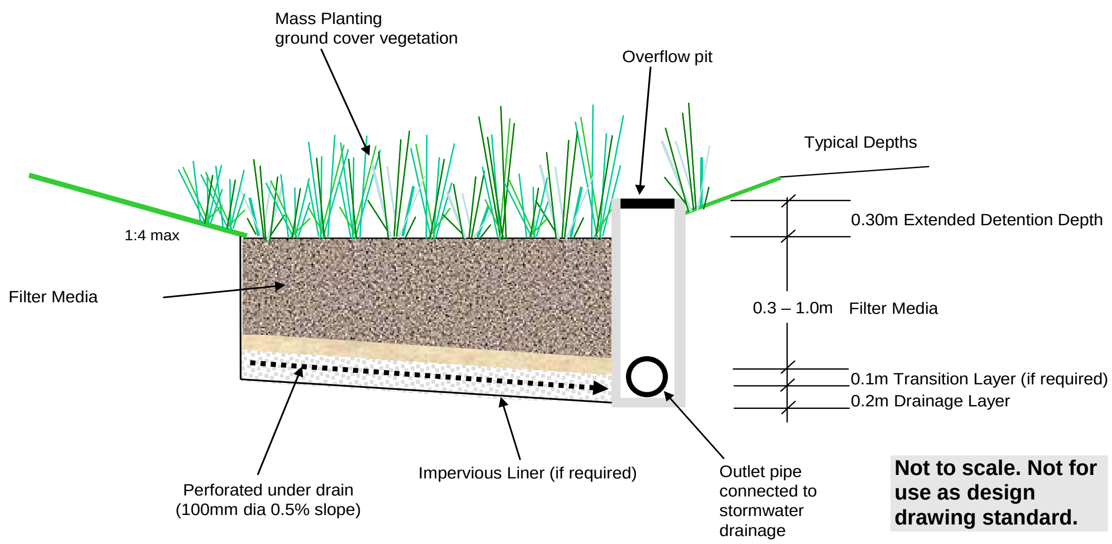


Figure 5-2: Typical Cross Section of a Bioretention Basin

5.2.6 Traffic Controls

Another design consideration is keeping traffic and building material deliveries off bioretention basins (particularly during the construction phase of a development). If bioretention basins are driven over or used for parking, the filter media will become compacted and the vegetation damaged. As they can cause filter media blockages, building materials and wash down wastes should also be kept out of the bioretention basin. To prevent vehicles driving on bioretention basins, and inadvertent placement of building materials, it is necessary to consider appropriate traffic control solutions as part of the design. These can include dense vegetation planting that will discourage the movement of vehicles onto the bioretention basin or providing physical barriers such as bollards and/ or tree planting.

Streetscape bioretention systems must be designed to satisfy local authority requirements with respect to traffic calming devices within particular street or road reserve widths. Where bioretention is incorporated into traffic calming or control devices, or directly adjacent to mountable kerbs, consideration should be given to protection of the area immediately behind the kerb where vehicles are likely to mount the kerb.

5.2.7 Services

Bioretention basins or cells located within road verges or within footpaths must consider the standard location for services within the verge and ensure access for maintenance of services without regular disruption or damage to the bioretention system.

5.3 Design Process

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

1. Confirm treatment performance of concept design

↓

2. Determine design flows

↓

3. Design inflow systems

a. Inlet scour protection
b. Coarse sediment forebay
c. Streetscape bioretention application

↓

4. Specify the bioretention filter media characteristics

a. Filter media
b. Drainage layer
c. Transition layer

↓

5. Design under-drain and undertake capacity checks

↓

6. Check requirement for impermeable lining

↓

7. Size overflow pit

↓

8. Specify vegetation

↓

9. Undertake verification checks

a. Vegetation scour velocity check
b. Confirm treatment performance

5.3.1 Step 1: Check Treatment Performance of Concept Design

Before commencing detailed design, the designer should first undertake a preliminary check to confirm the bioretention basin treatment area (i.e. the surface area of the filter media) from the concept design is adequate to deliver the required level of stormwater quality improvement. This design process assumes a conceptual design has been undertaken. The bioretention basin treatment performance curves shown in Figure 5-3 to Figure 5-5 can be used to undertake this verification check. These curves are intended to provide an indication only of appropriate sizing and do not substitute the need for a thorough conceptual design process.

The curves in Figure 5-3 to Figure 5-5 were derived using the *Model for Urban Stormwater Improvement Conceptualisation* (MUSIC), assuming the bioretention basin is a stand alone system (i.e. not part of a treatment train). The curves show the total suspended solid (TSS), total phosphorus (TP) and total nitrogen (TN) load removal performance for a typical bioretention basin design, being:

- Filter media saturated hydraulic conductivity (k) = 200 mm/hr
- Filter Media average particle size = 0.5 mm
- Filter Media Depth = 0.6 m
- Extended Detention Depth = 0.2 m.

It should be noted that the curves show the pollutant load reduction for this configuration, and are designed for comparison with SEQ load-based water quality targets. These curves should not be used to assess performance of a bioretention basin against concentration-based objectives.

The curves in Figure 5-3 to Figure 5-5 are generally applicable to bioretention basin applications within residential, industrial and commercial land uses. Curves are provided for four rainfall station locations selected as being broadly representative of the spatial and temporal climatic variation across South East Queensland. The shaded area on each of the curves indicates where the bioretention basin performance meets the Best Practice Pollutant Load Reduction Targets for South East Queensland.

If the configuration of the bioretention basin concept design is significantly different to that described above, or if the basin is part of a treatment train, then the curves in Figure 5-3 to Figure 5-5may not provide an accurate indication of treatment performance. In these cases, the detailed designer should use MUSIC to verify bioretention basin concept designs that are part of a "treatment train"(if not already undertaken as part of concept design process).

The curves in Figure 5-3 to Figure 5-5 provide the detailed designer with a useful visual guide to illustrate the sensitivity of bioretention basin performance to the ratio of bioretention basin treatment area and contributing catchment area. The curves allow the detailed designer to make a rapid assessment as to whether the bioretention basin concept design falls within the "optimal bioretention basin size range" or if it is potentially under or over-sized. An under-sized system might indicate the basin is part of a "treatment train" or that another supplementary treatment device may be located somewhere else within the catchment. This should be checked by the detailed designer. An over-sized system suggests the concept designer may have inadvertently sized the basin such that it is operating well beyond its point of "diminishing performance" (i.e. where incremental increases in bioretention basin size, and thus cost, result in only a marginal increase in treatment performance). In this instance, the detailed designer should confirm whether or not the bioretention basin size can be reduced or if additional treatment devices may be required.

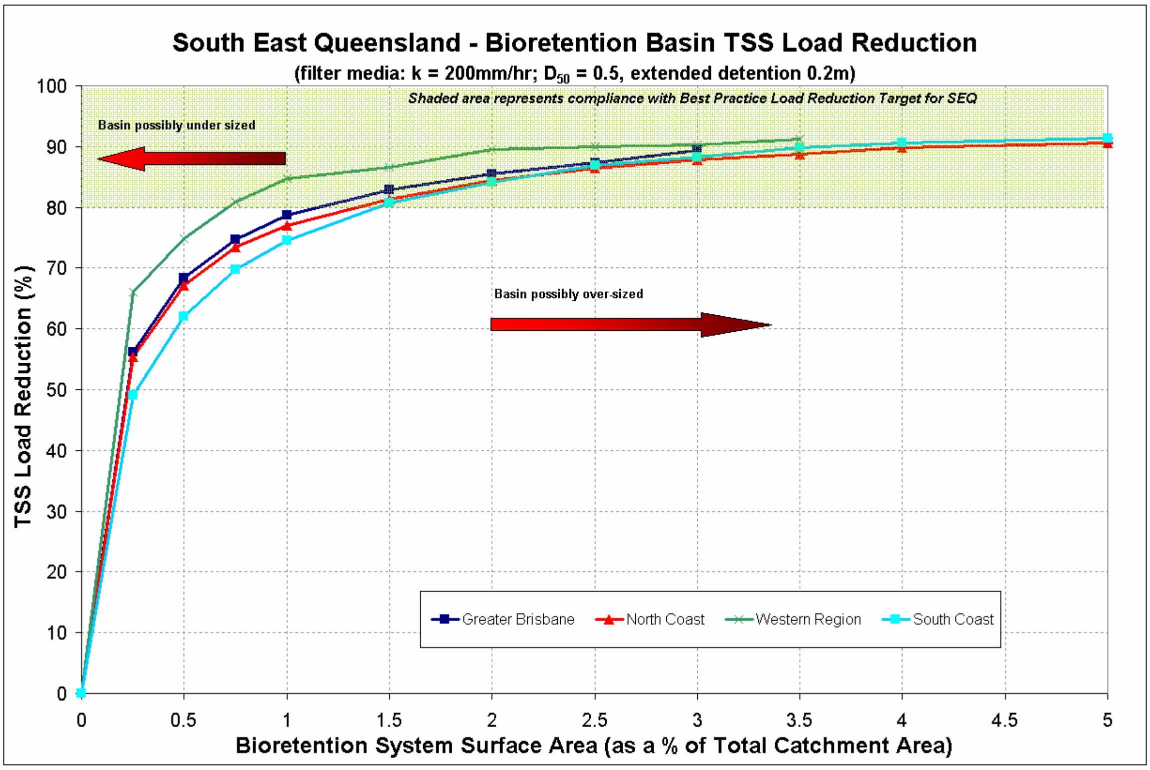


Figure 5-3: Bioretention Basin TSS Removal Performance

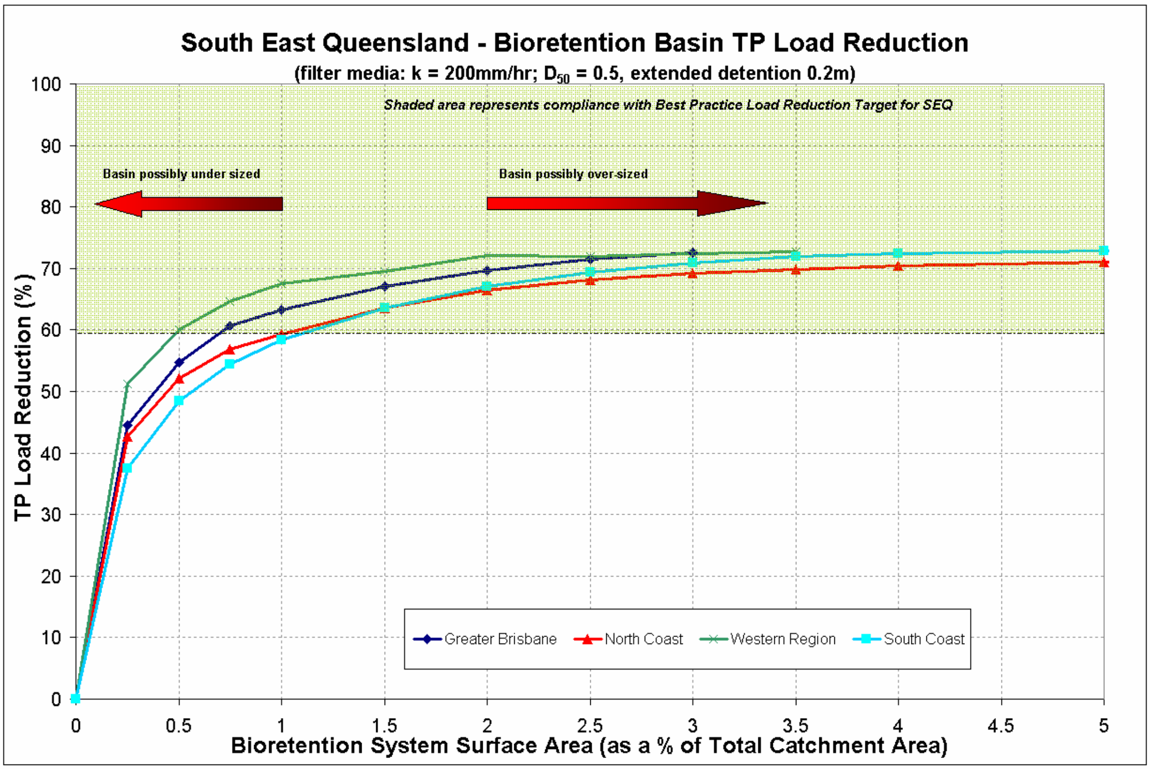


Figure 5-4: Bioretention Basin TP Removal Performance

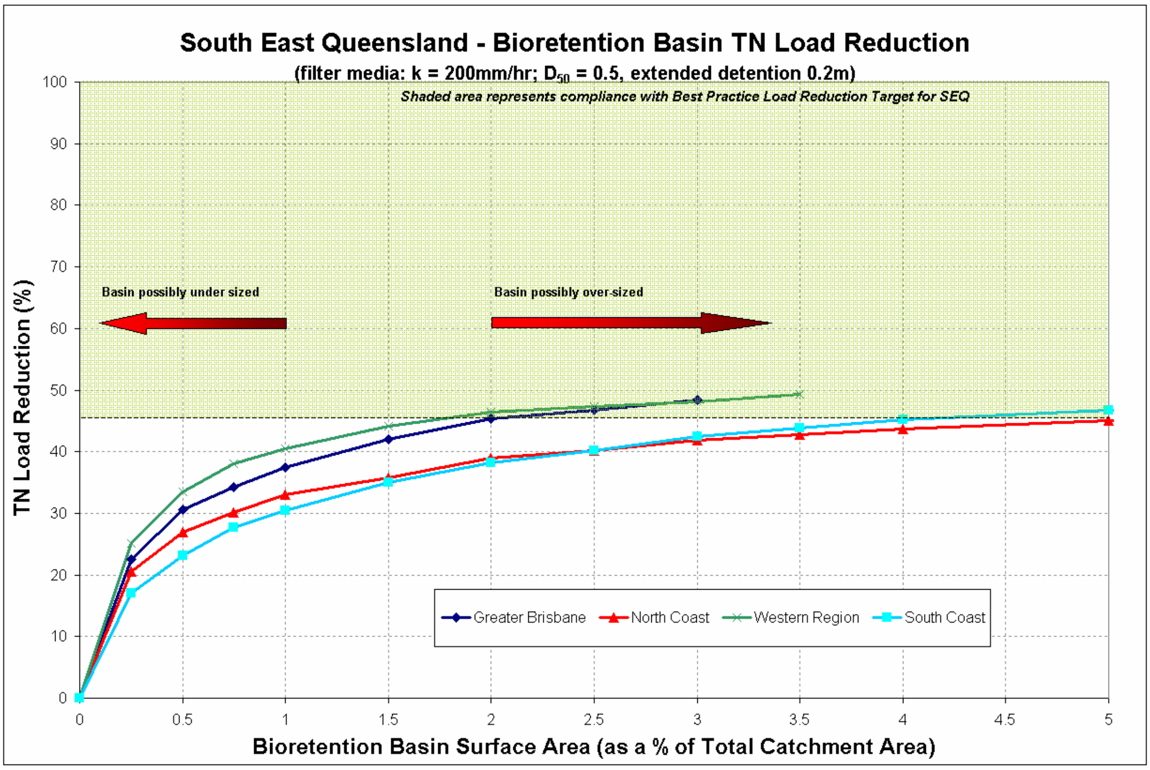


Figure 5-5: Bioretention Basin TN Removal Performance

5.3.2 Step 2: Determine Design Flows

5.3.2.1 Design Flows

The hydraulic design of the bioretention basin should be based on the following design flows:

- Minor Storm Event for sizing the inflow system and the overflow pit as well as undertaking the minor storm flow velocity check. The minor storm varies between the local governments in SEQ but is typically the 2, 5 or 10 yr ARI event.
- Major Storm Event for undertaking the major storm flow velocity check where the bioretention basin accepts the major storm event. The major storm varies between the local governments in SEQ but is typically the 50 or 100 yr ARI event.

5.3.2.2 Design Flow 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, if the bioretention system is to form part of a retention basin (Section 6.2.6) or if the catchment area to the bioretention system is large, then a full flood routing computation method needs to be used to estimate design flows.

5.3.3 Step 3: Design Inflow Systems

The design of the inflow systems to bioretention basins needs to consider a number of functions:

- Scour protection – In most bioretention applications stormwater flows will enter the bioretention basin as concentrated flow (piped, channel or open channel) and as such is it important to slow and spread flows using appropriate scour (rock) protection.

- Coarse sediment forebay – Where stormwater runoff from the catchment is delivered directly to the bioretention basin without any coarse sediment management (through vegetated swale or buffer treatment) a coarse sediment forebay is to be included in the design. The forebay is to remove coarse sediment (1mm +) from stormwater to minimise the risk of vegetation in the bioretention basin being smothered.
- Street hydraulics (streetscape applications only) – In streetscape applications, where stormwater is delivered directly from the kerb and channel to the bioretention basin, it is important to ensure the location and width of the kerb opening is designed such that flows enter the bioretention basin without adversely affecting trafficability of the road (QDUM, Table 5.09.01).

Each of these functions and the appropriate design responses are described in the following sections.

5.3.3.1 Inlet Scour Protection

Erosion protection should be provided for concentrated inflows to a bioretention basin. Typically, flows will enter the bioretention basin from either a surface flow system (i.e. roadside kerb, open channel) or a piped drainage system. Rock beaching is a simple method for dissipating the energy of concentrated inflow. Where the bioretention basin is receiving stormwater flows from a piped system (i.e. from larger catchments), the use of impact type energy dissipation may be required to prevent scour of the filter media. In most cases this can be achieved with rock protection and by placing several large rocks in the flow path to reduce velocities and spread flows as depicted in Figure 5-6 (with the 'D' representing the pipe diameter dimension). The rocks can form part of the landscape design of the bioretention component.

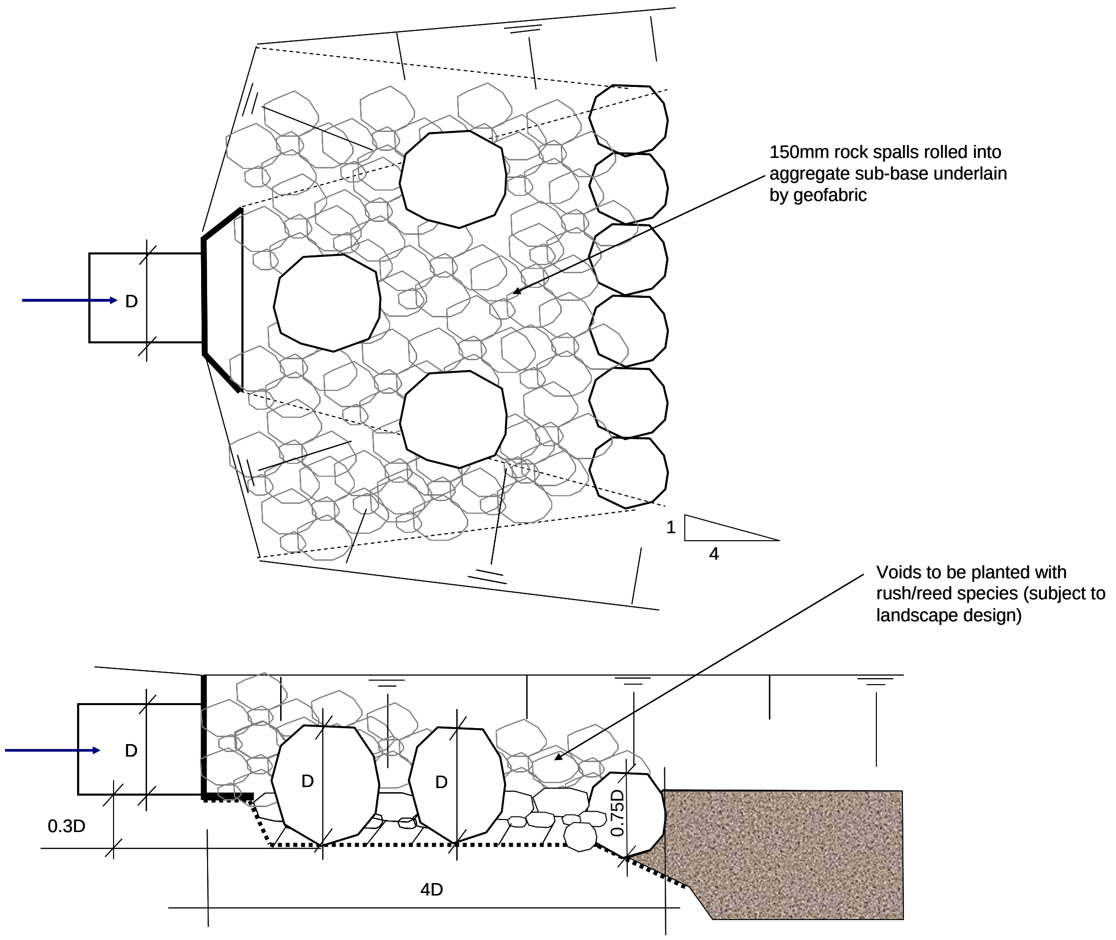


Figure 5-6: Typical Inlet Scour Protection Detail for Bioretention Basins Receiving Piped Flows

5.3.3.2 Coarse Sediment Forebay

Where stormwater runoff from the catchment is delivered directly to the bioretention basin without pre-treatment (through vegetated swale or buffer treatment), coarse sediment may accumulate near the basin inflow. This sediment may smother vegetation and reduce infiltration to the filter media. To mitigate these effects, either a sedimentation basin (see Chapter 4) must be located upstream or the bioretention basin inflow system should incorporate a coarse sediment forebay. The forebay should be designed to:

- Remove particles that are 1mm or greater in diameter from the 3 month ARI storm event.
- Provide appropriate storage for coarse sediment to ensure desilting is required once every year.

The size of the sediment forebay is established using the following:

$$V_s = A_c \cdot R \cdot L_o \cdot F_c$$ Equation 5.1

Where V_s = volume of forebay sediment storage required (m^3)

A_c = contributing catchment area (ha)

R = capture efficiency (assume 80 %)

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

F_c = desired cleanout frequency (years)

A catchment loading rate (L_o) of 1.6 m^3/ha/year for developed catchments can be used to estimate the sediment loads entering the basin. The area of the forebay is established by dividing the volume by the depth. The depth of the forebay should not be greater than 0.3m below the surface of the filter media.

$$A_s = \frac{V_s}{D_s}$$ Equation 5.2

Where D = depth of sediment forebay (max 0.3m below filter media surface)

The sediment forebay area should be checked to ensure it captures the 1mm and greater particles using the following expression (modified version of Fair and Geyer (1954)):

$$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A}\right]^{-n}$$ Equation 5.3

Where:

R = fraction of target sediment removed (80 %)

v_s = settling velocity of target sediment (100 mm/s or 0.1 m/s for 1 mm particle)

Q/A = applied flow rate divided by 'forebay' surface area ($m^3/s/m^2$)

n = turbulence or short-circuiting parameter (adopt 0.5)

The coarse sediment forebays will contain large rocks for energy dissipation and be underlain by filter material to promote drainage following storm events.

5.3.3.3 Kerb Opening Configuration (Streetscape Applications)

In streetscape applications where stormwater is delivered directly from a kerb and channel to a bioretention basin, the following design issues must be considered:

- The location of the kerb opening must be designed to ensure flows in the channel do not exceed the maximum allowance widths as defined by QUDM Table 5.09.01 (DPI, IMEA & BCC 1992) and the relevant local authority requirements.
- The width of the kerb opening is designed to allow the design flow to enter the bioretention basin.

Channel flow width at kerb opening

The width of channel flow at the kerb opening during a minor storm event (2-10 year ARI) needs to be checked to ensure it does not exceed the maximum allowable channel flow widths defined by QUDM Table 5.09.01 (DPI, IMEA & BCC 1992) and the local authority requirements. This check can be undertaken by applying Manning's equation or Izzard's equation and ensuring the flow depth does not exceed either the crest of the road or the top of kerb (whichever is lowest).

Design kerb opening width (where kerb and channel is used)

To determine the width of the opening in the kerb to allow flows to enter the bioretention basin, Manning's equation or Izzard's equation (*QUDM Section 5.09.2*) can be used with the kerb, channel and road profile to estimate the flow depth in the kerb and channel at the entry point. Once the flow depth for the minor storm (e.g. 2-10 year ARI) is known, this can then be used to calculate the required width of the opening in the kerb by applying a broad crested weir equation. The opening width is estimated by applying the flow depth in the channel (as *h*) and solving for *L* (opening width).

$$Q = C_w \cdot L \cdot h^{3/2} \qquad \text{Equation 5.4}$$

Where Q = flow (m^3/s)

C_w = weir coefficient (= 1.7)

L = length of opening (m)

h = depth of flow (m)

This method ensures the kerb opening does not result in an increase in the upstream channel flow depth, which in turn ensures the bioretention basin does not impact on the trafficability of the adjoining road pavement as required by the *QUDM*. To ensure the kerb opening width is adequate, additional width factors may be required to account for slope of the kerb and channel, and the angle at which flow meets the kerb opening. This will depend on the location and position of the bioretention system in relation to the kerb and channel. Design of the inflow system within the kerb and channel will need to consider maximizing flow into the bioretention system. The kerb opening can be made more effective by lowering the kerb opening below the channel, increasing the cross fall at the kerb opening or by providing deflectors at the kerb opening.

5.3.4 Step 4: Specify the Bioretention Filter Media Characteristics

At least two (and possibly three) types of media are required in bioretention basins (refer Figure 5-2).

5.3.4.1 Filter Media

The filter media layer provides the majority of the pollutant treatment function, through fine filtration and also by supporting vegetation. The vegetation enhances filtration, keeps the filter media porous, provides substrate for biofilm formation and provides some uptake of nutrients and other stormwater pollutants. As a minimum, the filter media is required to have sufficient depth to support vegetation. Typical depths are between 600-1000 mm with a minimum depth of 300mm accepted in depth constrained situations. It is important to note that if deep rooted plants such as trees are to be planted in bioretention basins, the filter media must have a minimum depth of 800 mm to avoid roots interfering with the perforated under-drain system.

The saturated hydraulic conductivity of the filter media is established by optimising the treatment performance of the bioretention system given site constraints of the particular site (using a continuous simulation model such as MUSIC). Saturated hydraulic conductivity should remain between 50-200 mm/hr (saturated hydraulic conductivity of greater than 500 mm/hr would not be accepted by most Councils). Once the saturated hydraulic conductivity of the filter media has been determined for a particular bioretention basin, the following process can then be applied to derive a suitable filter media soil to match the design saturated hydraulic conductivity:

- Identify available sources of a suitable base soil (i.e. topsoil) capable of supporting vegetation growth such as a sandy loam or sandy clay loam. In-situ topsoil should be considered first before importing a suitable base soil. Any base soil found to contain high levels of salt (see last bullet point), extremely low levels of organic carbon (< 5%), or other extremes considered retardant to plant growth and denitrification should be rejected. The base soil must also be structurally sound and not prone to structural collapse as this can result in a significant reduction in saturated hydraulic conductivity. The risk of structural collapse can be reduced by ensuring the soil has a well graded particle size distribution with a combined clay and silt fraction of < 12%.

- Using laboratory analysis, determine the saturated hydraulic conductivity of the base soil using standard testing procedures (AS 4419-2003 Appendix H Soil Permeability). A minimum of five samples of the base soil should be tested. Any occurrence of structural collapse during laboratory testing must be noted and an alternative base soil sourced.
- To amend the base soil to achieve the desired design saturated hydraulic conductivity either mix in a loose non-angular sand (to increase saturated hydraulic conductivity) or conversely a loose non-dispersive 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. The sand or clay content (by weight) that achieves the desired saturated hydraulic conductivity should then be adopted on-site. A minimum of five samples of the selected base soil and sand (or clay) content mix must be tested in the laboratory to ensure saturated hydraulic conductivity is consistent across all samples. If the average saturated hydraulic conductivity of the final filter media mix is within 20% of the design saturated hydraulic conductivity then the filter media can be adopted and installed in the bioretention system. Otherwise, further amendment of the filter media must occur through the addition of sand (or clay) and retested until the design saturated hydraulic conductivity is achieved.
- The base soil must have sufficient organic content to establish vegetation on the surface of the bioretention system. If the proportion of base soil in the final mix is less than 50%, it may be necessary to add organic material. This should not result in more than 10% organic content (measured in accordance with AS 1289.4.1.1-1997) and should not alter the saturated hydraulic conductivity of the final filter media mix.
- The pH of the final filter media is to be amended (if required) to between 6 and 7. If the filter media mix is being prepared off-site, this amendment should be undertaken before delivery to the site.
- The salt content of the final filter media (as measured by EC1:5) must be less than 0.63 dS/m for low clay content soils like sandy loam. (EC1:5 is the electrical conductivity of a 1:5 soil/ water suspension).

Imported soils must not contain Fire Ants. Visual assessment is required and any machinery should be free of clumped dirt. Soils must not be brought in from Fire Ant restricted areas. For further information on Fire Ant restrictions, contact the Department of Primary Industries and Fisheries.

5.3.4.2 Drainage Layer (if required)

The drainage layer is used to convey treated flows from the base of the filter media layer into the perforated under-drainage system. The composition of the drainage layer is to be considered in conjunction with the selection and design of the perforated under-drainage system (refer to Section 5.3.5) as the slot sizes in the perforated pipes may determine the minimum drainage layer particle size to avoid washout of the drainage layer into the perforated pipe system. Coarser material (e.g. fine gravel) is to be used for the drainage layer if the slot sizes in the perforated pipes are too large for use of a sand based drainage layer. Otherwise, sand is the preferred drainage layer media as it is likely to avoid having to provide a transition layer between the filter media and the drainage layer. The drainage layer is to be a minimum of 200 mm thick.

Ensure drainage media is washed prior to placement in bioretention system to remove any fines. Drainage media must also be free from Fire Ants and visually checked to confirm this. Drainage media must not be imported from a Fire Ant restricted area.

5.3.4.3 Transition Layer (if required)

The particle size difference between the filter media and the underlying drainage layer should be not more than one order of magnitude to avoid the filter media being washed through the voids of the drainage layer. Therefore, if fine gravels are used for the drainage layer (which will be at least two orders of magnitude coarser than the likely average particle size of the filter media), then a transition layer is recommended to prevent the filter media from washing into the perforated pipes. If a transition layer is required then the material must be sand/ coarse sand material. An example particle size distribution (% passing) is provided below (typical specification only):

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

The transition layer is recommended to be 100 mm thick.

The addition of a transition layer increases the overall depth of the bioretention system and may be an important consideration for some sites where total depth of the bioretention system may be constrained. In such cases, the use of a sand drainage layer and/ or perforated pipes with smaller slot sized may need to be considered (Section 5.3.5).

5.3.5 Step 5: Design Under-Drain and Undertake Capacity Checks(if required)

The maximum spacing of the perforated under-drains in bioretention basins located in streetscape zones and small public zones (i.e. bioretention < 100 m^2) is 1.5 m (centre to centre). This ensures that the distance water needs to travel horizontally toward the perforated pipes through the drainage layer does not hinder drainage of the filter media. The maximum spacing of the perforated pipes in bioretention basins located in local parks and large open space areas (i.e. bioretention > 100 m^2) can be increased to 2.5 - 3 m.

Where possible the perforated pipes are to grade at a minimum of 0.5% towards the overflow pit to ensure effective drainage. This is best achieved by grading the base of the bioretention system towards the pit and placing the perforated pipes (and the drainage layer) on this grade. Perforated pipes should not use a geofabric wrapping, as this is a potential location for blockage and would require a complete resetting of the bioretention system. Where perforated pipes are supplied with geofabric wrapping, it is to be removed before installation.

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 slotted PVC pipes or flexible perforated pipes (e.g. Ag pipe) can be used; however, care needs to be taken when selecting the type of pipe to consider the following:

- Ensure the slots in the pipes are not so large that sediment will freely flow into the pipes from the drainage layer. This is also a consideration when specifying drainage layer media.
- Minimise the potential for tree roots to enter the pipes in search of water. Generally, this is only a concern when the filter media has a low water holding capacity, or trees are planted in filter media whose depth is too shallow. In general, trees are not recommended if the filter media depth is less than 800 mm. Flexible ‘ribbed’ pipes are more likely, than PVC pipes, to retain ‘beads’ of moisture due to the small corrugations on the inside of the pipe. Therefore, a smooth surface perforated pipe system is recommended for use in bioretention basins exhibiting any of these characteristics.

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

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

The maximum filtration rate represents the maximum rate of flow through the bioretention filter media and is calculated by applying Darcy’s equation as follows:

$$Q_{max} = K_{sat} \cdot L \cdot W_{base} \cdot \frac{h_{max} + d}{d}$$ Equation 5.5

Where	Q_{max}	=	maximum filtration rate (m^3/s)
	K_{sat}	=	saturated hydraulic conductivity of the soil filter (m/s)
	W_{base}	=	base width of the ponded cross section above the soil filter (m)
	L	=	length of the bioretention zone (m)
	h_{max}	=	depth of pondage above the soil filter (m)
	d	=	depth of filter media (m)

The capacity of the perforated under-drains need to be greater than the maximum filtration rate to ensure the filter media drains freely and does not become the hydraulic 'control' in the bioretention system (i.e. to ensure the filter media sets the travel time for flows percolating through the bioretention system rather than the perforated under-drainage system).

To ensure the perforated under-drainage system has sufficient capacity to collect and convey the maximum filtration rate, it is necessary to determine the capacity for flows to enter the under-drainage system via the perforations in the pipes. To do this, orifice flow can be assumed and the sharp edged orifice equation 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, with the maximum driving head being the depth of the filter media if no extended detention is provided. If extended detention is provided in the design, then the maximum driving head is to the top of extended detention depth. It is conservative, but reasonable to use a blockage factor to account for partial blockage of the perforations by the drainage layer media. A 50% blockage of the perforations should be used.

The flow capacity of the perforations is thus:

$$Q_{perf} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h}$$ Equation 5.6

Where	Q_{perf}	=	flow through perforations (m^3/s)
	C_d	=	orifice discharge coefficient (0.6)
	A	=	total area of the orifice (m^2)
	g	=	gravity (9.80665 m/s^2)
	h	=	maximum depth of water above the pipe (m)
	B	=	blockage factor (0.5)

If the capacity of the drainage system is unable to collect the maximum filtration rate additional under-drains will be required.

After confirming the capacity of the under-drainage system to collect the maximum filtration rate, it is necessary to confirm the conveyance capacity of the underdrainage system is sufficient to convey the collected runoff. To do this, Manning's equation can be used (which assumes pipe full flow but not under pressure). The Manning's roughness used will be dependant on the type of pipe used (refer to QUDM Table 5.21.3 (DPI, IMEA & BCC 1992)).

Under-drains should be extended vertically to the surface of the bioretention system to allow inspection and maintenance when required. The vertical section of the under-drain should be unperforated and capped to avoid short-circuiting of flows directly to the drain. Reference is made to the drawings following the worked example (Section 5.9) for further guidance.

5.1.1 Step 6: Check Requirement for Impermeable Lining

The saturated hydraulic conductivity of the natural soil profile surrounding the bioretention system should be tested together with depth to groundwater, chemical composition and proximity to structures and other infrastructure. This is to establish if an impermeable liner is required at the base (only for systems designed to preclude ex-filtration to in-situ soils) and/or sides of the bioretention basin (refer also to discussion in Section 5.2.3). If the saturated hydraulic conductivity of the filter media in the bioretention system is more than one order of magnitude (10 times) greater than that of the surrounding in-situ soil profile, no impermeable lining is required.

5.1.2 Step 7: Size Overflow Pit

The intention of the overflow pit design is to safely convey the minor floods to the same level of protection that a conventional stormwater system would provide. Bioretention basins are typically served with either grated overflow pits or conventional side entry pits located downstream of an inlet. The location of the overflow pit is variable but must ensure that above design flows do not pass through the length of the bioretention system.

In bioretention basins, the overflow pit is designed with the pit crest raised above the level of the bioretention filter media, to establish the design extended detention depth (i.e. maximum ponding depth). Typically, grated pits are used. The allowable head for discharges into the pits is the difference in level between the pit crest and the maximum permissible water level to satisfy minimum freeboard requirements as defined in the *QUDM* and the relevant Council design guidelines. Depending on the location of the bioretention basin, the design flow to be used to size the overflow pit could be the minor flood flow (streetscape) or the major flood flow. There should be a minimum of 50 mm head over the overflow pit crest to facilitate discharge of the design flow into the overflow pit.

In streetscape bioretention applications, a level of conservatism is built into the design of grated overflow pits by placing their inverts at least 50 mm below the invert of the street channel (and therefore setting the maximum ponding depth). The head over the overflow pit crest is the sum of the 50 mm and the maximum ponding in the street channel under the minor storm (see Section 5.3.3.3). The overflow pit can be located near the inflow zone, and where designed for the minor storm, may be used in lieu of a traditional road gully pit. The overflow pit can also be external to the bioretention basin, potentially in the kerb and channel immediately downstream of the inlet to the basin in streetscape applications. In this way, the overflow pit can operate in the same way as a conventional side entry pit, with flows entering the pit only when the bioretention basin is at maximum ponding depth.

To size an overflow pit, two checks must be made to test 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 openings required in the grate cover (assuming drowned outlet conditions). The larger of the two pit configurations should be adopted (as per Section 5.10 QUDM (DPI, IMEA & BCC 1992)). In addition, a blockage factor that assumes the grate is 50% blocked is to be used.

For free overfall conditions (weir equation):

$$Q_{weir} = B \cdot C_w \cdot L \cdot h^{3/2}$$ Equation 5.7

Where	Q_{weir}	=	flow over weir (pit) (m^3/s)
	B	=	blockage factor (= 0.5)
	C_w	=	weir coefficient (= 1.66)
	L	=	Length of weir (m)
	h	=	flow depth above the weir (m)

Once the length of weir is calculated, a standard sized pit can be selected with a perimeter at least the same length of the required weir length.

For drowned outlet conditions (orifice equation):

$$Q_{orifice} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h}$$ Equation 5.8

Where *B*, *g* and *h* have the same meaning as above

$Q_{orifice}$ = flow rate under drowned conditions (m^3/s)

C_d = discharge coefficient (drowned conditions = 0.6)

A = area of orifice (perforations in inlet grate) (m^2)

When designing grated field inlet pits, reference is also to be made to the procedure described in QUDM Section 5.10.4.

In terms of the actual grate, letter box or dome type grates must be used in bioretention basins. An example of acceptable letter box solutions is provided in Brisbane City Council's Standard Drawings UMS 157 and UMS 337.

5.3.6 Step 8: Specify Vegetation

Refer to Section 5.4 and Appendix A for advice on selecting vegetation for bioretention basins in SEQ. Consultation with landscape architects is recommended when selecting vegetation to ensure the treatment system also compliments the landscape of the area.

5.3.7 Step 9: Undertake Verification Checks

Once the detailed design is complete, a final check should be undertaken to confirm that vegetation will be protected from scour during flood events and that the final design will achieve the required treatment performance.

5.3.7.1 Vegetation Scour Velocity Check

Scour velocities over the vegetation in the bioretention basin are determined by assuming the system flows at a depth equal to the maximum ponding depth across the full width of the system. By dividing the minor and major storm design flow rates by this cross sectional flow 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 major storm), however this will ensure the integrity of the vegetation.

Velocities should be kept below:

- 0.5 m/s for minor flood (2-10 year ARI) discharges.
- 2.0 m/s for major flood (50-100 year ARI) discharges.

If the inlet to the bioretention basin 'controls' the maximum inflow to the basin then it is appropriate to use this maximum inflow to check velocities. In this case, velocities should be maintained below 0.5 m/s.

5.3.7.2 Confirm Treatment Performance

If, during the course of undertaking detailed design of the bioretention basin, the basic design parameters established by the conceptual design phase have changed (e.g. area, filter media depth, etc.) then the designer should verify that the current design meets the required water quality improvement performance. This can be done by referring to Figure 5-3 to Figure 5-5 or simulating the design using MUSIC.

5.3.8 Design Calculation Summary

A calculation summary sheet for the key design elements of a bioretention basin is provided below.

BIORETENTION BASIN DESIGN CALCULATION SUMMARY

Calculation Task		CALCULATION SUMMARY Outcome		Check
	Catchment Characteristics			
	Catchment area		Ha	
	Catchment land use (i.e residential, commercial etc.)			
	Storm event entering inlet		yr ARI	
	Conceptual Design			
	Bioretention area		m^2	
	Filter media saturated hydraulic conductivity		mm/hr	
	Extended detention depth		mm	
1	**Verify size for treatment**			
	Bioretention area to achieve water quality objectives			
	Total suspended solids (Figure 5-3)		% of catchment	
	Total phosphorus (Figure 5-4)		% of catchment	
	Total nitrogen (Figure 5-5)		% of catchment	
	Bioretention area		m^2	
	Extended detention depth		m	
2	**Determine design flows**			
	Time of concentration			
	Refer to relevant local authority guidelines and QUDM		minutes	
	Identify rainfall intensities			
	Minor Storm ($I_{2\text{-}10\text{ year ARI}}$)		mm/hr	
	Major Storm ($I_{50\text{-}100\text{ year ARI}}$)		mm/hr	
	Design runoff coefficient			
	Minor Storm ($C_{2\text{-}10\text{ year ARI}}$)			
	Major Storm($C_{50\text{-}100\text{ year ARI}}$)			
	Peak design flows			
	Minor Storm (2-10 year ARI)		m^3/s	
	Major Storm (50-100 year ARI)		m^3/s	
3	**Design inflow systems**			
	Adequate erosion and scour protection?			
	Coarse Sediment Forebay Required?			
	Volume (V_s)		m^3	
	Area (A_s)		m^2	
	Depth (D)		m	
*	Check flow widths in upstream channel			
	Minor storm flow width		m	
	CHECK ADEQUATE LANES TRAFFICABLE			
*	Kerb opening width			
	Kerb opening length		m	
4	**Specify bioretention media characteristics**			
	Filter media hydraulic conductivity		mm/hr	
	Filter media depth		mm	
	Drainage layer media (sand or fine screenings)			
	Drainage layer depth		mm	
	Transition layer (sand) required			
	Transition layer depth		mm	
5	**Under-drain design and capacity checks**			
	Flow capacity of filter media		m^3/s	
	Perforations inflow check			
	Pipe diameter		mm	
	Number of pipes			
	Capacity of perforations		m^3/s	
	CHECK PERFORATION CAPACITY > FILTER MEDIA CAPACITY			
	Perforated pipe capacity			
	Pipe capacity		m^3/s	
	CHECK PIPE CAPACITY > FILTER MEDIA CAPACITY			

BIORETENTION BASIN DESIGN CALCULATION SUMMARY

	Calculation Task		CALCULATION SUMMARY Outcome		Check
6	Check requirement for impermeable lining				
		Soil hydraulic conductivity		mm/hr	
		Filter media hydraulic conductivity		mm/hr	
		MORE THAN 10 TIMES HIGHER THAN IN-SITU SOILS?			
7	Size overflow pit				
		System to convey minor floods (2-10yr ARI)		L x W	
8	Verification Checks				
		Velocity for Minor Storm (<0.5m/s)		m/s	
		Velocity for Major Storm (<2.0m/s)		m/s	
		Treatment performance consistent with Step 1			

* Relevant to streetscape application only

5.4 Landscape Design Notes

5.4.1 Objectives

Landscape design for bioretention basins has four key objectives:

- Addressing stormwater quality objectives by incorporating appropriate groundcover plant species for sediment removal, erosion protection, stormwater treatment (biologically active root zone) and preventing filter media blockages.
- Ensuring that the overall landscape design for the bioretention basin integrates with its surrounding environment.
- Incorporating Crime Prevention through Environmental Design (CPTED) principles and traffic visibility safety standards for roadside systems. This objective also needs to incorporate public safety.
- Providing other landscape values such as shade, amenity, character, buffers, glare reduction, place making and habitat.

Landscape treatments to bioretention basins will largely depend on their context and size. For example, planter box type systems in urban areas will require a different approach than larger systems located in open space areas. Comprehensive site analysis should inform the landscape design as well as road layouts, civil works and maintenance access requirements. Existing site factors such as roads, driveways, buildings, landforms, soils, plants, microclimates, services and views should be considered. Refer to *Water Sensitive Urban Design in the Sydney Region: 'Practice Note 2 – Site Planning'* (LHCCREMS 2002) for further guidance.

5.4.2 Bioretention Basin Vegetation

Planting for bioretention basin elements may consist of up to three vegetation types:

- Groundcovers for stormwater treatment and erosion protection
- Shrubbery for screening, glare reduction and character
- Trees for shading, character and other landscape values.

For specific guidance on plant species the designer is initially directed to relevant guidelines provided by the local authority. In the absence of local guidance the designer can refer to Appendix A Plant Selection for WSUD Systems which outlines plant species suitable for Brisbane.

The following sections describe the functional requirements of the different types of vegetation that can be applied to bioretention basins.

5.4.2.1 Groundcovers

Groundcover vegetation (e.g. sedges and tufted grasses) is an essential functional component of bioretention basins. Generally, the greater the density and height of vegetation planted in bioretention filter media, the better the treatment provided especially when extended detention is provided for in the design. This occurs when stormwater is temporarily stored and the contact between stormwater and vegetation results in enhanced sedimentation of suspended sediments and adsorption of associated pollutants.

Additionally, groundcover vegetation plays the primary role of continuously breaking up the surface of the bioretention filter media through root growth and wind induced agitation, which prevents surface clogging. The vegetation also provides a substrate for biofilm growth within the upper layer of the filter media, which facilitates biological transformation of pollutants (particularly nitrogen).

In general ground cover vegetation should:

- Cover the whole bioretention filter media surface.
- Possess high leaf density within the design extended detention depth to aid efficient stormwater treatment.
- A dense and uniform distribution to prevent preferred flow paths, to prevent scour/resuspension and to create a uniform root zone within the bioretention filter media.
- Where appropriate, be endemic to the area and as a minimum be local to SEQ. Species (including natives) that have the potential to become invasive weeds should be avoided.
- Tolerate short periods of inundation (and water logged soils) punctuated by longer dry periods.

5.4.2.2 Shrubs and Trees

Shrubs and trees are not a functional requirement for bioretention basins but can be designed into the systems to ensure integration within the wider landscape (streetscape or parkscape) and to provide amenity, character and habitat. When incorporating trees and shrubs into bioretention systems appropriate space should be allowed between the systems to promote an open canopy that allows sunlight to penetrate to groundcover plants. Additionally, trees and shrubs must be accompanied by shade tolerant groundcover species.

In general, trees and shrubs planted into bioretention basins should have the following features:

- Able to tolerate short periods of inundation (and water logged soils) punctuated by longer dry periods.
- Have relatively sparse canopies to allow light penetration to support dense groundcover vegetation.
- Root systems that are relatively shallow and are not known to be adventurous 'water seekers' to reduce the risk of root intrusion into under-drainage pipes.
- Preferably native to the SEQ region and not exotic or deciduous.

5.4.3 Other specific Landscape considerations

5.4.3.1 Planter Boxes

Planter boxes are relatively small WSUD elements that are most applicable to highly urbanised contexts. In well used areas, planter boxes are likely to be highly visible elements that could become local features. The urban landscape design principles of form, colour, texture and massing should apply to both plantings and raised containers. An irrigation system may be required to provide supplementary watering.

5.4.3.2 Parkland Bioretention Basins

Once the general location has been determined, it will be necessary to investigate how the elements of the bioretention system will be arranged within the open space including:

- opportunities and constraints presented by various siting options.
- if the device is to be visually prominent (perhaps for educational value) or merged with the surrounding parkland space using a consistent planting layout in the basin, embankment and parkland.
- a formal or informal style dependent on the setting and surrounding open space and urban design.

5.4.4 Safety Issues

The standard principles of informal surveillance, exclusion of places of concealment and open visible areas apply to the landscape design of bioretention basins. Regular clear sightlines should be provided between the roadway and footpaths/ property. Safety measures in accordance with the requirements of the relevant local authority should also be installed around structural components of bioretention basins where safety hazards exist.

5.4.4.1 Crime Prevention Through Environmental Design (CPTED)

Where planting may create places of concealment or hinder informal surveillance, groundcovers and shrubs should not generally exceed 1 m in height. For specific guidance on CPTED requirements the designer is initially directed to relevant guidelines provided by the local authority, however, in the absence of local guidance the designer can refer to BCC's CPTED Planning Scheme Policy in *Brisbane City Plan 2000* (BCC 2000b, vol. 2, app. 2, pp. 68a – 68f) and associated references.

5.4.4.2 Traffic Sightlines

The standard rules of sightline geometry apply. Planting designs should allow for visibility at pedestrian crossings, intersections, rest areas, medians and roundabouts. Refer to the *Road Landscape Manual* (DMR 1997) for further guidance.

5.5 Construction and Establishment

This section provides general advice for the construction and establishment of bioretention basins and key issues to be considered to ensure their successful establishment and operation. Some of the issues raised have been discussed in other sections of this chapter and are reiterated here to emphasise their importance based on observations from construction projects around Australia.

5.5.1 Staged Construction and Establishment Method

It is important to note that bioretention basin systems, like most WSUD elements that employ soil and vegetation based treatment processes, require approximately two growing seasons (i.e. two years) before the vegetation in the systems has reached its design condition (i.e. height and density). In the context of a large development site and associated construction and building works, delivering bioretention basins and establishing vegetation can be a challenging task. Therefore, bioretention basins require a careful construction and establishment approach to ensure the basin establishes in accordance with its design intent. The following sections outline a recommended staged construction and establishment methodology for bioretention basins (Leinster, 2006).

5.5.1.1 Construction and Establishment Challenges

There exist a number of challenges that must be appropriately considered to ensure successful construction and establishment of bioretention basins. These challenges are best described in the context of the typical phases in the development of a Greenfield or Infill development, namely the Subdivision Construction Phase and the Building Phase (see Figure 5.7).

- <u>Subdivision Construction</u> - Involves the civil works required to create the landforms associated with a development and install the related services (roads, water, sewerage, power etc.) followed by the landscape works to create the softscape, streetscape and parkscape features. The risks to successful construction and establishment of the WSUD systems during this phase of work have generally related to the following:
  - Construction activities which can generate large sediment loads in runoff which can smother vegetation and clog bioretention filter media
  - Construction traffic and other works can result in damage to the bioretention basins.

  Importantly, all works undertaken during Subdivision Construction are normally 'controlled' through the principle contractor and site manager. This means the risks described above can be readily managed through appropriate guidance and supervision.

Plate 5-3: Example of Building Phase destruction

- <u>Building Phase</u> - Once the Subdivision Construction works are complete and the development plans are sealed then the Building Phase can commence (i.e. construction of the houses or built form). This phase of development is effectively 'uncontrolled' due to the number of building contractors and sub-contractors present on any given allotment. For this reason the Allotment Building Phase represents the greatest risk to the successful establishment of bioretention basins.

5.5.1.2 Staged Construction and Establishment Method

To overcome the challenges associated within delivering bioretention basins a Staged Construction and Establishment Method should be adopted (Figure 5-7):

- <u>Stage 1: Functional Installation</u> - Construction of the functional elements of the bioretention basin at the end of Subdivision Construction (i.e. during landscape works) and the installation of temporary protective measures. For example, temporary protection of bioretention basins can been achieved by using a temporary arrangement of a suitable geofabric covered with shallow topsoil (e.g. 25mm) and instant turf, in lieu of the final basin planting.

- Stage 2: Sediment and Erosion Control – During the Building Phase the temporary protective measures preserve the functional infrastructure of the bioretention basins against damage whilst also providing a temporary erosion and sediment control facility throughout the building phase to protect downstream aquatic ecosystems.
- Stage 3: Operational Establishment - At the completion of the Building Phase, the temporary measures protecting the functional elements of the bioretention basins can be removed along with all accumulated sediment and the system planted in accordance with the design planting schedule.

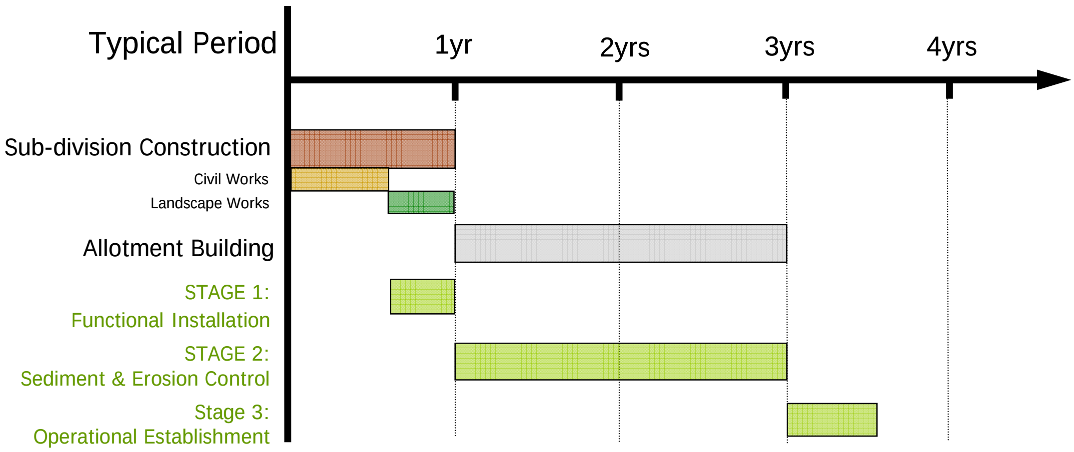


Figure 5-7: Staged Construction and Establishment Method

5.5.1.3 Functional Installation

Functional installation of bioretention basins occurs at the end of Subdivision Construction as part of landscape works and involves:

- Bulking out and trimming
- Installation of the outlet structures
- Placement of liner and installation of drainage layer (i.e. under-drains and drainage layer)
- Placement of filter media
- Placement of a temporary protective layer - Covering the surface of filtration media with geofabric and placement of 25mm topsoil and turf over geofabric. This temporary geofabric and turf layer will protect the bioretention basin during construction (Subdivision and Building Phases) ensuring sediment/litter laden waters do not enter the filter media causing clogging.
- Place silt fences around the boundary of the bioretention basin to exclude silt and restrict access.

Plate 5-4: Bioretention Basin Functional Installation

5.5.1.4 Sediment and Erosion Control

The temporary protective layers are left in place through the allotment building phase to ensure sediment laden waters do not clog the filtration media and allotment building traffic does not enter the bioretention system. Importantly the configuration of the bioretention basin and the turf vegetation allow the system to function effectively as a shallow sedimentation basin reducing the load of coarse sediment discharging to the receiving environment. Using this approach WSUD systems can operate effectively to protect downstream ecosystems immediately after construction.

5.5.1.5 Operational Establishment

At the completion of the Allotment Building Phase the temporary measures (i.e. geofabric and turf) are removed with all accumulated sediment and the bioretention system re-profiled and planted in accordance with the proposed landscape design. Establishment of the vegetation to design condition can require more than two growing seasons, depending on the vegetation types, during which regular watering and removal of weeds will be required.

5.5.2 Construction Tolerances

It is important to emphasise the significance of tolerances in the construction of bioretention basins (e.g. profiling of trench base and surface grades). Ensuring the base of the filtration trench and surface of the bioretention filter media is free from localised depressions resulting from construction is particularly important to achieve even distribution of stormwater flows across the surface and to prevent localised ponding on the surface, which may cause mosquito problems. In addition, to enable the perforated sub-surface drainage pipes to drain freely, the base of the trench should be sloped towards the outlet pit (min 0.5% longitudinal grade). Generally an earthworks tolerance of plus or minus 50 mm is considered acceptable.

Plate 5-5 : Bioretention Basin Sediment & Erosion Control

5.5.3 Sourcing Bioretention Vegetation

Notifying nurseries early for contract growing is essential to ensure the specified species are available in the required numbers and of adequate maturity in time for bioretention basin planting. When this is not done and the planting specification is compromised, poor vegetation establishment and increased initial maintenance costs may occur. The species listed in Appendix A are generally available commercially from local native plant nurseries. Availability is, however, dependent upon many factors including demand, season and seed availability. To ensure planting specification can be accommodated, the minimum recommended lead time for ordering is 3-6 months. This usually allows enough time for plants to be grown to the required size. The following pot sizes are recommended as the minimum:

- Viro Tubes - 50 mm wide x 85 mm deep
- 50 mm Tubes - 50 mm wide x 75 mm deep
- Native Tubes - 50 mm wide x 125 mm deep

5.5.4 Vegetation Establishment

The following weed control measures and watering schedule are recommended to ensure successful plant establishment. Regular general maintenance as outlined in Section 5.6 will also be required.

Plate 5-6: Plant Establishment Period in Bioretention Basin

5.5.4.1 Weed Control

Conventional surface mulching of bioretention basins with organic material like tanbark, should not be undertaken. Most organic mulch floats and runoff typically causes this material to be washed away with a risk of blocking drains. Adopting high planting densities and if necessary, applying a suitable biodegradable erosion control matting to the basin batters will help to combat weed invasion and reduce labour intensive maintenance requirements for weed removal. A heavy application of seedless hydro-mulch can also provide short term erosion and weed control prior to planting with nursery stock. No matting or hydro-mulch is to be applied to the surface of the bioretention basin following the construction phase (i.e. in its final design form, vegetated as per planting schedule), as this will hinder filtration of stormwater through the filter media.

5.5.4.2 Watering

Regular watering of bioretention basin vegetation is essential for successful establishment and healthy growth. The frequency of watering to achieve successful plant establishment is dependent upon rainfall, maturity of planting stock and the water holding capacity of the soil. The following watering program is generally adequate but should be adjusted (increased) to suit the site conditions:

- Week 1-2 3 visits/ week
- Week 3-6 2 visits/ week
- Week 7-12 1 visit/ week

After this initial three month period, watering may still be required, particularly during the first winter (dry period). Watering requirements to sustain healthy vegetation should be determined during ongoing maintenance site visits.

5.6 Maintenance Requirements

Vegetation plays a key role in maintaining the porosity of the filter media of a bioretention basin and a strong healthy growth of vegetation is critical to its performance. Therefore the most intensive period of maintenance is during the plant establishment period (first two years) when weed removal and replanting may be required.

Inflow systems and overflow pits require careful monitoring, as these can be prone to scour and litter build up. Debris can block inlets or outlets and can be unsightly, particularly in high visibility areas. Inspection and removal of debris should be done regularly, and debris should be removed whenever it is observed on a site. Where sediment forebays are adopted, regular inspection of the forebay is required (3 monthly) with removal of accumulated sediment undertaken as required.

For larger bioretention basins, it is essential that a maintenance access point is designed for and maintained in the bioretention basin. The size and complexity of the system will guide its design and may involve provision of a reinforced concrete ramp/ pad for truck or machinery access.

Typical maintenance of bioretention basin elements will involve:

- Routine inspection of the bioretention basin profile to identify any areas of obvious increased sediment deposition, scouring from storm flows, rill erosion of the batters from lateral inflows, damage to the profile from vehicles and clogging of the bioretention basin (evident by a 'boggy' filter media surface).
- Routine inspection of inflows systems, overflow pits and under-drains to identify and clean any areas of scour, litter build up and blockages.
- Removal of sediment where it is smothering the bioretention basin vegetation.
- Where a sediment forebay is adopted, removal of accumulated sediment.
- Repairing any damage to the profile resulting from scour, rill erosion or vehicle damage by replacement of appropriate fill (to match onsite soils) and revegetating.
- Tilling of the bioretention basin surface, or removal of the surface layer, if there is evidence of clogging.
- Regular watering/ irrigation of vegetation until plants are established and actively growing (see Section 5.5.4.2).
- Removal and management of invasive weeds (herbicides should not be used).
- Removal of plants that have died and replacement with plants of equivalent size and species as detailed in the plant schedule.
- Pruning to remove dead or diseased vegetation material and to stimulate growth.
- Vegetation pest monitoring and control.

Resetting (i.e. complete reconstruction) of the bioretention basin will be required if the system fails to drain adequately after tilling of the surface. Maintenance should only occur after a reasonably rain free period when the soil in the bioretention system is dry. Inspections are also recommended following large storm events to check for scour and other damage.

All maintenance activities must be specified in an approved Maintenance Plan (and associated maintenance inspection forms) to be documented and submitted to Council as part of the Development Approval process. Maintenance personnel and asset managers will use this Plan to ensure the bioretention basins continue to function as designed. An example operation and maintenance inspection form is included in the checking tools provided in Section 5.7.3. These forms must be developed on a site-specific basis as the nature and configuration of bioretention basins varies significantly.

5.7 Checking Tools

This section provides a number of checking aids for designers and Council development assessment officers. In addition, Section 5.5 provides general advice for the construction and establishment of bioretention basins and key issues to be considered to ensure their successful establishment and operation based on observations from construction projects around Australia. The following checking tools are provided:

- Design Assessment Checklist
- Construction Inspection Checklist (during and post construction)
- Operation and Maintenance Inspection Form
- Asset Transfer Checklist (following 'on-maintenance' period).

5.7.1 Design Assessment Checklist

The checklist on page 5-30 presents the key design features that are to be reviewed when assessing the design of a bioretention basin. These considerations include configuration, safety, maintenance and operational issues that need to be addressed during the design phase. If an item receives an 'N' when reviewing the design, referral is made back to the design procedure to determine the impact of the omission or error. A copy of the completed Design Calculation Summary from Section 5.3.10 should be provided as part of the application to assist in the design assessment. In addition to the checklist, a proposed design is to have all necessary permits for its installation. Council development assessment officers will require all relevant permits to be in place prior to accepting a design.

5.7.2 Construction Checklist

The checklist on page 5-31 presents the key items to be reviewed when inspecting the bioretention basin during and at the completion of construction. The checklist is to be used by Construction Site Supervisors and local authority Compliance Inspectors to ensure all the elements of the bioretention basin have been constructed in accordance with the design. If an item receives an 'N' in Satisfactory criteria then appropriate actions must be specified and delivered to rectify the construction issue before final inspection sign-off is given.

5.7.3 Operation and Maintenance Inspection Form

The example form on page 5-32 should be developed and 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 1 - 6 months depending on the size and complexity of the system. More detailed site specific maintenance schedules should be developed for major bioretention basins and include a brief overview of the operation of the system and key aspects to be checked during each inspection.

5.7.4 Asset Transfer Checklist

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. For details on asset transfer to specific to each Council, contact the relevant local authority. The table on page 5-33 provides an indicative asset transfer checklist.

BIORETENTION BASIN DESIGN ASSESSMENT CHECKLIST			
Asset I.D.		DA No.	
Basin 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?		
BIORETENTION MEDIA AND UNDER-DRAINAGE	**Y**	**N**
Design documents bioretention area and extended detention depth as defined by treatment performance requirements.		
Overall flow conveyance system sufficient for design flood event(s)?		
Where required, bypass sufficient for conveyance of design flood event?		
Where required scour protection provided at inflow point to bioretention?		
Bioretention media specification includes details of filter media, drainage layer and transition layer (if required)?		
Design saturated hydraulic conductivity included in specification?		
Transition layer provided where drainage layer consists of gravel (rather than coarse sand)?		
Perforated pipe capacity > infiltration capacity of filter media?		
Selected filter media hydraulic conductivity > 10 x hydraulic conductivity of surrounding soil?		
Liner provided if selected filter media hydraulic conductivity < 10 x hydraulic conductivity of surrounding soil?		
Maximum spacing of collection pipes <1.5m?		
Collection pipes extended to surface to allow inspection and flushing?		
*Maximum upstream flood conveyance complies with QUDM?		
*Overflow pit has set down of at least 50mm below kerb invert? (where conventional gully/lintel used downstream of bioretention then no overflow pit required)		
BASIN	**Y**	**N**
Bioretention area and extended detention depth documented to satisfy treatment requirements?		
Overflow pit crest set at top of extended detention?		
Maximum ponding depth will not impact on public safety?		
Maintenance access provided to surface of bioretention system (for larger systems)?		
Protection from coarse sediments provided (where required) with a sediment forebay?		
Protection from gross pollutants provided (where required)?		
LANDSCAPE	**Y**	**N**
Plant species selected can tolerate extended dry periods, periodic inundation and design velocities?		
Bioretention design and plant species selected integrate with surrounding landscape or built environment design?		
*Planting design conforms with acceptable sight line and safety requirements?		
COMMENTS		

*Streetscape application only

BIORETENTION BASIN CONSTRUCTION INSPECTION CHECKLIST			
Asset I.D.		Inspected By:	
Site:		Date:	
		Time:	
Constructed By:		Weather:	
		Contact During Visit:	

Items inspected	Checked		Satisfactory		Items inspected	Checked		Satisfactory	
	Y	N	Y	N		Y	N	Y	N
DURING CONSTRUCTION & ESTABLISHMENT									
A. FUNCTIONAL INSTALLATION					Structural components				
Preliminary Works					15. Location and configuration of inflow systems as designed				
1. Erosion and sediment control plan adopted					16. Location and levels of overflow pits as designed				
2. Temporary traffic/safety control measures					17. Under-drainage connected to overflow pits as designed				
3. Location same as plans					18. Concrete and reinforcement as designed				
4. Site protection from existing flows					19. Set down to correct level for flush kerbs (streetscape applications only)				
Earthworks and Filter Media					20. Kerb opening width as designed				
5. Bed of basin correct shape and slope									
6. Batter slopes as plans					B. SEDIMENT & EROSION CONTROL (IF REQUIRED)				
7. Dimensions of bioretention area as plans					21. Stabilisation immediately following earthworks and planting of terrestrial landscape around basin				
8. Confirm surrounding soil type with design					22. Silt fences and traffic control in place				
9. Confirm filter media specification in accordance with Step 4					23. Temporary protection layers in place				
9. Provision of liner (if required)									
10. Under-drainage installed as designed					C. OPERATIONAL ESTABLISHMENT				
11. Drainage layer media as designed					24. Temporary protection layers and associated silt removed				
12. Transition layer media as designed (if required)					Vegetation				
14. Extended detention depth as designed					25. Planting as designed (species and densities)				
					26. Weed removal and watering as required				

FINAL INSPECTION									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of banks				
2. Confirm structural element sizes					7. Under-drainage working				
3. Check batter slopes					8. Inflow systems working				
4. Vegetation as designed					9. Maintenance access provided				
5. Bioretention filter media surface flat and free of clogging									

COMMENTS ON INSPECTION
ACTIONS REQUIRED
Inspection officer signature:

BIORETENTION BASIN MAINTENANCE CHECKLIST			
Inspection Frequency:	1 to 6 monthly	Date of Visit:	
Location:			
Description:			
Asset I.D.			
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?			
Traffic damage present?			
Evidence of dumping (e.g. building waste)?			
Vegetation condition satisfactory (density, weeds etc)?			
Watering of vegetation required?			
Replanting required?			
Mowing/slashing 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			

BIORETENTION BASIN ASSET TRANSFER CHECKLIST		
Asset I.D.		
Asset Location:		
Construction by:		
'On-maintenance' Period:		
TREATMENT	Y	N
System appears to be working as designed visually?		
No obvious signs of under-performance?		
MAINTENANCE	Y	N
Maintenance plans and indicative maintenance costs provided for each asset?		
Vegetation establishment period completed (2 years)?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
ASSET INSPECTED FOR DEFECTS AND/OR MAINTENANCE ISSUES AT TIME OF ASSET TRANSFER	Y	N
Sediment accumulation at inflow points?		
Litter within basin?		
Erosion at inlet or other key structures?		
Traffic damage present?		
Evidence of dumping (e.g. building waste)?		
Vegetation condition satisfactory (density, weeds etc)?		
Watering of vegetation required?		
Replanting required?		
Mowing/slashing required?		
Clogging of drainage points (sediment or debris)?		
Evidence of ponding?		
Damage/vandalism to structures present?		
Surface clogging visible?		
Drainage system inspected?		
COMMENTS/ACTIONS REQUIRED FOR ASSET TRANSFER		
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 (e.g. drawings, survey, models) provided?		
Asset listed on asset register or database?		

5.8 Example Engineering Drawings

Where the relevant local authority has standard drawings appropriate to a bioretention basin application, these should be used to guide the design and construction of a bioretention basin. In the absence of local standards, BCC have developed a set of Standard Drawings (UMS 155, 156 and 337) that can be readily applied to bioretention basin applications throughout the local authorities of SEQ. These drawings relate specifically to inlet pits and sub-surface drains for bioretention swales but may be used to guide design for bioretention basins. These are not intended to be prescriptive drawings which must be adhered to, rather they are intended to provide detailed examples of bioretention system configurations. Standard drawings are available online at <http://www.brisbane.qld.gov.au/BCC:STANDARD:2073302232:pc=PC_1498>.

5.9 Bioretention Basin Worked Example

A series of bioretention basins, designed as landscaped 'out-stands', are to be retrofitted into a minor road in the greater Brisbane area. The street has a longitudinal grade of 1% and the adjacent allotments have an average slope of 8%. A proposed layout for the bioretention basins is shown in Figure 5-8 with an image of a similar system to that proposed shown in Plate 5.7.

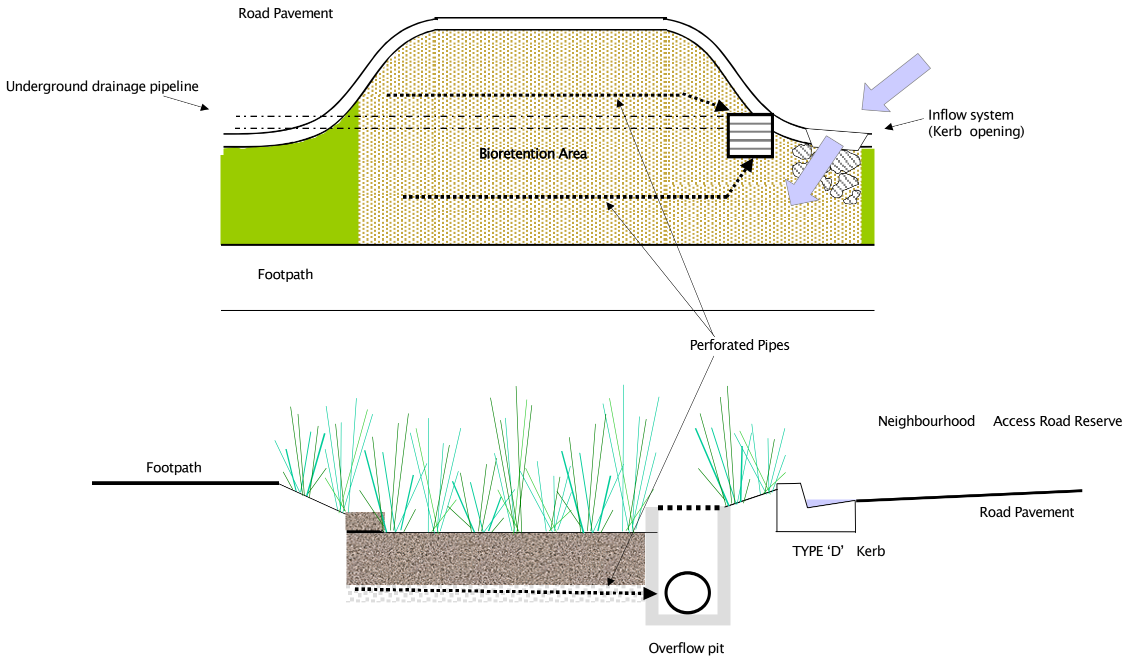


Figure 5-8: General Layout and Cross Section of Proposed Bioretention System

Plate 5-6: Retrofitted Bioretention System in a Street

The contributing catchment areas to each of the individual bioretention basins consist of 200 m^2 of road and footpath pavement and 400 m^2 of adjoining properties. Runoff from adjoining properties (approximately 60 % impervious) is discharged into the road channel and, together with road runoff, is conveyed along a conventional roadside channel to the bioretention basin.

The aim of the design is to facilitate effective treatment of stormwater runoff while maintaining a level of flood protection for the local street under the minor storm (2yr ARI in Brisbane). Conceptual design of the bioretention basins has been undertaken, with MUSIC used to ensure the stormwater discharges comply with the SEQ Best Practice Load Reduction Guidelines (80% TSS, 60% TP and 45% TN reductions). The bioretention basins have an area of 11 m^2 to meet both the landscape and stormwater treatment objectives with an extended detention depth of 200 mm and consisting of a modified sandy loam soil filtration medium (saturated hydraulic conductivity = 100 mm/hr). The width (measured perpendicular to the alignment of the road) of the bioretention basins is 2 m. The key design elements to ensure effective operation of the bioretention basins are listed below:

- road and channel details to convey water into the basins
- detailing inlet conditions to provide for erosion protection
- configuring and designing a system for 'above design' operation that will provide the required 2 year ARI flood protection for the local street
- detailing of the bioretention under-drainage system
- specification of the soil filter medium
- landscape layout and details of vegetation.

<u>Design Objectives</u>

Stormwater treatment to deliver the SEQ WQOs, which, in this case, equates to at least an 80% reduction in mean annual TSS load, 60% reduction in mean annual TP load and 45% reduction in mean annual TN load, whilst maintaining the minor event (i.e. 2 year ARI) level of flood protection for the local street.

<u>Constraints and Concept Design Criteria</u>

Analyses undertaken during a concept design established the following criteria:

- bioretention basin area of 11 m^2 required to achieve the landscape amenity and SEQ Best Practice Load Reduction Guidelines
- maximum width of each bioretention basin is to be 2 m
- extended detention depth is 200 mm
- filter media to have a saturated hydraulic conductivity of 100 mm/hr.

5.9.1 Step 1: Confirm Treatment Performance of Concept Design

It is assumed conceptual design of the bioretention basins included an assessment of the basin performance using MUSIC to ensure the configuration of the basins achieve the stated WQOs. Regardless, the design curves presented earlier in this chapter have been used to verify the size required to deliver the pollutant load reduction described above. Interpretation of Figure 5-3 to Figure 5-5 with the input parameters listed below provided an estimate of the reduction performance of the bioretention basin for the three key pollutants (TSS, TP and TN):

- 200 mm extended detention
- treatment area to catchment area ratio 1.8 % (i.e. 11 m^2 bioretention basin with 600 m2 catchment area).

The expected pollutant reductions are 85 %, 69 % and 45 % for TSS, TP and TN respectively, thus considered to meet the design objectives.

5.9.2 Step 2: Determine Design Flows

With a small catchment (in this case 600 m2), the Rational Method is considered an appropriate approach to estimate the design storm peak flow rates. The steps in this calculation follow below.

Time of concentration (t_c)

Adjacent allotment flow path length = 15 m

Time of concentration t_c = 10 mins (QUDM for land with 6 % < slope < 10 %)

Design runoff coefficient

Runoff Coefficients

C10 = 0.8 (from local authority guidelines)

	C Runoff		
ARI	2	10	50
QUDM Factor	0.85	1	1.15
C_{ARI}	0.68	0.8	0.92

Catchment Area, A = 600 m^2 (0.06 ha)

Rainfall Intensities, t_c = 10 mins

I_2 = 116 mm/hr

I_{50} = 227 mm/hr

Rational Method Q = CIA/360

$Q_{2yr\ ARI}$ = 0.013 m^3/s

$Q_{50yr\ ARI}$ = 0.035 m^3/s

5.9.3 Step 3: Design Inflow Systems

5.9.3.1 Inlet Scour Protection

Rock beaching is to be provided in the bioretention basins to manage flow velocities entering from the kerb opening. This detail is shown on the worked example drawings following Section 5.8.13.

5.9.3.2 Coarse Sediment Forebay

A bioretention system such as the one proposed here should incorporate a coarse sediment forebay to remove coarse sediment from stormwater prior to flowing across the surface of the filter media. The forebay should be designed to:

- Remove particles that are 1mm or greater in diameter from the 3mth ARI storm event.
- Provide appropriate storage for coarse sediment to ensure desilting is required once every year.

The size of the sediment forebay is established using the following:

$$V_s = A_c \cdot R \cdot L_o \cdot F_c$$

Where V_s = volume of forebay sediment storage required (m^3)

A_c = contributing catchment area (0.06 ha)

R = capture efficiency (assume 80%)

L_o = sediment loading rate (1.6 m^3/ha/year)

F_c = desired cleanout frequency (2 years)

V_s = 0.06 * 0.8 * 1.6 * 2

= 0.1536 m^3

The area of the forebay is established by dividing the volume by the depth. The depth of the forebay should not be greater than 0.3m below the surface of the filter media.

$$A_s = \frac{V_s}{D_s}$$

Where D = depth of sediment forebay (0.2+0.3)

A_s *= 0.1536 / 0.5*

= 0.3072 m^2

The sediment forebay area should be checked to ensure it captures the 1mm and greater particles using the following expression (modified version of Fair and Geyer (1954)):

$$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A}\right]^{-n}$$

Where

R = fraction of target sediment removed (80%)

v_s = settling velocity of target sediment (100mm/s or 0.1m/s for 1mm particle)

Q_{3mth} / A = applied flow rate divided by basin surface area (m^3/s/m^2)

n = turbulence or short-circuiting parameter (adopt 0.5)

Q_{3month} *= 0.5 * Q_1 (approx)*

I_1 *= 90 mm/hr*

Q_1 *= C*I*A / 360*

*= 0.8 * 90 * 0.06 / 360*

= 0.012 m^3/s

$Q_{3\text{-}month}$ *= 0.5 * 0.012*

= 0.006 m^3s

R *= 1 - [1 + 1/0.5 * 0.1 / 0.006 / 0.3072]$^{0.5}$*

= 1 – [11.24]$^{0.5}$

= 0.702

= 70 % of 1 mm particles

5.9.3.3 Kerb Opening Configuration (Streetscape Applications)

Channel flow width and kerb opening

The depth and width of channel flow at the locality of the kerb opening needs to be determined to establish the hydraulic head at the kerb opening. The kerb, channel (Figure 5-9) and road profile (Figure 5-10) is shown below as provided by the relevant local government guidelines. The longitudinal grade of the road is 1%.

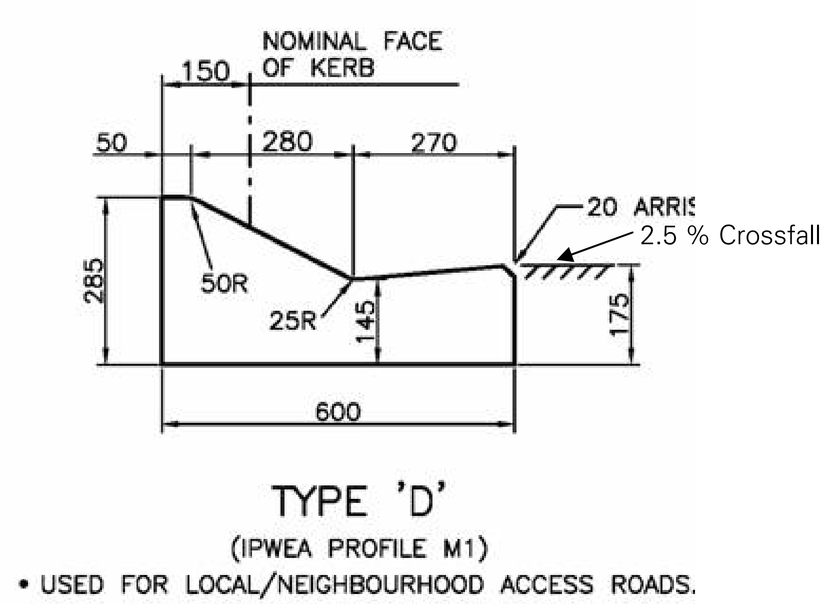


Figure 5-9: Typical Kerb and Channel Detail

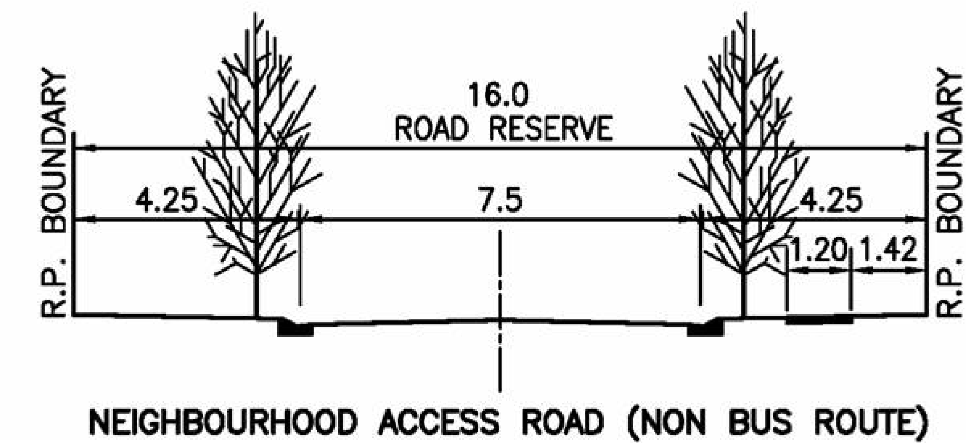


Figure 5-10: Typical Road Reserve Cross Section

The width and depth of channel flow is estimated using the procedure described in QDUM Section 5.09 with the 'Road Flow Capacity Chart Tables' provided in QUDM Volume 2 (DPI, IMEA & BCC 1992) allowing rapid calculation.

$Q_{2\ Year}$ = 0.013 m^3/s gives
Depth of Flow = 50 mm
Width of Flow = 1.3 m
Velocity = 0.57 m/s

The estimated channel flow width at the kerb opening during the $Q_{2\ Year}$ storm event is less than half road width during minor storm flow and thus complies with the relevant local government guidelines.

Kerb opening length

The flow depth in the channel estimated above is used to determine the required length of opening in the kerb to allow for the 2 year ARI flow to pass freely into the bioretention basin.

$Q_{2yr\ ARI} = 0.013\ m^3/s$

Assume broad crested weir flow conditions through the kerb opening and use Equation 5.1 to determine length of opening:

Hence

Where $Q = Q_{2yr\ ARI} = 0.013\ m^3/s$,

C_w = weir coefficient = 1.7

h = depth of (Q_2) flow (50mm) = 0.05 m

Solving gives L = 0.68 m, therefore adopt a 0.7 m long opening which ensures there will be no increase in channel flow depth and width upstream of the kerb opening.

5.9.4 Step 4: Specify the Bioretention Media Characteristics

As outlined in Section 5.3.4, the specification of the filter media and drainage layers requires consideration of the perforated under-drainage system. In this case, a perforated pipe with a slot width of 1.5 mm has been selected, meaning there is a risk that sand (typically 1 mm diameter and less) could wash into the pipe. Therefore, in this case three layers are to be used: an amended sandy loam as the filter media (600 mm), a coarse sand transition layer (100 mm) and a fine gravel drainage layer (200 mm).

5.9.4.1 Filter Media

The filter media is to be a sandy loam and will be formed through the procedure documented in Section 5.3.4.1. The filter media will generally meet the following geotechnical requirements:

- saturated hydraulic conductivity of 100 mm/hr determined from appropriate laboratory testing (see section 5.3.4.1)
- between 5% and 10% organic content, measured in accordance with AS1289 4.1.1
- pH neutral.

5.9.4.2 Drainage Layer

The drainage layer is to be 200 mm of 5 mm screenings graded at 0.5% toward the overflow pit.

5.9.4.3 Transition Layer

Transition layer material shall be 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 %

5.9.5 Step 5: Under-drain Design and Capacity Checks

Two under-drains are to be installed in the drainage layer approximately 1 m apart. This will ensure the drainage layer does not hinder drainage of the filter media. A standard perforated pipe was selected for the under-drain that has a slot clear opening of 2100 mm^2/m with the slots being 1.5 mm wide. The perforated pipes are to be laid on the base of the bioretention system which grades at 0.5 % towards the overflow pit.

The maximum filtration rate, or the flow reaching the perforated pipe in the drainage layer, is estimated by using the saturated hydraulic conductivity of the filter media (assuming no blockage of the media) and head above the base of the filter media and applying Darcy's equation (Equation 5.5).

Saturated hydraulic conductivity = 100 mm/hr = 0.1 m/hr

Area of bioretention basin = 11 m^2

Maximum ponding depth (h_{max}) = 200 mm

Filter media depth = 0.6 m

From Equation 5.5, the maximum filtration rate is:

Q_{max} = (0.1 m/hr x 11 m^2 x [0.2 m + 0.6 m]/0.6 m)/3600 s/hr = 0.00041 m^3/s

<u>Perforations inflow check</u>

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

Assume sub-surface drains with half of all slots blocked (*B* = 0.5)

Clear Opening = 2100 mm^2/m,

Hence, blocked openings = 1050 mm^2/m (50 %)

Slot Width = 1.5 mm

Slot Length = 7.5 mm

Pipe diameter = 100 mm

Number of slots per metre = (1050)/(1.5 x 7.5) = 93.3

Assume orifice flow conditions (Equation 5.6):

Where C_d = 0.61 (assume slot width acts as a sharp edged orifice)

h = 0.85 m (from above)

A = area of slots (=1.5 mm x 7.5 mm x 93.3 slots = 0.00105 m^2)

(note: this already allows for blockage, so *B* can be ignored in this case)

Inlet capacity per metre length of pipe:

= 0.0026 m^3/s

Inlet capacity per m x total length (two lengths of 5.5 m)

= 0.0026 x (2 x 5.5m) = 0.029 m^3/s >> 0.00031 (max filtration rate), hence OK.

<u>Perforated pipe capacity</u>

Manning's equation is applied to estimate the flow rate in the perforated pipes to confirm the capacity of the pipes is sufficient to convey the maximum filtration rate. Two 100 mm diameter perforated pipes are to be laid in parallel and at a grade of 0.5 % towards the overflow pit.

Applying the Manning's Equation assuming a Manning's n of 0.02 gives:

Q (flow per pipe) = 0.0024 m^3/s

Then Q_{Total} = 0.0048 m^3/s (for two pipes) > 0.00031 m^3/s, and hence OK.

5.9.6 Step 6: Check Requirement for Impermeable Lining

In the catchment, the surrounding soils are clay to silty clays with a saturated hydraulic conductivity of approximately 3.6 mm/hr. The sandy loam media that is proposed as the filter media has a hydraulic conductivity of 100 mm/hr, therefore the conductivity of the filter media is > 10 times (one order of magnitude) the conductivity of the surrounding soils and an impervious liner is not considered to be required.

5.9.7 Step 7: Size Overflow Pit

The overflow pit is required to convey 2 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 sizes of the pits are established using two calculations for drowned and free overfall conditions. For free overfall conditions, a broad crested weir equation (Equation 5.4) is used with the maximum headwater depth (h) above the weir being set by the level difference between the crest of the overflow pit and the invert level of the kerb opening (i.e. 100 mm for this design):

Where $Q = Q_{2yr\,ARI}$, B = 0.5, C_w = 1.7 and h = 0.1m and solving for L

Gives L = 0.48 m of weir length required (equivalent to 120 x 120 mm pit)

Now check for drowned conditions using Equation 5.5:

Where $Q = Q_{2yr\,ARI}$, B = 0.5, C_d = 0.6 and h = 0.1m and solving for A

Gives A = 0.030 m^2 (equivalent to 175 x 175 mm pit)

Hence, drowned outlet flow conditions dominate and the pit needs to be greater than 175 x 175 mm. In this case, a 600 x 600 mm pit is adopted as this is minimum pit size to accommodate underground pipe connections.

5.9.8 Step 9: Vegetation Specification

With such a small system, it is appropriate to have vegetation of a single species within the bioretention system. For this application, a Tall Sedge (*Carrex appressa*) is proposed with a planting density of 8 plants/m². Information on maintenance and establishment is provided in earlier sections of this chapter.

5.9.9 Step 8: Verification Checks

5.9.9.1 Vegetation Scour Velocity Checks

The location and sizing of the overflow pit precludes flows from minor and major storm events over the bioretention surface. Therefore, no scour velocity checks are required for this worked example.

5.9.9.2 Confirm Treatment Performance

The key functional elements of the bioretention basins developed as part of the conceptual design (i.e. area, filter media depth) were not adjusted as part of the detailed design. Therefore, the performance check undertaken in Step 1 (see Section 5.8.2) still applies.

5.9.10 Design Calculation Summary

The sheet below shows the results of the design calculations.

BIORETENTION BASIN DESIGN CALCULATION SUMMARY

	Calculation Task		CALCULATION SUMMARY Outcome		Check
	Catchment Characteristics				
		Catchment area	0.06	Ha	✓
		Catchment land use (i.e residential, commercial etc.)	Residential		
		Storm event entering inlet	2yr ARI	yr ARI	
	Conceptual Design				
		Bioretention area		m^2	✓
		Filter media saturated hydraulic conductivity		mm/hr	
		Extended detention depth		mm	
1	**Verify size for treatment**				
	Bioretention area to achieve water quality objectives				
		Total suspended solids (Figure 5-3)	1.2	% of catchment	
		Total phosphorus (Figure 5-4)	0.8	% of catchment	
		Total nitrogen (Figure 5-5)	1.8	% of catchment	
		Bioretention area	11	m^2	✓
		Extended detention depth	0.2	m	
2	**Determine design flows**				
	Time of concentration				
	Refer to relevant local authority guidelines and QUDM		10	minutes	✓
	Identify rainfall intensities				
		Minor Storm ($I_{2\text{-}10\ year\ ARI}$)	116	mm/hr	✓
		Major Storm ($I_{50\ year\ ARI}$)	227	mm/hr	
	Design runoff coefficient				
		Minor Storm ($C_{2\text{-}10\ year\ ARI}$)	0.8		✓
		Major Storm($C_{50\text{-}100\ year\ ARI}$)	0.925		
	Peak design flows				
		Minor Storm (2-10 year ARI)	0.013	m^3/s	✓
		Major Storm (50-100 year ARI)	0.035	m^3/s	
3	**Design inflow systems**				
	Adequate erosion and scour protection?		Yes		✓
	Coarse Sediment Forebay Required?		Yes		
		Volume (V_s)	0.15	m^3	
		Area (A_s)	0.31	m^2	
		Depth (D)	0.5	m	
*	Check flow widths in upstream channel				
		Minor storm flow width	1.3	m	✓
		CHECK ADEQUATE LANES TRAFFICABLE			
*	Kerb opening width				
		Kerb opening length	0.7	m	✓
4	**Specify bioretention media characteristics**				
		Filter media hydraulic conductivity	100	mm/hr	✓
		Filter media depth	600	mm	
	Drainage layer media (sand or fine screenings)				
		Drainage layer depth	200	mm	✓
		Transition layer (sand) required	Yes		
		Transition layer depth	100	mm	
5	**Under-drain design and capacity checks**				
		Flow capacity of filter media	0.00031	m^3/s	✓
	Perforations inflow check				
		Pipe diameter	100	mm	
		Number of pipes	2		
		Capacity of perforations	0.057	m^3/s	
		CHECK PERFORATION CAPACITY > FILTER MEDIA CAPACITY			
	Perforated pipe capacity				
		Pipe capacity	0.0048	m^3/s	✓
		CHECK PIPE CAPACITY > FILTER MEDIA CAPACITY			

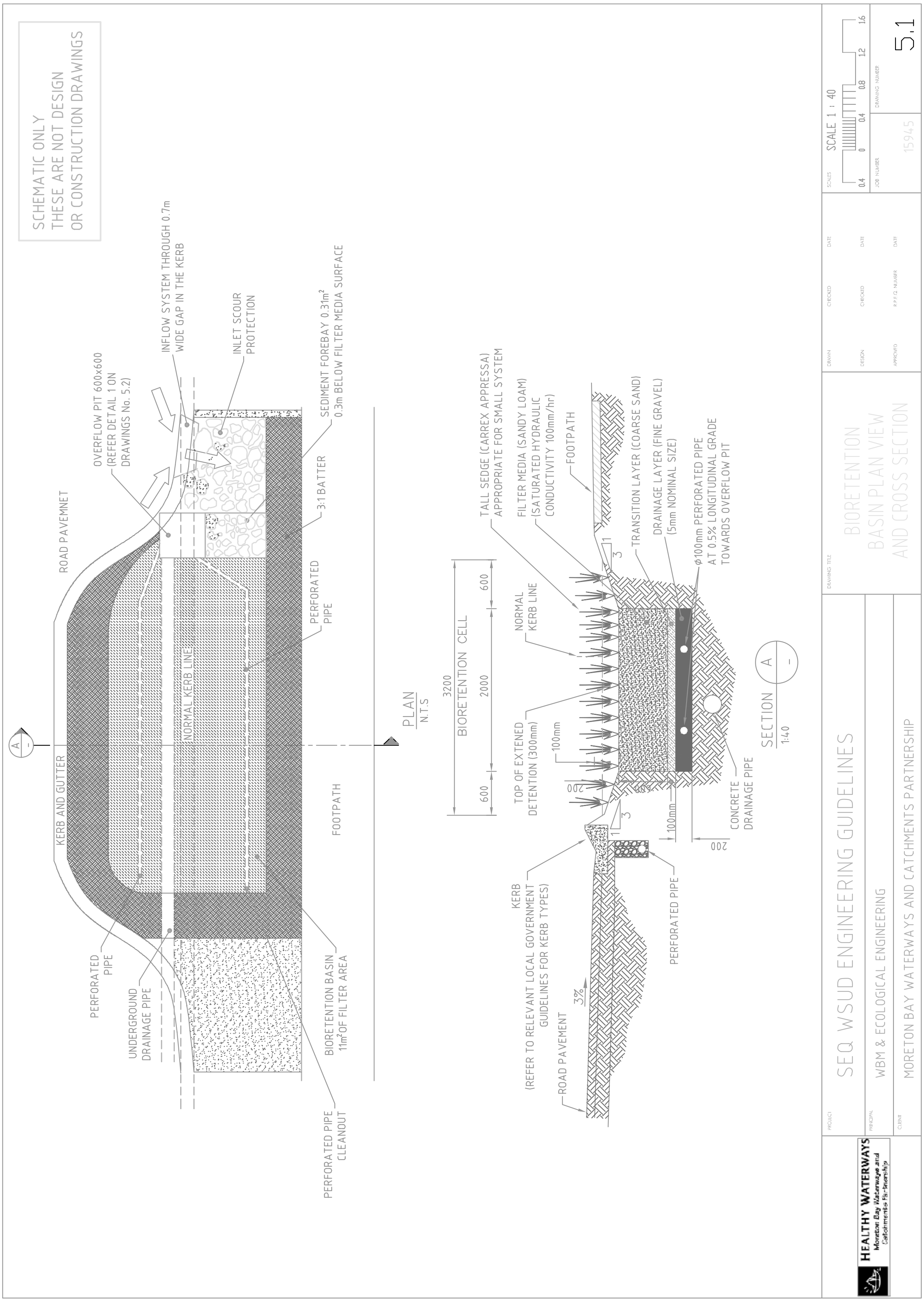
BIORETENTION BASIN DESIGN CALCULATION SUMMARY					
		CALCULATION SUMMARY			
	Calculation Task		Outcome		Check
6	Check requirement for impermeable lining				
		Soil hydraulic conductivity	3.6	mm/hr	✓
		Filter media hydraulic conductivity	100	mm/hr	
		MORE THAN 10 TIMES HIGHER THAN IN-SITU SOILS?			
7	Size overflow pit				
		System to convey minor floods (2-10yr ARI)	600 x 600	L x W	✓
8	Verification Checks				
		Velocity for Minor Storm (<0.5m/s)	N/A	m/s	✓
		Velocity for Major Storm (<2.0m/s)	N/A	m/s	
		Treatment performance consistent with Step 1	Yes		

* Relevant to streetscape application only

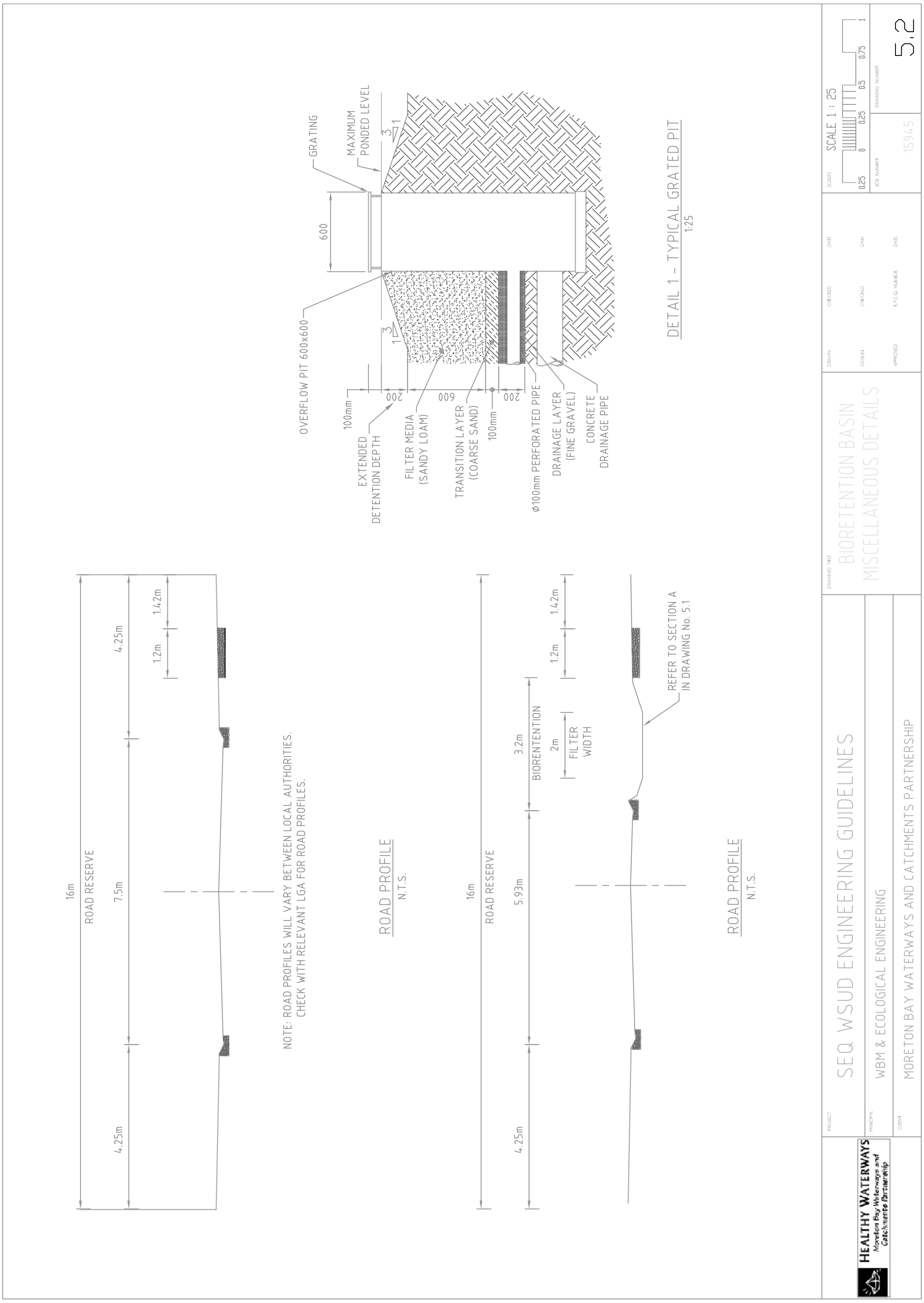
5.9.11 Worked Example Drawing

Drawings 5.1 and 5.2 illustrate the worked example bioretention basin layout.

Drawing 5.1 Bioretention Basin Plan View and Cross Section



Drawing 5.2 Bioretention Basin Miscellaneous Details



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[1] At the time of preparation of these guidelines, QUDM was under review and a significantly revised edition is expected to be released in 2006. These guidelines refer to and use calculations specified in the existing QUDM document, however the revised version of QUDM should be used as the appropriate reference document. It should be noted by users of this guideline that the structure and content of QUDM will change, and as such, the references to calculations and/or specific sections of QUDM may no longer be correct. Users of this guideline should utilise and adopt the relevant sections and/or calculations of the revised QUDM guideline.

Chapter 6 Constructed Stormwater Wetlands

6.1 Introduction 6-2

6.2 Design Considerations 6-2
6.2.1 Landscape Design 6-3
6.2.2 Detention Time and Hydrologic Effectiveness 6-3
6.2.3 Hydrodynamic Design 6-3
6.2.4 Inlet Zone Design Considerations 6-4
6.2.5 Macrophyte Zone Design Considerations 6-5
6.2.6 Wetlands Constructed within Retention (or Detention) Basins 6-5
6.2.7 Vegetation Types 6-5
6.2.8 Designing to Avoid Mosquitoes 6-6
6.2.9 Designing for Maintenance Access 6-6

6.3 Wetland Design Process 6-7
6.3.1 Step 1: Confirm Treatment Performance of Concept Design 6-8
6.3.2 Step 2: Determine Design Flows 6-10
6.3.3 Step 3: Design Inlet Zone 6-11
6.3.4 Step 4: Designing the Macrophyte Zone 6-12
6.3.5 Step 5: Design Macrophyte Zone Outlet 6-16
6.3.6 Step 6: Design High Flow Bypass Channel 6-18
6.3.7 Step 7: Verify Design 6-18
6.3.8 Step 8: Specify Vegetation 6-19
6.3.9 Step 9: Consider Maintenance Requirements 6-19
6.3.10 Design Calculation Summary 6-19

6.4 Landscape Design Notes 6-22
6.4.1 Objectives 6-22
6.4.2 Context and Site Analysis 6-22
6.4.3 Wetland Siting and Shapes 6-22
6.4.4 Specific Landscape Considerations 6-24
6.4.5 Constructed Wetland Vegetation 6-26
6.4.6 Safety Issues 6-27

6.5 Construction and Establishment 6-29
6.5.1 Staged Construction and Establishment Method 6-29
6.5.2 Construction Tolerances 6-31
6.5.3 Sourcing Wetland Vegetation 6-31
6.5.4 Topsoil Specification and Preparation 6-31
6.5.5 Vegetation Establishment 6-33

6.6 Maintenance Requirements 6-34

6.7 Checking Tools 6-35
6.7.1 Design Assessment Checklist 6-35
6.7.2 Construction Checklist 6-35
6.7.3 Operation and Maintenance Inspection Form 6-36
6.7.4 Asset Transfer Checklist 6-36

6.8 Constructed Wetland Worked Example 6-41
6.8.1 Step 1: Verify size for Treatment 6-42
6.8.2 Step 2: Determine Design Flows 6-42
6.8.3 Step 3: Design Inlet Zone 6-43
6.8.4 Step 4: Designing the Macrophyte Zone 6-45
6.8.5 Step 5: Design the Macrophyte Zone Outlet 6-47
6.8.6 Step 6: Design High Flow Bypass Channel 6-50
6.8.7 Step 7: Verification Checks 6-50
6.8.8 Step 8: Vegetation Specification 6-51
6.8.9 Step 9: Maintenance Plan 6-51
6.8.10 Design Calculation Summary 6-51
6.8.11 Worked Example Drawings 6-54

6.9 References 6-56

6.1 Introduction

Constructed wetland systems are shallow, extensively vegetated water bodies that use enhanced sedimentation, fine filtration and biological uptake processes to remove pollutants from stormwater. Water levels rise during rainfall events and outlets are configured to slowly release flows, typically over two to three days, back to dry weather water levels. In addition to treating stormwater, constructed wetlands can also provide habitat, passive recreation, improved landscape amenity and temporary storage of treated water for reuse schemes.

Wetlands generally consist of an inlet zone (sedimentation basin to remove coarse sediments (refer Chapter 4 – Sedimentation Basins)), a macrophyte zone (a shallow heavily vegetated area to remove fine particulates and uptake soluble pollutants) and a high flow bypass channel (to protect the macrophyte zone from scour and vegetation damage). Figure 6-1 shows the key elements of constructed wetland systems.

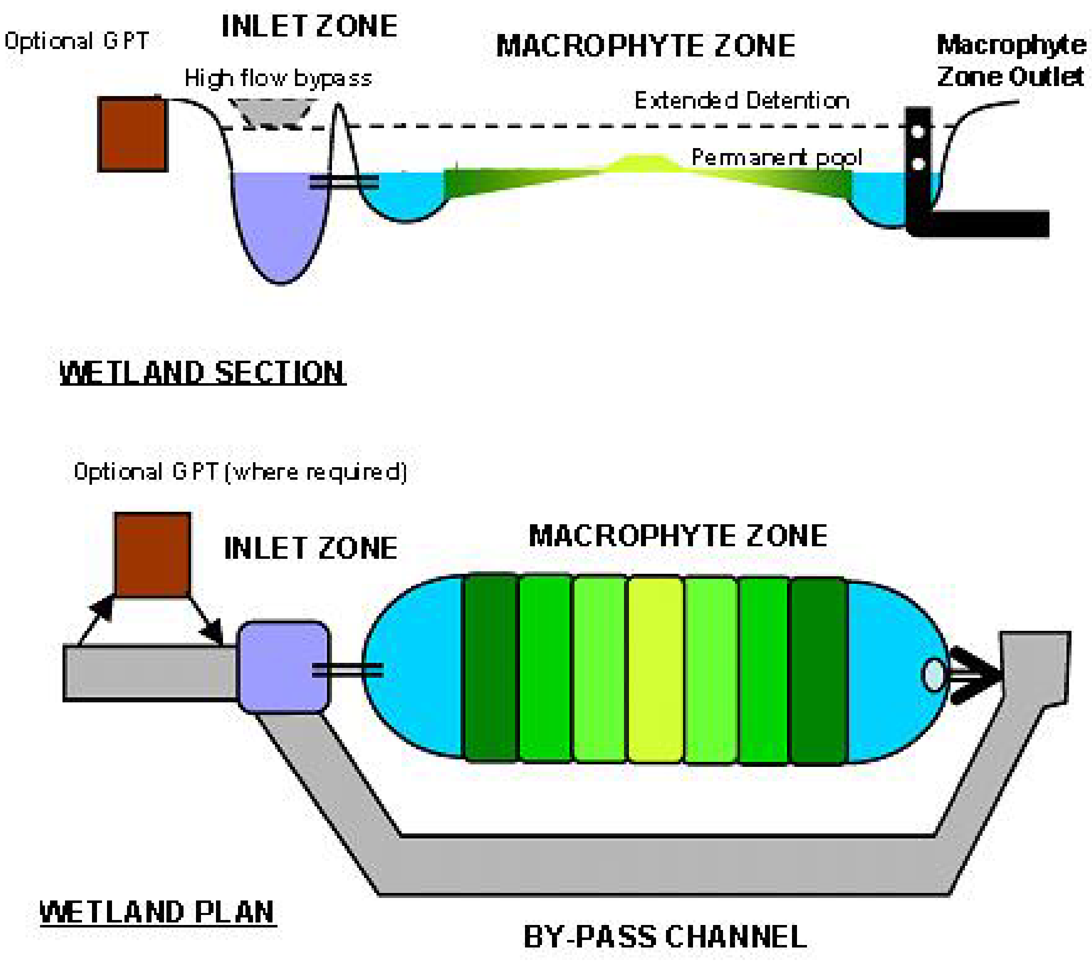


Figure 6-1: Schematic Layout of a Constructed Wetland System

6.2 Design Considerations

The operation of constructed wetlands involves the interaction between stormwater runoff, vegetation and hydraulic structures and the successful implementation of constructed wetlands requires appropriate integration into the landscape design. In this regard, the following sections provide an overview of the key design issues that must be considered when conceptualising and designing constructed wetlands.

6.2.1 Landscape Design

Constructed wetlands are often located within accessible open space areas and can become interesting community features. Landscape design aims to ensure that marsh planting fulfils the intended stormwater treatment function as well as integrating with their surrounds. Opportunities to enhance public amenity and safety with viewing areas, pathway links, picnic nodes and other elements should be exploited. Community education through signage and public art can also be explored. It is important that the landscape of constructed wetlands addresses stormwater quality objectives whilst being sensitive to these other important landscape aims.

Plate 6-1: Public Viewing Area on the Edge of a Landscaped Wetland

6.2.2 Detention Time and Hydrologic Effectiveness

Detention time is the time taken for each 'parcel' of water entering the wetland to travel through the macrophyte zone assuming 'plug' flow conditions. In highly constrained sites, simulations using computer models, such as the *Model for Urban Stormwater Improvement Conceptualisation* (MUSIC)(CRCCH 2005), are often required to optimise the relationship between wetland *detention time*[1] and wetland hydrologic effectiveness to maximise treatment performance. Hydrologic effectiveness is a measure of the mean annual volume of stormwater runoff captured and treated within the wetland and is expressed as a percentage of the mean annual runoff volume generated from the contributing catchment (it should be greater than 80 % for well designed wetlands).

The relationship between notional detention time and pollutant removal efficiency is largely influenced by the settling velocity of the target particulates, although defining the settling velocity of fine to colloidal particulates is not a straight forward exercise. Standard equations for settling velocities often do not apply for such fine particulates owing to the influence of external factors such as wind and water turbulence. It is therefore recommended that a notional detention time should preferably be 48 - 72 hours (and not less than 48 hours) to remove nutrients effectively from urban stormwater.

6.2.3 Hydrodynamic Design

Poor wetland hydrodynamics is often identified as a major contributor to wetland operational and management problems. A summary of desired hydrodynamic characteristics and design considerations is presented in Table 6-1.

---

1 It should be noted that detention time is rarely a constant and the term notional detention time is used throughout this chapter to provide a point of reference in modelling and determining the design criteria for riser outlet structures.

Table 6-1: Desired Wetland Hydrodynamic Characteristics and Associated Design Considerations

Hydrodynamic Characteristics	Design Considerations	Remarks
Uniform distribution of flow velocity	Wetland shape, inlet and outlet placement and bathymetrical design of wetland to eliminate short-circuit flow paths and poorly mixed zones.	Poor flow pattern within a wetland will lead to zones of stagnant pools which promote litter, oil and scum accumulation as well as potentially supporting mosquito breeding. Short circuit flow paths of high velocities 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 draining into the wetland. Bathymetry layout and outlet control design to compliment the botanical design and the hydrology of the wetland.	Regular flow through the wetland promotes flushing of the system thus maintaining a dynamic system and avoiding problems associated with stagnant water, e.g. algal blooms, mosquito breeding, oil and scum accumulation etc. Inadequate attention to the inundation depth, wetness gradient of the wetland and the frequency of inundation at various depth ranges would lead to sparse vegetation cover and/or dominance of certain plant species (especially weed species over time). This results in a deviation from the intended botanical layout of the wetland and reduced stormwater treatment performance. 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. Therefore, inclusion of ephemeral marsh zones in the bathymetric design is desirable if phosphorus is a targeted pollutant.
Uniform vertical velocity profile	Selection of plant species and location of inlet and outlet structures to promote uniform vertical 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 increasing the potential for algal blooms.
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.

6.2.4 Inlet Zone Design Considerations

The inlet zone of a constructed stormwater wetland is designed as a sedimentation basin (see Chapter 4) and has two key functional roles. The primary role is to remove coarse to medium sized sediment (i.e. 125 μm or larger) prior to flows entering the macrophyte zone. This ensures the vegetation in the macrophyte zone is not smothered by coarse sediment and allows the macrophyte zone to target finer particulates, nutrients and other pollutants.

The second role of the inlet zone is the control and regulation of flows entering the macrophyte zone and bypass of flows during 'above design flow' conditions. The outlet structures from the inlet zone (i.e. sedimentation basin) are designed such that flows up to the 'design flow' (typically the 1 year ARI) enter the macrophyte zone whereas 'above design flows' are bypassed around the macrophyte zone. In providing this function, the sedimentation basin protects the vegetation in the macrophyte zone against scour during high flows.

Chapter 4 presents the range of issues that should be considered when designing an inlet zone. Note that when the available space for a constructed wetland is constrained, it is important to ensure that the size of the inlet zone (i.e. sedimentation basin) is not reduced. This ensures the larger sediments are effectively trapped and prevented from smothering the macrophyte zone. When the site constrains the size of the constructed wetland it is the macrophyte zone of the wetland that should be reduced accordingly.

Large wetland systems usually require a gross pollutant trap (GPT) as part of the inlet zone to protect the wetland from litter and debris. The decision of whether a GPT is required or not depends on the presence of upstream GPT measures and catchment size. The relevant local authority should be consulted to determine if a GPT is required.

6.2.5 Macrophyte Zone Design Considerations

The layout of the macrophyte zone needs to be configured such that system hydraulic efficiency is optimised and healthy vegetation sustained. Design considerations include:

- The preferred extended detention depth is 0.5 m. Deeper extended detention depths up to a maximum of 0.75m may be acceptable where the wetland hydrologic effectiveness is greater than 80% (refer to Section 6.2.2) and where the botanic design uses plant species tolerant to greater depths of inundation.
- The bathymetry of the macrophyte zone should be designed to promote a sequence of ephemeral, shallow marsh, marsh and deep marsh zones in addition to small open water zones. The relative proportion of each zone will be dependent on the target pollutant and the wetland hydrologic effectiveness as discussed in Section 6.3.3.2.
- The macrophyte zone is required to retain water permanently and therefore the base must be of suitable material to retain water (eg. clay). If in-situ soils are unsuitable for water retention, a clay liner (e.g. compacted 300 mm thick) must be used to ensure there will be permanent water for vegetation and habitat.
- The bathymetry of the macrophyte zone should be designed so that all marsh zones are connected to a deeper open water zone to allow mosquito predators to seek refuge in the deeper open water zones during periods of extended dry weather.
- Particular attention should be given to the placement of the inlet and outlet structures, the length to width ratio of the macrophyte zone and flow control features to promote a high hydraulic efficiency within the macrophyte zone.
- Provision to drain the macrophyte zone for water level management during the plant establishment phase should also be considered.

The macrophyte zone outlet structure needs to be designed to provide a notional detention time (usually 48 to 72 hours) for a wide range of flow depths. The outlet structure should also include measures to exclude debris to prevent clogging.

6.2.6 Wetlands Constructed within Retention (or Detention) Basins

In many urban applications, wetlands can be constructed in the base of retention basins, thus reducing the land required for stormwater treatment. In these situations, wetland systems will occasionally become inundated to greater depths than the extended detention depth; however, the inundation duration is usually relatively short (hours) and is unlikely to affect the wetland vegetation provided there is a safe pathway to drain the wetland following flood events which avoids scour of the wetland vegetation and banks.

When designing a wetland within a retention basin, the outlet control structure of the retention basin (typically culverts) should be placed at the end of the wetland bypass channel. This ensures flood flows ‘backwater’ across the wetland thus protecting the macrophyte vegetation from scour by high velocity flows.

6.2.7 Vegetation Types

Vegetation planted in the macrophyte zone has an important functional role in treating stormwater flows, as well as adding aesthetic value. Dense planting of the littoral zone will inhibit public access to the macrophyte zone, minimising potential damage to wetland plants and reducing the safety risks posed by water bodies.

Plant species for the wetland area will be selected based on the hydrologic 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. The reader is referred to the Appendix A (Plant Selection for WSUD Systems) for a list of suggested plant species suitable for constructed wetland systems in SEQ. The planting densities recommended in the list should ensure that 70 - 80 % cover is achieved within two growing seasons (2 years). The distribution of the species within the wetland will relate to their structure, function, relationship and compatibility with other species.

6.2.8 Designing to Avoid Mosquitoes

To reduce the risk of high numbers of mosquitoes, there are a number of design features that can be considered. Not all of these will be feasible in any one situation, but they include:

- Providing access for mosquito predators, such as fish and predatory insects, to all parts of the water body (avoid stagnant isolated areas of water).
- Providing a deep sump of permanent water (for long dry periods or for when water levels are artificially lowered) so that mosquito predators can seek refuge and maintain a presence in the wetland.
- Maintaining natural water level fluctuations that disturb the breeding cycle of some mosquito species, but be aware that this may suit other mosquito species.
- Where possible, incorporating a steep slope into the water, preferably greater than 30° or 3:1 horizontal to vertical. Note that steep edges may be unacceptable for public safety reasons, and a slope of up to 8:1 horizontal to vertical is generally used.
- Wave action from wind over open water will discourage mosquito egg laying and disrupt the ability of larvae to breathe.
- 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.
- Providing ready access for field operators to monitor and treat mosquito larvae.
- Ensuring maintenance procedures do not result in wheel rut and other localised depressions that create isolated pools when water levels fall.
- Ensuring overflow channels don't have depressions that will hold water after a storm event.
- Water weeds such as Water Hyacinth and Salvinia can provide a breeding medium for some mosquito species whose larvae attach to these plants under water. These weeds should be removed immediately if encountered.

Each case has to be considered on its own merits. It may be possible that a well established constructed wetland will have no significant mosquito breeding associated with it; however, changes in climatic and vegetation conditions could change that situation rapidly. Maintaining awareness for mosquito problems and regular monitoring for mosquito activity should be considered as a component of the management of these sites. Effective and environmentally sound control products are available for control of mosquito larvae in these situations.

6.2.9 Designing for Maintenance Access

Access to all areas of a constructed wetland is required for maintenance. In particular inlet zones and gross pollutant traps require a track suitable for heavy machinery for removal of debris and desilting as well as an area for dewatering removed sediments (refer to Chapter 4). If sediment removal requires earthmoving equipment to enter the basin, then a stable ramp suitable for heavy plant will be required into the base of the inlet zone (maximum slope 1:10).

To aid maintenance, it is recommended that the inlet zone is constructed with a hard (i.e. rock) bottom. This is important if maintenance is performed by driving into the basin. It also serves an important role by allowing excavator operators to detect when they have reached the base of the inlet zone during desilting operations.

Macrophyte zones require access to the areas for weeding and replanting as well as regular inspections. Commonly, these access tracks can be incorporated with walking paths around a wetland system. Maintenance access to constructed wetland needs to be considered when determining the layout of a wetland system.

6.3 Wetland Design Process

The key design steps following the site planning and concept development stages are:

1. Confirm treatment performance of concept design

↓

2. Determine design flows

↓

3. Design inflow systems
(see Design Procedure for Sedimentation Basin in Chapter 4)

↓

4. Design macrophyte zone
a. Length to width ratio and hydraulic efficiency
b. Design the macrophyte zone bathymetry
c. Macrophyte zone edge design for safety

↓

5. Design macrophyte zone outlet
a. Riser outlet - Size and location of orifices
b. Maintenance drain
c. Discharge pipe

↓

6. Design high flow bypass channel

↓

7. Verify design
a. Macrophyte zone resuspension protection
b. Confirm treatment performance

↓

8. Specify vegetation

↓

9. Consider maintenance requirements

Each of these design steps is discussed in the following subsections. A worked example illustrating application of the design process on a case study site is presented in Section 6.8.

6.3.1 Step 1: Confirm Treatment Performance of Concept Design

Before commencing detailed design, the designer should first undertake a preliminary check to confirm the required wetland area (i.e. the macrophyte zone surface area) from the concept design is adequate to deliver the required level of stormwater quality improvement. This design process assumes a conceptual design has been undertaken. The wetland treatment performance curves shown in Figure 6-2 to Figure 6-4 can be used to undertake this verification check. These curves are intended to provide an indication only of appropriate sizing and do not substitute the need for a thorough conceptual design process.

The curves in Figure 6-2 to Figure 6-4 were derived using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC), assuming the constructed wetland is a stand alone system (i.e. not part of a treatment train). The curves show the total suspended solid (TSS), total phosphorus (TP) and total nitrogen (TN) load removal performance for a typical constructed wetland design, being:

- Average Depth = 0.25 m
- Extended Detention Depth = 0.5 m
- Notional Detention Time = 48 hrs

It should be noted that the curves show the pollutant load reduction for this configuration, and are designed for comparison with SEQ load-based water quality targets. These curves should not be used to assess performance of a constructed wetland against concentration-based objectives.

The curves in Figure 6-2 to Figure 6-4 are generally applicable to constructed wetland applications within residential, industrial and commercial land uses. Curves are provided for four rainfall station locations selected as being broadly representative of the spatial and temporal climatic variation across South East Queensland. The shaded area on each of the curves indicates where the wetland performance meets the Best Practice Pollutant Load Reduction Targets for South East Queensland.

If the configuration of the constructed wetland concept design is significantly different to that described above, or if the basin is part of a treatment train, then the curves in Figure 6-2 to Figure 6-4 may not provide an accurate indication of treatment performance. In these cases, the detailed designer should use MUSIC to verify concept designs that are part of a "treatment train"(if not already undertaken as part of concept design process).

The curves in Figure 6-2 to Figure 6-4 also provide the detailed designer with a useful visual guide to illustrate the sensitivity of constructed wetland performance to the ratio of macrophyte zone treatment area and contributing catchment area. The curves allow the detailed designer to make a rapid assessment as to whether the concept design falls within the "optimal size range" or if it is potentially under or over-sized. An under-sized system might indicate the wetland is part of a "treatment train" or that another supplementary treatment device may be located somewhere else within the catchment. This should be checked by the detailed designer. An over-sized system suggests the concept designer may have inadvertently sized the wetland such that it is operating well beyond its point of "diminishing performance" (i.e. where incremental increases in wetland size, and thus cost, result in only a marginal increase in treatment performance). In this instance, the detailed designer should confirm whether or not the wetland size can be reduced or if additional treatment devices may be required.

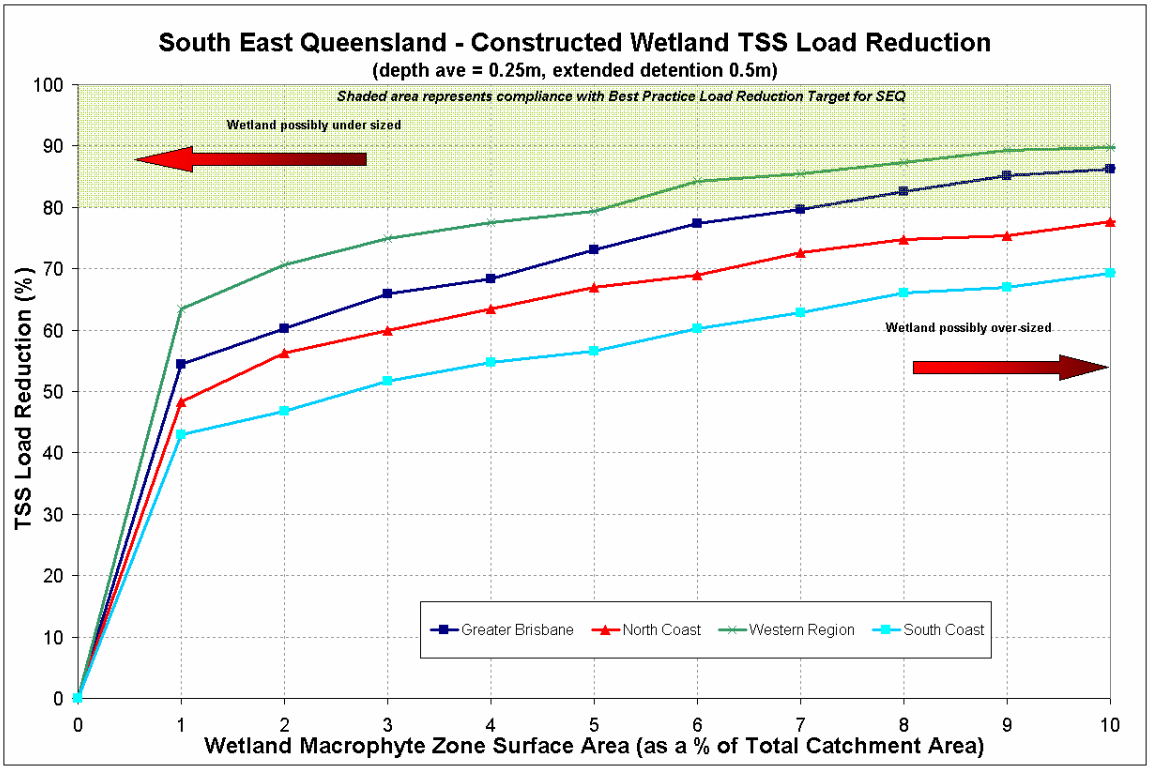


Figure 6-2:Constructed Wetland TSS Load Removal Performance

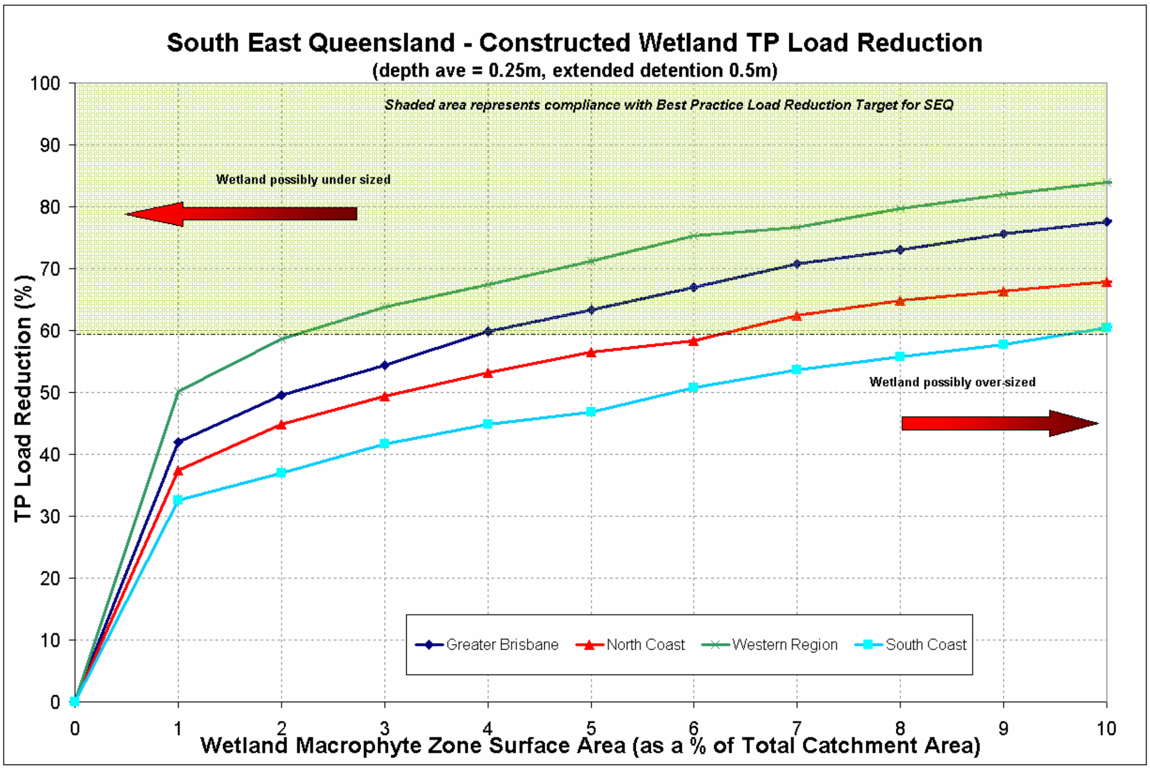


Figure 6-3: Constructed Wetland TP Load Removal Performance

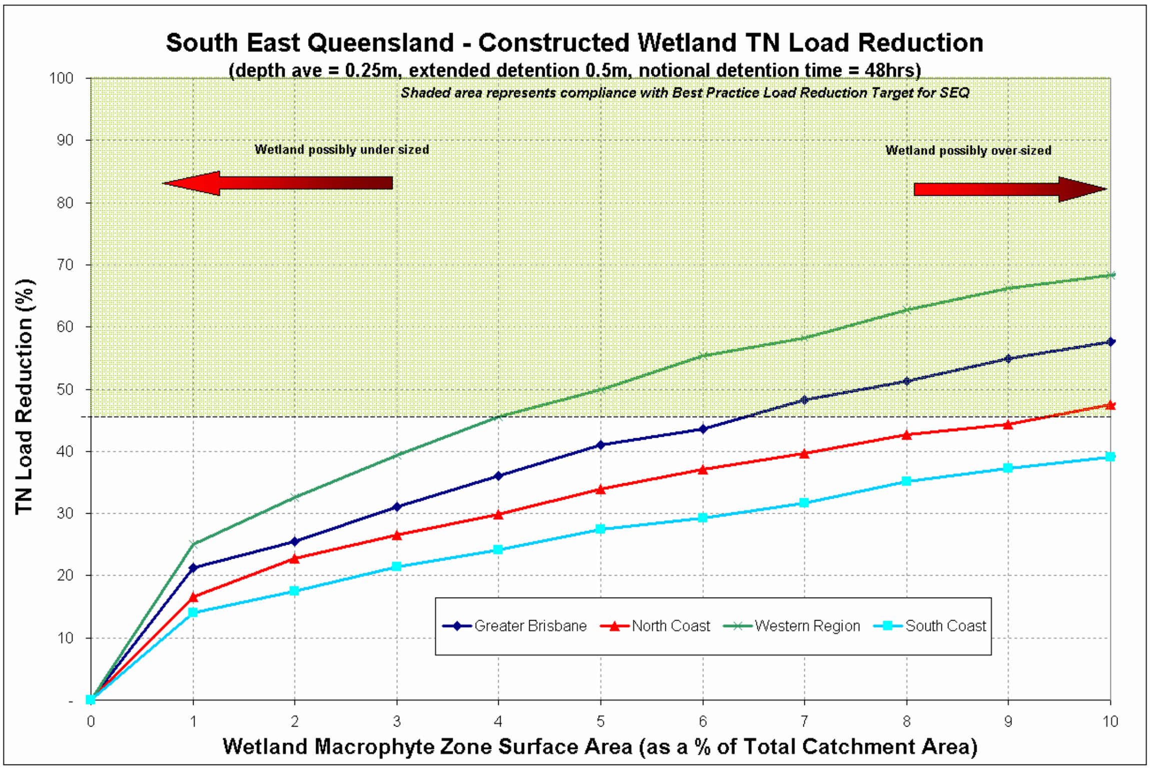


Figure 6-4: Constructed Wetland TN Load Removal Performance

6.3.2 Step 2: Determine Design Flows

6.3.2.1 Design Discharges

To configure the inlet zone and high flow bypass elements of a constructed wetland the following design flows apply:

- Design operation flow (1 year ARI) for sizing the inlet zone (i.e. sedimentation basin) and the 'control' outlet structure (i.e. overflow pit and pipe connection) discharging to macrophyte zone.
- Above design flow for design of the high flow bypass around the macrophyte zone. The discharge capacity for the bypass system may vary depending on the particular situation but will typically correspond to one of the following:
  - Minor design flow (2 or 10 year ARI) – for situations where only the minor drainage system is directed to the inlet zone. Relevant local government guidelines should be referred to for the required design event for the minor design flow.
  - Major flood flow (100 year ARI) – for situations where both the minor and major drainage system discharge into the inlet zone.

6.3.2.2 Design Flow 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, if the constructed wetland is to form part of a retention basin (Section 6.2.6) or if the catchment area to the wetland is large (> 50 ha), then a full flood routing computation method needs to be used to estimate design flows.

6.3.3 Step 3: Design Inlet Zone

As outlined in Section 6.2.4, the inlet zone of a constructed stormwater wetland is designed as a sedimentation basin (refer Chapter 4) and serves two functions: (1) pretreatment of inflow to remove coarse to medium sized sediment; and (2) the hydrologic control of inflows into the macrophyte zone and bypass of floods during 'above design' operating conditions. As depicted in Figure 6-5, the inlet zone consists of the following elements:

Plate 6-2: Inlet Zone of a Constructed Wetland in Brisbane

- Sedimentation basin 'pool' to capture coarse to medium sediment (125 µm or larger).
- Inlet zone connection to the macrophyte zone (or 'control' structure as defined in Chapter 4) normally consisting of an overflow pit within the inlet zone connected to one or more pipes through the embankment separating the inlet zone and the macrophyte zone.
- High flow bypass weir (or 'spillway' outlet structure as defined in Chapter 4) to deliver 'above design' flood flows to the high flow bypass channel.

For more information and design guidance for each of the inlet zone elements listed above, the reader is referred to Chapter 4 Sedimentation Basins. When applying the design procedure outlined in Chapter 4, the following should be used as a guide:

- The inlet zone typically must comprise a deep open water body (> 1.5 m) that operates essentially as a sedimentation basin designed to capture coarse to medium sized sediment (i.e. 125 µm or larger).
- It may be necessary for a GPT to be installed 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.
- The crest of the overflow pit must be set at the permanent pool level of the inlet zone (which is typically set 0.3 m above the permanent water level of the macrophyte zone).
- The dimension of the overflow pit (control structure) should be set at the permanent pool level of the inlet zone (which is typically set 0.3 m above the permanent water level of the macrophyte zone).
- The pipe that connects the sedimentation basin to the macrophyte zone needs to have sufficient capacity to convey a 1 year ARI flow, assuming the macrophyte zone is at the permanent pool level and without resulting in any flow over the high flow bypass weir.
- An energy dissipater is usually required at the end of the pipes to reduce velocities and distribute flows into the macrophyte zone.
- The inlet zone is to have a structural base (e.g. rock) to define the base when desilting and provide support for maintenance plant/ machinery when entering the basin for maintenance.
- The high flow bypass weir ('spillway' outlet) is to be set at the same level as the top of extended detention in the macrophyte zone.

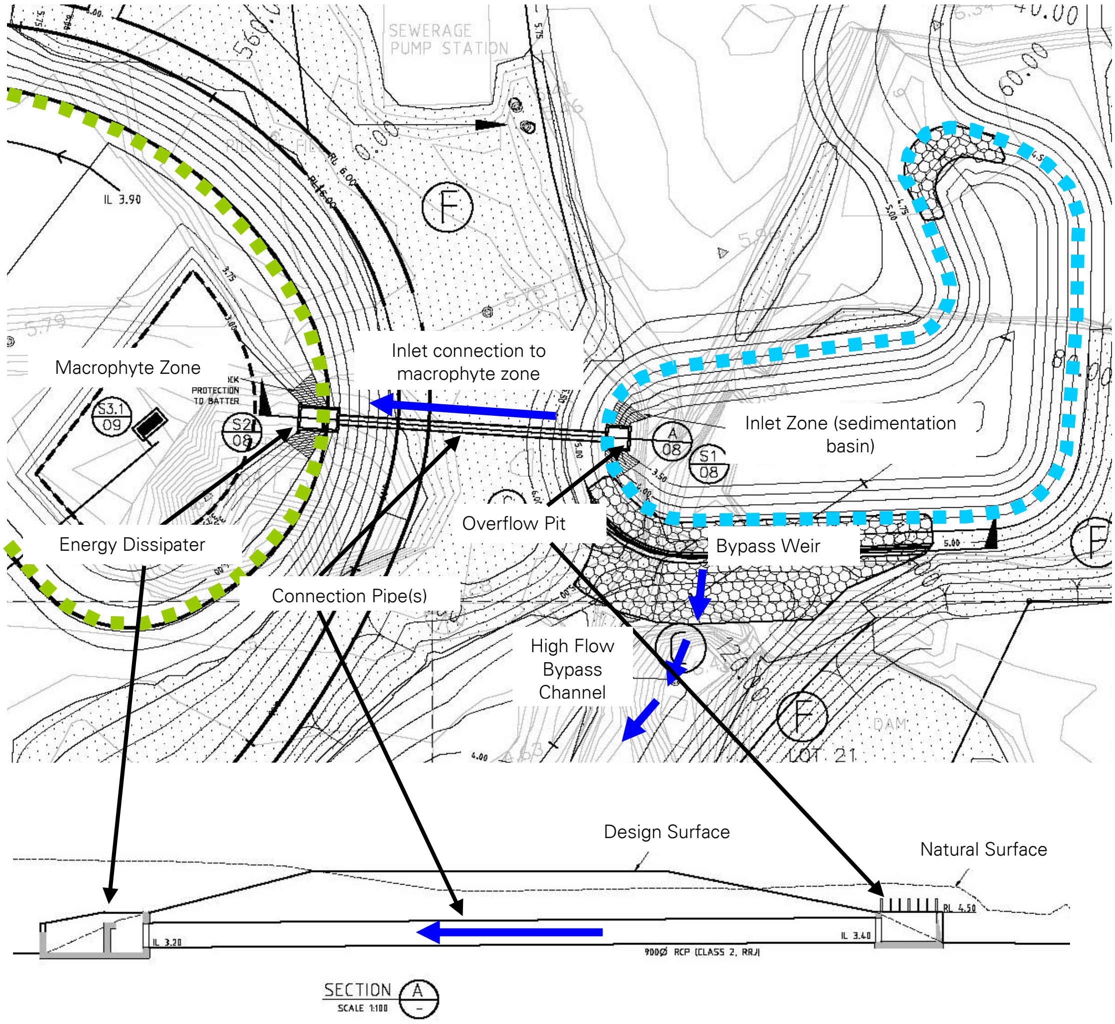


Figure 6-5: Example of Inlet Zone Connection to Macrophyte Zone

6.3.4 Step 4: Designing the Macrophyte Zone

6.3.4.1 Length to Width Ratio and Hydraulic Efficiency

To optimise wetland performance, it is important to avoid short circuit flow paths and poorly mixed regions within the macrophyte zone. One way to minimise this is to adopt a high length to width ratio not less than 5 to 1 for the macrophyte zone. Length to width ratios less than this can lead to poor hydrodynamic conditions and reduced water quality treatment performance.

Persson et al. (1999) used the term hydraulic efficiency (λ) to define the expected hydrodynamic characteristics for a range of configurations of stormwater detention systems (Figure 6-6). Engineers Australia (2006) recommend that constructed wetland systems should not have a hydraulic efficiency (λ) less than 0.5 and preferably should be greater than 0.7.

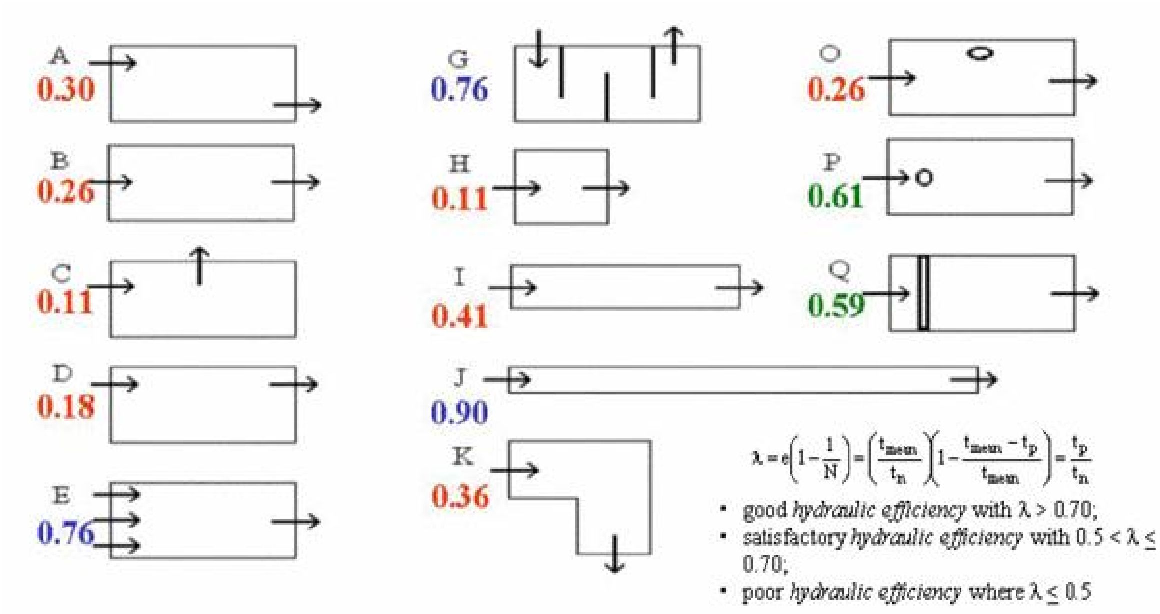


Figure 6-6: Hydraulic Efficiency (λ) Ranges

Hydraulic efficiency ranges from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment. The o in diagrams O and P represent islands in the waterbody and the double line in diagram Q represents a weir structure to distribute flows evenly (Persson et al. 1999).

6.3.4.2 Designing the Macrophyte Zone Bathymetry

It is good design practice to provide a range of habitat areas within the macrophyte zone to support a variety of plant species, ecological niches and perform a range of treatment processes. The macrophyte zone therefore typically comprises four marsh zones (defined by water depth) and an open water zone. The four marsh zones are ephemeral marsh, shallow marsh, marsh and deep marsh as depicted in Figure 6-1 and Figure 6-7. The bathymetry across the four marsh zones is to vary gradually ranging from 0.2 m above the permanent pool level (i.e. ephemeral marsh) to a maximum of 0.5 m below the permanent pool level (i.e. deep marsh). Appendix A provides further discussion on the macrophyte plants suited to each marsh zone.

The relative proportion of each marsh zone will be dependent on the specific pollutant(s) being targeted by the wetland. For example, a wetland targeting phosphorus removal would typically have a higher proportion of ephemeral marsh zone where the frequent wetting and drying cycle promotes the 'locking' of phosphorus onto the soil particles within the macrophyte zone substrate. Conversely, if nitrogen is the target pollutant, the macrophyte zone would typically have a higher proportion of marsh and deep marsh. The marsh and deep marsh zones facilitate nitrogen cycling within the aerobic and anaerobic substrate conditions as well as biological processing of soluble nitrogen from the water column by algal epiphytes and biofilms attached to the submerged part of the macrophytes in these zones.

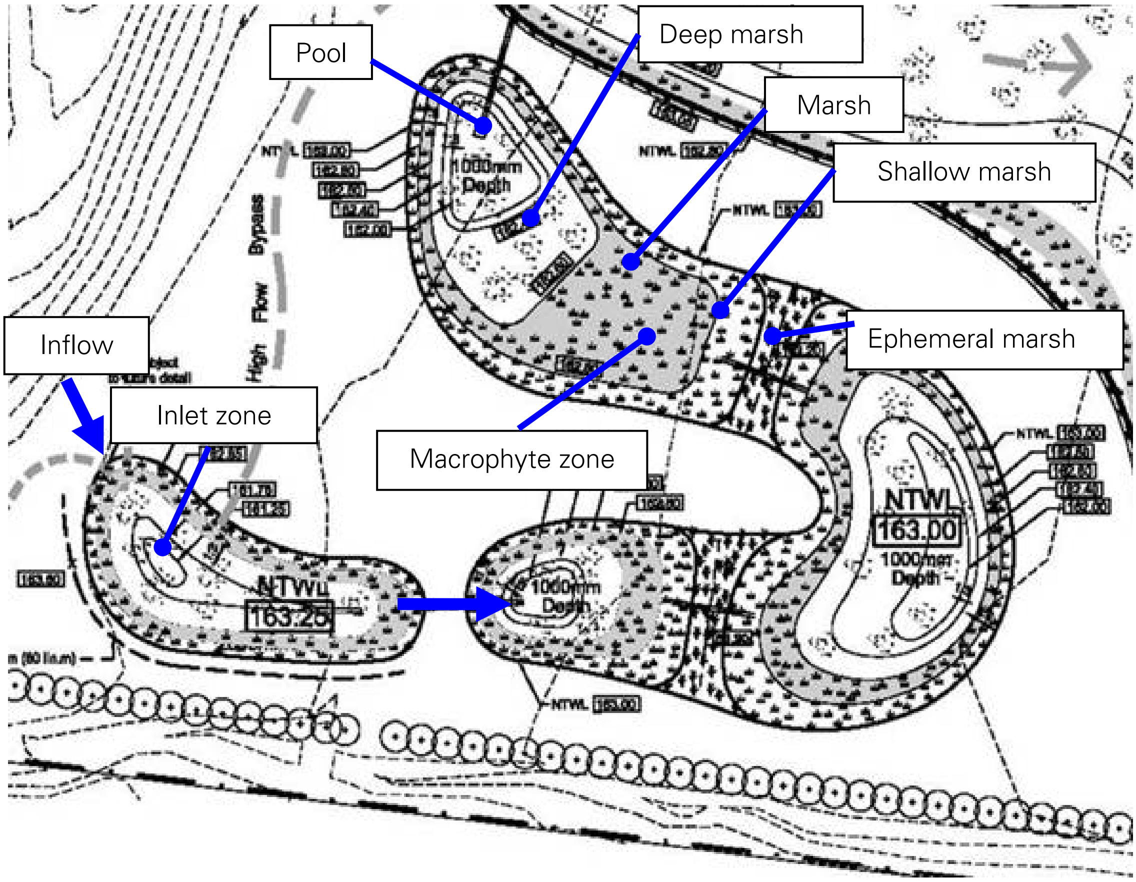


Figure 6-7: Example Bathymetry of a Constructed Wetland System (GBLA 2004)

Plate 6-3: Macrophyte Zone Planting and Bathymetry

The depth of the open water zones should be not less than 1 m below the permanent pool level to avoid colonisation by emergent macrophytes and typically not more than 1.5 m depth to allow for colonisation for submerged macrophytes.

To ensure optimal hydraulic efficiency of a wetland for a given shape and aspect ratio, wetland zones are arranged in bands running across (i.e. perpendicular to) the flow path (see Figure 6-1). The appropriate bathymetry, coupled with uniform plant establishment, ensures the macrophyte zone cross section has uniform hydraulic conveyance, thus reducing the risk of short circuiting.

6.3.4.3 Macrophyte Zone Edge Design for Safety

The batter slopes on approaches and immediately under the permanent water level have to be configured with consideration of public safety (refer Figure 6-8). 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.

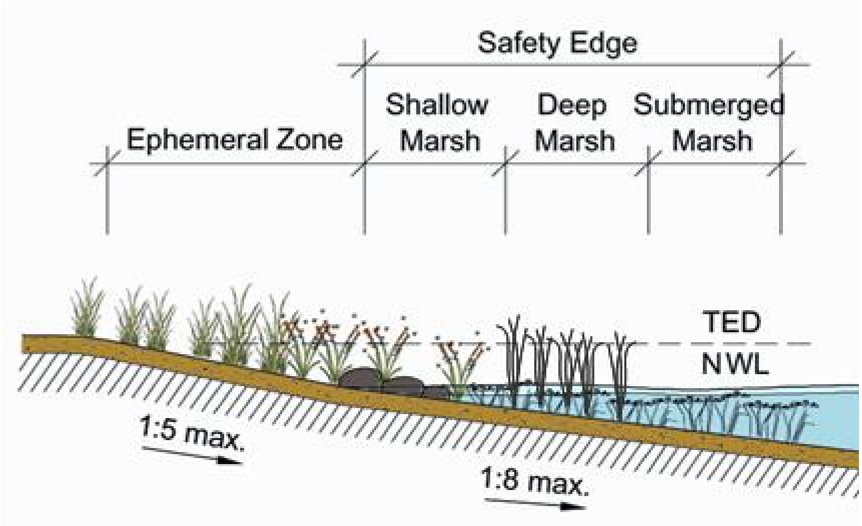


Figure 6-8: Example of Edge Design to a Constructed Wetland System

The safety requirements for individual wetlands will vary from site to site and requires careful consideration. The following requirements from the *Sediment Basin Design, Construction and Maintenance Guidelines* (BCC 2001) equally apply to constructed wetland systems:

- For water depths greater than 150 mm and maximum batter slope of 5:1 (H:V) or less, no fencing is required.
- For water depths greater than 150 mm and maximum batter slope greater than 5:1 (H:V) fencing is required.

In some cases, vertical edges are used for wetlands (refer to Section 6.4). When vertical edges are used, a safety fencing/ barrier should be considered on top of concrete or stone walls where:

- there is a risk of serious injury in the event of a fall (over 0.5 m high and too steep to comfortably walk up/ down or the lower surface has sharp or jagged edges)
- there is a high pedestrian or vehicular exposure (on footpaths, near bikeways, near playing/ sporting fields, near swings and playgrounds)
- where water ponds to a depth of greater than 300 mm on a constructed surface of concrete or stone
- where the water is expected to contain concentrated pollutants
- where mowed grassed areas abut the asset.

The type of fence/ barrier to be considered should be a:

- pool fence when there is a chance of drowning or infection from the asset and the surrounding area is specifically intended for use by small children (swings, playgrounds, sporting fields etc.)
- galvanised tubular handrail (in accordance with relevant Australian Standards) without chain wire elsewhere
- dense vegetation (hedge) at least 2 m wide and 1.2 m high (minimum) may be suitable if vandalism is not a demonstrated concern.

6.3.4.4 Macrophyte Zone Soil Testing

Constructed wetlands are permanent water bodies and therefore the soils in the base must be capable of retaining water. Geotechnical investigations of the suitability of the in-situ soils are required to establish the water holding capacity of the soils. Where the infiltration rates are too high for permanent water retention, tilling and compaction of in-situ soils may be sufficient to create a suitable base for the wetland. Where in-situ soils are unsuitable for water retention, a compacted clay liner may be required (eg. 300 mm thick). Specialist geotechnical testing and advice must be sought.

6.3.5 Step 5: Design Macrophyte Zone Outlet

A macrophyte zone outlet has two purposes: (1) hydrologic control of the water level and flows in the macrophyte zone to achieve the design detention time; and (2) to allow the wetland permanent pool to be drained for maintenance.

6.3.5.1 Riser Outlet – Size and Location of Orifices

The riser outlet is designed to provide a uniform notional detention time in the macrophyte zone over the full range of the extended detention depths. The target maximum discharge ($Q_{max\ riser}$) may be computed as the ratio of the volume of the extended detention to the notional detention time as follows:

$$Q_{max\ riser} = \frac{\text{extended detention storage volume } (m^3)}{\text{notional detention time (s)}}$$ Equation 6.1

The placement of orifices along the riser and determining their appropriate diameters is an iterative process. The orifice equation (Equation 6.2) is applied over discrete depths along the length of the riser starting at the permanent pool level and extending up to the riser maximum extended detention depth. This can be performed with a spreadsheet as illustrated in the worked example in Section 6.7.

$$A_o = \frac{Q}{C_d\sqrt{2 \cdot g \cdot h}}$$ (Small orifice equation) Equation 6.2

Where	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 achieve notional detention time (m^3/s) at the given h
	g	=	9.79 m/s^2

As the outlet orifices can be expected to be small, it is important that they are prevented from clogging by debris. Some form of debris guard is recommended as illustrated in Plate 6.5 below. An alternative to using a debris guard is to install a riser in a pit located in the embankment surrounding the wetland macrophyte zone (thus reducing any visual impact). A riser within the pit can also be configured with a weir plate (by drilling holes through the plate). An advantage of using a weir plate is that it provides an ability to drain the wetland simply by removing the weir plate entirely. Additionally, shorter weir plates may also be used during the vegetation establishment phase, thus providing more flexibility for water level manipulation.

Plate 6-4: Example Outlet Riser Assemblies with Debris Guards

The pit is connected to the permanent pool of the macrophyte zone via a submerged pipe culvert. The 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 (i.e. pipe obvert a minimum 0.3 m below permanent pool level), floating debris is generally prevented from entering the outlet pit, while heavier debris would normally settle onto the bottom of the wetland. The riser pipe should be mounted upright on a socketed and flanged tee with the top of the pipe left open to allow overtopping of waters if any of the riser orifices become blocked. Figure 6-9 and Plate 6-5shows one possible configuration for a riser outlet pit.

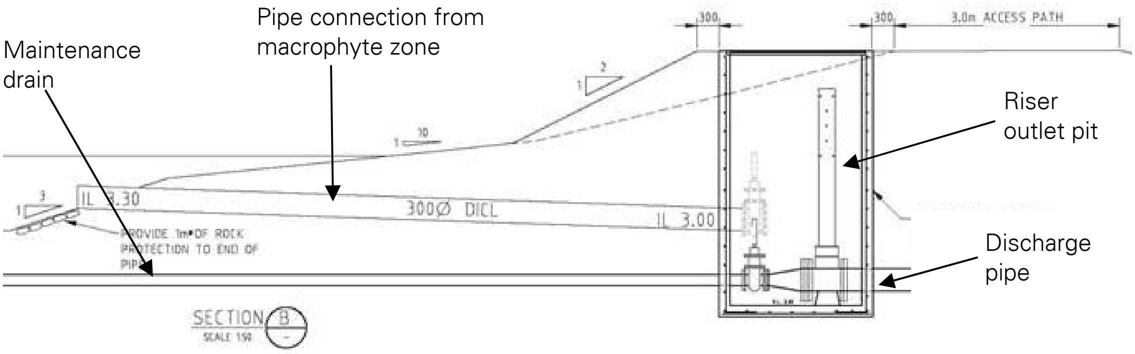


Figure 6-9: Typical Macrophyte Zone Outlet Arrangement

6.3.5.2 Maintenance Drains

To allow access for maintenance, the wetland should have appropriate allowance for draining. A maintenance drainage pipe should be provided that connects the low points in the macrophyte zone bathymetry to the macrophyte zone outlet. A valve is provided on the maintenance drainage pipe (typically located in the outlet pit as shown in Figure 6-9), which can be operated manually. The maintenance drainage pipe should be sized to draw down the permanent pool within 12 hours (i.e. overnight). If a weir plate is used as a riser outlet, provision should be made to remove the weir plate and allow drainage for maintenance.

Plate 6-5: Macrophyte Zone Outlet Arrangement

6.3.5.3 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 (i.e. maximum discharge from the riser or the maximum discharge from the maintenance drain).

6.3.6 Step 6: Design High Flow Bypass Channel

The bypass channel accepts 'above design flow' from the inlet zone of the wetland via the bypass weir (Section 6.3.3) and conveys these flows downstream around the macrophyte zone of the wetland. The bypass channel should be designed using standard methods (i.e. Manning's Equation) to convey the 'above design flow' (Section 6.3.2) and to avoid bed and bank erosion (see Chapter 2). Typically, a turf finish will provide appropriate protection for most bypass channel applications (but velocities need to be checked). Plate 6.7 shows typical high flow bypass channel configurations.

Plate 6-6: Constructed Wetland Bypass Weir and Channel Configurations

6.3.7 Step 7: Verify Design

6.3.7.1 Macrophyte Zone Resuspension Protection

The principle pathway for biological uptake of soluble nutrients in wetlands is through biofilms (epiphytes) attached to the surface of the macrophyte vegetation. The biofilms, being mostly algae and bacteria, are susceptible to wash out under high flow conditions. Further, wetland surveys indicate that up to 90 % of the total nutrients are stored in the sediments, therefore, the key to effective retention of pollutants is managing high velocity flows that could potentially resuspend and remobilise these stored pollutants.

A velocity check is to be conducted for design conditions, when the wetland water level is at the top of the extended detention level and the riser is operating at design capacity, to ensure velocities are less than 0.05 m/s through all zones of the wetland. The following condition must be met:

$$\frac{Q_{\text{max riser}}}{A_{\text{section}}} < 0.05\text{m/s} \qquad \text{Equation 6.3}$$

Where $Q_{max\ riser}$ = target maximum discharge (defined in equation 6.1) (m^3/s)

$A_{section}$ = wetland cross sectional area at narrowest point*, measured from top of extended detention (m^2)

* minimum wetland cross-section is used when undertaking this velocity check

6.3.7.2 Confirm Treatment Performance

If the basic wetland parameters established by the conceptual design phase have changed during the course of undertaking detailed design (e.g. macrophyte zone area, extended detention depth, etc.) then the designer should verify that the current design meets the required water quality improvement performance. This can be done by simulating the current design using MUSIC.

6.3.8 Step 8: Specify Vegetation

Refer to Section 6.4 and Appendix A for advice on selecting suitable plant species for constructed wetlands.

6.3.9 Step 9: Consider Maintenance Requirements

Consider how maintenance is to be performed on the wetland (e.g. how and where is access available, where is litter likely to collect etc.). A specific maintenance plan and schedule should be developed for the wetland, either as part of a maintenance plan for the whole treatment train, or for each individual asset. Guidance on maintenance plans is provided in Section 6.6.

6.3.10 Design Calculation Summary

Following is a design calculation summary sheet for the key design elements.

CONSTRUCTED WETLANDS DESIGN CALCULATION SUMMARY			
		CALCULATION SUMMARY	
	Calculation Task	Outcome	Check
	Catchment Characteristics		
	Catchment area	ha	
	Catchment land use (i.e residential, commercial etc.)		
	Storm event entering inlet pond (minor or major)		
	Conceptual Design		
	Macrophyte zone area	m^2	
	Permanent pool level of macrophyte zone	m AHD	
	Extended detention depth (0.25-0.5m)	m	
	Notional detention time	hrs	
1	**Confirm Treatment Performance of Concept Design**		
	Total suspended solids (Figure 6-2)	% removal	
	Total phosphorus (Figure 6-3)	% removal	
	Total nitrogen (Figure 6-4)	% removal	
2	**Determine design flows**		
	'Design operation flow' (1 year ARI)	year ARI	
	'Above design flow' (2-100 year ARI)	year ARI	
	Time of concentration		
	(Refer to relevant local government guidelines and QUDM)	minutes	
	Identify rainfall intensities		
	'Design operation flow' - $I_{1\ year\ ARI}$	mm/hr	
	'Above design flow'- $I_{2-100\ year\ ARI}$	mm/hr	
	Peak design flows		
	'Design operation flow' 1 year ARI	m^3/s	
	'Above design flow' – 2-100 year ARI	m^3/s	
3	**Design inlet zone**		
	Refer to sedimentation basin (Chapter 4) for detailed check sheet		
	Is a GPT required?		
	Suitable GPT selected and maintenance considered?		
	Inlet zone size		
	Target Sediment Size for Inlet Zone	μm	
	Capture efficiency	%	
	Inlet zone area (Figure 4.2 in Chapter 4)	m^2	
	$V_s > V_{s:5yr}$		
	Inlet zone connection to macrophyte zone		
	Overflow pit crest level	m AHD	
	Overflow pit dimension	L x W	
	Provision of debris trap		
	Connection pipe dimension	mm diam	
	Connection pipe invert level	m AHD	
	High flow by-pass weir		
	Weir Length	m	
	High flow by-pass weir crest level (top of extended detention)	m AHD	
4	**Designing the macrophyte zone**		
	Area of Macrophyte Zone	m^2	
	Aspect Ratio	L:W	
	Hydraulic Efficiency		

CONSTRUCTED WETLANDS DESIGN CALCULATION SUMMARY				
		CALCULATION SUMMARY		
	Calculation Task		Outcome	Check
5	Design macrophyte zone outlet			
	Riser outlet			
		Target maximum discharge (Q_{max})	m^3/s	
		Uniform Detention Time Relationship for Riser		
	Maintenance Drain			
		Maintenance drainage rate (drain over 12hrs)	m^3/s	
		Diameter of maintenance drain pipe	mm	
		Diameter of maintenance drain valve	mm	
	Discharge Pipe			
		Diameter of discharge pipe	mm	
6	Design high flow by-pass 'channel'			
		Longitudinal slope	%	
		Base width	m	
		Batter slopes	H:V	
7	Verification checks			
		Macrophyte zone re-suspension protection		
		Confirm treatment performance		

6.4 Landscape Design Notes

Whilst constructed wetlands play a significant role in delivering stormwater quality objectives, they can also play an important role in creating a community landscapes and urban ecology. The following sections outline some of the landscape design issues that should be considered when designing constructed wetland systems.

6.4.1 Objectives

Landscape design of wetlands generally requires consideration of the following objectives:

- Integrated planning and design of constructed wetlands within the built and landscape environments ensuring that the overall landscape design for the wetland integrates with its host natural and/ or built environment.
- Ensuring that a wetland planting strategy based on wetland design depths/zones addresses stormwater quality objectives and has the structural characteristics to perform particular treatment processes (e.g. well distributed flows, enhance sedimentation, maximise surface area for the adhesion of particles and provide a substratum for algal epiphytes and biofilms).
- Providing appropriate fringe plantings that promote habitat for fauna
- Addressing stormwater quality objectives by incorporating appropriate plant species that suit the depth range of a wetland zone and have the structural characteristics to perform particular treatment processes
- Incorporating Crime Prevention Through Environmental Design (CPTED) principles.
- Providing other landscape values, such as shade, amenity, character and place making.

Comprehensive site analysis should inform the landscape design as well as road layouts, maintenance access points and civil works. Existing site factors such as roads, buildings, landforms, soils, plants, microclimates, services and views should be considered. Refer to *Water Sensitive Urban Design in the Sydney Region: 'Practice Note 2 – Site Planning'* (LHCCREMS 2002) for further guidance.

If sited within accessible open space, constructed wetlands can be significant features within the built environment. Landscape design also has a key role in overcoming the negative perceptions that permanent water bodies, like sedimentation basins, have in some communities. In the past this may have been due to legitimate pest and safety concerns that have arisen from poorly designed and/ or managed systems, particularly remnant swamps and lagoons. Creative landscape design can enhance the appeal and sense of tranquillity that wetlands provide.

6.4.2 Context and Site Analysis

Constructed wetlands can have some impact on the available open space within new developments and considerable landscape planning needs to ensure that a balanced land use outcome is provided. Opportunities to enhance public amenity and safety with viewing areas, pathway links, picnic nodes, interpretive signage/art and other elements should be explored to further enhance the social context of constructed wetlands. Landscape treatments should respond to the local context of the site, in particular planting types as they relate to the different vegetation communities in the shire.

6.4.3 Wetland Siting and Shapes

Constructed wetlands need to be arranged to meet hydrological and stormwater quality requirements, but also to integrate effectively into the surrounding existing landscape. The arrangement of wetland, basin and high flow bypass should be designed early in the concept design phase, to ensure that amenity of open space is enhanced.

The final shape of a wetland should provide landscape opportunities to create alternate useable spaces/recreation areas. Often different shapes to wetland edges can make pathway connections through and around these recreation areas more convenient and enhances the community perception of constructed wetlands. Pathways and bridges across planted earth bunds can be the best way of getting across or around wetlands. The materials on the bridge and pathways are important to be low maintenance and do not impede hydrological flows. Ease of access to the inlet basin for sediment and trash removal is also important to consider.

The area required for the high flow bypass can be manipulated to provide open spaces that only periodically convey stormwaters. Further discussion of high flow bypass configuration is provided in Section 6.4.4.3.

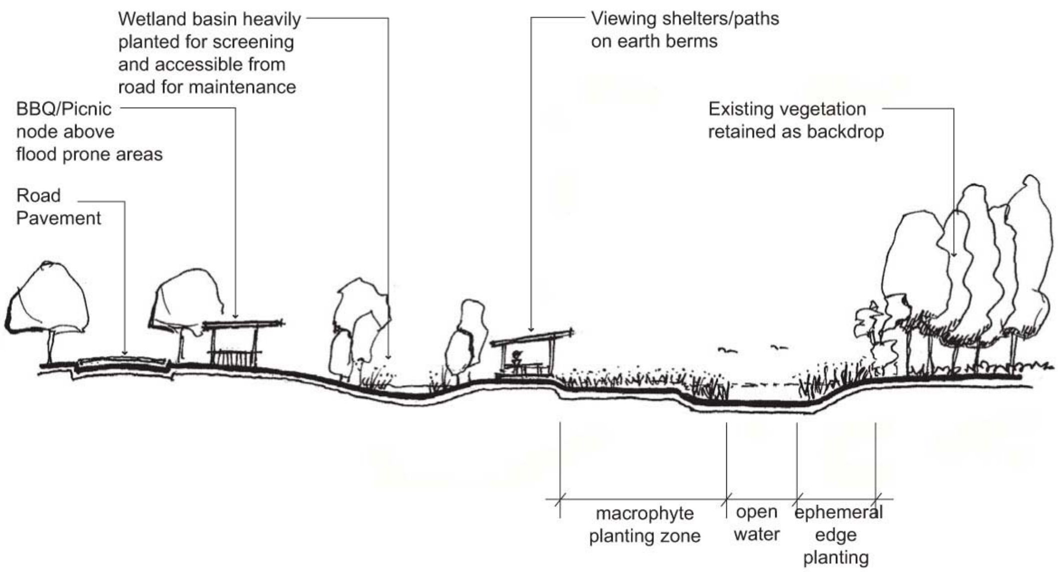


Figure 6-10: Typical landscape treatments to constructed wetlands in open space areas

Plate 6-7: Boardwalk treatment over wetland (right) and integration of urban art with wetland setting (left)

6.4.4 Specific Landscape Considerations

Numerous opportunities are available for creative design solutions for specific elements. Close collaboration between landscape designer, hydraulic designer, civil/ structural engineer and maintenance personnel is essential. In parklands and residential areas, the aim is to ensure elements are sympathetic to their surroundings and are not overly engineered or industrial in style and appearance. Additionally, landscape design to specific elements should aim to create places that local residents and visitors will come to enjoy and regard as an asset.

6.4.4.1 Crossings

Given the size and location of wetland systems, it is important to consider if access is required across the wetland as part of an overall pathway network and maintenance requirement. Factors that should be considered include:

- The appropriateness of hardwood timber board walks given their life-cycle costs. Where walkway footings are in contact with water, Council will not accept timber piers.
- If boardwalks are used, they should not be located near open water where they could encourage the public to feed wildlife.
- The use of earth bunds as crossings with culverts below. This approach allows some cut material (non-dispersive soils only) to be used on site and can be planted as a shaded walkway. They should be located within the ephemeral marsh zone of constructed wetlands or between the sedimentation basin and first macrophyte zone. Earth bund crossings can be shaped and planted to discourage wildlife feeding. Figure 6-11 illustrates a conceptual earth bund walkway.

6.4.4.2 Wetland Embankments

The landscape design approach for the wetland embankments is similar to the approach taken for embankments in sedimentation basins. Refer to Chapter 4, Section 4.4.3.3 for guidance.

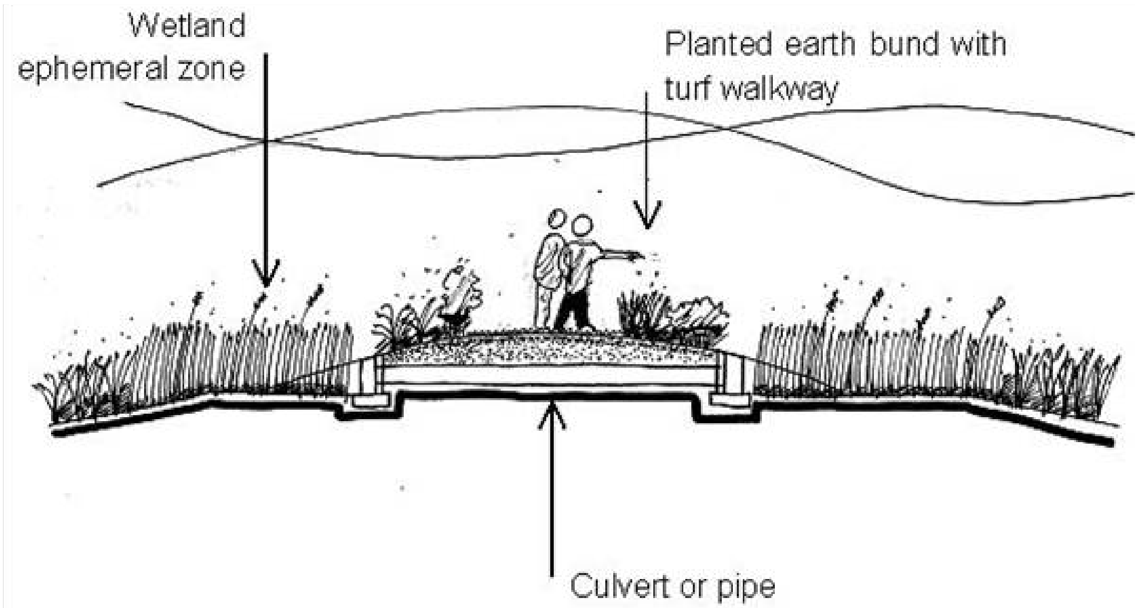


Figure 6-11: Earth Bund Structure as Wetland Crossing

6.4.4.3 High Flow Bypass Channel

The high flow bypass channel will convey stormwaters during above design flow and in some situations can form a large element in the landscape. Therefore the design of the high flow bypass needs to be carefully considered to provide recreational and landscape opportunities during times outside of above design flow events.

The key considerations for design of the high flow bypass area are as follows:

- No major park infrastructure including playgrounds, barbeques and amenity buildings to be located within the high flow bypass areas. Passive recreation infrastructure including seating and picnic tables are suitable provided they are of robust design.
- In many cases, the high flow bypass will be formed through the use of turf and in these cases the opportunity for creating more active spaces should be investigated.
- Designers should investigate the opportunities for locating trees and other vegetation types within the bypass channel. Provided hydraulic efficiencies can be accommodated, grassed mounds and landform grading of the embankment edge could also be explored to add variation and interest.
- Where groundcover species over than truf is adopted, the species should be selected to ensure appropriate response after periodic flooding, for example using *Lomandra* species.
- Areas of large revegetation or garden beds that cut through the high flow bypass zone should use thick matting mulch types that bind well to the surface to minimize loss.
- The relationship between the high flow bypass channel and the permanent water bodies should be considered in order to create interesting spaces and forms within the open space. For example, after consideration of site constraints and hydraulic parameters, designers could investigate options to separate the elements from each other or to channel both elements alongside each other. Opportunities should also be sought to achieve balanced cut and fill earthworks. Figure 6-12 (following page) provides an illustration of creation of open spaces through configuration of key wetland components.

6.4.4.4 Macrophyte Zone Outlet Structure

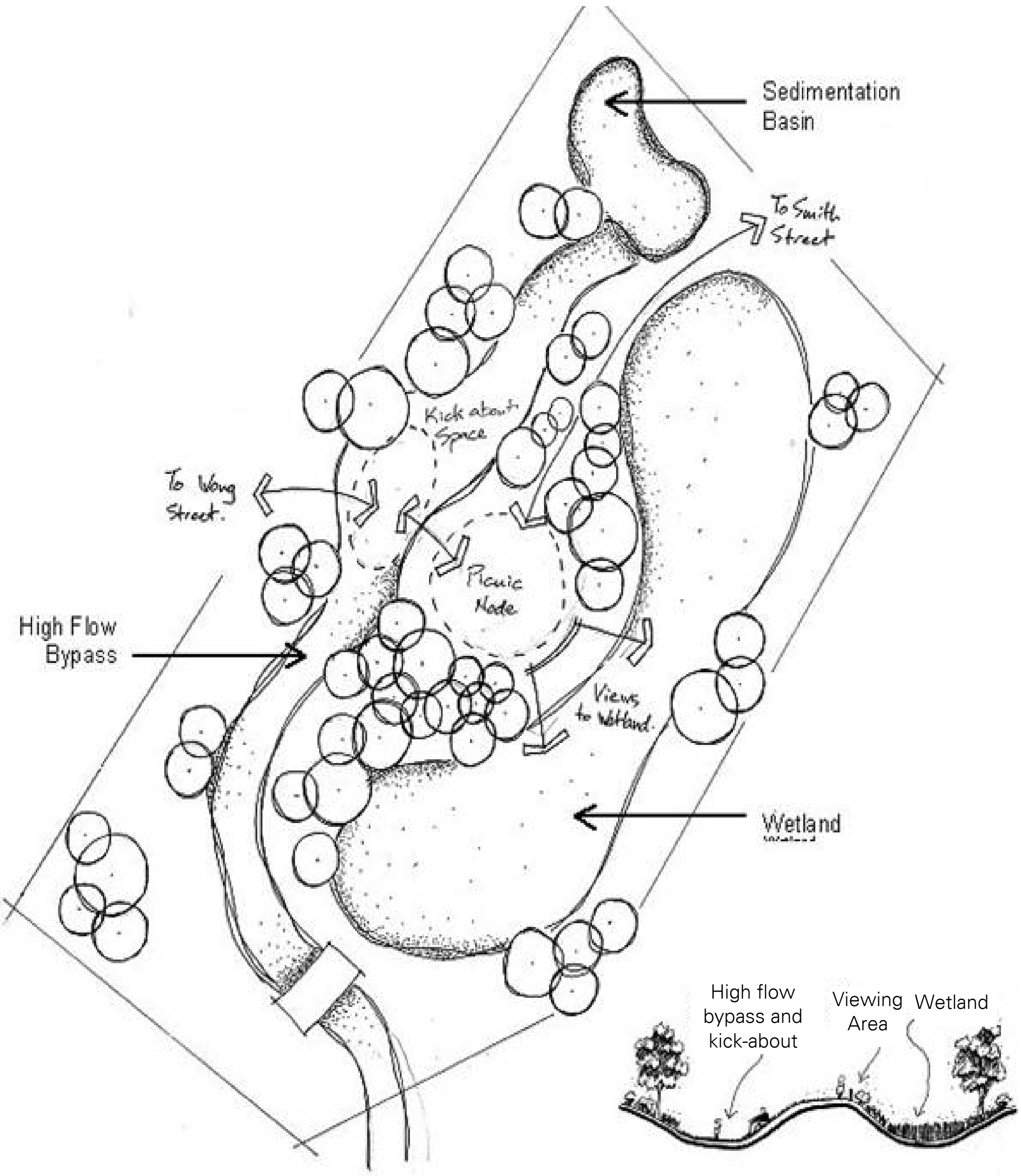
Landscape design approach for the macrophyte outlet zone is similar to the approach taken for overflow pits in sedimentation basins. Refer to Chapter 4, Section 4.4.3.5 for further guidance.

6.1.1.6 Viewing Areas

Refer to Chapter 4, Section 4.4.3.7 for guidance.

6.4.4.5 Fencing

Refer to Chapter 4, Section 4.4.3.8 for guidance.



Landscape design should explore options for siting the bypass, wetland and basin and analyse the potential for enhanced amenity. This process should initially take place at the concept development phase and can be refined during the detailed design.

Figure 6-12: Example Relationship between High Flow Bypass, Wetland and Basin and the Creation of Open Space

6.4.5 Constructed Wetland Vegetation

Planting for constructed wetlands systems may consist of up to three vegetation types:

- Macrophyte zone planting consisting of ephemeral marsh, shallow marsh, marsh and deep marsh (from 1.0 m below to 0.2 m above design water level)
- Embankment (littoral) vegetation (greater than 0.2 m above design water level)
- Terrestrial plants, including existing vegetation, adjacent to the embankment edge.

6.4.5.1 Macrophyte Zone Planting (from 1.0 m below to 0.2 m above design water level)

Appendix A provides guidance on selecting suitable plant species and cultivars that deliver the desired stormwater quality objectives for constructed wetlands. Often the most effective way to meet those objectives with the macrophyte planting is to create large bands of planting perpendicular to flow that respond to designed depth zones and local biodiversity. This reflects natural wetland systems that are often dominated by one single species.

In general, macrophyte vegetation should provide:

- well distributed flows
- enhanced sedimentation
- maximum surface area for the adhesion of particles
- a substratum for algal epiphytes and biofilms.
- habitat and refuge for fauna, both terrestrial and aquatic.

When selecting suitable species it is important to also note the ability of some species to be highly self-sustaining. Macrophytes that distribute themselves across new wetlands quickly by producing large quantities of seed material are great for colonizing and minimizing costs of replacements. Additionally, ephemeral marsh planting should provide a dense buffer between the water body and publicly accessible open space to discourage contact with the water.

6.4.5.2 Embankment (Littoral) Vegetation (greater than 0.2 m above design water level) and Parkland Vegetation

Between the macrophyte zone and the top of the embankment, establishment of trees, shrubs and groundcovers can occur in consideration of the following:

- Selecting groundcovers, particularly for slopes greater than 1 in 3, with matting or rhizomataceous root systems to assist in binding the soil surface during the establishment phase. Example species include *Imperata cylindrica, Lomandra sp.* and *Cyndodacton sp*.
- Preventing macrophyte zone plants from being shaded out by minimising tree densities at the water's edge and choosing species such as *Melaleuca* that allow sunlight to penetrate the tree canopy.
- Locating vegetation to allow views of the wetland and its surrounds whilst discouraging the public from accessing the water body.

Parkland vegetation may be of a similar species to the embankments littoral vegetation and layout to visually integrate the sedimentation basin with its surrounds. Alternatively, vegetation of contrasting species and/ or layout may be selected to highlight the water body as a feature within the landscape. Turf may be considered to achieve this goal.

6.4.6 Safety Issues

6.4.6.1 General

Constructed wetlands need to be generally consistent with public safety requirements for new developments. These include reasonable batter profiles for edges to facilitate public egress from areas with standing water and fencing where water depths and edge profile requires physical barriers to public access. The constructed wetlands can be substituted where possible by using dense edge plantings to deter public access to areas of open water. A dense hedge using local species such as Lillypillys and Bottlebrush that can get to around 2 m high and 1.5 m wide are effective in deterring public access.

6.4.6.2 Crime Prevention Through Environmental Design (CPTED)

The standard principles of informal surveillance, exclusion of places of concealment and open visible areas apply to the landscape design of wetlands. Where planting may create places of concealment or hinder informal surveillance, groundcovers and shrubs should not generally exceed 1 meter in height. For further guidance on CPTED standards refer to relevant local government guidelines.

6.4.6.3 Restricting Access to Open Water

Fences or vegetation barriers to restrict access should be incorporated into wetland areas, particularly on top of concrete or stone walls where:

- there is a risk of serious injury in the event of a fall (over 0.5 m high and too steep to comfortably walk up/ down or the lower surface or has sharp or jagged edges)
- there is a high pedestrian or vehicular exposure (on footpaths, near bikeways, near playing/ sporting fields, near swings and playgrounds etc)
- where water ponds to a depth of greater than 300 mm on a constructed surface of concrete or stone. Natural water features are exempt
- where the water is expected to contain concentrated pollutants
- where grassed areas requiring mowing abut the asset.

Fences considered appropriate are:

- pool fences (for areas adjacent to playgrounds/ sports fields where a child drowning or infection hazard is present)
- galvanised tubular handrails (without chain wire) in other areas
- dense vegetative hedges.

Dense littoral planting around the wetland and particularly around the deeper open water pools of the inlet zone (with the exception of any maintenance access points), will deter public access to the open water and create a barrier to improve public safety. Careful selection of plant species (e.g. tall, dense or 'spiky' species) and planting layouts can improve safety as well as preventing damage to the vegetation by trampling.

Dense vegetation (hedge) at least 2 m wide and 1.2 m high (minimum) may be suitable if vandalism is not a demonstrated concern (this may be shown during the initial 12 month maintenance period). A temporary fence (e.g. 1.2 m high silt fence) will be required until the vegetation has established and becomes a deterrent to pedestrians/ cyclists.

An alternative to the adoption of a barrier/ fence is to provide a 2.4 m safety bench that is less than 0.2 m deep below the permanent pool level around the waterbody. This is discussed in Chapter 4 Section 4.3.3.3 with respect to appropriate batter slopes.

6.5 Construction and Establishment

This section provides general advice for the construction and establishment of constructed wetlands and key issues to be considered to ensure their successful establishment and operation. Some of the issues raised have been discussed in other sections of this chapter and are reiterated here to emphasise their importance based on observations from construction projects around Australia.

6.5.1 Staged Construction and Establishment Method

It is important to note that constructed wetlands, like most WSUD elements that employ soil and vegetation based treatment processes, require approximately two growing seasons (i.e. two years) before the vegetation in the systems has reached its design condition (i.e. height and density). In the context of a large development site and associated construction and building works, delivering constructed wetlands and establishing vegetation can be a challenging task. Therefore, constructed wetlands require a careful construction and establishment approach to ensure the wetland establishes in accordance with its design intent. The following sections outline a recommended staged construction and establishment methodology for constructed wetlands (Leinster, 2006).

6.5.1.1 Construction and Establishment Challenges

There exist a number of challenges that must be appropriately considered to ensure successful construction and establishment of wetlands. These challenges are best described in the context of the typical phases in the development of a Greenfield or Infill development, namely the Subdivision Construction Phase and the Building Phase (see Figure 6-13).

- Subdivision Construction - Involves the civil works required to create the landforms associated with a development and install the related services (roads, water, sewerage, power etc.) followed by the landscape works to create the softscape, streetscape and parkscape features. The risks to successful construction and establishment of the WSUD systems during this phase of work have generally related to the following:
  - Construction activities which can generate large sediment loads in runoff which can smother wetland vegetation
  - Construction traffic and other works can result in damage to the constructed wetlands.

  Importantly, all works undertaken during Subdivision Construction are normally 'controlled' through the principle contractor and site manager. This means the risks described above can be readily managed through appropriate guidance and supervision.

- Building Phase - Once the Subdivision Construction works are complete and the development plans are sealed then the Building Phase can commence (i.e. construction of the houses or built form). This phase of development is effectively 'uncontrolled' due to the number of building contractors and sub-contractors present on any given allotment. For this reason the Allotment Building Phase represents the greatest risk to the successful establishment of constructed wetlands.

6.5.1.2 Staged Construction and Establishment Method

To overcome the challenges associated within delivering constructed wetlands a Staged Construction and Establishment Method should be adopted (see Figure 6-13):

- Stage 1: Functional Installation - Construction of the functional elements of the constructed wetland at the end of Subdivision Construction (i.e. during landscape works) and the installation of temporary protective measures.

- Stage 2: Sediment and Erosion Control – During the Building Phase the temporary protective measures preserve the functional infrastructure of the constructed wetland against damage whilst also providing a temporary erosion and sediment control facility throughout the building phase to protect downstream aquatic ecosystems.

- Stage 3: Operational Establishment - At the completion of the Building Phase, the temporary measures protecting the functional elements of the constructed wetland can be removed along with all accumulated sediment.

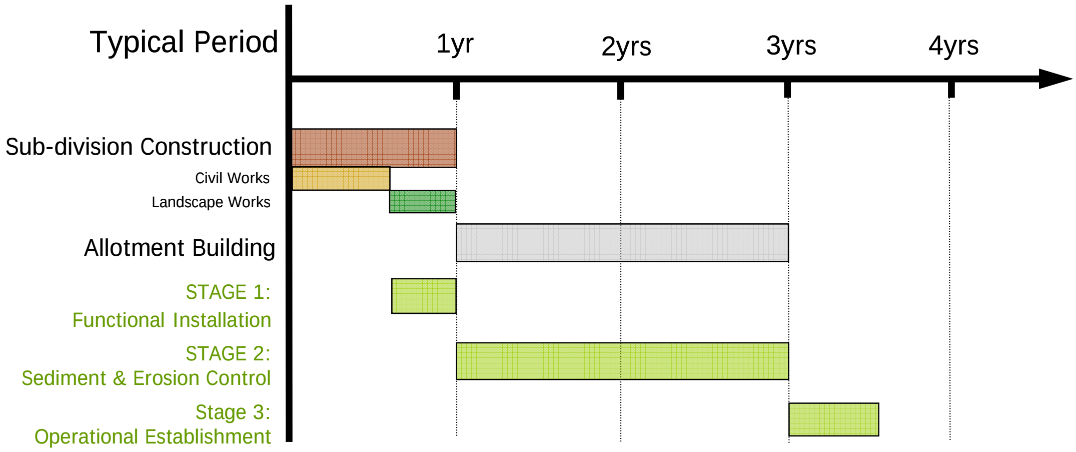


Figure 6-13: Staged Construction and Establishment Method

6.5.1.3 Functional Installation

Functional installation of constructed wetlands occurs at the end of Subdivision Construction as part of landscape works and involves:

- Earthworks to configure the bathymetry of the wetland.
- Installation of the hydraulic control structures including inlet/outlet control and the high flow bypass weir
- Placement of topsoil, trimming and profiling
- Placement of turf in the High Flow Bypass channel to protect against erosion.
- Disconnecting the Inlet Zone from Macrophyte Zone and allowing all stormwater to flow along High Flow Bypass. This effectively isolates the Macrophyte Zone from catchment flows and allows the establishment of wetland plants without the risk of being smothered with coarse sediment during the Subdivision Construction and Allotment Building Phases.
- Planting of the Macrophyte Zone once the disconnection is in place. Water level in the Macrophyte Zone can be varied as required by the rate of wetland plant maturity by opening the connection for short periods or opening the outlet control.

Plate 6-8: Constructed Wetland Functional Installation

6.5.1.4 Sediment and Erosion Control

During Allotment Building Phases the Inlet Zone will essentially form a sedimentation basin reducing the load of coarse sediment discharging to receiving environment. The disconnection will remain in place to ensure the majority of flows from the catchment continue to bypass the Macrophyte Zone thus allowing the wetland plants to reach full maturity without the risk of being smothered with coarse sediment. This means the Macrophyte Zone can be fully commissioned and made ready for operation once the Allotment Building Phase is complete.

Plate 6-9: Constructed Wetland Sediment & Erosion Control Operation

6.5.1.5 Operational Establishment

At the completion of the Allotment Building Phase the Inlet Zone is de-silted, the disconnection between the Inlet Zone and Macrohpyte Zone is removed and the constructed wetland allowed to operate in accordance with the design.

Plate 6-10: Constructed Wetland Operation Establishment

6.5.2 Construction Tolerances

It is important to emphasise the significance of tolerances in the construction of constructed wetland systems. Ensuring the relative levels of the control structures (inlet connection to microphyte zone, bypass weir and macrophyte zone outlet) are correct is particularly important to achieve appropriate hydraulic functions. Generally control structure tolerance of plus or minus 5 mm is considered acceptable.

Additionally the bathymetry of the macrophyte zone must be free from localized depressions and low points resulting from earthworks. This is particularly important to achieve a well distributed flow path and to prevent isolated pools from forming (potentially creating mosquito habitat) when the wetland drains. Generally an earthworks tolerance of plus or minus 25 mm is considered acceptable.

6.5.3 Sourcing Wetland Vegetation

To ensure the specified plant species are available in the required numbers and of adequate maturity in time for wetland planting, it is essential to notify nurseries early for contract growing. When early ordering is not undertaken, the planting specification may be compromised due to sourcing difficulties, resulting in poor vegetation establishment and increased initial maintenance costs. The species listed in Table A.2 (Appendix A) are generally available commercially from local native plant nurseries but availability is dependent upon many factors including demand, season and seed availability. To ensure the planting specification can be accommodated the minimum recommended lead time for ordering is 3-6 months. This generally allows adequate time for plants to be grown to the required size. The following sizes are recommended as the minimum:

- Viro Tubes 50 mm wide x 85 mm deep
- 50 mm Tubes 50 mm wide x 75 mm deep
- Native Tubes 50 mm wide x 125 mm deep

A system of interlocking plantings/ containers is recommended for initial wetland planting, particularly for deep marsh and marsh zones. This involves a series of plants (usually 5) grown together in a single ‘strip’ container. Generally, more mature plants with developing rhizomes (for rhizomatous species), are grown together creating interlocking roots. This has been used very successfully in wetland planting previously because the larger more mature plants, often with a thick rhizome system, can survive in deeper water and are more tolerant to fluctuations in water level. The structure of this system slows the movement of water and binds the substrate, helping to reduce erosion. The weight of the interlocking plants also prevents birds from removing them, a common problem encountered during wetland plant establishment. Nurseries require a minimum lead time of 6 months for supply of these systems.

6.5.4 Topsoil Specification and Preparation

The provision of suitable topsoil in wetlands is crucial to successful macrophyte establishment and to the long term functional performance of the wetland. Wetland macrophytes typically prefer medium textured silty to sandy loams that allow for easy rhizome and root penetration. Although there are a few plants that can grow in in-situ heavy clays (e.g. Phragmites), growth is slow and the resulting wetland system will have low species richness, which is undesirable. The wetland must therefore have a layer of topsoil no less than 200 mm deep.

During the wetland construction process, topsoil is to be stripped and stockpiled for possible wetland reuse as a plant growth medium. Most terrestrial topsoils provide a good substratum for wetlands, nonetheless laboratory soil testing (using Australian Standard testing procedures) of the in-situ topsoil is necessary to ensure the topsoil will support plant and microbial growth and have a high potential for nutrient retention. Typically, standard horticultural soil analysis, which includes major nutrients and trace elements, is suitable for topsoils intended for wetland use. The laboratory report will indicate the soils suitability as a plant growth medium and if any amendments are required.

If the in-situ topsoil is found to contain high levels of salt, extremely low levels of organic carbon (<< 5 %), or any other extremes that may be considered a retardant to plant growth, it should be rejected.

If the in-situ topsoil is not suitable and soil amendment is considered impractical or not cost effective, sandy loam topsoil should be purchased from a soil supplier. If the local topsoil is suitable but very shallow, mixing with an imported soil will be necessary to reach the required volume to ensure a minimum 200 mm deep topsoil for wetland planting.

Imported topsoils are generally suitable as wetland plant growth medium, however as for in-situ soils (above), testing is required to determine the appropriate gypsum or lime dosing rate. If the local topsoil was tested and found to be suitable but then mixed with an imported soil to meet the required volume, laboratory soil testing should be repeated.

Any imported soils must not contain Fire Ants. A visual assessment of the soils is required and any machinery should be free of clumped dirt. Soils must not be brought in from Fire Ant restricted areas.

6.5.4.1 Topsoil Treatments

The wetland topsoil should be tested in accordance with AS 4419-2003: *Soils for landscaping and garden use* to ensure it is appropriate for growth of vegetation. If testing finds the topsoil is not appropriate then an alternative source should be found.

Topsoils for wetlands generally do not require fertiliser treatment. Imported foreign loam will contain sufficient nutrients for vegetation growth and local terrestrial topsoil will release nutrients after the wetting process. Submersion of terrestrial soils in water causes a shift from aerobic to anaerobic processes, prompting mineralisation and decomposition of organic matter contained in the soil, thus increasing available nitrogen. When soils become anaerobic, reduction processes cause iron oxides to be released from the surface of soil particles leading to increased availability of phosphorus. The addition of nutrients (fertiliser application) can facilitate the growth of algae (including cyanobacteria (blue-green) algae), particularly when the competing macrophytes and submerged plants are in the early stages of development, increasing the likelihood of algal blooms.

The topsoil within the wetland (macrophyte zones and open water zones) may need to be treated with gypsum or lime. The application of gypsum is standard on most construction sites for the purpose of securing or flocculating dispersive soils if entrained in runoff. The use of gypsum in wetland should only occur within catchments with dispersive soils and applied at a maximum rate of 0.4 kg/m^2. The application of lime may be required where the AS4419 (2003) soil testing identifies a potential soil pH problem (pH < 5) or where acid sulfate soils (ASS) exist in the vicinity of the wetland. The rate of lime application should be guided by soil test results, an ASS Management Plan and water quality (pH) monitoring of the wetland and inflow.

Gypsum/ lime should be applied about one week prior to vegetation planting. Subsequent application may be required at intervals depending on water quality monitoring. Application of gypsum/ lime too far in advance of planting may lead to aquatic conditions that promote algal growth (i.e. clear water with no aquatic plants competing for resources).

6.5.5 Vegetation Establishment

6.5.5.1 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. October and November are considered ideal times to plant vegetation in treatment elements. This allows for adequate establishment/ root growth before the heavy summer rainfall period but also allows the plants to go through a growth period soon after planting, resulting in quicker establishment. Planting late in the year also avoids the dry winter months, reducing maintenance costs associated with watering. Construction planning and phasing should endeavour to correspond with suitable planting months wherever possible. However, as lead times from earthworks to planting can often be long, temporary erosion controls (e.g. use of matting or sterile grasses to stabilise exposed batters) should always be used prior to planting.

6.5.5.2 Water Level Manipulation

To maximise the chances of successful vegetation establishment, the water level of the wetland system is to 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 be able to exist in their prescribed water depths (depending on the maturity of the plant stock provided). Macrophytes intended for the deep marsh sections will need to have half of their form above the water level, which may not be possible if initially planted at their intended depth. Similarly, if planted too deep, 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.

The water depth must be controlled in the early establishment phase. This can be achieved by closing off the connection between the inlet zone and the macrophyte zone (i.e. covering the overflow pit) and opening the maintenance drain. The deep marsh zones should have a water depth of approximately 0.2 m for at least the first 6 - 8 weeks. This will ensure the deep marsh and marsh zones of the wetland are inundated to shallow depth and the shallow marsh zone remains moist (muddy) providing suitable conditions for plant establishment. Seedlings planted in the ephemeral marsh and littoral zones of the wetland will require ongoing watering at a similar rate as the terrestrial landscape surrounding the wetland (Section 6.4.6.2). When it is evident that the plants are establishing well and growing actively, a minimum of 6 - 8 weeks following planting, the plants should be of sufficient stature to endure deeper water. At this time, the connection between the inlet pond and the macrophyte zone can be temporarily opened to allow slow filling of the wetland to the design operating water level.

6.5.5.3 Weed Control

Weed management in constructed wetlands is important to ensure that weeds do not out compete the species planted for the particular design requirements. This may also include some native species like Phragmites that naturally can appear in constructed wetlands and out-compete other more important planted species.

Conventional surface mulching of the wetland littoral berms with organic material like tanbark is not recommended. Most organic mulch floats and water level fluctuations and runoff typically causes this material to be washed into the wetland with a risk of causing blockages to outlet structures. Mulch can also increase the wetland organic load, potentially increasing nutrient concentrations and the risk of algal blooms. Adopting high planting density rates and if necessary applying a suitable biodegradable erosion control matting to the wetland batters (where appropriate), will help to combat weed invasion and will reduce maintenance requirements for weed removal. If the use of mulch on the littoral zones is preferred, it must be secured in place with appropriate mesh or netting (e.g. jute mesh).

6.5.5.4 Watering

Regular watering of the littoral and ephemeral marsh zone vegetation during the plant establishment phase is essential for successful establishment and healthy growth. The frequency of watering to achieve successful plant establishment is dependent upon rainfall, maturity of planting stock and the water level within the wetland. However, the following watering program is generally adequate but should be adjusted (i.e. increased) as required to suit site conditions:

- Week 1-2 3 visits/ week
- Week 3-6 2 visits/ week
- Week 7-12 1 visit/ week

After this initial three month period, watering may still be required, particularly during the first winter (dry period). Watering requirements to sustain healthy vegetation should be determined during ongoing maintenance site visits.

6.5.5.5 Bird Protection

During the early stages of wetland establishment, water birds can be a major nuisance due to their habit of pulling out recently planted species. Interlocking planting systems (i.e. where several plants are grown together in a single container such as 'floral edges') can be used, as water birds find it difficult to lift the interlocking plants out of the substrate unlike single plants grown in tubes.

6.6 Maintenance Requirements

Wetlands treat runoff by filtering it through vegetation and providing extended detention to allow sedimentation to occur. 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 healthy vegetation and adequate flow conditions in a wetland are the key maintenance considerations. Weeding, planting, mowing and debris removal are the dominant tasks (but should not include use of herbicides as this affects water quality). 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 4). Routine maintenance of wetlands should be carried out once a month.

The most intensive period of maintenance is during 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. Debris removal is an ongoing maintenance function. If not removed, debris can block inlets or outlets, and can be unsightly if in a visible location. Inspection and removal of debris should be done regularly. Typical maintenance of constructed wetlands will involve:

- desilting the inlet zone following the construction/ building period
- routine inspection of the wetland to identify any damage to vegetation, scouring, formation of isolated pools, litter and debris build up or excessive mosquitoes
- routine inspection of inlet and outlet points to identify any areas of scour, litter build up and blockages
- removal of litter and debris
- removal and management of invasive weeds
- repair to wetland profile to prevent the formation of isolated pools
- periodic (usually every 5 years) draining and desilting of the inlet pond
- regular watering of littoral vegetation during plant establishment
- water level control during plant establishment
- replacement of plants that have died (from any cause) with plants of equivalent size and species as detailed in the planting schedule
- vegetation pest monitoring and control.

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

All maintenance activities must be specified in a maintenance plan (and associated maintenance inspection forms) to be developed as part of the design procedure (Step 9). Maintenance personnel and asset managers will use this plan to ensure the wetlands continue to function as designed. To ensure maintenance activities are appropriate for the wetland as it develops, maintenance plans should be updated a minimum of every three years. The maintenance plans and forms must address the following:

- inspection frequency
- maintenance frequency
- data collection/ storage requirements (i.e. during inspections)
- detailed clean-out procedures (main element of the plans) including:
  - equipment needs
  - maintenance techniques
  - occupational health and safety
  - public safety
  - environmental management considerations
  - disposal requirements (of material removed)
  - access issues
  - stakeholder notification requirements
  - data collection requirements (if any)
- design details.

An approved maintenance plan is required prior to asset transfer to the local authority. Refer to the guidelines or direction from the relevant local authority for more specific guidance on requirements for asset transfer.

An example operation and maintenance inspection form is included in the checking tools provided in Section 6.7.3. These forms must be developed on a site specific basis as the configuration and nature of constructed wetlands varies significantly.

6.7 Checking Tools

This section provides a number of checking aids for designers and Council development assessment officers. In addition, Section 6.6.5 provides general advice for the construction and establishment of wetlands and key issues to be considered to ensure their successful establishment and operation, based on observations from construction projects around Australia. The following checking tools are provided:

- Design Assessment Checklist;
- Construction Inspection Checklist (during and post);
- Operation and Maintenance Inspection Form; and
- Asset Transfer Checklist (following 'on-maintenance' period).

6.7.1 Design Assessment Checklist

The checklist on page 6-39 presents the key design features to be reviewed when assessing a design of a wetland. 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 must have all necessary permits for its installations. Council development assessment officers will require supporting evidence/ proof from the developer that all relevant permits are in place.

6.7.2 Construction Checklist

The checklist on page 6-40 presents the key items to be reviewed when inspecting the bioretention basin during and at the completion of construction. The checklist is to be used by Construction Site Supervisors and local authority Compliance Inspectors to ensure all the elements of the bioretention basin have been constructed in accordance with the design. If an item receives an 'N' in Satisfactory criteria then appropriate actions must be specified and delivered to rectify the construction issue before final inspection sign-off is given.

6.7.3 Operation and Maintenance Inspection Form

The example form on page 6-41 should be developed and 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 1 - 6 months depending on the size and complexity of the system. More detailed site specific maintenance schedules should be developed for major constructed wetland systems and include a brief overview of the operation of the system and key aspects to be checked during each inspection.

6.7.4 Asset Transfer Checklist

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. The table on page 6-42 provides an indicative asset transfer checklist.

WETLAND DESIGN ASSESSMENT CHECKLIST		
Asset I.D.		
Wetland Location:		
Hydraulics:	Design operational flow (m^3/s):	Above design flow (m^3/s):
Area:	Catchment Area (ha):	Wetland Area (ha):

TREATMENT	Y	N
MUSIC modelling performed?		
INLET ZONE	**Y**	**N**
Discharge pipe/structure to inlet zone sufficient for maximum design flow?		
Scour protection provided at inlet for inflow velocities?		
Configuration of inlet zone (aspect, depth and flows) allows settling of particles >125µm?		
Bypass weir incorporated into inlet zone?		
Bypass weir length sufficient to convey 'above design flow' ?		
Bypass weir crest at macrophyte zone top of extended detention depth?		
Bypass channel has sufficient capacity to convey 'above design flow'?		
Bypass channel has sufficient scour protection for design velocities?		
Inlet zone connection to macrophyte zone overflow pit and connection pipe sized to convey the design operation flow?		
Inlet zone connection to macrophyte zone allows energy dissipation?		
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 safety design considerations included in inlet zone design?		
Where required, gross pollutant protection measures provided on inlet structures (both inflows and to macrophyte zone)		
MACROPHYTE ZONE	**Y**	**N**
Extended detention depth >0.25m and <0.5m?		
Vegetation bands perpendicular to flow path?		
Appropriate range of macrophyte vegetation (ephemeral, shallow, marsh, deep marsh)?		
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?		
Public safety design considerations included in macrophyte zone (i.e. batter slopes less than 5(H):1(V)?		
Maintenance access provided into areas of the macrophyte zone (especially open water zones)?		
Safety audit of publicly accessible areas undertaken?		
Freeboard provided above extended detention depth to define embankments?		
OUTLET STRUCTURES	**Y**	**N**
Riser outlet provided in macrophyte zone?		
Notional detention time of 48-72 hours?		
Orifice configuration allows for a linear storage-discharge relationship for full range of the extended detention depth?		
Maintenance drain provided?		
Discharge pipe has sufficient capacity to convey maximum of either the maintenance drain flows or riser pipe flows with scour protection?		
Protection against clogging of orifice provided on outlet structure?		
COMMENTS		

WETLAND CONSTRUCTION INSPECTION CHECKLIST

Asset I.D.		Inspected by:	
Site:		Date:	
Constructed by:		Time:	
		Weather:	
		Contact During Visit:	

Items inspected	Checked		Satisfactory		Items inspected	Checked		Satisfactory	
	Y	N	Y	N		Y	N	Y	N
DURING CONSTRUCTION									
A. FUNCTIONAL INSTALLATION					Structural components cont				
Preliminary Works					22. Ensure spillway is level				
1. Erosion and sediment control plan adopted					23. Provision of maintenance drain(s)				
2. Limit public access					24. Collar installed on pipes				
3. Location same as plans					25. Low flow channel is adequate				
4. Site protection from existing flows					26. Protection of riser from debris				
5. All required permits in place					27. Bypass channel stabilised				
Earthworks					28. Erosion protection at macrophyte outlet				
6. Integrity of banks					Vegetation				
7. Batter slopes as plans					29. Vegetation appropriate to zone (depth)				
8. Impermeable (eg. clay) base installed					30. Weed removal prior to planting				
9. Maintenance access to whole wetland					31.Provision for water level control				
10. Compaction process as designed					32. Vegetation layout and densities as designed				
11. Placement of adequate topsoil					33. Provision for bird protection				
12. Levels as designed for base, benches, banks and spillway (including freeboard)					34. By-pass channel vegetated				
13. Check for groundwater intrusion									
14. Stabilisation with sterile grass					B. EROSION AND SEDIMENT CONTROL				
Structural components					35. Disconnect inlet zone from macrophyte zone (flows via high flow bypass)				
15. Location and levels of outlet as designed					36. Inlet zone to be used as sediment basin during construction				
16. Safety protection provided					37. Stabilisation immediately following earthworks and planting of terrestrial landscape around basin				
17. Pipe joints and connections as designed					38. Silt fences and traffic control in place				
18. Concrete and reinforcement as designed									
19. Inlets appropriately installed					C. OPERATIONAL ESTABLISHMENT				
20. Inlet energy dissipation installed					39. Inlet Zone desilted				
21. No seepage through banks					40Inlet zone disconnection removed				

FINAL INSPECTION									
1. Confirm levels of inlets and outlets					8. Public safety adequate				
2. Confirm structural element sizes					9. Check for uneven settling of banks				
3. Check batter slopes					10. Evidence of stagnant water, short circuiting or vegetation scouring				
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					13. Evidence of debris in high flow bypass				
7. Maintenance access provided					14. Macrophyte outlet free of debris				

COMMENTS ON INSPECTION

ACTIONS REQUIRED
1.
2.
3.
4.
Inspection officer signature:

WETLAND MAINTENANCE CHECKLIST			
Asset I.D.			
Inspection Frequency:	1 to 6 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			

ASSET TRANSFER CHECKLIST		
Asset ID:		
Asset Description:		
Asset Location:		
Construction by:		
'On-maintenance' Period:		
TREATMENT	Y	N
System appears to be working as designed visually?		
No obvious signs of under-performance?		
MAINTENANCE	Y	N
Maintenance plans and indicative maintenance costs provided for each asset?		
Vegetation establishment period completed (2 years?)		
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 (e.g. drawings, survey, models) provided?		
Asset listed on asset register or database?		
COMMENTS		

6.8 Constructed Wetland Worked Example

As part of a residential development in the greater Brisbane area, stormwater runoff is to be delivered to a constructed wetland for water quality treatment. An illustration of the site and proposed layout of the wetland is shown in Figure 6-14. This worked example describes the design process for each component of the constructed wetland: inlet zone (including the bypass weir), macrophyte zone, macrophyte zone outlet and high flow bypass channel.

Catchment Characteristics

The development is a typical detached housing estate (15 lots/ hectare) served by 14 m wide local road reserves. Due to the moderate to steep gradient through the contributing catchment (10 ha), stormwater runoff is collected and conveyed to the wetland inlet zone via conventional piped drainage with minor storm (2 year ARI) flows discharged to the wetland inlet zone via a 975 mm diameter pipe and major storm (50 year ARI) entering via overland flow.

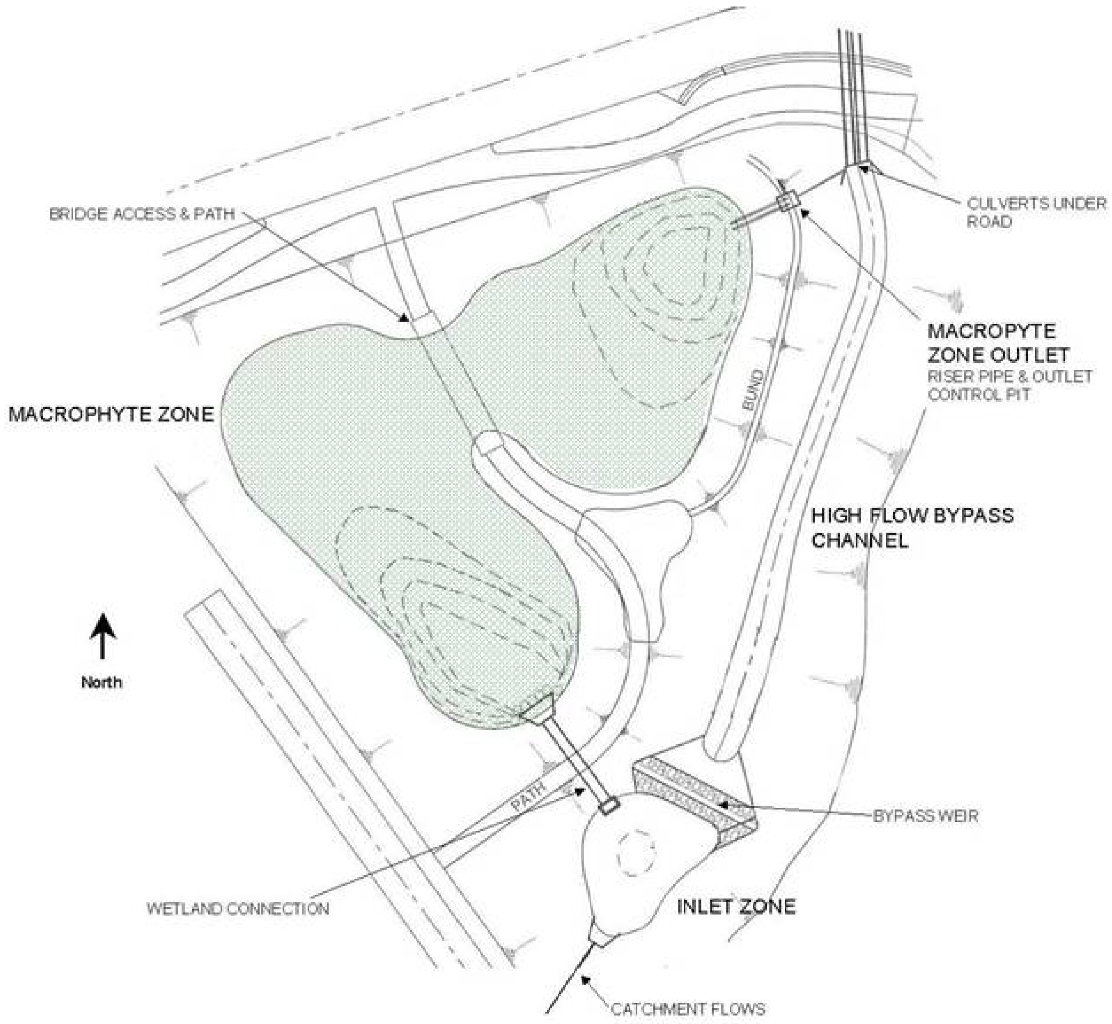


Figure 6-14: Layout of Proposed Wetland System

Site Characteristics

The site has a moderate fall of 2.5 m from south to north and is constrained by roads to the west and north and by steeper grades to the east. Soils through the site have been classified as clay.

Conceptual Design

The conceptual design of the constructed wetland (as shown in Figure 6-14) established the following key design elements to ensure effective operation:

- wetland macrophyte zone extended detention depth of 0.5 m, permanent pool level of 11.5 m AHD and an area of 7000 m^2
- inlet zone permanent pool level of 11.7 m AHD, which is 0.2 m above the permanent pool level of the macrophyte zone
- bypass weir ('spillway' outlet) level of 12 m AHD set at the top of extended detention in the wetland macrophyte zone and 0.3 m above the inlet zone permanent pool level
- high flow bypass channel longitudinal grade of 1.5%.

During the conceptual design phase, the configuration described above and shown in Figure 6-14 was modelled using MUSIC to ensure the stormwater discharges from the site comply with local authority water quality objectives (WQOs). In this case, delivering the local authority WQOs equates to an 80 % reduction in mean annual TSS load, more than 60 % reduction in mean annual TP load and 45 % reduction in mean annual TN load. To achieve these objectives, the wetland concept required a macrophyte zone area of 7000 m^2, extended detention depth of 0.5 m and detention time of 72 hours.

6.8.1 Step 1: Verify size for Treatment

The design curves presented earlier in this chapter have been used to verify the wetland size required to deliver the pollutant load reduction described above. From Figures 6.2 to 6.4, the wetland size to deliver the required load reductions (based on 0.5 m extended detention depth) is 7 % of the catchment area, equating to 7000 m^2 (macrophyte zone area). This verifies the MUSIC modelling results undertaken during the concept design phase and confirms the wetland conceptual design can be now progressed to detailed design and documentation.

6.8.2 Step 2: Determine Design Flows

The site has a contributing catchment of 10 ha which is drained via conventional pipe drainage. Both the minor storm (2 year ARI) and the major storm (50 year ARI) flows enter the inlet zone of the wetland. Therefore, the 50 year ARI peak flow sets the 'above design flow'. The 'design operation flow', which is required to size the inlet zone and the inlet zone connection to the macrophyte zone, is the 1 year ARI peak flow.

Design flows are established using the Rational Method using QUDM (DPI, IMEA & BCC, 1992) and local government guidelines. The time of concentration (t_c) was calculated using the procedures outlined in Section 5.05 of QUDM and found to be 10 minutes. The coefficient of runoff was taken from local government guidelines as follows:

C_{10} = 0.8 (from local government guidelines)

	C Runoff		
ARI	1	10	50
QUDM Factor	0.8	1	1.15
C_{ARI}	0.64	0.8	0.92

Catchment area, A = 10 ha
Rainfall Intensities, t_c = 10 mins
I_1 = 90 mm/hr
I_{50} = 227 mm/hr

Rational Method Q = $CIA/360$
'Design operation flow' (1-year ARI) = 1.60 m^3/s
'Above design flow' (50-year ARI) = 5.80 m^3/s

6.8.3 Step 3: Design Inlet Zone

The design of the inlet zone is undertaken in accordance with the design procedures outlined in Chapter 4 with a summary of the key inlet zone elements provided below.

6.8.3.1 Inlet Zone (Sedimentation Basin) Size

An initial estimate of the inlet zone area can be established using the curves provided in Figure 4.3 of Chapter 4. Assuming a notional permanent pool depth of 2 m and an inlet zone extended detention depth of 0.3 m (i.e. 0.5 m macrophyte zone extended detention depth – 0.2 m level difference between the permanent pools), a sedimentation basin area of 360 m^2 is required to capture 90% of the 125 μm particles for flows up to the 'design operation flow' (1 year ARI = 1.6 m^3/s). Confirmation of the sedimentation basin area is provided by using Equation 4.1 in Chapter 4.

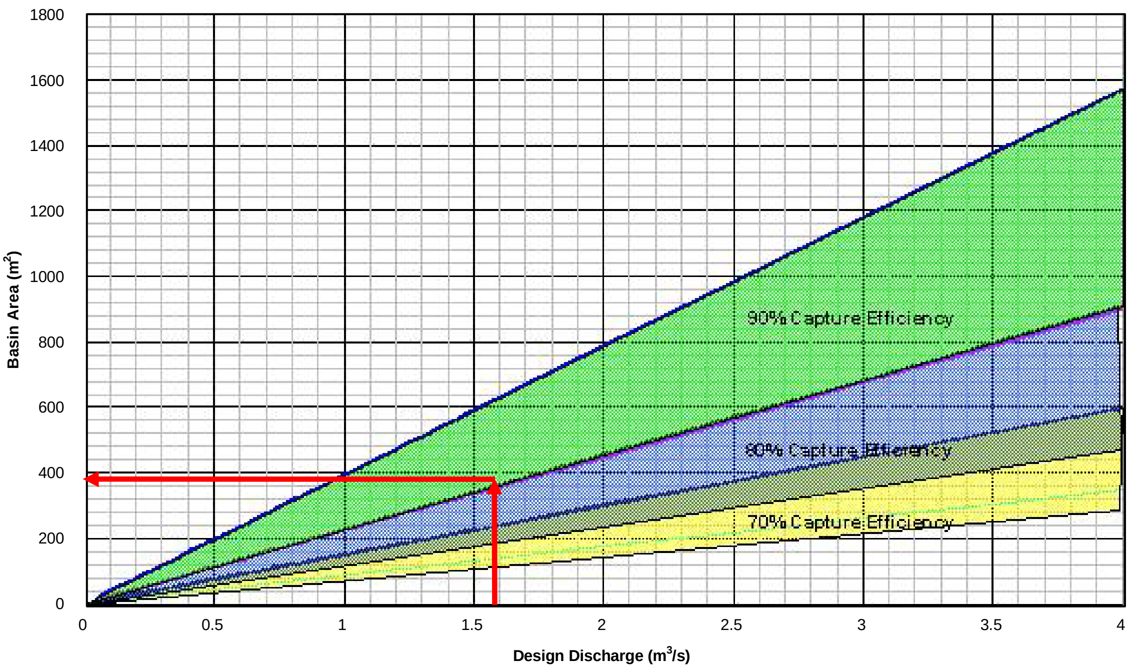


Figure 6-15: Figure 4.3 from Chapter 4 (reproduced for this example)

A further consideration in the design of the inlet zone is the provision of adequate storage for settled sediment to prevent the need for frequent desilting. A desirable frequency of basin desilting is once every five years. To ensure this storage zone is appropriate the following must be met (refer to Chapter 4):

Sedimentation Basin Storage Volume (V_s) > Volume of accumulated sediment over 5 years ($V_{s:5yr}$)

The sedimentation basin storage volume (V_s) is defined as the storage available in the bottom half of the inlet zone permanent pool. Considering the internal batters of the inlet zone will be 2:1 (H:V) below the permanent water level the area of the basin at 1 m depth is 153 m^2 and at 2 m depth 17 m^2. Therefore, the sedimentation basin storage volume V_s is 85 m^3.

The volume of accumulate sediments over 5 years ($V_{s:5yr}$) is established using Equation 4.3 from Chapter 4 (using a sediment discharge rate of 1.6 m^3/Ha/yr):

$$V_{s:5yr} = A_c \quad R \quad L_o \quad F_c \qquad = 10 \times 0.9 \times 1.6 \times 5 = 72\ m^3$$

Therefore, $V_s > V_{s:5yr}$, hence OK.

6.8.3.2 Inlet Zone Connection to Macrophyte Zone

The configuration of the hydraulic structure connecting the inlet zone to the macrophyte zone consists of an overflow pit (in the inlet zone) and a connection pipe with the capacity to convey the 'design operation flow' (1-year ARI = 1.60 m^3/s). As defined by the conceptual design (Section 6.7.1.2) the follow design elements apply:

- Inlet zone permanent pool level (overflow pit crest level) = 11.7 m AHD which is 0.2 m above the permanent pool level of the macrophyte zone
- Bypass weir ('spillway' outlet) crest level = 12 m AHD which is the top of extended detention for the wetland and 0.3 m above the inlet zone permanent pool level.

It is common practice to allow for 0.3 m of freeboard above the afflux level when setting the top of embankment elevation.

<u>Overflow Pit</u>

According to Section 4.3.5 in Chapter 4, two possible flow conditions need to be checked: weir flow conditions (with extended detention of 0.3 m) and orifice flow conditions.

Weir Flow Conditions

From Equation 4.4 (Chapter 4), the required perimeter of the outlet pit to pass 1.6 m^3/s with an afflux of 0.3 m can be calculated assuming 50% blockage:

$$P = \frac{Q_{des}}{B \cdot C_w \cdot h^{3/2}} = \frac{1.6}{0.5 \cdot 1.66 \cdot 0.3^{3/2}} = 11.7\,\mathrm{m}$$

Orifice Flow Conditions

From Equation 4.5 (Chapter 4), the required area of the outlet pit can be calculated as follows:

$$A_o = \frac{Q_{des}}{B \;\; C_d \sqrt{2 \;\; g \;\; h}} = \frac{1.6}{0.5 \;\; 0.6\sqrt{2 \;\; g(0.3)}} = 2.2\ m^2$$

In this case the weir flow condition is limiting. Considering the overflow pit is to convey the 'design operation flow' (1 year ARI) or slightly greater, a 2000 x 4000 mm pit size is adopted providing a perimeter of 12 m which is greater than the 11.7 m calculated using the weir flow equation above. The top of the pit is to be fitted with a letter box grate. This will ensure large debris does not enter the 'control' structure while avoiding the likely of blockage of the grate by smaller debris.

<u>Connection Pipe(s)</u>

As the connection pipe (i.e. between the inlet zone and the macrophyte zone) is to be submerged, the size can be determined by firstly estimating the required velocity in the connection pipe using the following:

$$h = \frac{2 \cdot V^2}{2 \cdot g}$$

Where *h* = maximum available head level driving flow through the pipe (defined as the bypass weir spillway outlet crest level minus the normal water level in the macrophyte zone = 0.5 m)

V = pipe velocity (m/s)

g = 9.79 m/s^2

Note: the coefficient of 2 in the equation is a conservative estimate of the sum of entry and exit loss coefficients ($K_{in} + K_{out}$).

Hence, $V = (9.79 \times 0.5)^{0.5} = 2.21$ m/s

The area of pipe required to convey the 1 year ARI is then calculated using the continuity equation by dividing the 1 year ARI flow (Q_1 = 1.60 m^3/s) by the velocity:

$$A_{pipe} = \frac{Q}{V} = \frac{1.60}{2.21} = 0.724\ \text{m}^2$$

This area is equivalent to two (2) 675 mm reinforced concrete pipes (RCPs). The obvert of the pipes is to be set below the permanent water level in the wetland macrophyte zone (11.5 m AHD) meaning the invert is at 10.80 m AHD.

6.8.3.3 High Flow Bypass Weir

All flows in excess of the 'design operation flow' and up to the 'above design flow' are to bypass the wetland macrophyte zone. This is facilitated by a high flow bypass weir ('spillway' outlet) designed to convey the 'above design flow' (50 year ARI) with the weir crest level 0.3 m above the permanent pool of the inlet pond.

Assuming a maximum afflux of 0.3 m, the weir length is calculated using the weir flow equation (Equation 4.4 in Chapter 4):

$$L = \frac{Q_{des}}{C_w \cdot H^{3/2}} = \frac{5.8}{1.66 \cdot 0.3^{3/2}} = 21.3\ \text{m (adopt 22 m)}$$

To ensure no flows breach the embankment separating the inlet zone and the macrophyte zone the embankment crest level is to be set at 12.6 m AHD (i.e. 0.3 m freeboard on top of the maximum afflux level over the high flow bypass weir).

Inlet Zone Area	= 360 m^2 set at 11.7 m AHD
Overflow pit	= 2000 x 4000 mm with letter box grate set at 11.7 m AHD
Pipe connection (to wetland)	= 2 x 675 mm RCPs at 10.80 m AHD
High flow bypass weir	= 22 m length set at 12.0 m AHD

6.8.4 Step 4: Designing the Macrophyte Zone

6.8.4.1 Length to Width Ratio and Hydraulic Efficiency

A macrophyte zone area of 7000 m^2 was established as part of the conceptual design and verified as part of Step 1. The layout of the wetland as presented in Figure 6-14 represents 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 6-6 (but closer to Case G). Thus, the expected hydraulic efficiency (λ) is 0.6-0.7.

Aspect Ratio	= 6(L) to 1(W)
Hydraulic Efficiency	~ 0.6-0.7

6.8.4.2 Designing the Macrophyte Zone Bathymetry

Being a typical residential catchment, the wetland macrophyte zone has been configured to target sediment and nutrient capture. Therefore, the macrophyte zone of the wetland is divided into four marsh zones and an open water zone as depicted in Figure 6-16 and described below:

- The bathymetry across the four marsh zones is to vary gradually over the length of the macrophyte zone, ranging from 0.2 m above the permanent pool level (ephemeral zone) to 0.5 m below the permanent pool level (see Figure 6-16 and Table 6-2). The ephemeral marsh zone is to be located adjacent to the pathway and bridge crossing mid way along the wetland.
- The permanent pools upstream and downstream of the ephemeral zone are to be connected via the maintenance drain to ensure the upstream permanent pool can drain down to 11.5 m AHD following a rainfall event.
- The depth of the open water zone in the vicinity of the outlet structure is to be 1 m below the permanent pool level.
- The marsh zones are arranged in bands of equal depth running across the flow path to optimise hydraulic efficiency and reduce the risk of short-circuiting.

Table 6-2: Indicative Break of Marsh Zones

Zone	Depth Range (m)	Proportion of Macrophyte Zone Surface Area (m)
Open Water (Pool)	>1.0 below permanent pool	10%
Transition	0.5 – 1.0 below permanent pool	10%
Deep Marsh	0.35 – 0.5 below permanent pool	20%
Marsh	0.2 – 0.35 below permanent pool	20%
Shallow Marsh	0.0 – 0.2 below permanent pool	20%
Ephemeral Marsh	0.2 – 0.0 above permanent pool	20%

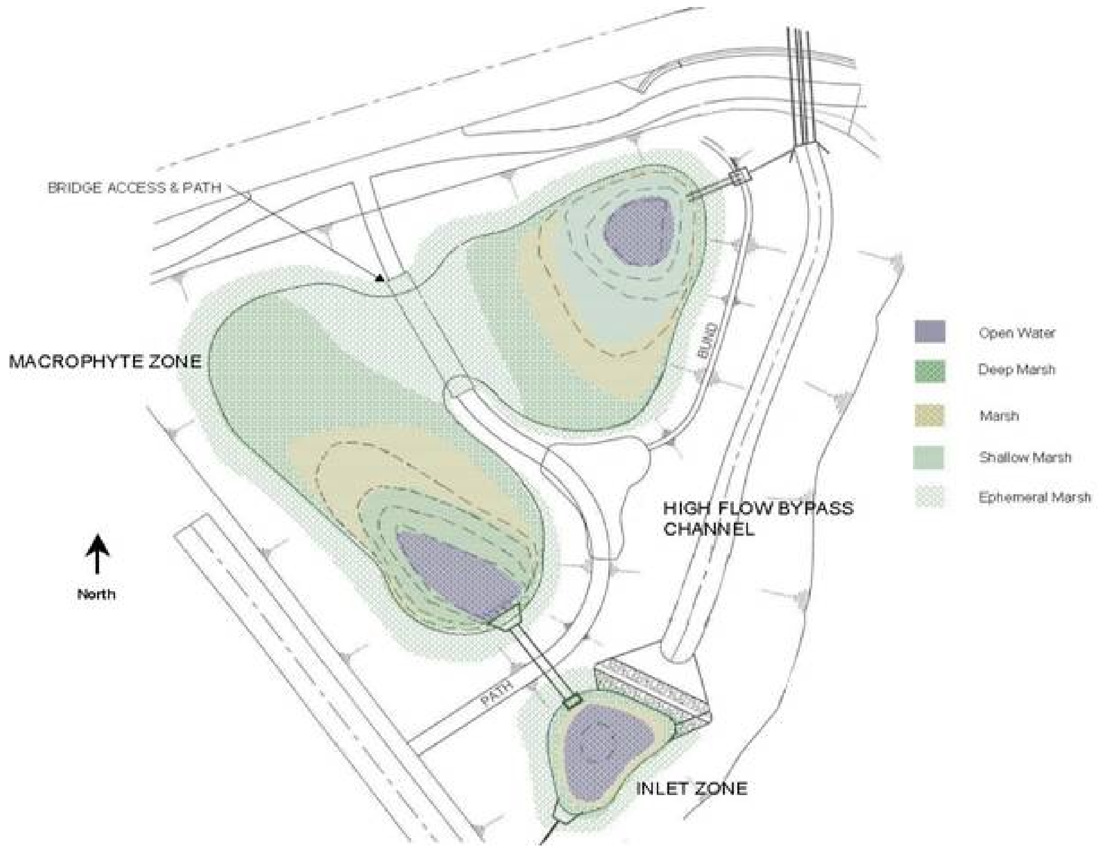


Figure 6-16: Layout of Marsh Zones

6.8.4.3 Macrophyte Zone Edge Design for Safety

The batter slopes on approaches and immediately under the permanent water level have to been configured with consideration of public safety:

- Generally, batter slopes of 1(V):8(H) from the top of the extended detention depth to 0.3 m beneath the water line has been adopted.
- The general grade through the wetland below the waterline is 1(V):8(H) or flatter.
- The batters directly adjacent and within the open water zones of the macrophyte are limited to 1(V):8(H).

Reference is made to the construction drawings in Section 6.7.12 for typical long and cross sections of the macrophyte zone.

6.8.5 Step 5: Design the Macrophyte Zone Outlet

6.8.5.1 Riser Outlet – Size and Location of Orifices

The riser outlet is designed to provide a uniform notional detention time in the macrophyte zone for the full range of possible extended detention depths. The target maximum discharge from the riser is computed as the ratio of the volume of the extended detention to the notional detention time as follows (Equation 6.1):

$$Q_{\text{max riser}} = \frac{\text{extended detention storage volume } (m^3)}{\text{notional detention time (s)}}$$

Extended detention storage	= 7000 m^2 x 0.5 m extended detention
	= 3500 m^3
Notional detention time	= 72 hrs x 3600 s/hr
Therefore, Q_{max}	= 3500/(72 x 3600) = 0.0135 m^3/s = 13.5 L/s

The placement of orifices along the riser and determining their appropriate diameters involves iterative calculation using the orifice equation (Equation 6.2) over discrete depths along the length of the riser.

Equation 6.2 is given as:

$$A_o = \frac{Q}{C_d\sqrt{2 \cdot g \cdot h}} \qquad \text{(Small orifice equation)}$$

Where 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

The size of each orifice is sized to achieve the notional detention time (72 hrs) over the full range of extended detention depths. This was performed in a spreadsheet application and the resulting riser configuration can be described as follows:

- Orifices are located at 0.125 m intervals along the length of riser at 0 m, 0.125 m, 0.250 m and 0.375 m above the permanent pool level (11.5 m AHD).
- Two orifice diameters of 30 mm and 40 mm were selected and the numbers required at each level are summarised in Table 6-3 and Figure 6-17 below.

Table 6-3: Iterative Spreadsheet Calculations for Stage-Discharge Relationship

Orifice Positions (m above 11.5m AHD)		0	0.125	0.25	0.0375		
Orifice Diameter (mm)		40	30	30	30		
Number of orifices		3	3	2	2		
Extended Det. Depth (m above 11.5m AHD)	Extended Det. Volume (m3)	Flow at given Ext. Det. Depths (L/s)				Total Flow (L/s)	Not. Detention Time (hrs)
0	0	0.00				0.00	
0.125	875	3.25	0.00			3.25	74.87
0.25	1750	4.81	1.87	0.00		6.67	72.83
0.375	2625	5.97	2.73	1.25	0.00	9.95	73.30
0.5	3500	6.94	3.38	1.82	1.25	13.39	72.61

The stage-discharge relationship of the riser is plotted in the chart below (Figure 6.14) and shows that the riser maintains a linear stage discharge relationship.

At the top of extended detention the high flow bypass is activated; therefore, the riser pipe has no role in managing of flows greater than the Q_{max} (13.5 L/s) of riser pipe. An upstand riser pipe diameter of 225 mm is selected.

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, as per Figure 6.9, with a pipe connection to the permanent pool of the macrophyte zone. The connection is via a 225 mm diameter pipe. The pit is accessed via the locked screen on top of the pit.

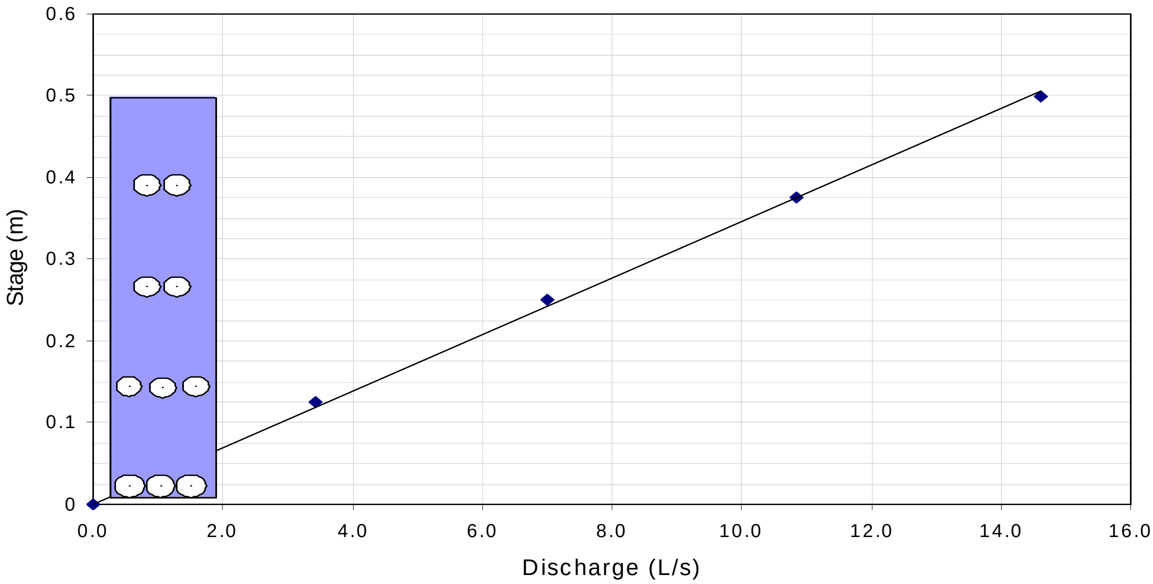


Figure 6-17: Riser Pipe Configuration Showing Discharge Stage Relationship

6.8.5.2 Maintenance Drains

To allow access for maintenance, the wetland is to be drained via a maintenance drain (i.e. pipe) that connects the low points in the macrophyte bathymetry. The drain must be sized to draw down the permanent pool of the macrophyte zone in 12 hours with allowance for manual operation (i.e. inclusion of valve).

The mean flow rate to draw down the macrophyte zone over a notional 12 hour period is as follows:

Permanent Pool Volume ~ 1750 m^3 (assuming approximate 0.25 m nominal depth)

Q = 1750/(12 x 3.6) = 40.5 L/s

The size of the of maintenance drain can be established using the Manning's equation assuming the drain/ pipe is flowing full and at 0.5 % grade:

$$Q = \frac{A \cdot R^{2/3} \cdot S^{1/2}}{n}$$

Where	A	= cross sectional area of drain (m^2)
	R	= hydraulic radius (m) (pipe area/wetted perimeter)
	S	= 0.5% (0.005m/m)
	n	= 0.012

Giving pipe diameter of 240 mm – adopt 225 mm diameter pipe meaning a notional draining time of 14 hrs.

The size of the valve can be established using the orifice equation, assuming the orifice operates under inlet control (Equation 6.2):

$$A_o = \frac{Q}{C_d\sqrt{2 \cdot g \cdot h}}$$

Where Q = 40.5 L/s (0.0405 m^3/s)

C_d = 0.6

h = 0.33 m (one third of permanent pool depth)

So A_o = 0.0104 m^2 corresponding to an orifice diameter of 115 mm – adopt 150mm

6.8.5.3 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 the larger of the discharges from the riser (13.5 L/s) or the maintenance drain (30.5 L/s). Considering the maintenance drain flow is the larger of the two flows the discharge pipe size is set to the size of the maintenance drain (225 mm pipe at 0.5% as calculated above).

Riser outlet = 225 mm diameter pipe with following orifice detail:

Level	Orifices	Orifice Diameter
11.5 m AHD	3	40 mm
11.625 m AHD	3	30 mm
11.75 m AHD	2	30 mm
11.875 m AHD	2	30 mm

Maintenance drain = 225 mm diameter pipe at 0.5 % grade

Maintenance control = 150 mm diameter valve

Discharge pipe = 225 mm diameter at 0.5 % grade

6.8.6 Step 6: Design High Flow Bypass Channel

The bypass channel accepts 'above design flow' (50 year ARI = 5.80 m/s) from the inlet zone (via the bypass weir) and conveys this flow around the macrophyte zone of the wetland. The configuration of the bypass channel can be designed using Manning's Equation:

Manning's $$Q = \frac{A \cdot R^{2/3} \cdot S^{1/2}}{n}$$

Where Q = 'above design flow' (50-year ARI = 5.80 m^3/s)

A = cross section area (m^2)

R = hydraulic radius (m)

S = channel slope (1.5%)

n = Manning's roughness factor

A turf finish is to be adopted for the bypass channel and a Manning's n of 0.03 is considered appropriate for flow depths more than double the height of the grass.

Assuming there is a 0.3 m drop from the bypass weir crest to the upstream invert of the bypass channel and 5(H):1(V) batters, the base width of the bypass channel can be established by setting the maximum flow depth in the bypass channel at 0.3 m. This ensures flow in the channel does not backwater (i.e. submerge) the bypass weir.

For base width = 16 m, Q = 5.9 m^3/s > 'Above Design flow' (5.8m^3/s)

High flow bypass channel – Base width of 16 m, batters of 5(H):1(V) and longitudinal slope of 1.5%.

6.8.7 Step 7: Verification Checks

6.8.7.1 Macrophyte Zone Resuspension Protection

A velocity check is to be conducted for when the wetland is at the top of the extended detention level and the riser is operating at design capacity. This check is to ensure velocities through the macrophyte zone ($V_{macrophyte\ zone}$) are less than 0.05 m/s to avoid potential scour of biofilms from the wetland plants (macrophytes) and resuspension of the sediments (Equation 6.3):

$$\frac{Q_{max\ riser}}{A_{section}} \langle 0.05 m/s$$

Where $Q_{max\ riser}$ = target maximum discharge (defined in equation 6.1) (m^3/s)

$A_{section}$ = wetland cross sectional area at narrowest point*, measured from top of extended detention (m^2)

* minimum wetland cross section is used when undertaking this velocity check

Wetland width (W) = 34 m (based on the 6 (L) : 1 (W) length to width ratio)

Minimum depth at top of extended detention depth is within the ephemeral marsh = 0.3 m depth

Giving $A_{section}$ = 34 m x 0.3m = 10.2 m^2

$Q_{max\ riser}$ = 13.5 L/s (0.0135 m^3/s)

Therefore, $V_{macrophyte\ zone}$ = 0.0135/0.3/34 = 0.0013 m/s < 0.05 m/s (OK)

6.8.7.2 Confirm Treatment Performance

The key functional elements of the constructed wetland developed as part of the conceptual design (i.e. area, extended detention depth) were not adjusted as part of the detailed design. Therefore, the performance check undertaken in Step 1 (see Section 6.3.1) still applies.

6.8.8 Step 8: Vegetation Specification

The vegetation specification and recommended planting density for the macrophyte zone have been adapted from Appendix A and are summarised in Table 6.5 below.

The reader is referred to Appendix A for further discussion and guidance on vegetation establishment and maintenance.

Table 6-4: Worked Example Vegetation List

Zone	Plant Species	Planting Density (plants/m^2)
Ephemeral marsh	*Carex appressa* *Isolepis nodosa*	8 8
Shallow Marsh	*Eleocharis equisetina* *Juncus usitatus*	10 10
Marsh	*Schoenoplectus mucronatus* *Baumea rubiginosa*	6 6
Deep Marsh	*Baumea articulata* *Schoenoplectus validus*	4 4

6.8.9 Step 9: Maintenance Plan

A maintenance plan for the wetland is to be prepared in accordance with Section 6.5.

6.8.10 Design Calculation Summary

The sheet below shows the results of the design calculations.

CONSTRUCTED WETLANDS DESIGN CALCULATION SUMMARY

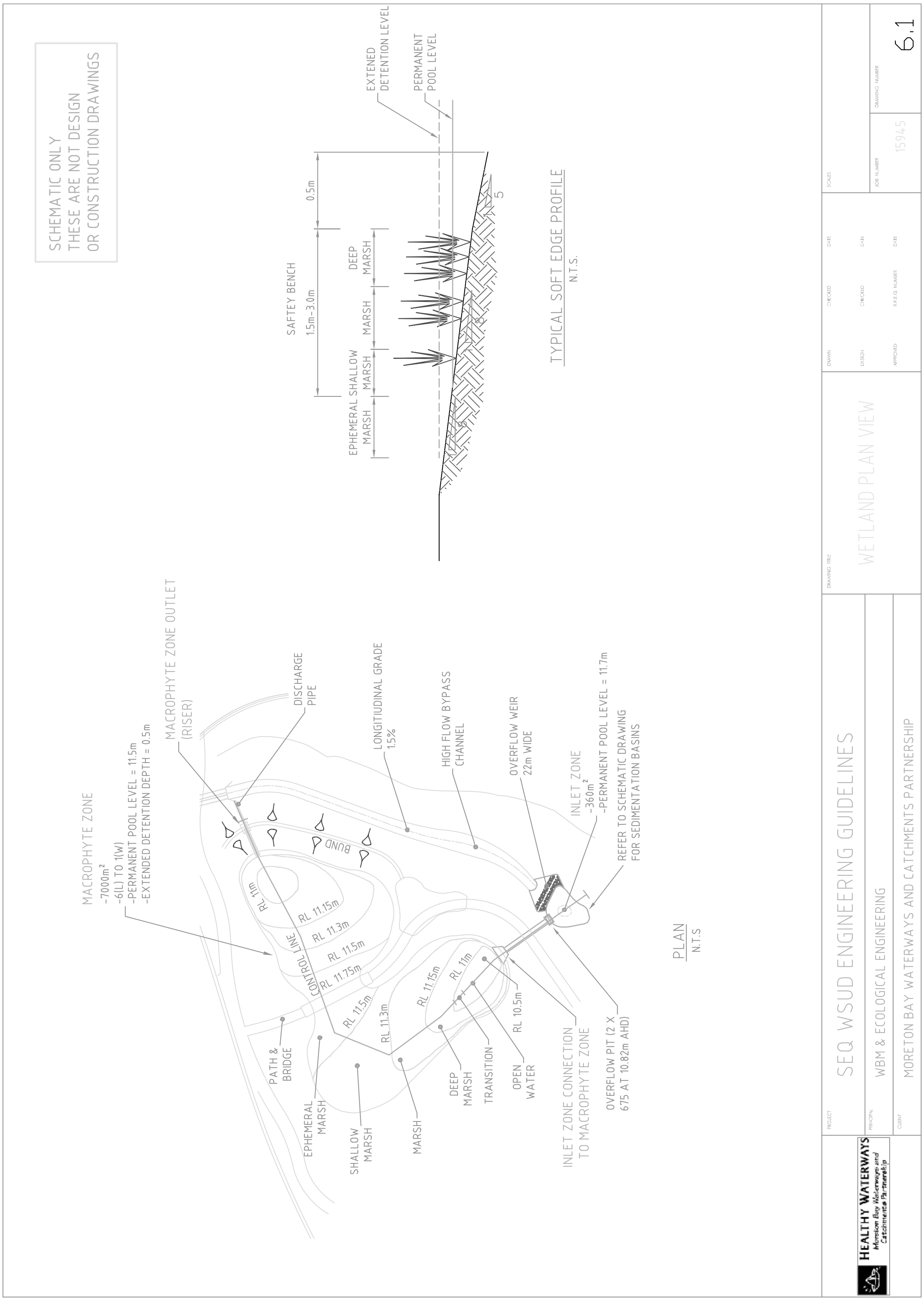
	Calculation Task		CALCULATION SUMMARY Outcome		Check
	Catchment Characteristics				
		Catchment area	10	ha	✓
		Catchment land use (i.e residential, commercial etc.)	Residential		
		Storm event entering inlet pond (minor or major)	50yr ARI		
	Conceptual Design				
		Macrophyte zone area	7000	m^2	✓
		Permanent pool level of macrophyte zone	11.5	m AHD	
		Extended detention depth (0.25-0.5m)	0.5	m	
		Notional detention time	72	hrs	
1	**Confirm Treatment Performance of Concept Design**				
		Total suspended solids (Figure 6-2)	81	% removal	✓
		Total phosphorus (Figure 6-3)	67	% removal	
		Total nitrogen (Figure 6-4)	45	% removal	
	Macrophyte Area		7000	m^2	✓
2	**Determine design flows**				
		'Design operation flow' (1 year ARI)	1	year ARI	✓
		'Above design flow' (either 2, 10, 50 or 100 year ARI)	50	year ARI	
	Time of concentration				
	(Refer to relevant local government guidelines and QUDM)		10	minutes	✓
	Identify rainfall intensities				
		'Design operation flow' - $I_{1\ year\ ARI}$	90	mm/hr	✓
		'Above design flow'- $I_{2\ year\ ARI}$ or I_{10} or $I_{100\ year\ ARI}$	227	mm/hr	
	Peak design flows				
		'Design operation flow' - 1 year ARI	1.6	m^3/s	✓
		'Above design flow' – 2, 10 or 100 year ARI	5.8	m^3/s	
3	**Design inlet zone**				
	Refer to sedimentation basin (Chapter 4) for detailed check sheet				
	Is a GPT required?				
		Suitable GPT selected and maintenance considered?	No		✓
	Inlet zone size				
		Target Sediment Size for Inlet Zone	125	μm	✓
		Capture efficiency	90	%	
		Inlet zone area (Figure 4.2 in Chapter 4)	360	m^2	
		$V_s > V_{s:5yr}$	Yes		
	Inlet zone connection to macrophyte zone				
		Overflow pit crest level	11.7	m AHD	✓
		Overflow pit dimension	4000 x 2000	L x W	
		Provision of debris trap	Yes		
		Connection pipe dimension	2 x 675	mm diam	✓
		Connection pipe invert level	10.8	m AHD	
	High flow by-pass weir				
		Weir Length	22	m	✓
		High flow by-pass weir crest level (top of extended detention)	12.0	m AHD	
4	**Designing the macrophyte zone**				
		Area of Macrophyte Zone	7000	m^2	✓
		Aspect Ratio	6:1	L:W	
		Hydraulic Efficiency	0.6-0.7		

CONSTRUCTED WETLANDS DESIGN CALCULATION SUMMARY					
			CALCULATION SUMMARY		
	Calculation Task		Outcome		Check
5	Design macrophyte zone outlet				
	Riser outlet				
		Target maximum discharge (Q_{max})	13.5	m^3/s	✓
		Uniform Detention Time Relationship for Riser	Yes		
	Maintenance Drain				
		Maintenance drainage rate (drain over 12hrs)	40.5	m^3/s	✓
		Diameter of maintenance drain pipe	225	mm	
		Diameter of maintenance drain valve	150	mm	
	Discharge Pipe				
		Diameter of discharge pipe	225	mm	✓
6	Design high flow by-pass 'channel'				
		Longitudinal slope	1.5	%	✓
		Base width	16	m	
		Batter slopes	5:1	H:V	
7	Verification checks				
		Macrophyte zone re-suspension protection			✓
		Confirm treatment performance			✓

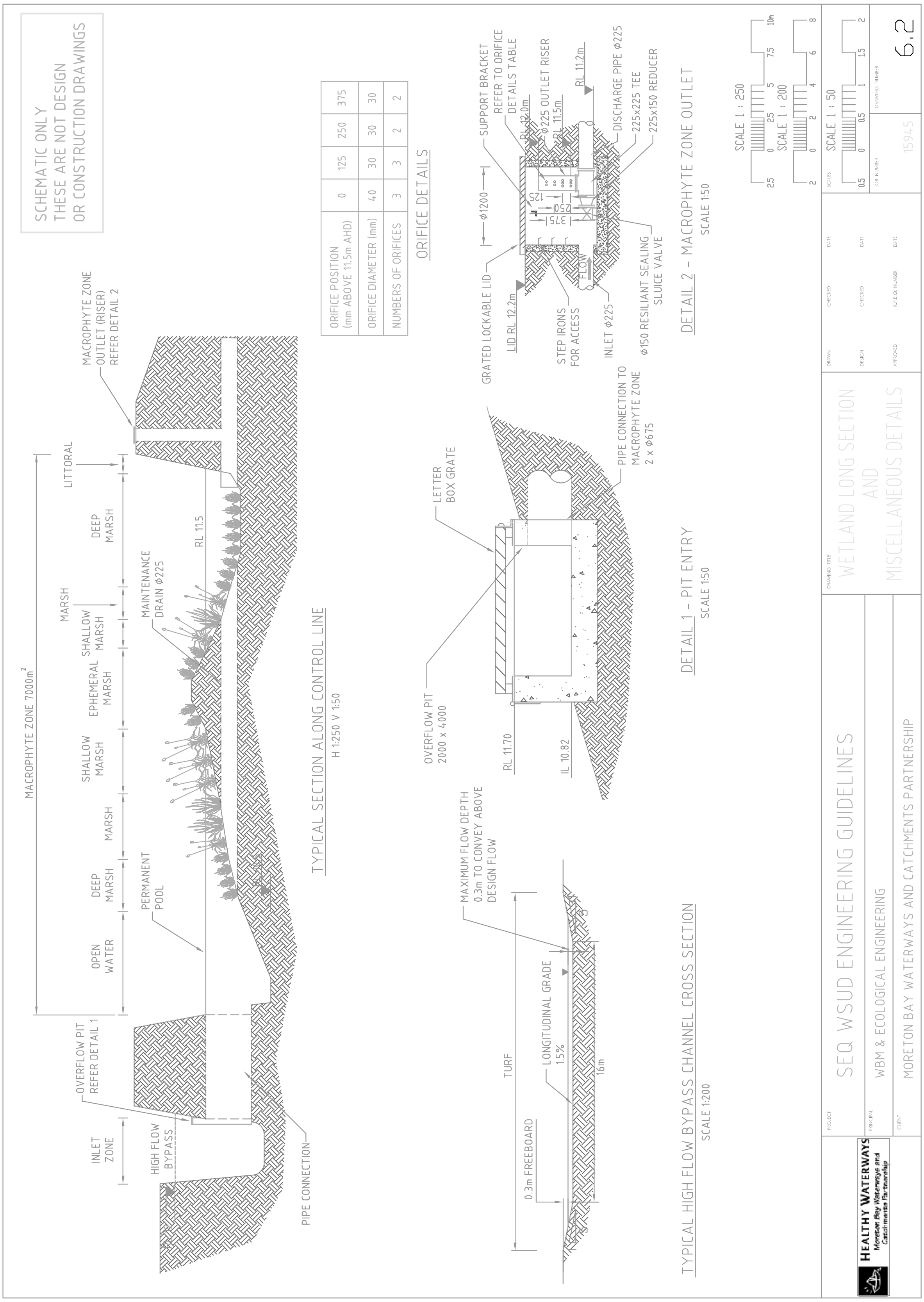
6.8.11 Worked Example Drawings

Drawings 6.1 and 6.2 illustrate the worked example wetland layout.

Drawing 6.1 Wetland Plan View



Drawing 6.2 Wetland Long Section and Miscellaneous Details



ORIFICE POSITION (mm ABOVE 11.5m AHD)	0	125	250	375
ORIFICE DIAMETER (mm)	40	30	30	30
NUMBERS OF ORIFICES	3	3	2	2

6.9 References

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Standards Australia 2003, AS 4419-2003: Soils for landscaping and garden use, Standards Australia

[2] At the time of preparation of these guidelines, QUDM was under review and a significantly revised edition is expected to be released in 2006. These guidelines refer to and use calculations specified in the existing QUDM document, however the revised version of QUDM should be used as the appropriate reference document. It should be noted by users of this guideline that the structure and content of QUDM will change, and as such, the references to calculations and/or specific sections of QUDM may no longer be correct. Users of this guideline should utilise and adopt the relevant sections and/or calculations of the revised QUDM guideline.

Chapter 7 Infiltration Measures

7.1 Introduction 7-2

7.2 Design Considerations 7-4
7.2.1 Design Objectives 7-4
7.2.2 Selecting the Type of Infiltration System 7-4
7.2.3 Design (Sizing) Methods 7-5
7.2.4 Pretreatment of Stormwater 7-6
7.2.5 Site Terrain 7-6
7.2.6 In-Situ Soils 7-7
7.2.7 Groundwater 7-8
7.2.8 Building Setbacks (Clearances) 7-8
7.2.9 Flow Management 7-8

7.3 Design Process 7-9
7.3.1 Step 1: Site and Soil Evaluation 7-10
7.3.2 Step 2: Confirm Design Objectives 7-10
7.3.3 Step 3: Select Infiltration System Type 7-10
7.3.4 Step 4: Pretreatment Design 7-10
7.3.5 Step 5: Determine Design Flows 7-10
7.3.6 Step 6: Size Infiltration System 7-11
7.3.7 Step 7: Locate Infiltration System 7-18
7.3.8 Step 8: Set Infiltration Depths (sub-surface systems only) 7-18
7.3.9 Step 9: Specify Infiltration 'Detention Volume' Elements 7-18
7.3.10 Step 10: Flow Management Design 7-18
7.3.11 Step 11: Consider Maintenance Requirements 7-21
7.3.12 Design Calculation Summary 7-21

7.4 Construction and Establishment 7-24
7.4.1 Construction and Establishment Challenges 7-24
7.4.2 Staged Construction and Establishment Method 7-24

7.5 Maintenance Requirements 7-26

7.6 Checking Tools 7-26
7.6.1 Design Assessment Checklist 7-26
7.6.2 Construction Checklist 7-26
7.6.3 Operation and Maintenance Inspection Form 7-27
7.6.4 Asset Transfer Checklist 7-27

7.7 Infiltration Measure Worked Example 7-32
7.7.1 Step 1: Site and Soil Evaluation 7-33
7.7.2 Step 2: Confirm Design Objectives 7-33
7.7.3 Step 3: Select Infiltration System Type 7-34
7.7.4 Step 4: Pretreatment Design 7-34
7.7.5 Step 5: Determine Design Flows 7-34
7.7.6 Step 6: Size Infiltration System 7-35
7.7.7 Step 7: Locate Infiltration System 7-35
7.7.8 Step 8: Set Infiltration Depths (Sub-surface Systems Only) 7-36
7.7.9 Step 9: Specify Infiltration 'Detention Volume' Elements 7-36
7.7.10 Step 10: Hydraulic Control Design 7-37
7.7.11 Design Calculation Summary 7-40
7.7.12 Worked Example Drawings 7-43

7.8 References 7-44

7.1 Introduction

Stormwater infiltration systems capture stormwater runoff and encourage infiltration into surrounding in-situ soils and underlying groundwater. This has the benefit of reducing stormwater runoff peak flows and volumes, reducing downstream flooding, managing the hydrologic regime entering downstream aquatic ecosystems and improving groundwater recharge.

The purpose of infiltration systems in a stormwater management strategy is as a conveyance measure (to capture and infiltrate flows), NOT as a stormwater treatment system. Appropriate pretreatment of stormwater entering infiltration systems is required to avoid clogging and to protect groundwater quality.

Infiltration systems generally consist of a 'detention volume' and an 'infiltration area' (or infiltration surface):

- The 'detention volume' can be located above or below ground and is designed to detain a certain volume of runoff and make it available for infiltration. When the 'detention volume' is exceeded, the system is designed to overflow to the downstream drainage systems and the receiving environment.
- The 'infiltration area' is the surface or interface between the detention volume and the in-situ soils through which the collected water is infiltrated.

The application of infiltration systems is best suited to moderately to highly permeable in-situ soils (i.e. sandy loam to sandy soils); however, infiltration systems can still be applied in locations with less permeable soils by providing larger detention volumes and infiltration areas.

As outlined in *Australian Runoff Quality* (Engineers Australia 2006) and *Practice Note 5: Infiltration Devices* (LHCCREMS 2002) there are four basic types of infiltration systems:

Leaky Well

A leaky well is typically used in small scale residential applications and consists of a vertical perforated pipe (concrete or PVC) and an open base (Figure 7-1). Pretreated stormwater enters via an inlet pipe at the top of the well and when the detention volume is full, an overflow pipe delivers excess waters to the downstream drainage system. The perforations in the open pipe and the base are covered with a geotextile (non-woven) and the pipe is surrounded by a ring of clean gravel (5 - 10 mm particle size diameter).

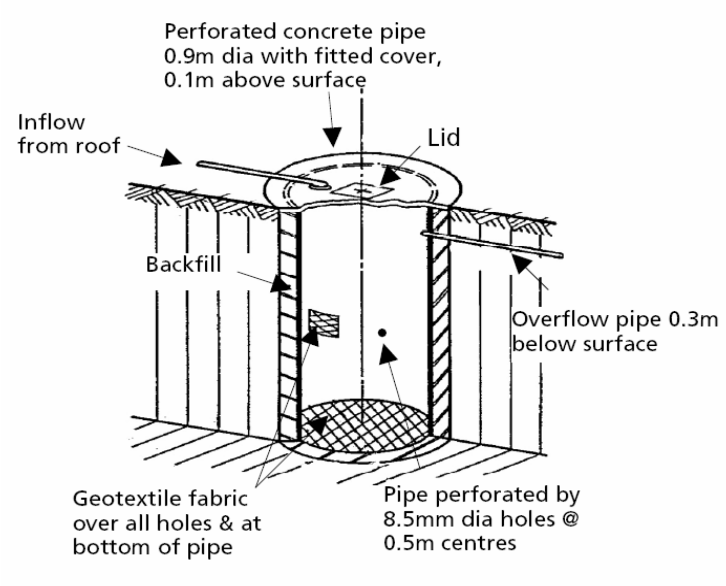


Figure 7-1: 'Leaky Well' Infiltration System (Engineers Australia 2006 and LHCCREMS 2002)

Infiltration Trench

Infiltration trenches can be applied across a range of scales and consist of a trench, typically 0.5 - 1.5 m deep, filled with gravel or modular plastic cells lined with geotextile (non-woven) and placed under 300 mm of backfill (topsoil or sandy loam). Pretreated runoff enters the trench (detention volume) either directly or via an inlet control pit, with excess waters delivered downstream via an overflow pipe. If the trench contains gravel fill then a perforated distribution pipe is incorporated into the system to ensure effective distribution of stormwater into the detention volume. A typical configuration of an infiltration trench is shown in Figure 7-2.

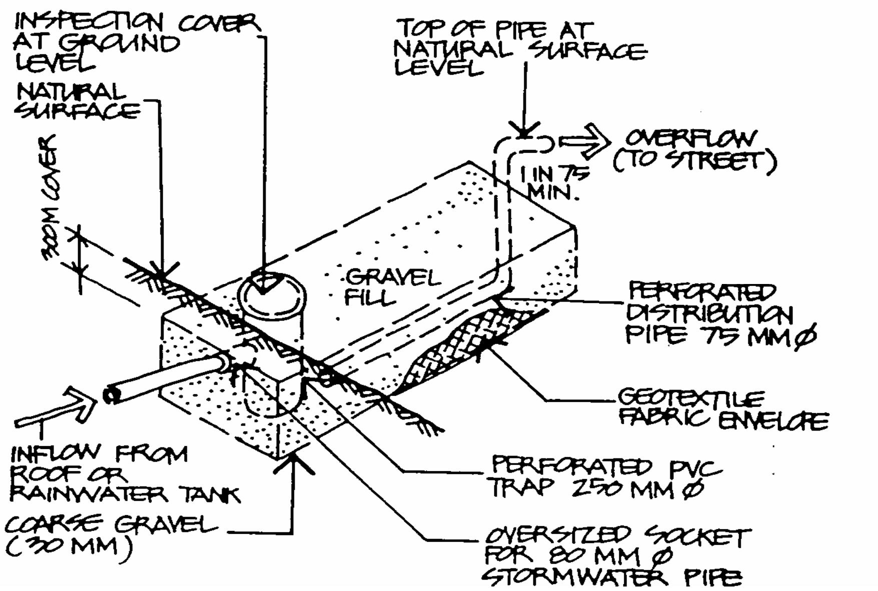


Figure 7-2: Infiltration Trench (Engineers Australia 2006)

Infiltration 'Soak-away'

Soak-aways are similar to trenches in operation but have a larger plan area, being typically rectangular, and of shallower depth (Figure 7-3). Infiltration soak-aways can be applied across a range of scales from residential allotments through to open space or parklands.

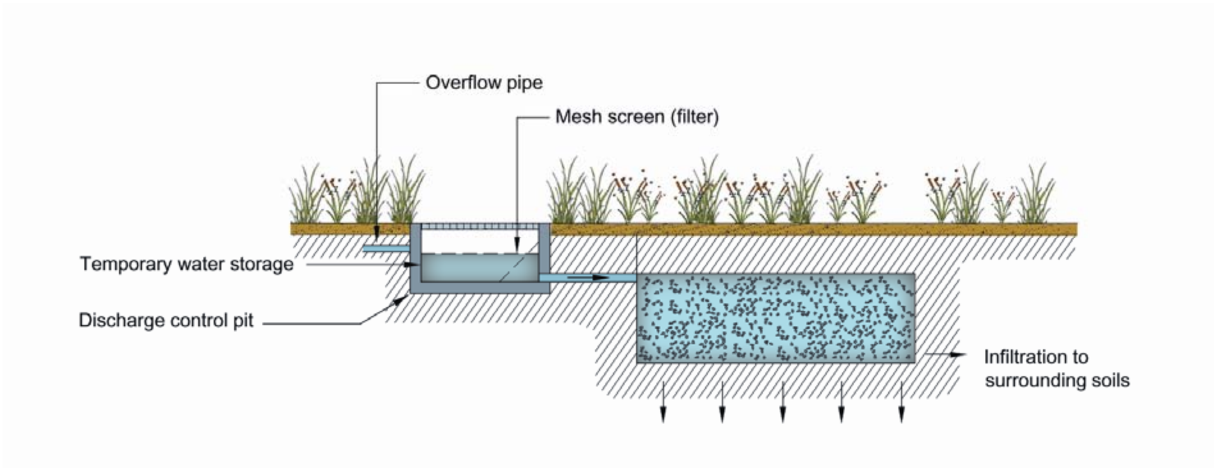


Figure 7-3: Operation of a Gravel Filled 'Trench' or Soak-away' Type Infiltration System

Infiltration Basin

Infiltration basins are typically used in larger scale applications where space is not a constraint (e.g. parklands). They consist of natural or constructed depressions designed to capture and store stormwater runoff on the surface (i.e. detention volume located above ground) prior to infiltration into the in-situ soils. A typical section through an infiltration basin is provided in Figure 7-4. Infiltration basins are best suited to sand or sandy-clay in-situ soils and can be planted out with a range of vegetation to blend into the local landscape. Pretreatment of stormwater entering infiltration basins is required with the level of pretreatment varying depending on in-situ soil type and basin vegetation. Further guidance in this regard is provided in Section 7.2.4.

Plate 7-1: Infiltration Basin

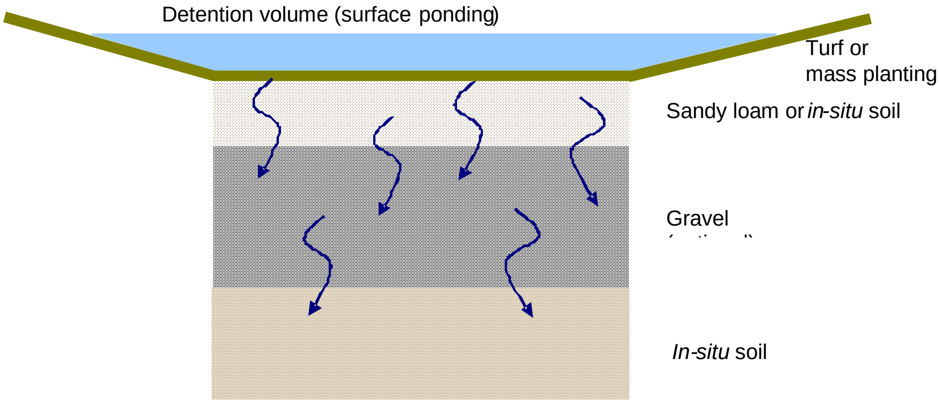


Figure 7-4: Infiltration Basin Typical Section

7.2 Design Considerations

7.2.1 Design Objectives

Infiltration systems can be designed to achieve a range of objectives including:

- Minimising the volume of stormwater runoff from a development
- Preserving predevelopment hydrology
- Capturing and infiltrating flows up to a particular design flow
- Enhancing groundwater recharge or preserving predevelopment groundwater recharge.

The design objective will vary from one location to another and will depend on site characteristics, development form and the requirements of the receiving ecosystems. It is essential that these objectives are established as part of the conceptual design process and approved by the local authority prior to commencing the engineering design.

7.2.2 Selecting the Type of Infiltration System

Selection of the type of infiltration system for a particular application must occur as part of the conceptual design process (i.e. Site Based Stormwater Management Plan) by assessing the site conditions against the relative merits of the four basic types of infiltration systems described in Section 7.1. There is a range of resources available to assist with this selection process, including *Australian Runoff Quality* (Engineers Australia 2006), *Water Sensitive Urban Design: Basic Procedures for 'Source Control' of Stormwater* (Argue 2004) and *Water Sensitive Urban Design: Technical Guidelines for Western Sydney* (UPRCT 2004).

In general, selection of the type of infiltration system is determined by the size of the contributing catchment. Table 7-1 provides guidance on selection by listing the type of infiltration systems against typical scales of application.

Table 7-1: Infiltration Types and Associated Application Scales

Infiltration Type	Allotment Scale (< 0.1 ha)*	Medium Scale (0.1 - 10 ha)*	Large Scale (> 10 ha)*
Leaky Wells	✓		
Infiltration Trenches	✓	✓	
Infiltration Soak-aways		✓	
Infiltration Basins		✓	✓

* Catchment area directing flow to the infiltration system

7.2.3 Design (Sizing) Methods

Establishing the size of an infiltration system requires consideration of the volume and frequency of runoff discharged into the infiltration system, the available 'detention volume' and the infiltration rate (product of 'infiltration area' and hydraulic conductivity of in-situ soils). The approach for establishing these design elements depends on the design objectives as outlined in Section 7.2.1. For the purposes of these guidelines, the infiltration system design objectives can be addressed by two design methods: the hydrologic effectiveness method and the design storm method. These methods are summarised in Table 7-2 and discussed in the following sections.

Table 7-2: Design (Sizing) Methods to Deliver Infiltration System Design Objectives

Infiltration Design objective	*Hydrologic Effectiveness Method	*Design Storm Method
Minimise the volume of stormwater runoff from a development	✓	
Preserve pre-development hydrology	✓	
Capture and infiltrate flows up to a particular design flow		✓
Enhance groundwater recharge or preserve pre-development groundwater recharge	✓	

*Unless otherwise approved by the Local Authority, the hydrologic effectiveness method must be used when designing infiltration systems.

7.2.3.1 Hydrologic Effectiveness Method

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 conditions, the hydrologic effectiveness of an infiltration system is determined by the combined effect of the nature/ quantity of runoff, the 'detention volume', in-situ soil hydraulic conductivity and 'infiltration area'.

The hydrologic effectiveness of an infiltration system requires long term continuous simulation which can be undertaken using the *Model for Urban Stormwater Improvement Conceptualisation* (MUSIC) (CRCCH 2005). However, in most situations, where a number of the design considerations can be fixed (i.e. frequency of runoff, depth of detention storage, saturated hydraulic conductivity), hydrologic effectiveness curves can be generated and used as the design tool for establishing the infiltration system size.

The hydrologic effectiveness curves derived for infiltration systems (with defined parameters) located in the four climatic zones of SEQ are presented in Section 7.3.6.1 and represent Step 6 in the design steps required for infiltration measures.

7.2.3.2 Design Storm Method

Where the design objective for a particular infiltration system is peak discharge attenuation or the capture and infiltration of a particular design storm event (e.g. 3 month ARI event), then the design storm approach can be adopted for sizing the infiltration system.

This method involves defining the required 'detention volume' by relating the volume of inflow and outflow for a particular design storm, and then deriving the 'infiltration area' to ensure the system empties prior to the commencement of the next storm event. Details of the approach for defining the detention volume and infiltration area are presented in Section 7.3.6.2. However, unless otherwise approved by local authority, the Hydrologic Effectiveness Method described in Section 7.3.2.1 must be used.

7.2.4 Pretreatment of Stormwater

Pretreatment of stormwater entering an infiltration system is primarily required to minimise the potential for clogging of the infiltration media and to protect groundwater quality. In line with these requirements, there are two levels of stormwater pretreatment required:

- Level 1 Pretreatment - Stormwater should be treated to remove coarse and medium sized sediments and litter to prevent blockage of the infiltration system. Level 1 Treatment applies to all four types of infiltration system.
- Level 2 Pretreatment - To protect groundwater quality, pretreatment is required to remove fine particulates and associated pollutants, such as nutrients and metals. This second level of treatment is the most stringent as any stormwater infiltrated must be of equal, or preferably superior, quality to that of the receiving groundwater to ensure the groundwater quality and values are protected. To determine an appropriate level of pretreatment, assessment of the groundwater aquifer quality, values, possible uses and suitability for recharge is required to the satisfaction of the local authority.

Level 2 pretreatment applies to leaky wells, infiltration trenches and infiltration soak-aways. It also applies to most infiltration basin applications, however, there are situations where pretreatment is not required. For example, where basins are located on sandy clay to clay soils (hydraulic conductivity <180 mm/hr) and the depth to groundwater is greater than 1.0 m, the system can be planted out with rush and reed species and allowed to function in a similar manner to a bioretention system prior to waters entering the underlying groundwater. A summary of pretreatment requirements for each of the infiltration system types is presented in Table 7-3.

Table 7-3: Pretreatment Requirements for Each Type of Infiltration System

Infiltration Type	Level 1 Pretreatment	Level 2 Pretreatment
Leaky Well	✓	✓
Infiltration Trench	✓	✓
Infiltration Soak-away	✓	✓
Infiltration Basin		
- Sandy clay to clay soils (K_{sat} < 180 mm/hr) + dense ground cover	✓	
- Sandy clay to clay soils (K_{sat} < 180 mm/hr) + turf ground cover	✓	✓
- Sandy soils (K_{sat} > 180 mm/hr)	✓	✓

Note: K_{sat} = saturated hydraulic conductivity (mm/hr) of in-situ soil (see Section 7.2.5.1)

7.2.5 Site Terrain

Infiltration into steep terrain can result in stormwater re-emerging onto the surface at some point downslope. The likelihood of this pathway for infiltrated water is dependent on the soil structure. Duplex soils and shallow soil over rock create situations where re-emergence of infiltrated water to the surface is most likely to occur. These soil conditions do 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 to ensure hazards or nuisance to downstream sites are avoided.

Additionally, the introduction of infiltration systems on steep terrain can increase the risk of slope instability. Installation of infiltration systems on slopes greater than 10 % will not be approved by the local authority unless a detailed engineering assessment has been undertaken.

7.2.6 In-Situ Soils

7.2.6.1 Hydraulic Conductivity

Hydraulic conductivity of the in-situ soil, being the rate at which water passes through a water-soil interface, influences both the suitability of infiltration systems and the size of the infiltration area. Therefore, it is essential that field measurement of hydraulic conductivity be undertaken to confirm assumptions of soil hydraulic conductivity adopted during the concept design stage (i.e. site based Stormwater Management Plan). The determination of hydraulic conductivity must be undertaken in accordance with procedures outlined in Appendix 4.1F of AS/NZS 1547:2000, which provides an estimate of saturated hydraulic conductivity (K_{sat})(i.e. the hydraulic conductivity of a soil when it is fully saturated with water). The typical ranges of saturated hydraulic conductivities for homogeneous soils are provided in Table 7-4.

Table 7-4: Typical Soil Types and Associated Saturate Hydraulic Conductivity (Engineers Australia 2006)

Soil Type	Saturated Hydraulic Conductivity	
	m/s	mm/hr
Coarse Sand	$>1 \times 10^{-4}$	>360
Sand	5×10^{-5} to 1×10^{-4}	180 – 360
Sandy Loam	1×10^{-5} to 5×10^{-5}	36 to 180
Sandy Clay	1×10^{-6} to 1×10^{-5}	3.6 to 36
Medium clay	1×10^{-7} to 1×10^{-6}	0.36 – 3.6
Heavy Clay	1×10^{-7}	0.0 to 0.36

When assessing the appropriateness of infiltration systems and the type of in-situ soils, the following issues must be considered:

Soils with a saturated hydraulic conductivity of 3.6 mm/hr to 360 mm/hr are preferred for infiltration application.

Infiltration systems will not be accepted by the local authority where the in-situ soils are very heavy clays (i.e. < 0.36 mm/hr).

Soils with a low hydraulic conductivity (0.36 - 3.6 mm/hr) do not necessarily preclude the use of infiltration systems even though the required infiltration/ storage area may become prohibitively large. However, soils with lower hydraulic conductivities will be more susceptible to clogging and will therefore require enhanced pretreatment.

7.2.6.2 Soil Salinity

Infiltration systems must be avoided in areas with poor soil conditions, in particular sodic/ saline and dispersive soils, and shallow saline groundwater. If the 'Site and Soil Evaluation' (refer to Section 7.3.1) identifies poor soil conditions, then the local authority will not approve the use of infiltration systems.

7.2.6.3 Impermeable Subsoil, Rock and Shale

Infiltration systems must not be placed in locations where soils are underlain by rock or a soil layer with little or no permeability (i.e. $K_{sat} < 0.36$ mm/hr). In locations where fractured or weathered rock prevail, the use of infiltration systems may be approved by the local authority provided detailed engineering testing has been carried out to ensure the rock will accept infiltration.

7.2.7 Groundwater

7.2.7.1 Groundwater Quality

As outlined in Section 7.2.4, the suitability of infiltrating stormwater and the necessary pretreatment requires assessment of the groundwater quality. The principle legislation governing the management of groundwater quality is the *Environmental Protection (Water) Policy 1997* and the overriding consideration is that there should be no deterioration in groundwater quality. This means the stormwater being infiltrated must be of equal or preferably superior quality to that of the receiving groundwater in order to ensure the groundwater quality and values are protected. To determine an appropriate level of pretreatment for stormwater, assessment of the groundwater aquifer quality, values, possible uses and suitability for recharge is required and must be approved by the local authority.

7.2.7.2 Groundwater Table

A second groundwater related design consideration is to ensure that the base of an infiltration system is always above the groundwater table. It is generally recommended that the base of the infiltration system be a minimum of 1.0 m above the seasonal high water table.

If a shallow groundwater table is likely to be encountered, investigation of the seasonal variation of groundwater levels is essential. This should include an assessment of potential groundwater mounding (i.e. localised raising of the water table in the immediate vicinity of the infiltration system) that in shallow groundwater areas could cause problems with nearby structures.

7.2.8 Building Setbacks (Clearances)

Infiltration systems should not be placed near building footings to avoid the influence of continually wet sub-surface or greatly varying soil moisture content on the structural integrity. *Australian Runoff Quality* (Engineers Australia 2006) recommends minimum distances from structures and property boundaries (to protect possible future buildings in neighbouring properties) for different soil types. These values are shown in Table 7-5.

Table 7-5: Minimum Setback Distances (adapted from Engineers Australia 2006)

Soil Type	Saturated Hydraulic Conductivity (mm/hr)	Minimum distance from structures and property boundaries
Sands	>180	1.0 m
Sandy Loam	36 to 180	2.0 m
Sandy Clay	3.6 to 36	4.0 m
Medium to Heavy Clay	0.0 to 3.6	5.0 m

7.2.9 Flow Management

The following issues should be considered when designing the flow control structures within infiltration systems:

- For large scale systems (i.e. infiltration basins), the surface of the 'infiltration area' must be flat or as close to this as possible to ensure uniform distribution of flow and to prevent hydraulic overloading on a small portion of the 'infiltration area'.
- For gravel filled infiltration systems, flow should be delivered to the 'detention volume' via a perforated pipe(s) network that is located and sized to convey the design flow into the infiltration systems and allow distribution of flows across the entire infiltration area.
- Where possible, 'above design' flows will bypass the infiltration systems. This can be achieved in a number of ways. For smaller applications, an overflow pipe or pit, which is connected to the downstream drainage system, can be used. For larger applications, a discharge control pit can be located upstream of the infiltration system. This will function much like the inlet zone of a constructed wetland to regulate flows (1 year ARI) into the infiltration systems and bypass above design flows (> 1 year ARI).

7.3 Design Process

The following sections detail the design steps required for infiltration measures. Key design steps are:

1. Site and soil evaluation

↓

2. Confirm design objectives

↓

3. Select infiltration system type

↓

4. Determine pre-treatment and design measures

↓

5. Determine design flows

↓

6. Size infiltration system
a. Hydrologic effectiveness method
b. Design storm method

↓

7. Locate infiltration sysytem

↓

8. Set infiltration depths (sub-surface systems only)

↓

9. Specify infiltration 'detention volume' elements

↓

10. Flow management design
a. Pipe flows (inflow pipe and overflow pipe)
b. Perforated inflow pipes
c. Overflow pit
d. Overflow weir

↓

11. Consider maintenance requirements

7.3.1 Step 1: Site and Soil Evaluation

As outlined in Section 7.2, there are a range of site and soil conditions which influence infiltration system design. To define the site's capability to infiltrate stormwater, a 'Site and Soil Evaluation' must be undertaken in accordance with AS/NZS 1547:2000 Clause 4.1.3. The evaluation should provide the following:

- Soil type
- Hydraulic conductivity (must be measured in accordance with AS/NZS 1547:2000 Appendix 4.1F)
- Presence of soil salinity (where applicable)
- Presence of rock shale
- Slope of terrain (%)
- Groundwater details (depth, quality and values).

7.3.2 Step 2: Confirm Design Objectives

This step involves confirming the design objectives, defined as part of the conceptual design, to ensure the correct infiltration system design method is selected (refer to Table 7-2).

7.3.3 Step 3: Select Infiltration System Type

This step involves selecting the type of infiltration system by assessing the site conditions against the relative merits of the four infiltration systems described in Section 7.1. In general, the scale of application dictates selection of the infiltration system. Table 7-1 provides guidance in this regard.

For further guidance in selecting infiltration systems, designer should refer to *Australian Runoff Quality* (Engineers Australia 2006), *Water Sensitive Urban Design: Basic Procedures for 'Source Control' of Stormwater* (Argue 2004) and the *Water Sensitive Urban Design: Technical Guidelines for Western Sydney* (UPRCT 2004).

7.3.4 Step 4: Pretreatment Design

As outlined in Section 7.2.4 and Table 7-3, both Level 1 Pretreatment (minimising risk of clogging) and Level 2 Pretreatment (groundwater protection) are required for all infiltration systems except for specific infiltration basin applications. To determine Level 2 requirements, an assessment of the groundwater must be undertaken to define existing water quality, potential uses (current and future) and suitability for recharge.

Pretreatment measures include the provision of leaf and roof litter guards along the roof gutter, sediment basins, vegetated swales, bioretention systems or constructed wetland as outlined in the other chapters of this guideline.

7.3.5 Step 5: Determine Design Flows

7.3.5.1 Design Flows

To configure the inflow system and high flow bypass elements of the infiltration system the following design flows generally apply:

- 'Design operation flow' for sizing the inlet to the infiltration system. This may vary depending on the particular situation but will typically correspond to one of the following:
  - 1 year ARI – for situations where a discharge control pit is used to regulate flows into the infiltration system and bypass larger flows
  - 2 - 10 year ARI (minor design flow) – for situations where the minor drainage system is directed to the infiltration system.
- 'Above design flow' for design of the high flow bypass around the infiltration system. The discharge capacity for the bypass system may vary depending on the particular situation but will typically correspond to one of the following:
  - 2 - 10 year ARI (minor design flow) – for situations where only the minor drainage system is directed to the infiltration system.
  - 50 - 100 year ARI (major design flow) – for situations where both the minor and major drainage system discharge to the infiltration system.

7.3.5.2 Design Flow Estimation

A range of hydrologic methods can be applied to estimate design flows. If typical catchment areas are relatively small, the Rational Method design procedure is considered suitable. However, if the infiltration system is to form part of a detention basin or if the catchment area to the system is large (> 50 ha) then a full flood routing computation method should be used to estimate design flows.

7.3.6 Step 6: Size Infiltration System

As outlined in Section 7.2.3, there are two design methods available for establishing the size of the detention volume and infiltration area of infiltration systems: the hydrologic effectiveness method and the design storm method. Unless otherwise approved by the local authority, the hydrologic effectiveness method must be used when designing infiltration systems.

7.3.6.1 Hydrologic Effectiveness Method

Figure 7-5 to Figure 7-12 below show the relationship between the hydrologic effectiveness, infiltration area and detention storage for a range of soil hydraulic conductivities, detention storage depths and detention storage porosities. The curves are based on the performance of an infiltration system in a typical residential suburb of Brisbane (i.e. with an annual volumetric runoff coefficient (AVRC) of 0.38). The curves were derived using the *Model for Urban Stormwater Improvement Conceptualisation* (MUSIC)(CRCCH 2005).

The curves in Figure 7-5 to Figure 7-12 are generally applicable to infiltration measure applications within residential, industrial and commercial land uses. Curves are provided for four rainfall station locations selected as being broadly representative of the spatial and temporal climatic variation across South East Queensland. If the configuration of the infiltration measure concept design is significantly different to that described below then the curves in Figure 7-5 to Figure 7-12 may not provide an accurate indication of performance. In these cases, the detailed designer should use MUSIC to size the infiltration system.

The curves were derived (conservatively) assuming the systems have the following characteristics:

- varying in-situ soil hydraulic conductivity
- ‘infiltration area’ = ‘detention volume’ area
- ‘detention volume’ depth of 1.0 m and porosity of 1.0 (i.e. an open detention volume with no fill media)
- ‘detention volume’ depth of 1.0 m and porosity of 0.35 (gravel filled detention volume)

These curves can be used to establish the size of both the ‘detention volume’ and ‘infiltration area’ of the infiltration systems to achieve a particular hydrologic effectiveness. The designer is required to select the relevant hydrologic effectiveness curve by establishing the likely configuration and form of the infiltration system, namely whether it will be an open void detention volume (porosity = 1.0) or gravel filled (porosity = 0.35).

Linear interpolation between the curves may be used to estimate the infiltration area required for systems with hydraulic conductivities not shown on the charts. However, it should be noted that the relationship between the curves is not linear and as a result, these interpolations do not provide an exact representation of the size of infiltration area (as a % of catchment area). Designers must be careful not to under size infiltration areas through this process.

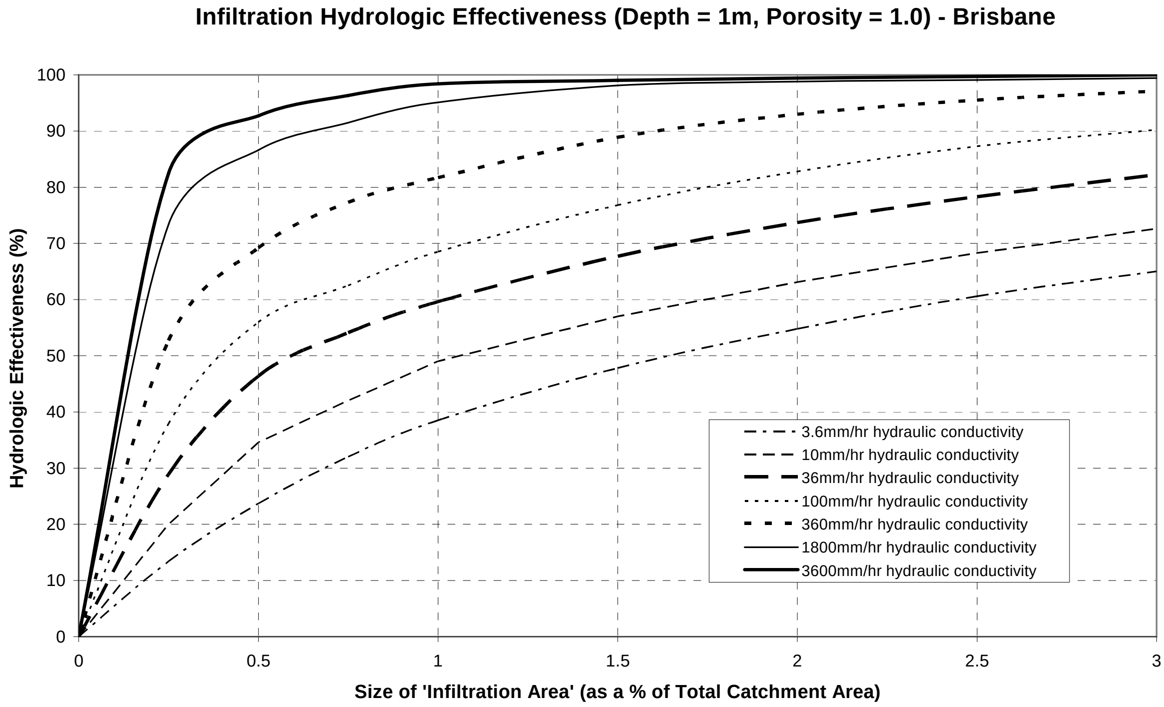


Figure 7-5: Hydrologic Effectiveness of 'Detention Storage' for Infiltration Systems for Greater Brisbane (Depth = 1 m and Porosity = 1.0)

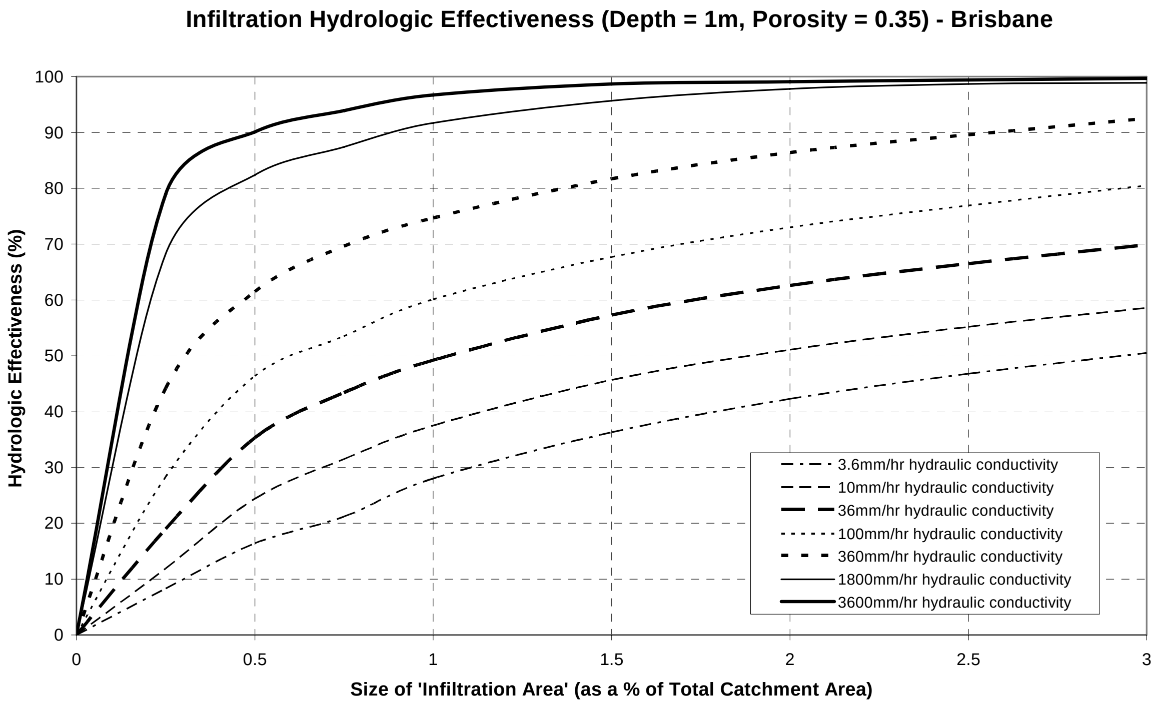


Figure 7-6: Hydrologic Effectiveness of 'Detention Storage' for Infiltration Systems for Greater Brisbane (Depth = 1 m and Porosity = 0.35)

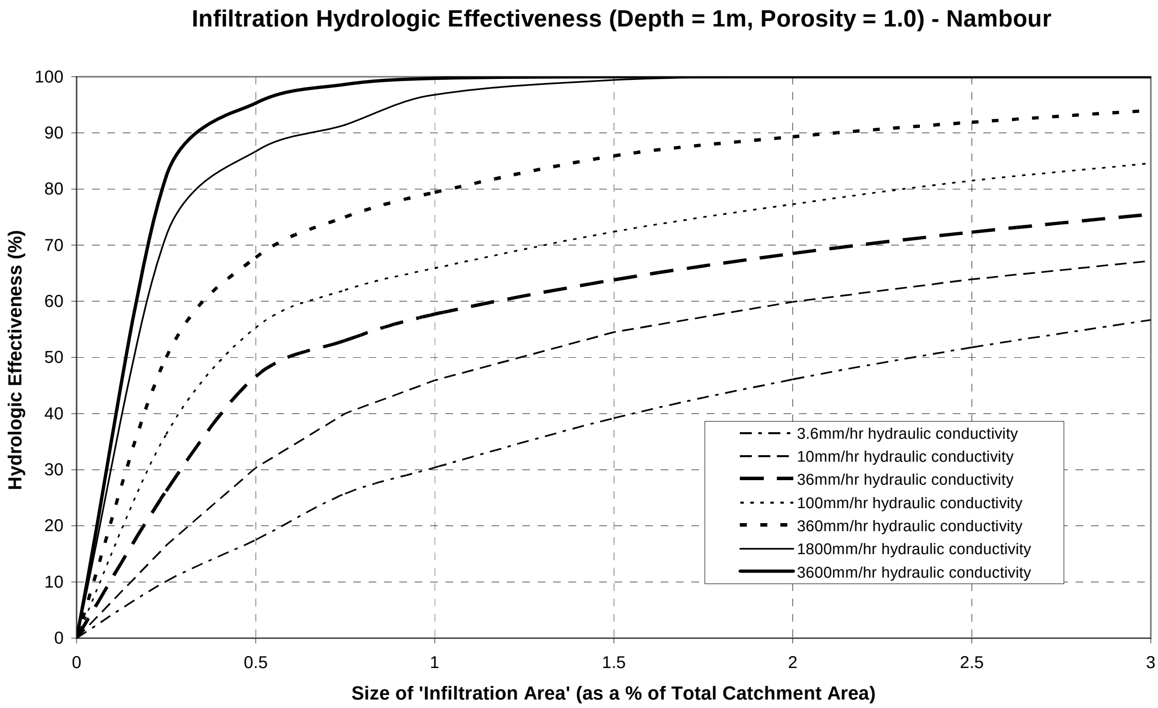


Figure 7-7: Hydrologic Effectiveness of 'Detention Storage' for Infiltration Systems for the North Coast (Depth = 1 m and Porosity = 1.0)

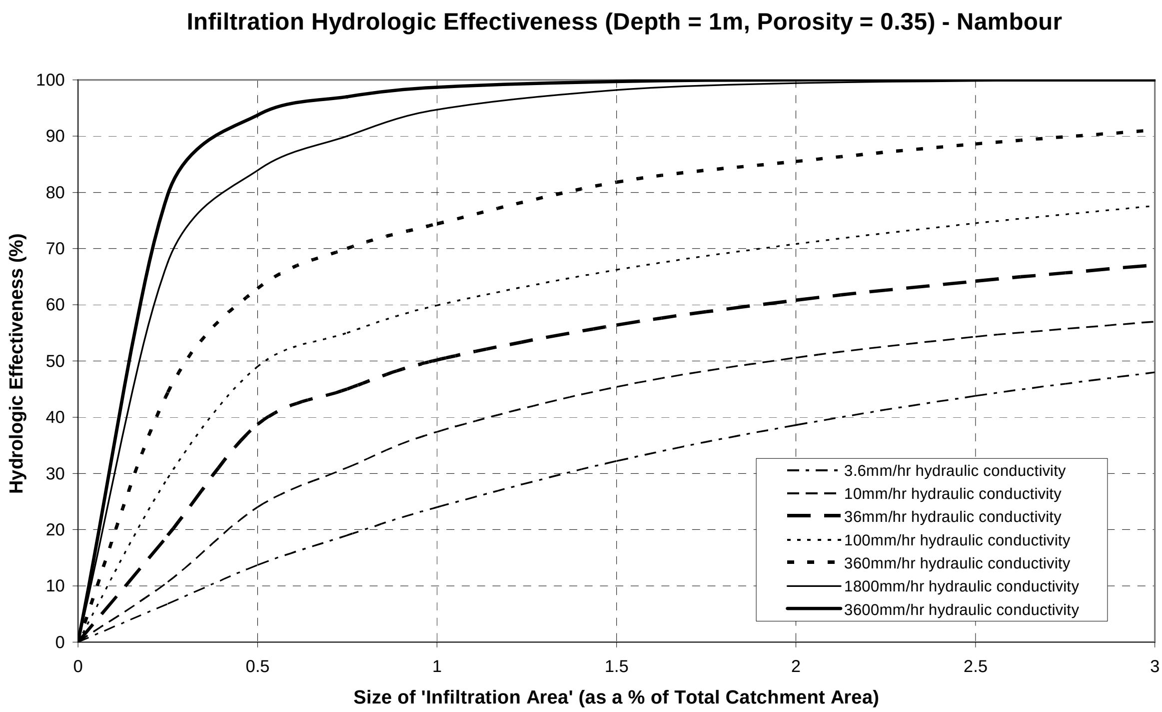


Figure 7-8: Hydrologic Effectiveness of 'Detention Storage' for Infiltration Systems the North Coast (Depth = 1 m and Porosity = 0.35)

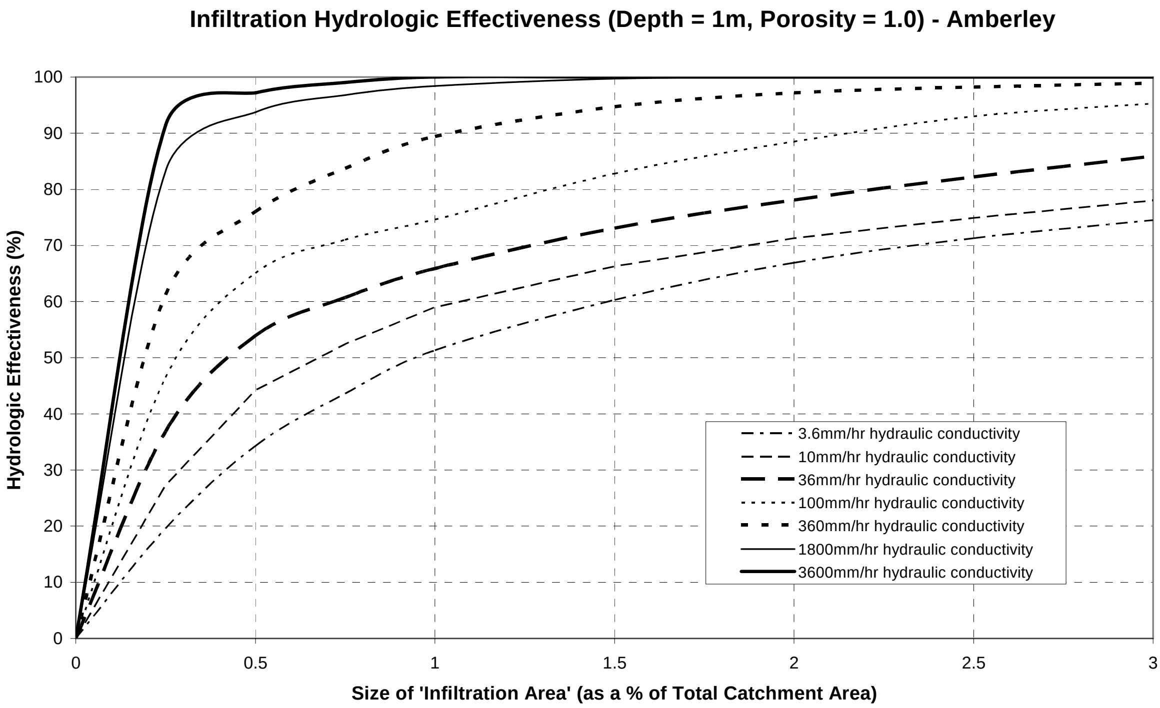


Figure 7-9: Hydrologic Effectiveness of 'Detention Storage' for Infiltration Systems for the Western Region (Depth = 1 m and Porosity = 1.0)

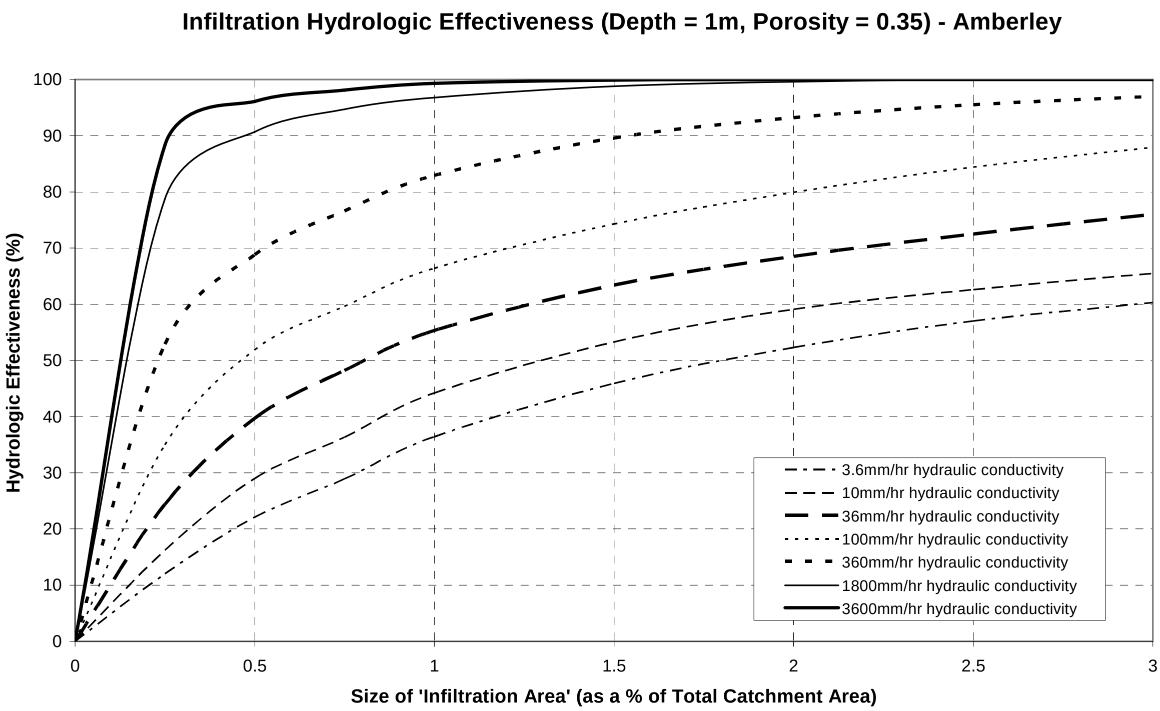


Figure 7-10: Hydrologic Effectiveness of 'Detention Storage' for Infiltration Systems for the Western Region (Depth = 1 m and Porosity = 0.35)

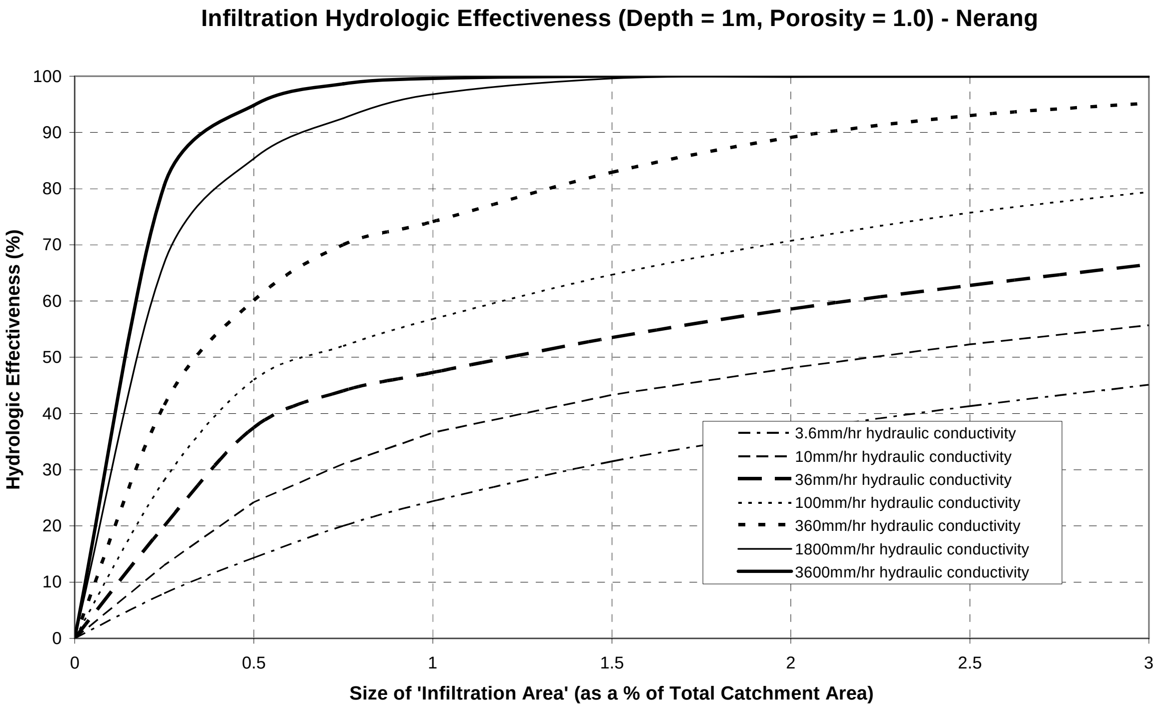


Figure 7-11: Hydrologic Effectiveness of 'Detention Storage' for Infiltration Systems for the South Coast (Depth = 1 m and Porosity = 1.0)

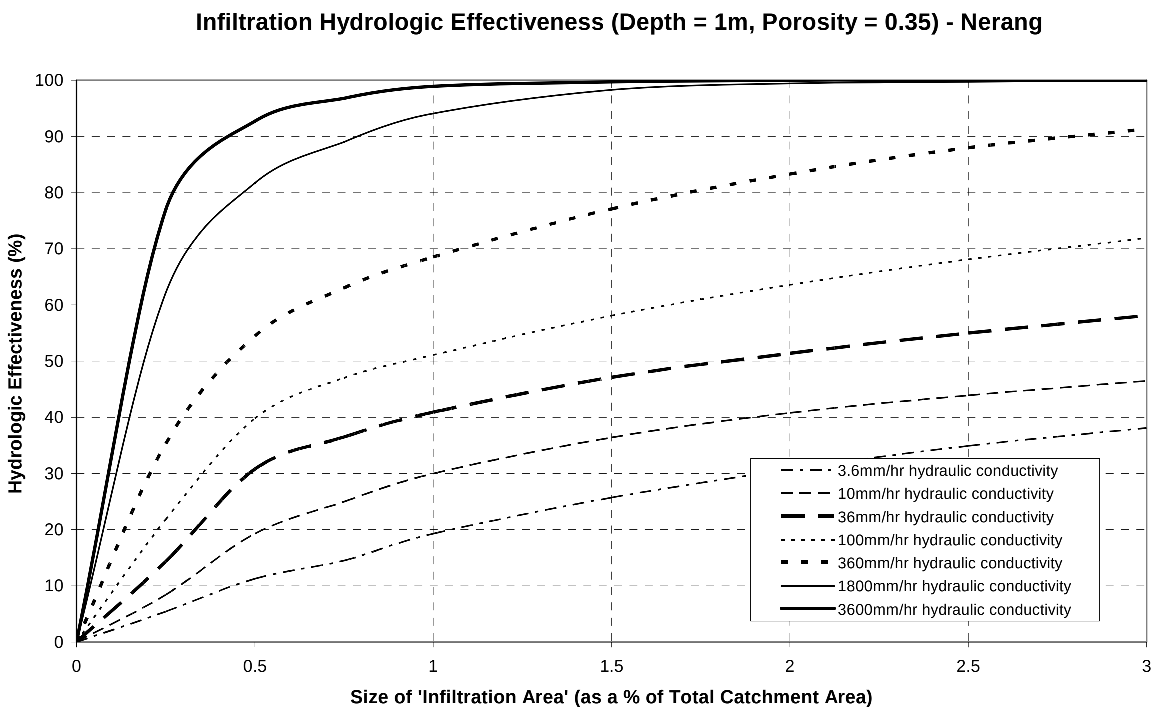


Figure 7-12: Hydrologic Effectiveness of 'Detention Storage' for Infiltration Systems for the South Coast (Depth = 1 m and Porosity = 0.35)

7.3.6.2 Design Storm Approach

Where the design objective for a particular infiltration system is peak discharge attenuation or the capture and infiltration of a particular design storm event, then the design storm approach may be adopted for sizing the infiltration system. Use of the design storm approach must be approved by the local authority for sizing infiltration systems.

Design Storm Selection (Q_{des})

The first step in the design storm approach to sizing the infiltration system is selecting the design storm for capture and infiltration. This must occur in consultation with the local authority and will generally relate to 3 month ARI and 1 year ARI design storms.

Detention Volume

The required 'detention volume' of an infiltration system is defined by the difference in inflow and outflow (or infiltrated) volumes for the duration of a storm.

The inflow volume (V_i) is determined, in accordance with Section 6 (Detention Basins) of QUDM, as the product of the design storm flow and the storm duration:

$$V_i = Q_{des} \cdot D \qquad \text{Equation 7.1}$$

Where V_i = inflow volume (for storm duration D) (m^3)

Q_{des} = design storm flow for sizing as outlined in Section 7.3.5

(Rational Method, $Q = CIA/360$ (m^3/s)

D = storm duration (hrs x 3600 s/hr)

As outlined in Section 7.3.5.2, if the infiltration system is to form part of a detention basin or if the catchment area to the system is large (> 50 ha) then a full flood routing computation method should be used to estimate design flows.

Outflow from the infiltration system is via the base and sides of the infiltration media and is dependent on the area and depth of the structure. 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:

$$V_o = \left[A_{inf} + \left(P \cdot \frac{d}{2}\right)\right] \cdot \frac{U \cdot K_{sat} \cdot D}{1000} \qquad \text{Equation 7.2}$$

Where V_o = outflow volume (for storm duration D) (m^3)

K_{sat} = saturated hydraulic conductivity (mm/hr) as provided in Step 1.

A_{inf} = infiltration area (m^2)

P = perimeter length of the infiltration area (m)

d = depth of the infiltration system (m)

U = soil hydraulic conductivity moderating factor (see Table 7.5)

D = storm duration (hrs)

Thus, the required detention volume (V_d) of an infiltration system can be computed as follows:

$$V_d = \frac{V_i - V_o}{p} \qquad \text{Equation 7.3}$$

Where	V_d	= required detention volume (m^3)
	V_i	= inflow volume (m^3)
	V_o	= outflow volume (m^3)
	p	= porosity (void = 1, gravel = 0.35)

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 that which results in the highest required storage. A spreadsheet application (using equations 7.1 to 7.3) is the most convenient way of doing this. It is important to note that some storm events result in double peaks in the hyetograph for the particular storm and these may affect the size of detention storage required.

Soil Hydraulic Conductivity Moderating Factor

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 suggests that field tests of 'point' soil hydraulic conductivity (as defined by Step 1) can often under estimate the areal hydraulic conductivity of clay soils and over estimate in sandy soils. As a result, *Australian Runoff Quality* (Engineers Australia 2005) recommends that moderation factors for hydraulic conductivities determined from field test be applied as shown in Table 7-6.

Table 7-6: Moderation Factors to Convert Point to Areal Conductivities (after Engineers Australia 2005)

Soil Type	Moderation Factor (*U*) (to convert "point" K_{sat} to areal K_{sat})
Sandy soil	0.5
Sandy clay	1.0
Medium and Heavy Clay	2.0

Emptying Time

Emptying time is defined as the time taken to fully empty a detention volume associated with an infiltration system following the cessation of rainfall. This is an important design consideration as the computation procedure associated with Equation 7.3 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. For detention basins (surface systems) the emptying time must be limited to 72 hours to reduce the risk of mosquito breeding.

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):

$$t_e = \frac{1000 \cdot V_d \cdot P \cdot p}{A_{inf} \cdot K_{sat}} \qquad \text{Equation 7.4}$$

Where	t_e	= emptying time (hours)
	V_d	= detention volume (m^3)
	P	= perimeter length of the infiltration area (m)
	A_{inf}	= infiltration area (m^2)
	K_{sat}	= saturated hydraulic conductivity (mm/hr) as provided in Step 1
	p	= porosity (void = 1, gravel = 0.35)

7.3.7 Step 7: Locate Infiltration System

This step involves locating the infiltration system in accordance with the requirement set out in Section 7.2.8 and Table 7-5 to minimise the risk of damage to structures from wetting and drying of soils (i.e. swelling and shrinking of soils and slope stability).

7.3.8 Step 8: Set Infiltration Depths (sub-surface systems only)

For sub-surface infiltration systems, selection of the optimum depth requires consideration of the seasonal high water table and the appropriate cover to the surface.

- Seasonal groundwater table - As outlined in Section 7.2.6.2, it is generally recommended that the base of the infiltration system be a minimum of 1 m above the seasonal high water table.
- Cover (i.e. depth of soil above top of infiltration system) – Minimum cover of 0.3 m. For systems created using modular plastic cell storage units, an engineering assessment is required.

7.3.9 Step 9: Specify Infiltration 'Detention Volume' Elements

The following design and specification requirements must be documented as part of the design process for 'leaky wells', infiltration trenches and 'soak-aways'.

7.3.9.1 Gravel

Where the infiltration 'detention volume' is created through the use of a gravel-filled trench then the gravel must be clean (free of fines) stone/ gravel with a uniform size of between 25 - 100 mm diameter.

7.3.9.2 Modular Plastic Cells

Where the infiltration detention volume is created through the use of modular plastic cells (similar to a milk crate), the design must be accompanied by an engineering assessment of the plastic cells and their appropriateness considering the loading above the infiltration system. A minimum 150 mm thick layer of coarse sand or fine gravel must underlie the base of the plastic cells.

7.3.9.3 Geofabric

Geofabric must be installed along the side walls and through the base of the infiltration detention volume to prevent the migration of in-situ soils into the system. For infiltration system application, Council will only accept the use of non-woven geofabric with a minimum perforation or mesh of 0.25 mm.

7.3.9.4 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 for infiltration rates to be confirmed over time.

7.3.10 Step 10: Flow Management Design

The design of the hydraulic control for infiltration systems varies for the different types of systems. For smaller applications, all pretreated flows will directly enter the infiltration system and an overflow pipe or pit will be used to convey excess flow to the downstream drainage system. For larger applications, a discharge control pit will be located upstream of the infiltration systems to function similar to the inlet zone of a constructed wetland to regulate flows (1 year ARI) into the infiltration systems and bypass above design flows (> 1 year ARI). Table 7-7 summarises the typical hydraulic control requirements for the different types of infiltration system.

Table 7-7: Typical Hydraulic Control Requirements for Infiltration Systems

Infiltration Type	Inflow		Overflow/ Bypass	
	Direct inflow	Discharge control pit	Overflow pipe/ pit	Discharge control pit
Leaky Wells	✓		✓	
Infiltration Trenches	✓		✓	
Infiltration Soak-aways		✓		✓
Infiltration Basins	✓	✓	✓	✓

Note: For gravel filled infiltration systems, flow should be delivered to the 'detention volume' via a perforated pipe network.

The hydraulic control measures described in Table 7-7 are designed using the following techniques.

7.3.10.1 Pipe Flows (Inflow Pipe and Overflow Pipe)

Pipe flows are to be calculated in accordance with the QUDM and the relevant local authority guidelines which use standard pipe equations that account for energy losses associated with inlet and outlet conditions and friction losses within the pipe. For most applications, the pipe or culvert will operate under outlet control with the inlet and outlet of the pipe/ culvert being fully submerged. With relatively short pipe connections, friction losses are typically small and can be computed using Manning's equation. The total energy (head) loss (ΔH) of the connection is largely determined by the inlet and outlet conditions and the total losses can be computed using the expression as provided in QUDM:

$$\Delta H = h_f + h_s \qquad \text{Equation 7.5}$$

where

h_f = $S_f \cdot L$ = head loss in pipe due to friction (m)

h_s = $(K_{in} + K_{out}) \cdot V^2/2g$ = head loss at entry and exit (m)

S_f = friction slope which is computed from Manning's Equation (m/m)

L = is the length pipe (m)

$K_{in} + K_{out}$ = the head loss coefficients for the inlet and outlet conditions (typically, and conservatively, assumed to be 0.5 and 1.0 respectively)

V = velocity on flow in pipe (m/s)

g = gravity (9.79 m/s^2)

7.3.10.2 Perforated Inflow Pipes

To ensure the perforated inflow pipes to gravel filled infiltration systems have sufficient capacity to convey the 'design operation flow' (Section 7.3.4.1) and distribute this flow into the infiltration system, there are two design checks required:

- Ensure the pipe itself has capacity to convey the 'design operation flow'
- Ensure the perforations are adequate to pass the 'design operation flow'.

It is recommended that the maximum spacing of the perforated pipes is 3 m (centres) and that the minimum grade is 0.5 % from the inflow point. The inflow pipes should be extended to the surface of the infiltration system to allow inspection and maintenance when required. The base of the infiltration system must remain flat.

Perforated Pipe Conveyance

To confirm the capacity of the perforated pipes to convey the 'design operation flow', Manning's equation can be used (which assumes pipe full flow but not under pressure). When completing this calculation it should be noted that installing multiple perforated pipes in parallel is a means of increasing the capacity of the perforated pipe system.

Perforated Pipe Slot Conveyance

The capacity of the slots in the perforated pipe needs to be greater than the maximum infiltration rate to ensure the slots does not become the hydraulic 'control' in the infiltration system (i.e. to ensure the in-situ soils and 'detention volume' set the hydraulic behaviour rather than the slots in the perforated pipe). To do this, orifice flow can be assumed to occur through the slots and the sharp edged orifice equation used to calculate the flow through the slots for the full length of perforated pipe. Firstly, the number and size of perforations needs to be determined (typically from manufacturer's specifications) and used to estimate the flow rate out of the pipes, with the driving head being the difference between the overflow level and the invert of the perforated pipe. It is conservative, but reasonable, to use a blockage factor to account for partial blockage of the perforations. A 50 % blockage should be used.

$$Q_{perf} = B \cdot C_d \cdot A \cdot \sqrt{2 \cdot g \cdot h}$$ Equation 7.6

Where
- Q_{perf} = flow through perforations (m^3/s)
- B = blockage factor (0.5)
- C_d = orifice discharge coefficient (assume 0.61 for sharp edge orifice)
- A = total area of the perforations (m^2)
- g = gravity (9.79 m/s^2)
- h = head above the centroid of the perforated pipe (m)

If the capacity of the perforated pipe system is unable to convey the 'design operation flow' then additional perforated pipes will be required.

7.3.10.3 Overflow Pit

To size an overflow pit, two checks should be made to test 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 openings required in the grate cover (assuming drowned outlet conditions). The larger of the two pit configurations should be adopted (as per Section 5.10 QUDM). In addition, a blockage factor is to be used that assumes the grate is 50 % blocked.

For free overfall conditions (weir equation):

$$Q_{weir} = B \cdot C_w \cdot L \cdot h^{3/2}$$ Equation 7.7

Where
- Q_{weir} = flow into pit (weir) under free overfall conditions (m^3/s)
- B = blockage factor (= 0.5)
- C_w = weir coefficient (= 1.66)
- L = length of weir (perimeter of pit) (m)
- h = flow depth above the weir (pit) (m)

Once the length of weir is calculated, a standard sized pit can be selected with a perimeter at least the same length of the required weir length.

For drowned outlet conditions (orifice equation):

$$Q_{orifice} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h}$$ Equation 7.8

Where B, g and h have the same meaning as above

$Q_{orifice}$ = flow rate into pit under drowned conditions (m^3/s)

C_d = discharge coefficient (drowned conditions = 0.6)

A = area of orifice (perforations in inlet grate) (m^2)

When designing grated field inlet pits, reference is to be made to the procedure described in QUDM Section 5.10.4 and the requirements of the local authority.

7.3.10.4 Overflow Weir

In applications where infiltration systems require a discharge control pit, a 'spillway' outlet weir will form part of the high flow bypass system to convey the 'above design flow'. The 'spillway' outlet weir level will be set at the top of the 'detention storage' to ensure catchment flows bypass the infiltration system once the 'detention volume' is full. The length of the 'spillway' outlet weir is to be sized to safely pass the maximum flow discharged to the discharge control pit (as defined the 'above design flow' in Section 7.3.4).

The required length of the 'spillway' outlet weir can be computed using the weir flow equation (Equation 7.7) and the 'above design flow' (Section 7.3.4).

7.3.11 Step 11: Consider Maintenance Requirements

Consider how maintenance is to be performed on the infiltration system (e.g. how and where is access available, where sediment likely to collect etc.). A specific maintenance plan and schedule should be developed for the infiltration system, either as part of a maintenance plan for the whole treatment train, or for each individual asset. Guidance on maintenance plans is provided in Section 7.5.

7.3.12 Design Calculation Summary

Following is a design calculation summary sheet for the key design elements of an infiltration system to aid the design process.

INFILTRATION SYSTEMS DESIGN CALCULATION SUMMARY

	Calculation Task	CALCULATION SUMMARY Outcome		Check
	Catchment Characteristics			
	Catchment area		ha	
	Catchment landuse (i.e residential, commercial etc.)			
	Storm event entering infiltration system (minor or major)		year ARI	
1	**Site and soil evaluation**			
	Site and Soil Evaluation' undertaken in accordance with AS1547-2000 Clause 4.1.3			
	Soil type			
	Hydraulic conductivity (K_{sat})		mm/hr	
	Presence of soil salinity			
	Presence of rock/shale			
	Infiltration site terrain (% slope)			
	Groundwater level		m AHD	
			m below surface	
	Groundwater quality			
	Groundwater uses			
2	**Confirm design objectives**			
	Confirm design objective as defined by conceptual design			
3	**Select infiltration system type**			
	Leaky Well			
	Infiltration Trench			
	Infiltration 'Soak-away'			
	Infiltration Basin			
4	**Pre-treatment design**			
	Level 1 Pre-treatment (avoid clogging)			
	Level 2 Pre-treatment (groundwater protection)			
5	**Determine design flows**			
	'Design operation flow' (1 year ARI)		year ARI	
	'Above design flow' (2 - 100 year ARI)		year ARI	
	Time of concentration			
	Refer to GCC *Land Development Guidelines* and *QUDM*		minutes	
	Identify rainfall intensities			
	'Design operation flow' - $I_{1\ \text{year ARI}}$		mm/hr	
	'Above design flow'- $I_{2-100\ \text{year ARI}}$		mm/hr	
	Design runoff coefficient			
	'Design operation flow' - $C_{1\ \text{year ARI}}$			
	'Above design flow'- $C_{2-100\ \text{year ARI}}$			
	Peak design flows			
	'Design operation flow' - 1 year ARI		m^3/s	
	'Above design flow' (2-100 year ARI)		m^3/s	
6	**Size infiltration system**			
	Hydrologic effectiveness approach			
	Hydrologic effectiveness objective		%	
	Depth		m	
	Porosity (void = 1.0, gravel filled = 0.35)			
	'Infiltration Area'		m^2	
	'Detention Volume'		m^3	

INFILTRATION SYSTEMS DESIGN CALCULATION SUMMARY

	Calculation Task		CALCULATION SUMMARY Outcome		Check
	Design storm approach				
		Design storm flow		m^3/s	
		Inflow volume		m^3	
		Outflow volume		m^3	
		Depth		m	
		'Infiltration Area'		m^2	
		'Detention Volume'		m^3	
7	Locate infiltration system				
		Minimum distance from boundary (Table 7-5)		m	
		Width		m	
		Length		m	
8	Set infiltration depths (sub-surface systems only)				
		Ground surface level		m AHD	
		Groundwater level		m AHD	
				m below surface	
		Infiltration system depth		m	
		Top of infiltration system		m AHD	
		Base of infiltration system		m AHD	
		Cover		m	
		Depth to water table		m	
9	Specify infiltration 'detention volume' elements				
		Gravel size		mm diam.	
		Modular plastic cells			
		Geofabric			
10	Flow management design				
	Inflow/Overflow structures				
		Direct inflow			
		Overflow pit/pipe			
		Discharge control pit			
	Discharge pipe				
		Pipe capacity		m^3/s	
		Pipe size		mm diam.	
	Inflow pipe				
		Pipe capacity		m^3/s	
		Pipe size		mm diam.	
	Overflow pipe				
		Pipe capacity		m^3/s	
		Pipe size		mm diam.	
	Overflow pit				
		Pit capacity		m^3/s	
		Pit size		mm x mm	
	Perforated inflow pipes				
		No. of pipes			
		Pipe size		mm	
	Discharge control pit				
		Pit size		mm x mm	
		Weir length		m	

7.4 Construction and Establishment

It is important to note in the context of a development site and associated construction/building works, delivering infiltration measures can be a challenging task. A careful construction and establishment approach to ensure the system is delivered in accordance with its design intent. The following sections outline a recommended staged construction and establishment methodology for infiltration measures based on the guidance provided in Leinster (2006).

7.4.1 Construction and Establishment Challenges

There exist a number of challenges that must be appropriately considered to ensure successful construction and establishment of infiltration measures. These challenges are best described in the context of the typical phases in the development of a Greenfield or Infill development, namely the Subdivision Construction Phase and the Building Phase (see Figure 7-13).

- Subdivision Construction - Involves the civil works required to create the landforms associated with a development and install the related services (roads, water, sewerage, power etc.) followed by the landscape works to create the softscape, streetscape and parkscape features. The risks to successful construction and establishment of the WSUD systems during this phase of work have generally related to the following:
  - Construction activities which can generate large sediment loads in runoff which can clog infiltration measures
  - Construction traffic and other works can result in damage to the infiltration measures.

Importantly, all works undertaken during Subdivision Construction are normally 'controlled' through the principle contractor and site manager. This means the risks described above can be readily managed through appropriate guidance and supervision.

- Building Phase - Once the Subdivision Construction works are complete and the development plans are sealed then the Building Phase can commence (i.e. construction of the houses or built form). This phase of development is effectively 'uncontrolled' due to the number of building contractors and sub-contractors present on any given allotment. For this reason the Allotment Building Phase represents the greatest risk to the successful establishment of infiltration measures.

7.4.2 Staged Construction and Establishment Method

To overcome the challenges associated within delivering infiltration measures a Staged Construction and Establishment Method should be adopted (see Figure 7-13):

- Stage 1: Functional Installation - Construction of the functional elements of the infiltration measure at the end of Subdivision Construction (i.e. during landscape works) and the installation of temporary protective measures (i.e. stormwater bypass system).

- Stage 2: Sediment and Erosion Control – During the Building Phase the temporary protective measures preserve the functional infrastructure of the infiltration measure against damage.

- Stage 3: Operational Establishment - At the completion of the Building Phase, the temporary measures protecting the functional elements of the infiltration measure can be removed and the system allowed to operation in accordance with the design intent.

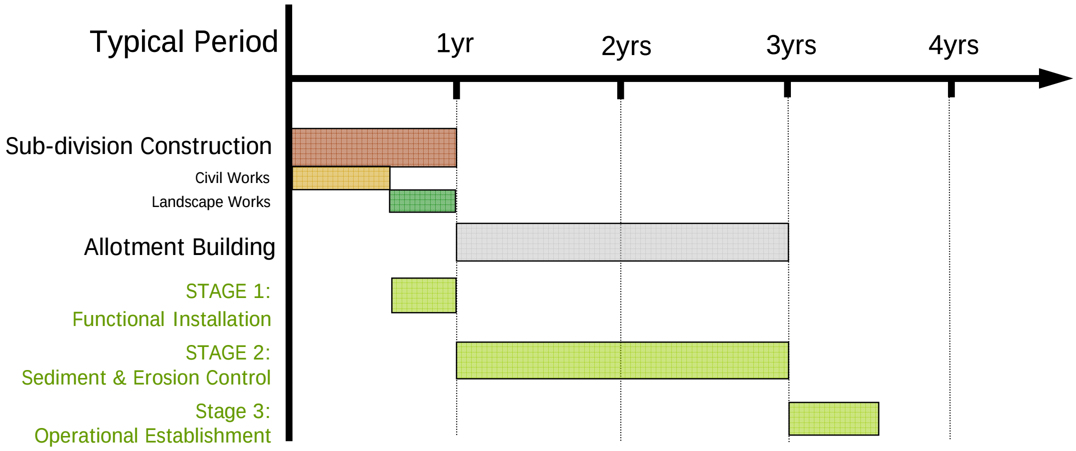


Figure 7-13: Staged Construction and Establishment Method

7.4.2.1 Functional Installation

Functional installation of infiltration measure occurs at the end of Subdivision Construction as part of landscape works and involves:

- Bulking out and trimming
- Installation of the control and pipe structures
- Placement of non-woven geofabric to sides and base
- Placement of gravel (if part of design)
- Where infiltration system is located underground, the inlet should be blocked to ensure sediment laden stormwater flows 'bypass' the system.
- Where the system is an infiltration basin, placement of a temporary protective layer - Covering the surface of filtration media with geofabric and placement of 25 mm topsoil and turf over geofabric. This temporary geofabric and turf layer will protect the infiltration measure during construction (Subdivision and Building Phases) ensuring sediment/litter laden waters do not cause clogging.
- Place silt fences around the boundary of the infiltration measure to exclude silt and restrict access.

7.4.2.2 Sediment and Erosion Control

The temporary protective measures are left in place through the allotment building phase to ensure sediment laden waters do not enter and clog the infiltration measure.

7.4.2.3 Operational Establishment

At the completion of the Allotment Building Phase the temporary measures (i.e. stormwater bypass) can be removed and the infiltration measure allowed to operate. It is critical to ensure that the pretreatment system for an infiltration measure is fully operational before flows are introduced.

7.5 Maintenance Requirements

Maintenance for infiltration measures aims at 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 elements are operating as designed to avoid blockage of the infiltration measure and to prevent groundwater contamination.

To ensure the system is operating as designed, the infiltration zone should be inspected every 1 - 6 months (or after each major rainfall event) depending on the size and complexity of the system. Typical maintenance of infiltration systems will involve:

- Routine inspection to identify any surface ponding after the design infiltration period (refer to Section 7.3.6.2 for appropriate emptying times), which would indicate clogging/ blockage of the underlying aggregate or the base of the trench.
- Routine inspection of inlet points to identify any areas of scour, litter build up, sediment accumulation or blockages.
- Removal of accumulated sediment and clearing of blockages to inlets.
- Tilling of the infiltration surface, or removing the surface layer, if there is evidence of clogging.
- Maintaining the surface vegetation (if present).

7.6 Checking Tools

This section provides a number of checking aids for designers and Council development assessment officers. In addition, Section 7.5.5 provides general advice on the construction and establishment of infiltration measures and key issues to be considered to ensure their successful establishment and operation based on observations from construction projects around Australia.

The following checking tools are provided:

- Design Assessment Checklist
- Construction Inspection Checklist (during and post)
- Operation and Maintenance Inspection Form
- Asset Transfer Checklist (following 'on-maintenance' period).

7.6.1 Design Assessment Checklist

The checklist on page 7-29 presents the key design features that are to be reviewed when assessing the design of an infiltration system. These considerations include configuration, safety, maintenance and operational issues that need to be addressed during the design phase. If an item receives an 'N' when reviewing the design, referral is to 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. Council development assessment officers will require that all relevant permits are in place prior to accepting a design.

7.6.2 Construction Checklist

The checklist on page 7-30 presents the key items to be reviewed when inspecting the infiltration measure during and at the completion of construction. The checklist is to be used by Construction Site Supervisors and local authority Compliance Inspectors to ensure all the elements of the infiltration measure have been constructed in accordance with the design. If an item receives an 'N' in Satisfactory criteria then appropriate actions must be specified and delivered to rectify the construction issue before final inspection sign-off is given.

7.6.3 Operation and Maintenance Inspection Form

In addition to checking and maintaining the function of pretreatment elements, the form on page 7-31 can be used during routine maintenance inspections of the infiltration measure and kept as a record on the asset condition and quantity of removed pollutants over time. Inspections should occur every 1 - 6 months depending on the size and complexity of the system. More detailed site specific maintenance schedules should be developed for major infiltration systems and include a brief overview of the operation of the system and key aspects to be checked during each inspection.

7.6.4 Asset Transfer Checklist

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. For details on asset transfer to specific to each Council, contact the relevant local authority. The table on page 7-32 provides an indicative asset transfer checklist.

INFILTRATION MEASURE DESIGN ASSESSMENT CHECKLIST				
Asset I.D.				
Infiltration Measure Location:				
Hydraulics:	Design operational flow (m^3/s):		Above design flow (m^3/s):	
Area:	Catchment Area (ha):	Infiltration Area (m^2):	Detention Volume (m^3):	
SITE AND SOIL EVALUATION			Y	N
Site and Soil Evaluation undertaken in accordance with AS1547-2000				
Soil types appropriate for infiltration (K_{sat} > 0.36mm/hr, no salinity problems, no rock/shale)?				
PRE-TREATMENT				
Groundwater conditions assessed and objectives established?				
Level 1 Pre-Treatment provided?				
Level 2 Pre-Treatment provided?				
INFILTRATION SYSTEM			Y	N
Design objective established?				
Has the appropriate design approach been adopted?				
Infiltration system setbacks appropriate?				
Base of infiltration system >1m above seasonal high groundwater table?				
Has appropriate cover (soil depth above infiltration system) been provided?				
If placed on >10% terrain (ground slope), has engineering assessment been undertaken?				
FLOW MANAGEMENT			Y	N
Overall flow conveyance system sufficient for design flood event?				
Are the inflow systems designed to convey design flows?				
Bypass/ overflow sufficient for conveyance of design flood event?				
COMMENTS				

INFILTRATION MEASURES CONSTRUCTION INSPECTION CHECKLIST

Asset I.D.		Inspected by:	
Site:		Date:	
Constructed by:		Time:	
		Weather:	
		Contact during visit:	

Items inspected	Checked Y	Checked N	Satisfactory Y	Satisfactory N	Items inspected	Checked Y	Checked N	Satisfactory Y	Satisfactory N
DURING CONSTRUCTION									
A. FUNCTIONAL INSTALLATION					Structural components				
Preliminary Works					10. Location and levels of infiltration system and overflow points as designed				
1. Erosion and sediment control plan adopted					11. Pipe joints and connections as designed				
2. Traffic control measures					12. Concrete and reinforcement as designed				
3. Location same as plans					13. Inlets appropriately installed				
4. Site protection from existing flows					14.Provision of geofabric to sides and base				
Earthworks					15. Correct fill media/modular system used				
5. Excavation as designed					B. SEDIMENT & EROSION CONTROL (if required)				
6. Side slopes are stable					16. Stabilisation immediately following earthworks				
Pre-treatment					17. Silt fences and traffic control in place				
7. Maintenance access provided					18. Temporary protection layers in place				
8. Invert levels as designed					C. OPERATIONAL ESTABLISHMENT				
9. Ability to freely drain					19. Temporary protection layers and associated silt removed				

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. Gravel 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.
Inspection officer signature:

INFILTRATION MEASURES MAINTENANCE CHECKLIST			
Asset I.D.			
Inspection Frequency:	1 to 6 monthly	Date of Visit:	
Location:			
Description:			
Site Visit by:			

INSPECTION ITEMS	Y	N	ACTION REQUIRED (DETAILS)
Sediment accumulation in pre-treatment zone?			
Erosion at inlet or other key structures?			
Evidence of dumping (eg building waste)?			
Evidence of extended ponding times (eg. algal growth)?			
Evidence of silt and clogging within 'detention volume'?			
Clogging of flow management systems (sediment or debris)?			
Damage/vandalism to structures present?			
Drainage system inspected?			
Resetting of system required?			
COMMENTS			

INFILTRATION MEASURE ASSET TRANSFER CHECKLIST		
Asset Description:		
Asset ID:		
Asset Location:		
Construction by:		
'On-maintenance' Period:		
TREATMENT	Y	N
System appears to be working as designed visually?		
No obvious signs of under-performance?		
MAINTENANCE	Y	N
Maintenance plans and indicative maintenance costs provided for each asset?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
ASSET INSPECTED FOR DEFECTS AND/OR MAINTENANCE ISSUES AT TIME OF ASSET TRANFSFER	Y	N
Sediment accumulation at inflow points?		
Litter present?		
Erosion at inlet or other key structures?		
Traffic damage present?		
Evidence of dumping (e.g. building waste)?		
Evidence of ponding?		
Surface clogging visible?		
Damage/vandalism to structures present?		
COMMENTS		
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?		

7.7 Infiltration Measure Worked Example

An infiltration system is to be installed to infiltrate stormwater runoff from an industrial allotment in Brisbane. The allotment is 1.0 ha in area on a rectangular site (200 m x 50 m) with an overall impervious surface area of 0.48 ha (48 % impervious). All stormwater runoff is to be pretreated through swale bioretention systems prior to entering the infiltration system to ensure sustainable operation of the infiltration system and protection of groundwater. An illustration of the proposed allotment and associated stormwater management scheme is shown in Figure 7-14.

Treated flows from the swale bioretention systems are to be delivered to the infiltration system via traditional pipe drainage sized to convey the minor storm event (2 year ARI).

The allotment is located within a catchment that drains to a natural wetland that has been defined by the local authority as being hydrologically sensitive to increases in catchment flow. Therefore, there is to be no increase in mean annual runoff as a result of the development.

This worked example focuses on the design of an infiltration 'soak-away' system for the allotment based on the site characteristics and design objectives listed below.

<u>Site Characteristics</u>

The site characteristics are summarised as follows:

- Catchment area	2,400 m^2 (roof)
	2,400 m^2 (ground level paved)
	5,200 m^2 (pervious)
	10,000 m^2(total)
- Predevelopment mean annual runoff = 2.2 ML/yr
- Post development mean annual runoff = 6.1 ML/yr
- Soil type - sandy clay
- Saturated hydraulic conductivity (K_{sat}) = 80 mm/hr
- Topography - flat to moderate grades towards the road (2 - 4 %).

<u>Design Objectives</u>

As outlined in Section 7.6.1, the allotment is located within a catchment that drains to a natural wetland that has been defined by BCC as being hydrologically sensitive to increases in catchment flow and BCC require that there be no increase in mean annual runoff as a result of the development. Considering the predevelopment mean annual runoff is 2.2 ML/yr and the post-development mean annual runoff is 6.1 ML/yr, the design objective of the infiltration system is the capture and infiltration of 3.9 ML/yr (equal to 64 % hydrologic effectiveness).

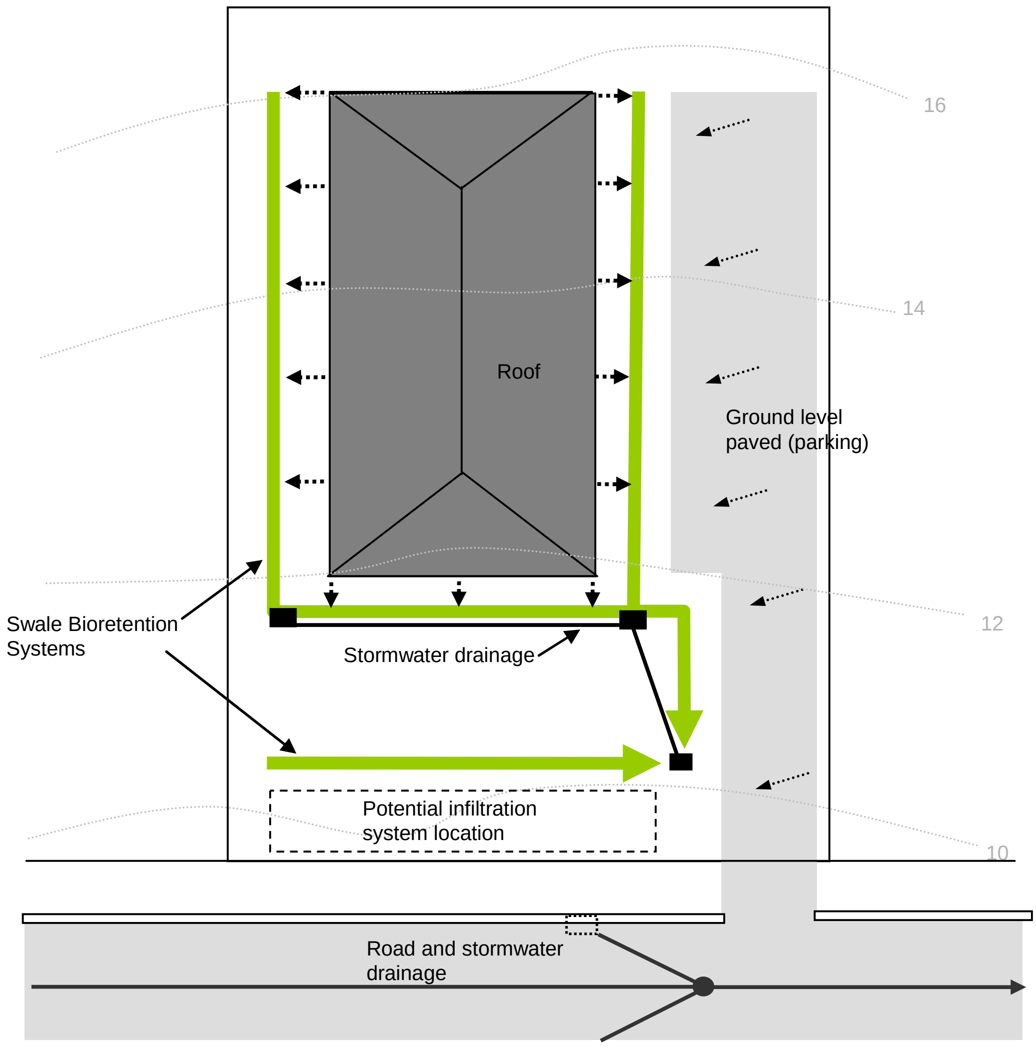


Figure 7-14: Site Layout (see Figure 7.10 for Drainage Detail)

7.7.1 Step 1: Site and Soil Evaluation

To define the site's suitability for infiltration of stormwater a 'Site and Soil Evaluation' was undertaken in accordance with AS1547-2000 Clause 4.1.3. The key information from the evaluation is presented below:

- soil type = sandy clay
- hydraulic conductivity = 80 mm/hr
- presence of soil salinity = no problems discovered
- presence of rock or shale = no rock or shale discovered
- slope/ terrain (%) = 2 – 4 %, ground level 10 m AHD in infiltration location
- groundwater details (depth, quality and values) = water table 5 m below surface (5 m AHD), moderate water quality with local bores used for irrigation.

Field tests found the soil to be suitable for infiltration, consisting of sandy clay with a saturated hydraulic conductivity of 80 mm/hr.

7.7.2 Step 2: Confirm Design Objectives

As outlined in Section 7.7.1.2, the design objective for the infiltration system is no increase in mean annual runoff as a result of the development, which requires the system to achieve 64 % hydrologic effectives. The hydrologic effectiveness approach will be used to establish the size of the infiltration system.

Design objective = no increase in mean annual runoff post- development (i.e. 64 % hydrologic effectiveness).

7.7.3 Step 3: Select Infiltration System Type

Based on the site attributes, the scale of the infiltration application (i.e. 1.0 ha) and Table 7-1, an infiltration 'soak-away' system is selected for the industrial allotment.

7.7.4 Step 4: Pretreatment Design

As an infiltration 'soak-away' has been selected for the site, reference to Section 7.2.4 and Table 7-3 indicates both Level 1 and 2 Pretreatment is required. Considering the groundwater is of moderate quality and is currently used for irrigation purposes, best practice treatment (80 % reduction in TSS and 45 % reduction in TP and TN) was proposed and approved by BCC based on meeting the BCC water quality objectives. This is being achieved through the use of swale bioretention systems strategically located through the allotment to capture runoff before it enters the traditional drainage systems (see Figure 7.9).

7.7.5 Step 5: Determine Design Flows

As described in Section 7.3.5, the 'design operation flow' is required to size the inlet to the infiltration system, which may vary depending on the particular situation. In this case, flows into the infiltration system are to be regulated through a discharge control pit, which will deliver flows up to the 1 year ARI into the infiltration system. Flows greater than 1 year ARI, or when the infiltration system is full, will bypass the infiltration system by overtopping the overflow weir in the discharge control pit. Considering only traditional drainage will enter the discharge control pit, the 'above design flow' is the 2 year ARI event. Therefore:

- 'design operation flow' = 1 year ARI
- 'above design flow' = 2 year ARI

Design flows are established using the Rational Method and the procedures provided in the relevant local government guidelines and QUDM (DPI, IMEA & BCC, 1992). The site has one contributing catchment being 1.0 ha in area, 200 m long and drained by swale bioretention systems and stormwater pipes.

<u>Time of concentration (t_c)</u>

Time of Concentration t_c = 10 mins

Design runoff coefficient

Runoff Coefficients

C_{10} = 0.88 (*Supplement to QUDM*)

	***C* Runoff**		
ARI	1	2	10
QUDM Factor	0.8	0.85	1.0
C_{ARI}	0.7	0.75	0.88

Catchment Area, A = 10,000 m^2 (1.0 ha)

Rainfall Intensities (BCC) t_c = 10 mins

I_1 = 90 mm/hr

I_2 = 116 mm/hr

Rational Method $Q = CIA/360$

$Q_{1yr\,ARI}$ = 0.175 m^3/s

$Q_{2yr\,ARI}$ = 0.242 m^3/s

'Design operation flow' = 0.175 m^3/s

'Above design flow' = 0.242 m^3/s

7.7.6 Step 6: Size Infiltration System

The design objective for the infiltration basin is to achieve a hydrologic effectiveness of 64 %. This objective is to be delivered through use of an infiltration 'soak-away' created using gravel and being 1.0 m in depth.

Referring to Figure 7-6 (depth = 1.0 m and porosity = 0.35) and estimating the position of the 80 mm/hr hydraulic conductivity curve (by carrying out a simple interpolation between the 36 mm/hr and the 100 mm/hr curves), the 'infiltration area' must be approximately 1.5 % of the catchment area to achieve a hydrologic effectiveness when the in-situ soil hydraulic conductivity is 80 mm/hr. Therefore, the 'infiltration area' is 150 m^2 and the 'detention volume' is 150 m^3 (gravel filled).

Note: The relationship between the curves is not linear and as a result, interpolations do not provide an exact representation of the size of infiltration area (as a % of catchment area). Designers must be careful not to undersize infiltration areas through this process.

Gravel filled infiltration 'soak-away'

'Infiltration Area' = 150 m^2

'Detention Volume' = 150 m^3

7.7.7 Step 7: Locate Infiltration System

With a sandy clay soil profile, the minimum distance of the infiltration system from structures and property boundary is 2 m (Table 7-5). 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.

The infiltration 'soak-away' is to be rectangular in shape, being 30 m long by 5 m wide and located 2 m from the front boundary as shown in Figure 7-15.

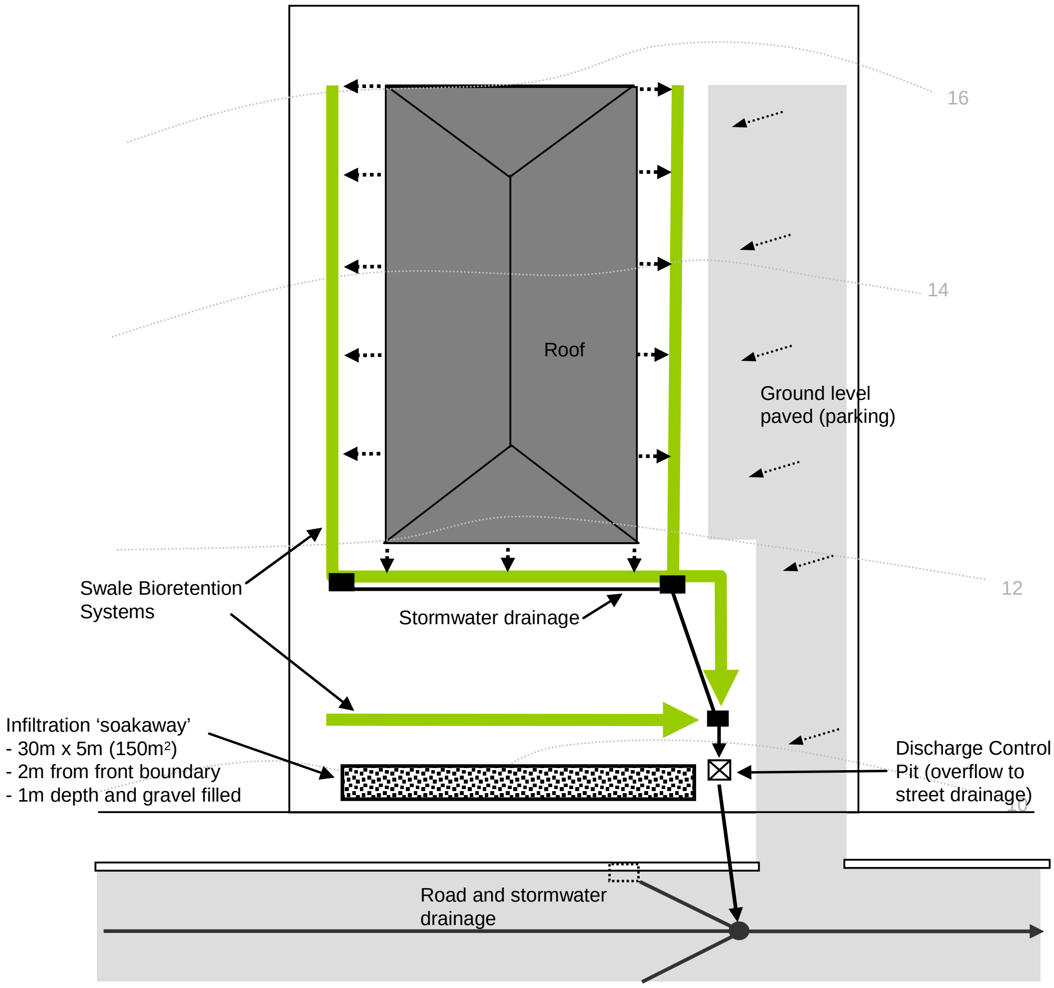


Figure 7-15: Location of Infiltration System

7.7.8 Step 8: Set Infiltration Depths (Sub-surface Systems Only)

The depth of the infiltration systems must be set to ensure the base is a minimum of 1.0 m above the seasonal high water table and there is a minimum of 0.3 m cover. Considering the water table sits 5 m below surface (5 m AHD), an infiltration depth of 1.0 m is adopted with a cover of 0.5 m. This means the base of the infiltration system sits at 8.5 m AHD which is 3.5 m above the water table.

Infiltration depth = 1.0 m
Cover = 0.5 m
Top of infiltration system = 9.5 m AHD
Base of infiltration system = 8.5 m AHD
Depth to water table = 3.5 m

7.7.9 Step 9: Specify Infiltration 'Detention Volume' Elements

The following design specification applies to the infiltration 'soak-away':

- Gravel - clean (fines free) stone/ gravel with a uniform size of 50 mm diameter.
- Geofabric - Geofabric must to be installed along the side walls and through the base of the infiltration detention volume to prevent the migration of in-situ soils into the system. Geofabric must be non-woven type with a minimum perforation or mesh size of 0.25 mm.

7.7.10 Step 10: Hydraulic Control Design

Flow into the infiltration 'soak-away' will be regulated through a discharge control pit with overflow or bypass flows being directed into the piped drainage system located in the road reserve. As depicted in Figure 7-16 (over page), the discharge control pit consists of the following:

- discharge pipe – discharge 'above design flow' (2 year ARI) into the pit
- inflow pipe - connection between the pit and the infiltration basin sized to convey 'design operation flow' (1 year ARI)
- perforated inflow pipes - to distribute ' design operation flow' (1 year ARI) into the gravel filled 'detention volume'
- overflow weir – to bypass 'above design flow' (2 year ARI).

7.7.10.1 Discharge pipe

The discharge pipe into the control pit is sized to convey the 'above design flow' (2 year ARI = 0.242 m^3/s) into the discharge control pit using Equation 7.5 in accordance with QUDM (DPI, IMEA & BCC, 1992). The resulting pipe size is a 375 mm diameter reinforced concrete pipe (RCP) at 2 % grade (calculation not presented). The pipe will enter the pit at 9.2 m AHD therefore the invert of the discharge control pit is set at 9.0 m AHD.

Discharge Pipe = 375 mm diameter RCP at 2 % grade

Invert Level at Pit = 9.0 m AHD.

7.7.10.2 Inflow Pipe (Connection to Infiltration System)

The size of the inflow pipe connecting the discharge pit to the infiltration system is calculated by estimating the velocity in the connection pipe using a simplified version of Equation 7.5:

$$h = \frac{2 \cdot V^2}{2 \cdot g}$$

Where h = head level driving flow through the pipe (defined as the overflow weir crest level minus the invert level of the inflow pipe)

= 9.5 m AHD – 9.0 m AHD = 0.5 m

V = pipe velocity (m/s)

g = gravity (9.79 m/s^2)

Note: the coefficient of 2 in the equation is a conservative estimate of the sum of entry and exit loss coefficients (K_{in} + K_{out}).

Hence, V = (9.79 x 0.5) 0.5 = 2.21 m/s

The area of pipe required to convey the 'design operation flow' (1 year ARI) is then calculated by dividing the above 'design operation flow' by the velocity:

A = 0.175/2.21 = 0.079 m^2

This area is equivalent to ~ 300 mm RCP. The obvert of the pipe is to be set at 9.0 m AHD.

Inflow pipe = 300 mm diameter RCP

Invert Level at Pit = 9.0 m AHD

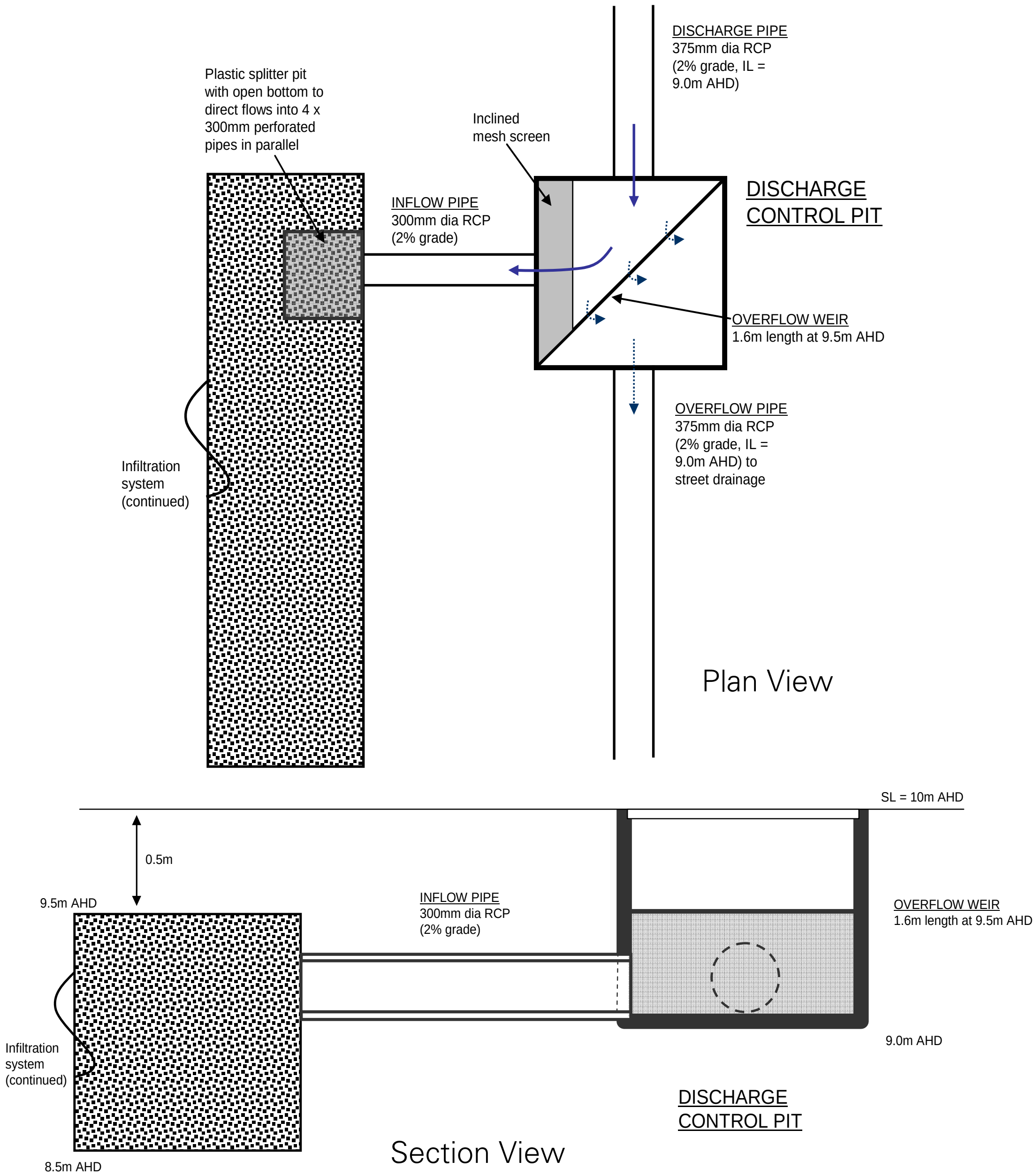


Figure 7-16: Discharge Control Pit Configuration

7.7.10.3 Perforated Inflow Pipes

To ensure appropriate distribution of flows into the gravel filled 'detention volume', four 300 mm diameter perforated pipes laid in parallel (1.0 m apart) are to accept flows from the 300 mm diameter RCP. The perforated pipes have a slot clear opening of 3150 mm^2/m with the slots being 1.5 mm wide and are to be placed at 0.5 % grade.

Two design checks are required:

- Ensure the pipe has capacity to convey the 'design operation flow' (0.175 m^3/s).
- Ensure the perforations are adequate to pass the 'design operation flow'.

Perforated Pipe Conveyance

Manning's equation is applied to estimate the flow rate in the perforated pipes and confirm the capacity of the pipes is sufficient to convey the 'design operation flow' (0.175 m^3/s). The four 300 mm diameter perforated pipes are to be laid in parallel at a grade of 0.5 %.

Applying the Manning's Equation assuming a Manning's *n* of 0.015 finds:

Q (flow per pipe) = 0.044 m/s

Q_{Total} = 0.176 m^3/s (for four pipes) > 0.175 m^3/s, and hence OK.

Perforated Pipe Slot Conveyance

To ensure the perforated pipe slots are not a hydraulic choke in the system, the flow capacity of perforated pipe slots is estimated and compared with the 'design operation flow' (0.175 m^3/s). To estimate the flow rate, an orifice equation (Equation 7.6) is applied as follows:

$$Q_{orifice} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h}$$

Where Head *(h)* = 0.5 m

Blockage *(B)* = 0.5 (50 % blocked)

Area *(A)* = 2100 mm^2/m clear perforations, hence blocked area

= 1050 mm^2/m

Slot Width = 1.5 mm

Slot Length = 7.5 mm

Pipe diameter = 300 mm

Coefficient *(C_d)* = 0.61 (assume slot width acts as a sharp edged orifice).

Number of slots per metre = (1050)/(1.5x7.5) = 93.3

Note: blockage factor (*B*) already accounted for in 'Area' calculation above

Inlet capacity /m of pipe $= [0.61 \times (0.0015 \times 0.0075) \times \sqrt{2 \times 9.81 \times 0.5}] \times 93.3$

= 0.002 m^3/s

Inlet capacity/m x total length (4 lengths of 30 m) = 0.002 x (4 x 30) = 0.24 m^3/s > 0.175, hence OK.

Perforated pipes = 4 x 300 mm diameter perforated pipe laid in parallel, 1.0 m apart and at 0.5 % grade.

7.7.10.4 Overflow Weir

An overflow weir (internal weir) located within the discharge control pit separates the inflow pipe to the infiltration system from the overflow pipe connecting to the street drainage. The overflow weir is to be sized to convey the 'above design flow' of 0.242 m^3/s and surcharge 0.2 m above the weir.

The weir flow equation (Equation 7.7) is used to determine the required weir length:

$$Q_{weir} = B \cdot C_w \cdot L \cdot h^{3/2}$$

So $$L = \frac{Q_{weir}}{B \cdot C \cdot h^{3/2}}$$

Using the 'above design operation' flow (0.242 m^3/s), B = 1.0 (no blockage for internal weir), C_w = 1.66 and h = 0.2 m we have L = 1.6 m.

If the weir is located diagonally across the discharge control pit, a 1200 x 1200 mm pit can be used. The crest of the weir must be set at the top of the 'detention volume' of the infiltration system (i.e. 9.5 m AHD).

Overflow weir = 1.6 m length at 9.5 m AHD

Discharge control pit = 1200 x 1200 mm

7.7.11 Design Calculation Summary

The sheet below summarises the results of the design calculations.

INFILTRATION SYSTEMS DESIGN CALCULATION SUMMARY

			CALCULATION SUMMARY		
	Calculation Task		Outcome		Check
	Catchment Characteristics				
		Catchment area	1.0	ha	✓
		Catchment landuse (i.e residential, commercial etc.)	Industrial		
		Storm event entering infiltration system (minor or major)	2	year ARI	
1	Site and soil evaluation				
	Site and Soil Evaluation' undertaken in accordance with AS1547-2000 Clause 4.1.3				
		Soil type	Sandy Clay		✓
		Hydraulic conductivity (*K_{sat}*)	80	mm/hr	
		Presence of soil salinity	No		
		Presence of rock/shale	No		
		Infiltration site terrain (% slope)	2-4%		
		Groundwater level	5	m AHD	
			5	m below surface	
		Groundwater quality	Moderate		
		Groundwater uses	Irrigation		
2	Confirm design objectives				
		Confirm design objective as defined by conceptual design	No increase in mean annual runoff. 64% hydrologic effectiveness		✓
3	Select infiltration system type				
		Leaky Well			✓
		Infiltration Trench			
		Infiltration 'Soak-away'	✓		
		Infiltration Basin			
4	Pre-treatment design				
		Level 1 Pre-treatment (avoid clogging)	✓		✓
		Level 2 Pre-treatment (groundwater protection)	✓		
5	Determine design flows				
		'Design operation flow' (1 year ARI)	1	year ARI	✓
		'Above design flow' (2 - 100 year ARI)	2	year ARI	
	Time of concentration				
		Refer to GCC *Land Development Guidelines* and *QUDM*	10	minutes	✓
	Identify rainfall intensities				
		'Design operation flow' - $I_{1\ \text{year ARI}}$	90	mm/hr	✓
		'Above design flow'- $I_{2-100\ \text{year ARI}}$	116	mm/hr	
	Design runoff coefficient				
		'Design operation flow' - $C_{1\ \text{year ARI}}$	0.7		✓
		'Above design flow'- $C_{2-100\ \text{year ARI}}$	0.75		
	Peak design flows				
		'Design operation flow' - 1 year ARI	0.175	m^3/s	✓
		'Above design flow' (2 - 100 year ARI)	0.242	m^3/s	
6	Size infiltration system				
	Hydrologic effectiveness approach				
		Hydrologic effectiveness objective	64	%	✓
		Depth	1	m	
		Porosity (void = 1.0, gravel filled = 0.35)	0.35		
		Size of infiltration area (from Figure 7-5 to Figure 7-12)	1.5	% of catchment area	
		'Infiltration Area'	150	m^2	
		'Detention Volume'	150	m^3	

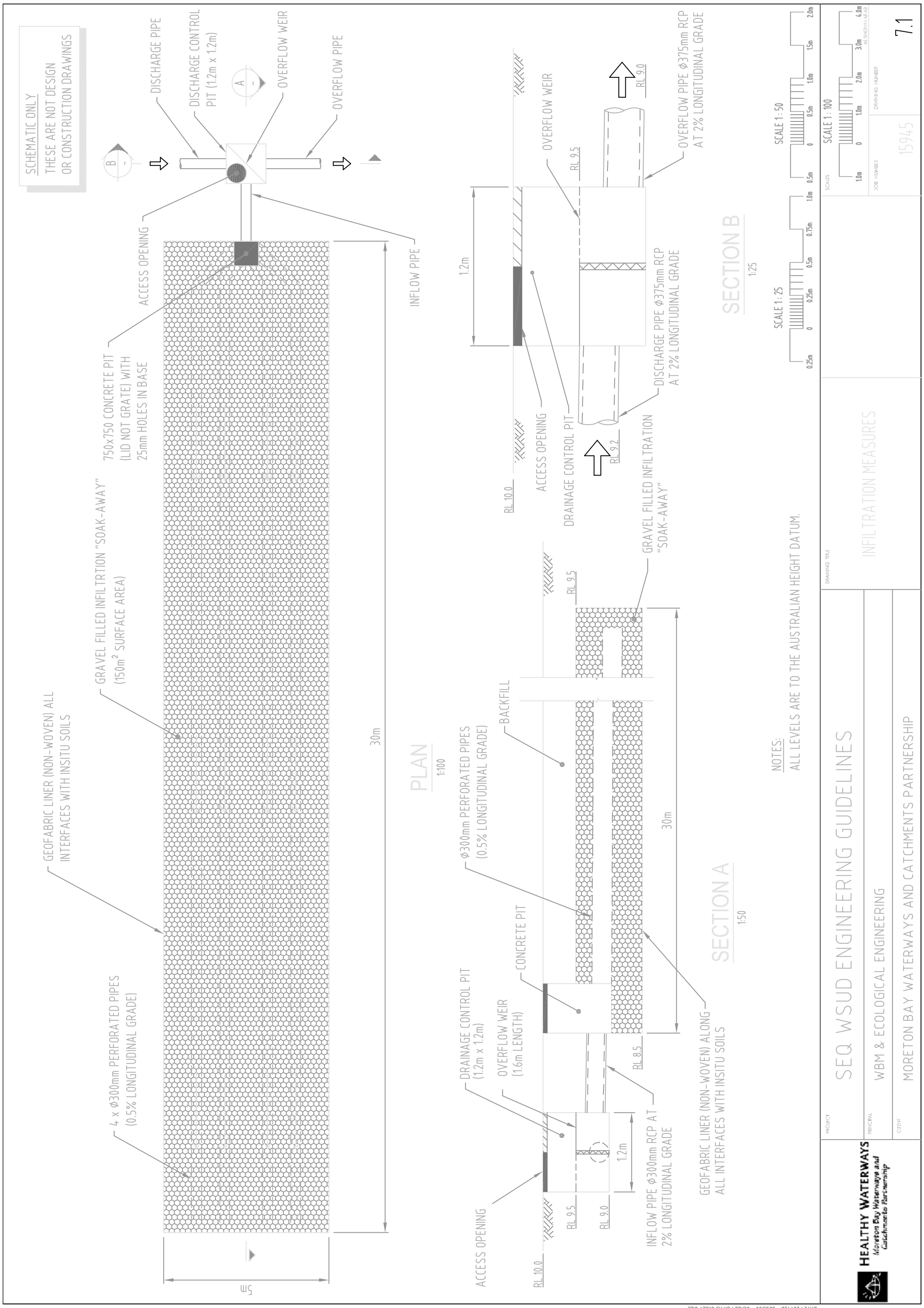
INFILTRATION SYSTEMS DESIGN CALCULATION SUMMARY

			CALCULATION SUMMARY		
	Calculation Task		Outcome		Check
	Design storm approach				
		Design storm flow	-	m^3/s	✓
		Inflow volume	-	m^3	
		Outflow volume	-	m^3	
		Depth	-	m	
		'Infiltration Area'	-	m^2	
		'Detention Volume'	-	m^3	
7	Locate infiltration system				
		Minimum distance from boundary (Table 7-5)	2	m	✓
		Width	5	m	
		Length	30	m	
8	Set infiltration depths (sub-surface systems only)				
		Ground surface level	10	m AHD	✓
		Groundwater level	5	m AHD	
			5	m below surface	
		Infiltration system depth	1	m	
		Top of infiltration system	9.5	m AHD	
		Base of infiltration system	8.5	m AHD	
		Cover	0.5	m	
		Depth to water table	3.5	m	
9	Specify infiltration 'detention volume' elements				
		Gravel size	50	mm diam.	✓
		Modular plastic cells			
		Geofabric	✓		
10	Flow management design				
	Inflow/Overflow structures				
		Direct inflow			✓
		Overflow pit/pipe			
		Discharge control pit	✓		
	Discharge pipe				
		Pipe capacity	0.242	m^3/s	✓
		Pipe size	375	mm diam.	
	Inflow pipe				
		Pipe capacity	0.175	m^3/s	✓
		Pipe size	300	mm diam.	
	Overflow pipe				
		Pipe capacity	0.242	m^3/s	✓
		Pipe size	375	mm diam.	
	Overflow pit				
		Pit capacity	-	m^3/s	✓
		Pit size	-	mm x mm	
	Perforated inflow pipes				
		No. of pipes	4		✓
		Pipe size	300	mm	
	Discharge control pit				
		Pit size	1200 x 1200	mm x mm	✓
		Weir length	1.5	m	

7.7.12 Worked Example Drawings

Drawing 7.1 details the construction of the infiltration system designed in the worked example.

Drawing 7.1 Infiltration Methods



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[1] At the time of preparation of these guidelines, QUDM was under review and a significantly revised edition is expected to be released in 2006. These guidelines refer to and use calculations specified in the existing QUDM document, however the revised version of QUDM should be used as the appropriate reference document. It should be noted by users of this guideline that the structure and content of QUDM will change, and as such, the references to calculations and/or specific sections of QUDM may no longer be correct. Users of this guideline should utilise and adopt the relevant sections and/or calculations of the revised QUDM guideline.

Chapter 8 Sand Filters

8.1 Introduction 8-2

8.2 Design Considerations 8-2
8.2.1 Configuration 8-2
8.2.2 Maintenance 8-4

8.3 Design Process 8-6
8.3.1 Step 1: Confirm Treatment Performance of Concept Design 8-6
8.3.2 Step 2: Determine Design Flows 8-8
8.3.3 Step 3: Design Sedimentation Chamber 8-9
8.3.4 Step 4: Specify the Filter Media Characteristics 8-9
8.3.5 Step 5: Under-drain Design and Capacity Checks 8-10
8.3.6 Step 6: Size Overflow Weir 8-11
8.3.7 Design Calculation Summary 8-11

8.4 Construction Advice 8-14
8.4.1 Building Phase Damage 8-14
8.4.2 Sediment Basin Drainage 8-14
8.4.3 Inspection Openings (flushing points) for Perforated Pipes 8-14
8.4.4 Clean Filter Media 8-14

8.5 Maintenance Requirements 8-15

8.6 Checking Tools 8-15
8.6.1 Design Assessment Checklist 8-15
8.6.2 Construction Checklist 8-15
8.6.3 Operation and Maintenance Inspection Form 8-16
8.6.4 Asset transfer checklist 8-16

8.7 Sand Filter Worked Example 8-21
8.7.1 Step 1: Confirm Treatment Performance of Concept Design 8-22
8.7.2 Step 2: Estimating Design Flows 8-22
8.7.3 Step 3: Design Sedimentation Chamber 8-23
8.7.4 Step 4: Specify Filter Media Characteristics 8-25
8.7.5 Step 5: Under-drain Design 8-26
8.7.6 Step 7: Size Overflow Weir 8-27
8.7.7 Design Calculation Summary 8-27
8.7.8 Worked Example Drawings 8-30

8.8 References 8-31

8.1 Introduction

Sand filters operate in a similar manner to bioretention systems, with the exception that stormwater passes through a filter media (typically sand) that has no vegetation growing on the surface. Sand filters do not incorporate vegetation because the filter media does not retain sufficient moisture to support plant growth and they are often installed underground (therefore light limits plant growth). The absence of vegetation and the associated biologically active soil layer typically created around the root zone of vegetation planted in bioretention systems, means sand filters have a reduced stormwater treatment performance compared to bioretention systems.

Sand filters should only be considered where site conditions, such as space or drainage grades, limit the use of bioretention systems. This is most likely related to retrofit situations where the surrounding urban environment is already developed. Treatment can then be achieved underground with sand filters, in areas such as high density developments with little or no landscape areas. Their lack of vegetation requires more regular maintenance than bioretention systems to ensure the surface of the sand filter media remains porous and does not become clogged with accumulated sediments. This typically involves regular inspections and routine removal of fine sediments that have formed a 'crust' on the sand filter surface.

Prior to entering a sand filter, flows must be subjected to pre-treatment to remove litter, debris and coarse sediments (typically via an 'inlet chamber', which is designed as part of the system). Following pretreatment, flows are spread over the sand filtration media and water percolates downwards and is intercepted by 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 that can be treated. Very high flows are diverted around sand filters to protect the sand media from scour.

8.2 Design Considerations

8.2.1 Configuration

A sand filter system typically consists of three chambers: an inlet chamber that allows sedimentation and removal of gross pollutants, a sand filter chamber and a high flow bypass chamber, as illustrated in Figure 8-1. The shape of a sand filter can be varied to suit site constraints and maintenance access, provided each of the chambers is adequately sized.

8.2.1.1 Sedimentation Chamber

Water firstly enters the sedimentation (inlet) chamber where gross pollutants and coarse to medium-sized sediments are retained. Stormwater enters this chamber either via a conventional side entry pit or through an underground pipe network.

The sedimentation chamber can be designed to either have permanent water between events or to drain between storm events with weep holes. There are advantages and disadvantages with each approach. The decision of which type of system is most appropriate must be made based on catchment runoff characteristics and downstream receiving environment, likely maintenance programs (and available equipment) and site accessibility.

Having a permanent water body reduces the likelihood of re-suspension of sediments at the start of subsequent rainfall events as inflows do not fall and scour collected sediments. This system requires the removal of wet material from the sedimentation chamber during maintenance, which is more costly than for drained material. However, where appropriate maintenance machinery (such as vacuum trucks) is available, these costs may be manageable. A potential issue with these systems arises from the stagnant water and potentially high organic loads that can lead to anaerobic conditions. This may cause the release of soluble pollutants (such as phosphorus) and generation of odorous gases. This transformation of particulate bound pollutants to soluble forms can lead to a reduced treatment performance by the sand filter as soluble forms of nutrients and metals are more difficult to retain and process within the sand filtration chamber. The subsequent discharge of soluble forms of pollutants can cause water quality problems downstream (such as excessive algal growth).

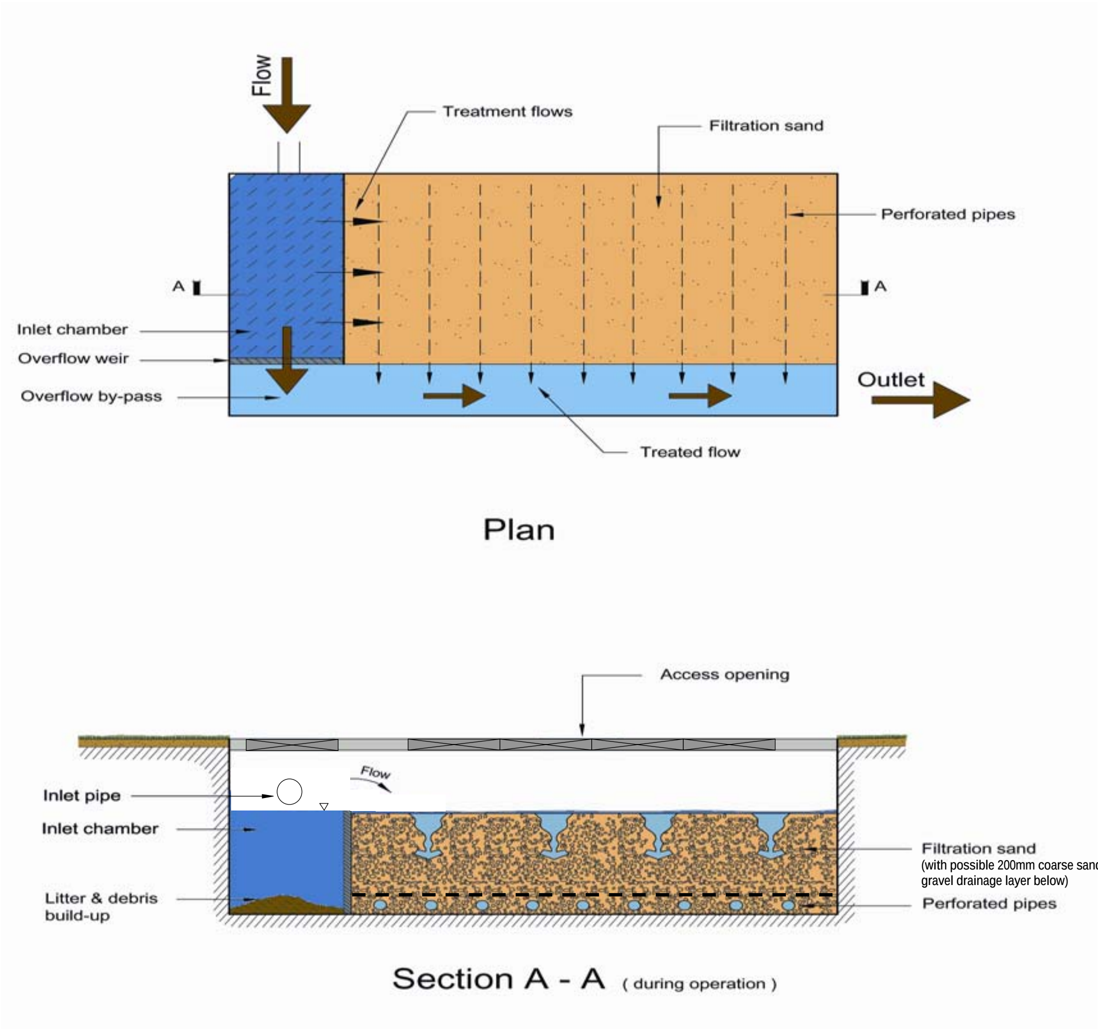


Figure 8-1: Typical Sand Filter Configuration during Operation

Allowing the sedimentation chamber to drain during inter-event periods (by the installation of weep holes) reduces the likelihood of pollutant transformation during the inter-event period. The challenge with this system is to design weep holes such that they do not block and can continue to drain as material (litter, organic material and sediment) accumulates. Drained sediment chambers are also prone to re-suspension of accumulated material as the initial flows from subsequent rainfall events enter the chamber. Where re-suspension is expected to be an issue, a baffle arrangement or other structure to manage incoming flow velocities may be constructed across the inlet flow path to minimise turbulence as flow enters the sediment chamber and thus reduce potential for re-suspension of sediments.

It should also be noted that free-draining dry chambers result in a portion of the detained stormwater (particularly low-flows) being discharged without receiving treatment by the sand filter. To reduce the amount of untreated flow from the chamber, drainage holes between the sediment chamber and the sand filter chamber can be provided to drain the sediment chamber into the sand filter media. Alternatively, additional treatment could be provided to this discharged stormwater through another treatment device (e.g. bioretention).

The sedimentation chamber requires sufficient access space for manual removal of sediment and accumulated debris during maintenance operations. These factors need to be considered when designing the sedimentation chamber.

8.2.1.2 Sand Filter Chamber

Stormwater flows from the sedimentation chamber into the sand filter chamber via a weir. Water then percolates through the sand filtration media (typically 400-600 mm depth) and perforated under-drain pipes collect filtered water in a similar manner to bioretention systems. Provision for temporary ponding is provided within the sand filter chamber. When water levels reach the maximum ponding depth, flows spill over to an overflow (bypass) chamber (usually via the sedimentation chamber). The bypass chamber protects the surface of the sand filter media from scour during high flow events. The high saturated hydraulic conductivity of the sand filtration media means that only a small (~200 mm) extended detention (temporary ponding) depth is required.

The sand filter media will typically have a saturated hydraulic conductivity between 1×10^{-4} m/s (360 mm/hr) to 1×10^{-3} m/s (3600 mm/hr) depending on the selected sand particle size distribution. The material should be free of fines and have a relatively uniform grain size distribution. Example particle size distributions are provided in Section 8.3.4.1. The surface of the sand filter media should be set at the crest of the weir connecting the sedimentation chamber to the sand filter chamber. This minimises potential scouring of the sand surface as water flows into the sand filter chamber. Alternatively, where the crest of the sediment chamber weir (treatment flow weir) is elevated above the sand filter surface, appropriate scour protection must be used.

The sand filter chamber typically comprises two layers, a drainage layer consisting of a clean washed river sand with saturated hydraulic conductivity >4000mm/hr overlain by the sand filter media described above. The drainage layer contains either flexible perforated pipes (e.g. ag pipes) or slotted PVC pipes, however care needs to be taken to ensure the slots in the pipes are not so large that particles can freely flow into the pipes from the drainage layer. The slotted or perforated collection pipes at the base of the sand filter collect treated water for conveyance downstream. They should be sized so that the filtration media freely drains and the collection system does not become a 'choke' in the system.

In some circumstances it may be desirable to restrict the discharge capacity of sand filter chamber so as to promote a longer detention period within the sand filter media and therefore allow for increased biological treatment from longer contact time. One such circumstance is when depth constraints may require a shallower filter media depth and a larger surface area, leading to a higher than desired maximum infiltration rate. In such circumstances it is recommended that the drainage layer and under drainage pipe network be designed so as to not become the hydraulic "choke" in the system (as above) and that a control valve be used at the outfall of the underdrainage system to regulate the detention time in the sand filter media. In this way greater control over detention time can be achieved.

8.2.1.3 Overflow Chamber

The overflow chamber provides a bypass during flood events to downstream drainage infrastructure. When water levels in the sedimentation and sand filter chambers exceed the extended detention depth, water overflows a weir into the bypass chamber and is conveyed into the downstream drainage system. The overflow weir is sized to ensure that it has sufficient capacity to convey the minor storm flow (typically the 2-10 year ARI).

8.2.2 Maintenance

Sand filters have no vegetation to break up the filter surface (unlike bioretention systems); therefore, maintenance is critical to ensuring continued performance, particularly in preserving the hydraulic conductivity of the filtration media. Without regular maintenance (e.g. 3-6 months with more frequent inspections to determine clean out requirements), collected fine material will create a 'crust' on the surface that significantly decreases infiltration capacity. Regular maintenance involves removing the surface layer of fine sediments that can tend to clog the filtration media.

Inspections of the sedimentation chamber need to be performed every 1-6 months (as for the sand filter chamber); however, sediment and/ or gross pollutant cleanout may only be required annually. The frequency will ultimately depend on upstream catchment activities and will be linked to seasonal rainfall (i.e. high summer rainfall may require more frequent cleanouts). Of particular importance are regular inspections during and immediately following construction and these should be conducted after the first few significant rainfall events. Records of all inspections and maintenance activities should be documented and filed for future use.

There are several key decisions during design that significantly impact on ease of maintenance for a sand filter. Easy access for maintenance is fundamental to long term performance and needs to be considered early 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 (including less frequent access to the overflow chamber).

Direct physical access to the whole surface of the sand filter chamber will be required to remove fine sediments from the surface layer of the filter media (top 25-50 mm) as they accumulate forming a crust. Depending on the scale of the system, this may require multiple entry points to the chamber to enable access with a shovel or vacuum machinery. If maintenance crews cannot access part of the sand filter chamber, it will quickly become blocked and thus reduce water quality improvement.

The sedimentation chamber needs to be drained for maintenance purposes (unless appropriate wet extraction equipment is available). A drainage valve or gate should be incorporated into systems that have no weep holes so that this chamber can fully drain. Having freely drained material significantly reduces the removal and disposal maintenance costs. Alternatively, water in the sediment chamber can be pumped into the sand filter and then pollutants removed.

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 by extending the under-drains to the surface of the sand filter to allow for inspection and maintenance when required. The vertical section of the under-drain should be either solid pipe or wrapped in impermeable geotextile and a cap placed on the end of the pipe to avoid short circuiting of flows directly in to the drain. A temporary filter sock or equivalent should also be placed over the outlet pipe in the overflow chamber to capture flushed sediment during maintenance activities.

8.3 Design Process

The following sections detail the design steps required for sand filters. Key design steps are:

1. Confirm treatment performance of concept design
2. Determine design flows
3. Design sedimentation chamber
4. Specify filter media characteristics
5. Design under-drain and undertake capacity checks
6. Size overflow weir

8.3.1 Step 1: Confirm Treatment Performance of Concept Design

This step ensures the detailed designer of the sand filter system first checks the general dimensions and configuration of the concept design layout before proceeding to detailed design. The curves presented below indicate the expected pollutant removal performance of a typical sand filter system and allow a rapid assessment of the adequacy of the treatment area dimension of the concept design layout against the expected performance of the system.

The curves in Figure 8-2 to Figure 8-4 show the pollutant removal performance expected for a sand filter with the following characteristics:

- saturated hydraulic conductivity of 3600 mm/hr
- filter media depth of 600 mm
- filter particle effective diameter of 1.0 mm
- extended detention depth of 200 mm.

The curves are based on the performance of a sand filter in a typical residential suburb in South East Queensland (i.e. with an annual volumetric runoff coefficient (AVRC) of 0.38). The curves were derived using the *Model for Urban Stormwater Improvement Conceptualisation* (MUSIC) (CRCCH 2005).

Where local data are available, or if the configuration of the system varies to that described below, a suitable stormwater quality model such as MUSIC should be used in preference to the curves to estimate removal performances. Model results will always supersede the curves.

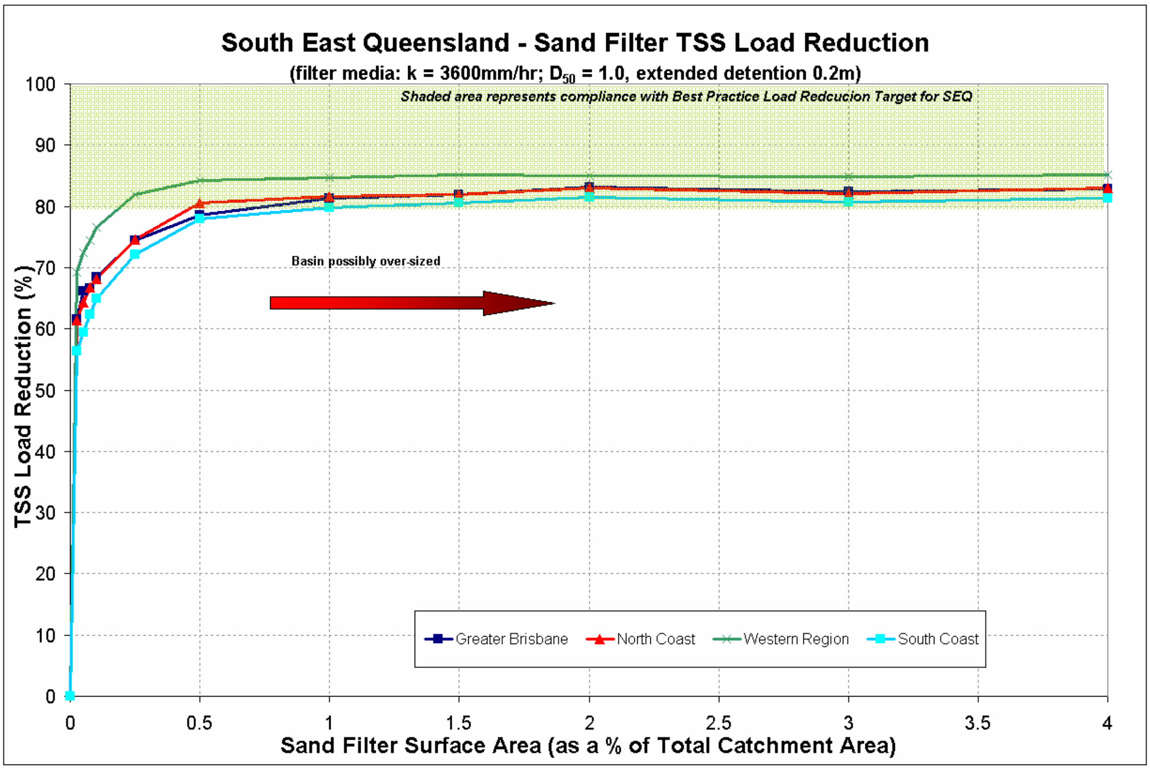


Figure 8-2: Sand Filter TSS Load Reduction

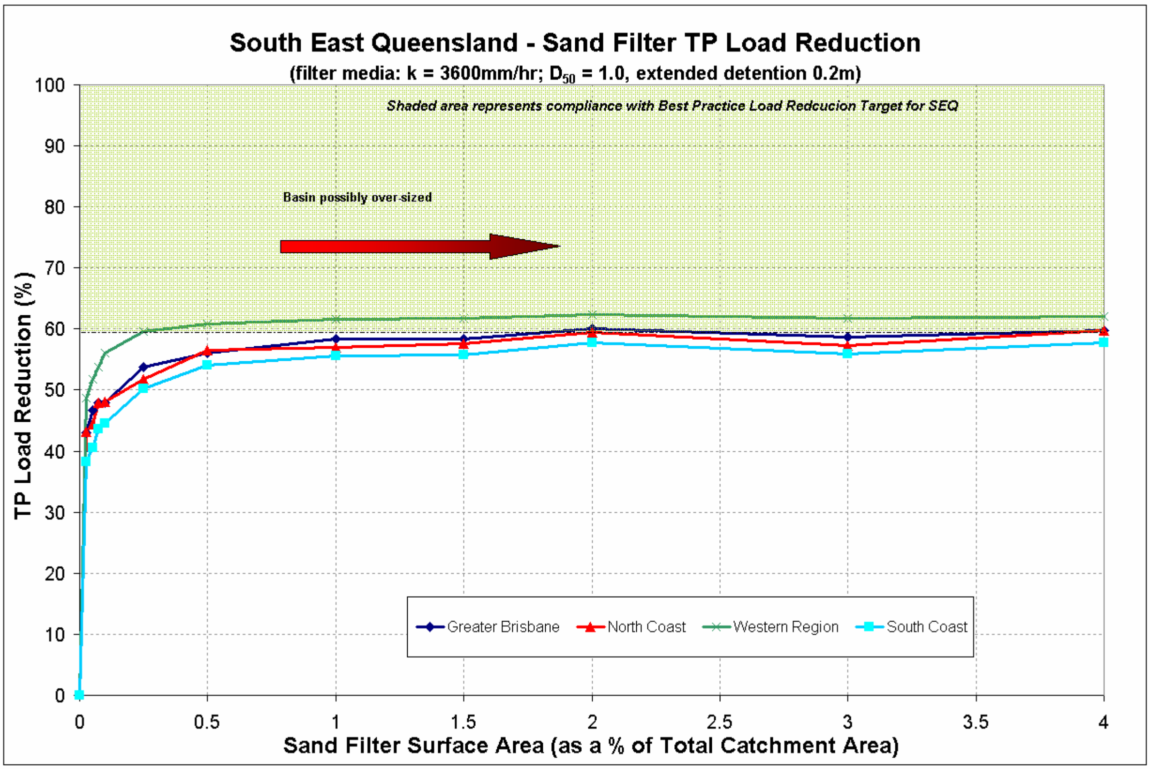


Figure 8-3: Sand Filter TP Load Reduction

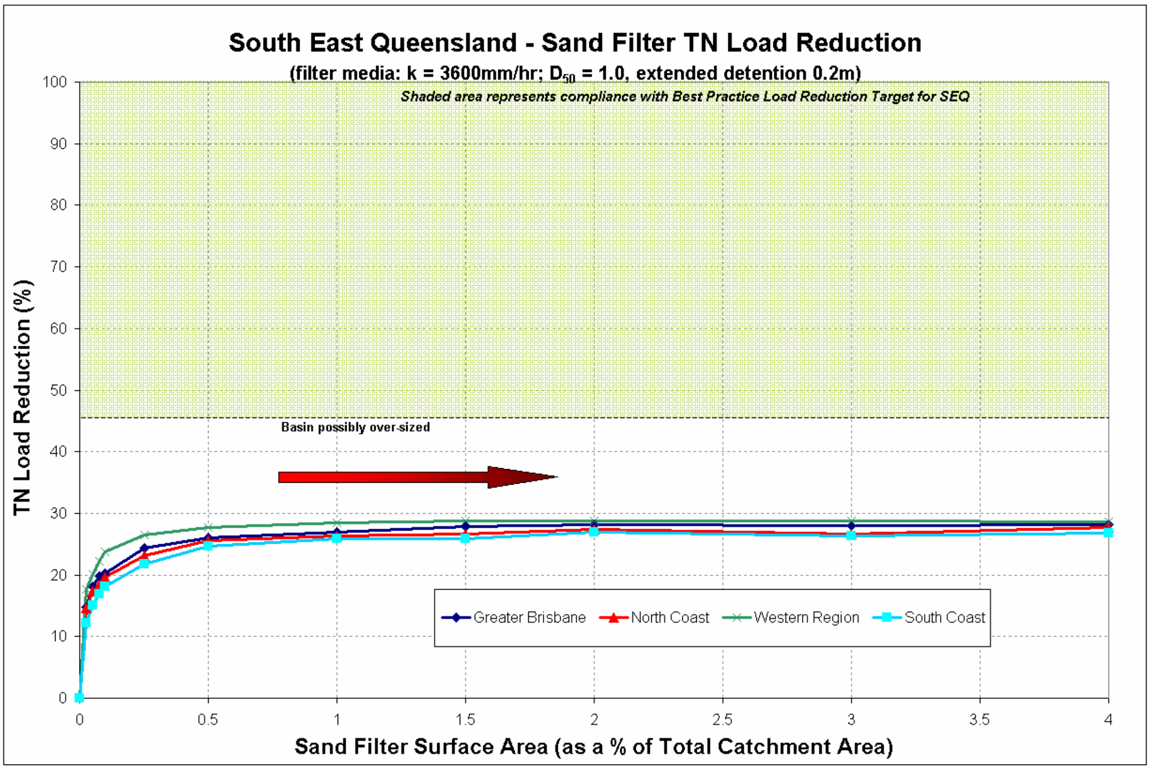


Figure 8-4: Sand Filter TN Load Reduction

8.3.2 Step 2: Determine Design Flows

Three design flows are required for sand filters:

- 'Sedimentation chamber design flow' – this would normally correspond to a 1 year ARI peak discharge as standard practice for sedimentation basins.
- 'Sand filter design flow (or maximum infiltration rate)' – this is the product of the maximum infiltration rate and the surface area of the sand filter, used to determine the minimum discharge capacity of the under-drains to allow the filter media to freely drain.
- 'Sedimentation chamber above design flow' – this is for design of the weir connecting the sand filter to the overflow chamber 'spillway' to allow for bypass of high flows safely around the sand filter chamber. Defined by either:
  - Minor design flow (2 to 10 year ARI) – required for situations where only the minor drainage system is directed to the sedimentation basin. Refer to relevant local government guidelines for the required design event for the minor design flow.
  - Major flood flow (50 to 100 year ARI) – required for situations where the major drainage system discharges into the sedimentation basin.

8.3.2.1 Design Flow Estimation

QUDM identifies the Rational Method as the procedure most commonly used to estimate peak flows from small catchments in Queensland. As sand filters are only recommended for small catchments (e.g. less than 10 hectares), the Rational Method is recognized as an appropriate method to use in the determination of peak design flows.

8.3.2.2 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 does not become a 'choke' in the system.

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

$$Q_{max} = K_{sat} \cdot A \cdot \frac{h_{max} + d}{d} \qquad \text{Equation 8.1}$$

where K_{sat} = hydraulic conductivity of the soil filter (m/s)
A = surface area of the sand filter (m^2)
h_{max} = depth of pondage above the sand filter (m)
d = depth of the filter media (m)

8.3.3 Step 3: Design Sedimentation Chamber

The dimensions of the sedimentation chamber should be sized to retain sediments larger than 125 µm for the sedimentation chamber design flow (typically 1 year ARI peak flow) and to have adequate capacity to retain settled sediment (and gross pollutants) such that the cleanout frequency is a minimum of once per year. A target sediment capture efficiency 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 4). With a sand filter, lower capture efficiencies can be supported because of the maintenance regime of the filter media (inspections and either scraping or removal of the surface of the sand filter twice a year) and particle size range in the sand filter being of a similar order of magnitude as the target sediment size of 125 µm.

During storm events, stormwater in the sedimentation chamber is discharged (via surcharge) over a weir into the sand filter chamber. This weir will have a maximum discharge capacity that is equal to the sand filter design flow.

The overflow weir to the bypass channel is also typically located within the sedimentation chamber. The sizing of the overflow weir is covered in detail in Step 6 (Section 8.3.6).

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 bypass channel. A maximum flow velocity of 0.2 m/s is recommended through the sedimentation chamber before bypass occurs and 0.5 m/s for the overflow design flow rate. Velocities are estimated by dividing the cross section area by the design flow rate.

The reader is referred to Chapter 4 for guidance on the sizing the sedimentation chamber allowing for the recommended 70 % capture efficiency for sediments.

8.3.4 Step 4: Specify the Filter Media Characteristics

Filter media in the sand filter chamber consist of two layers: (1) drainage layer consisting of clean washed river sand or gravel material to encase the perforated under-drains and (2) a sand filtration layer.

8.3.4.1 Filter media

A range of particle sizes 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 being 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 m	2-10 %

This grading is based on TP10 (ARC 2003).

Alternatively, finer material can be used (described below), however this will require more attention to maintenance to ensure the material maintains sufficient hydraulic conductivity and does not become blocked. Inspections should be carried out every 1-6 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 %

8.3.4.2 Drainage Layer

A drainage layer is used to convey treated flows from the base of the filter media layer into the perforated under-drainage system. The particle size of the drainage layer is selected with consideration of the perforated under-drainage system (refer to Section 8.3.5) as the slot sizes in the perforated pipes may determine the minimum drainage layer particle size that will not be washed into the perforated pipes. Coarser material (e.g. fine gravel) must be used for the drainage layer if the slot sizes in the perforated pipes are too large for use of a sand based drainage layer. Otherwise, a clean washed river sand is the preferred drainage layer media. The drainage layer must be a minimum of 200 mm thick.

8.3.5 Step 5: Under-drain Design and Capacity Checks

Treated water that has passed through the filtration media is directed into slotted pipes located within the 'drainage layer' or at the base of the sand filtration layer (when a drainage layer is not required). The maximum spacing of the slotted or perforated collection pipes is 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. Collection pipes are to be a maximum of 100 mm diameter. 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.

8.3.5.1 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.

$$Q_{perf} = B \cdot C_d \cdot A_{perf} \cdot \sqrt{2 \cdot g \cdot h}$$ Equation 8.2

where Q_{perf} = flow through perforations (m^3/s)
B = blockage factor (0.5-0.75)
C_d = orifice discharge coefficient (~0.6)
A = total area of the perforations (m^2)
g = gravity (9.81 m/s^2)
h = depth of water over the collection pipe (m)

The combined discharge capacity of the perforations in the collection pipe(s) must 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.

8.3.5.2 Perforated Pipe Capacity

The discharge capacity of the collection pipe (Q_{pipe}) can be calculated using an orifice flow equation similar to that expressed in Equation 8.2, assuming that the pipe has no blockage restricting the flow (i.e. B = 1). This equation is used in preference to Manning's equation, as the pipe is completely submerged while discharging:

$$Q_{pipe} = C_d \cdot A_{pipe} \sqrt{2 \cdot g \cdot h}$$ Equation 8.3

where Q_{pipe} = flow through pipe(s) (m^3/s)
C_d = orifice discharge coefficient (~0.6)
A = area of the pipe(s) (m^2)
g = gravity (9.81 m/s^2)
h = depth of water over the collection pipe (m)

The capacity of this pipe must exceed the maximum infiltration rate.

8.3.6 Step 6: Size Overflow Weir

The overflow weir is typically located in the sedimentation chamber. The overflow weir must be sized to ensure that it has sufficient capacity to convey the design discharge from the sedimentation chamber (typically 2-10 year ARI peak flow).

When water levels in the sedimentation and sand filter chambers exceed the extended detention depth, water will overflow directly from the sedimentation chamber (bypassing the sand filter) into the overflow/bypass chamber and be conveyed into the downstream drainage system. Water levels in the overflow chamber must remain below ground when operating at the design discharge for the minor stormwater drainage system.

A broad crested weir equation can be used to determine the length of the overflow weir:

$$Q_{weir} = C_w \cdot L \cdot h^{3/2}$$ Equation 8.4

where Q_{weir} = flow rate over weir (m^3/s)
C_w = weir coefficient (~1.7)
L = length of the weir (m)
h = depth of water above the weir (m)

8.3.7 Design Calculation Summary

Below is a design calculation summary sheet for key design elements of sand filters to aid the design process.

SAND FILTER DESIGN CALCULATION SUMMARY

	Calculation Task	CALCULATION SUMMARY Outcome		Check
	Catchment Characteristics			
	Catchment area		Ha	
	Catchment landuse (i.e. residential, commercial etc.)			
	Storm event entering inlet		yr ARI	
	Conceptual Design			
	Sand filter area		m^2	
	Filter media saturated hydraulic conductivity		mm/hr	
	Extended detention depth		mm	
1	**Verify size for treatment**			
	Sand filter area to achieve water quality objectives			
	Total suspended solids (Figure 8-2)		% of catchment	
	Total phosphorus (Figure 8-3)		% of catchment	
	Total nitrogen (Figure 8-4)		% of catchment	
	Sand filter area		m^2	
	Extended detention depth		m	
2	**Determine design flows**			
	'Sedimentation chamber design flow' (1 year ARI)		year ARI	
	'Sedimentation chamber above design flow' (2 to 100 year ARI)		year ARI	
	Time of concentration		minutes	
	(Refer to local Council's Development Guidelines/ QUDM)			
	Identify rainfall intensities			
	'Sedimentation chamber design flow' - $I_{1\ year\ ARI}$		mm/hr	
	'Sedimentation chamber above design flow' - $I_{2\ year\ ARI}$ to $I_{100\ year\ ARI}$		mm/hr	
	Design runoff coefficient			
	'Sedimentation chamber design flow' - $C_{1\ year\ ARI}$			
	'Sedimentation chamber above design flow' - $I_{2\ year\ ARI}$ to $I_{100\ year\ ARI}$			
	Peak design flows			
	'Sedimentation chamber design flow' - 1 year ARI		m^3/s	
	'Sedimentation chamber above design flow' – 2 to 100 year ARI		m^3/s	
	$Q_{infiltration}$		m^3/s	
3	**Design sedimentation chamber**			
	Required surface area?		m^2	
	length x width		m	
	depth		m	
	Design particle size		mm	
	CHECK SCOUR VELOCITY (<0.5 m/s)?		m/s	
	CHECK OVERFLOW CAPACITY?		m^3/s	
4	**Specify sand filter media characteristics**			
	Filter media hydraulic conductivity		mm/hr	
	Filter media depth		mm	
	Drainage layer depth		mm	
	Provided specification for sand media?			
5	**Under-drain design and capacity checks**			
	Flow capacity of filter media		m^3/s	

SAND FILTER DESIGN CALCULATION SUMMARY				
		CALCULATION SUMMARY		
	Calculation Task	Outcome		Check
	Perforations inflow check			
	Pipe diameter		mm	
	Number of pipes			
	Capacity of perforations		m^3/s	
	CHECK PERFORATION CAPACITY > FILTER MEDIA CAPACITY			
	Perforated pipe capacity			
	Pipe capacity		m^3/s	
	CHECK PIPE CAPACITY > FILTER MEDIA CAPACITY			
6	Size overflow weir			
	Design storm for overflow (e.g. 2yr ARI)			
	weir length		m	

8.4 Construction Advice

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

8.4.1 Building Phase Damage

Protection of sand filtration media is very important during the 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 building site runoff. If a sand filter is not protected during the building phase, it is likely to require replacement of the sand filter media. A recommended approach during the building phase is to "block" the weir between the sedimentation chamber and the sand filter chamber so that only the sedimentation chamber is engaged by stormwater flows. Once building is complete and the catchment is stabilised the weir can be re-opened to allow stormwater flows into the sand filter chamber.

8.4.2 Sediment Basin Drainage

When a sediment chamber is designed to drain between storms (so that pollutants are stored in a drained state), blockage of the weep holes can be avoided by constructing a protective sleeve (to protect the holes from debris blockage, e.g. 5 mm 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. 10-15 mm diameter).

8.4.3 Inspection Openings (flushing points) for Perforated Pipes

It is good design practice to have inspection openings (flushing points) at the ends of the perforated pipes. This allows for inspection of sediment build within the under-drainage system and when required an easy access point for flushing out accumulated sediments. Sediment controls downstream should be used when flushing out sediments from the under drainage system to prevent sediments reaching downstream waterways.

8.4.4 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.

8.5 Maintenance Requirements

Maintenance of sand filters is primarily concerned with:

- Regular inspections (1-6 monthly) to inspect the sedimentation chamber and the sand filter media surface, particularly immediately after construction.
- Checking for blockage and clogging.
- Removal of accumulated sediments, litter and debris from the sedimentation chamber.
- Checking to ensure the weep holes (if provided) and overflow weirs are not blocked.

Maintaining the flow through a sand filter relies on 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. 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 in the sedimentation chamber needs to be monitored. Depending on catchment activities (e.g. building phase), sediment deposition can overwhelm the chamber and reduce flow capacities.

Debris removal is an ongoing maintenance function. If not removed, debris can block inlets or outlets, and be unsightly if located in a visible location. Inspection and removal of debris/ litter should be carried out regularly.

8.6 Checking Tools

This section provides a number of checking tools for designers and Council development assessment officers. In addition, Section 8.5.5 provides general advice on the construction of sand filters and key issues to be considered to ensure their successful establishment and operation based on observations from construction projects around Australia.

The following checking tools are provided:

- Design Assessment Checklist
- Construction Inspection Checklist (during and post)
- Operation and Maintenance Inspection Form
- Asset Transfer Checklist (following "on-maintenance" period).

8.6.1 Design Assessment Checklist

The checklist on page 8-19 presents the key design features to be reviewed when assessing the design of a sand filter. These considerations include configuration, safety, maintenance and operational issues that need to be addressed during the design phase. If an item receives an 'N' when reviewing the design, referral is made to the design procedure to determine the impact of the omission or error.

In addition to the checklist, a proposed design is to have all necessary permits for its installations. Council development assessment officers will require that all relevant permits are in place prior to accepting a design.

8.6.2 Construction Checklist

The checklist on page 8-20 presents the key items to be reviewed when inspecting the sand filter during and at the completion of construction. The checklist is to be used by construction site supervisors and the local authority compliance inspectors to ensure all the elements of the sand filter have been constructed in accordance with the design. If an item receives an 'N' in Satisfactory criteria, appropriate actions must be specified and delivered to rectify the construction issues before inspection sign-off is given.

8.6.3 Operation and Maintenance Inspection Form

The form on page 8-21 should be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

8.6.4 Asset transfer checklist

Land ownership and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design is to clearly identify the ultimate asset owner and who is responsible for its maintenance. The local government authority will use the asset transfer checklist on page 8-22 below when the asset is to be transferred to them.

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 using MUSIC?			
INLET ZONE/HYDRAULICS		Y	N
Station selected for IFD appropriate for location?			
Configuration of sediment chamber (aspect, depth and flows) allows settling of particles >125 μm?			
Sediment chamber capacity sufficient for desilting period >=1 year?			
Scour protection provided at inlet?			
Maintenance access allowed for sediment chamber?			
Public access to system prevented?			
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 <1.5 m?			
Drainage layer >200 mm?			
Transition layer provided to prevent clogging of drainage layer?			
FILTER BASIN		Y	N
Maximum ponding depth will not impact on public safety?			
Collection pipes extended to surface of sand to allow inspection and flushing?			
Selected filter media hydraulic conductivity > 10 x hydraulic conductivity of surrounding soil? if not, impermeable liner provided?			
Maintenance access provided to base of filter media (where reach to any part of a basin >6 m)?			
Sand media specification included in design?			
COMMENTS			

SAND FILTER CONSTRUCTION INSPECTION CHECKLIST

Site:		Inspected by:	
		Date:	
		Time:	
Constructed by:		Weather:	
		Contact during visit:	

Items inspected	Checked		Satisfactory		Items inspected	Checked		Satisfactory	
	Y	N	Y	N		Y	N	Y	N
DURING CONSTRUCTION									
A. Preliminary works					C. Sedimentation Chamber				
1. Erosion and sediment control plan adopted					12. Invert level correct				
2. Temporary traffic/safety control measures					13. Ability to freely drain (weep holes)				
3. Location same as plans					D. Structural Components				
4. Site protection from existing flows					14. Location and levels of pits as designed				
B. Earthworks					15. Safety protection provided				
5. Level bed					16. Pipe joints and connections as designed				
6. Side slopes are stable					17. Concrete and reinforcement as designed				
7. Provision of liner (if required)					18. Inlets appropriately installed				
8. Perforated pipe installed as designed					E. Filtration System				
9. Drainage layer media as designed					19. Provision of liner				
10. Sand media specifications checked					20. Adequate maintenance access				
11. Adequate maintenance access					21. Inlet and outlet as designed				
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					7. No surface clogging				
2. Traffic control in place					8. Maintenance access provided				
3. Confirm structural element sizes					9. Construction generated sediment removed				
4. Sand filter media as specified					10. Provision of removed sediment drainage area				
5. Sedimentation chamber freely drains					11. Evidence of litter or excessive debris				
6. Check for uneven settling of sand									

COMMENTS ON INSPECTION

ACTIONS REQUIRED
1.
2.
3.
4.
5.
Inspection officer signature:

SAND FILTER MAINTENANCE CHECKLIST			
Inspection Frequency:	1-6 monthly	Date of Visit:	
Location:			
Description:			
Site Visit by:			

INSPECTION ITEMS	Y	N	ACTION REQUIRED (DETAILS)
Litter within filter area?			
Scour present within sediment chamber or filter?			
Sediment requires removal (record depth, remove if >50%)?			
All structures in satisfactory condition (pits, pipes etc)?			
Traffic damage evident?			
Evidence of dumping (e.g. building waste)?			
Clogging of drainage weep holes or outlet?			
Evidence of ponding (in sedimentation chamber or sand filter)?			
Damage/vandalism to structures present?			
Surface clogging visible?			
Drainage system inspected?			
Removal of fine sediment required?			
COMMENTS			

ASSET TRANSFER CHECKLIST		
Asset Description:		
Asset ID:		
Asset Location:		
Construction by:		
Defects and Liability Period:		
TREATMENT	Y	N
System appears to be working as designed visually?		
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?		
COMMENTS		

8.7 Sand Filter Worked Example

A concrete encased sand filter system is proposed to treat stormwater runoff from a courtyard/ plaza area along the coastal strip of the Gold Coast. The site is nested amongst a number of tall buildings and is to be fully paved as a multi-purpose courtyard. Stormwater runoff from the surrounding building is to be directed to bioretention planter boxes while runoff from this 3500 m^2 courtyard will be directed into an underground sand filter. Provision for overflow from the sand filter into the underground piped drainage system ensures that the site is not subjected to flood inundation for storm events up to the 50 year ARI. The existing stormwater drainage system has sufficient capacity to accommodate the 50 year ARI peak discharge from this relatively small catchment.

Key functions of the sand filter include the following:

- promote the capture of gross pollutants
- promote sedimentation of 70 % of particles larger than 125 μm within the inlet zone for flows up to a 1 year ARI peak discharge
- promote filtration of stormwater following sedimentation pre-treatment through a sand layer
- provide for high flow bypass operation by configuring and designing the bypass chamber.

The concept design suggests that the sand filter system will remove 80 %, 56 % and 26 % of TSS, TP and TN respectively. Therefore additional treatment will be required downstream of the sand filter in order to meet best practice pollutant load reduction discharge standards. The concept design has suggested a required area of the sand filter chamber is 30 m^2, depth of the sand filter 600 mm, saturated hydraulic conductivity of 3600mm/hr and extended detention depth of 0.2 m. Larger sand filter treatment areas did not provide any additional treatment benefits when modelled in MUSIC. 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 (overflow chamber). Flows in excess of the 0.2 m extended detention depth would overflow and discharge into the underground stormwater pipe and bypass the sand filter.

Design Objectives

Design objectives include the following:

- Sand filter to consist of three chambers: a sedimentation (and gross pollutant trapping) chamber, a sand filter chamber and an overflow chamber.
- The sedimentation chamber will be designed to capture particles larger than 125 μm for flows up to the peak 1 year ARI design flow with a capture efficiency of 70 %. Flows up to the Maximum Infiltration Rate through the sand filter will be conveyed from the sedimentation chamber to the sand filter chamber by a series of "slot" weirs.
- 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 peak flow associated with the Maximum Infiltration Rate through the sand filter media can enter the pipes and that the pipes have sufficient conveyance capacity.
- The overflow weir (located in the sedimentation chamber) and the overflow chamber will be designed to receive and convey flows up to the 50 year ARI peak discharge (i.e. the Major Storm).
- The sedimentation chamber will retain sediment and gross pollutants in a dry state and have sufficient storage capacity to limit sediment cleanout frequency to once a year.
- Inlet/ outlet pipes to be sized to convey the 50 year ARI peak discharge.

Site Characteristics

The site characteristics are summarised as follows:

catchment area	3,500 m^2 (70 m x 50 m)
land use/ surface type	paved courtyard
fraction impervious	0.90
overland flow travel path	50 m
overland flow slope	6.0 %

8.7.1 Step 1: Confirm Treatment Performance of Concept Design

The nominated area of the sand filter from the conceptual design is 30 m^2 (i.e. approx 1% of contributing catchment area). This treatment area is checked using the charts in Figure 8-2 to Figure 8-4 which confirms the appropriateness of the concept design.

8.7.2 Step 2: Estimating Design Flows

With a small catchment (in this case 3,500 m^2), the Rational Method is considered an appropriate approach to estimate the design storm peak flow rates. The steps in this calculation follow below.

<u>Time of Concentration (t_c)</u>

Approach:

The time of concentration is estimated assuming overland flow across the paved courtyard. From procedures documented in QUDM (DPI et al. 1992) and the local Council's development guidelines, the overland sheet flow component should be limited to 50 m in length and determined using the Kinematic Wave Equation:

$$t = 6.94\ (L.n^*)^{0.6}/I^{0.4}\ S^{0.3}$$

where: t = overland sheet flow travel time (mins)

L = overland sheet flow path length (m)

n^* = surface roughness/retardance coefficient

I = rainfall intensity (mm/hr)

S = slope of surface (m/m)

In urban areas, QUDM notes that sheet flow will typically be between 20 to 50 m, after which the flow will become concentrated against fences, gardens or walls or intercepted by minor channel or piped drainage (DPI et al. 1992). Therefore when calculating remaining overland flow travel times, it is recommended that stream velocities in Table 5.05.4 of QUDM be used.

Assuming: Predominant slope = 6%

Overland sheet flow = 50m

Flow path is predominately paved, with a typical n* = 0.013 (QUDM)

10 year ARI:

$$t_{sheet\ flow} = 6.94\ (50 \times 0.013)^{0.6}/(I^{0.4} \times 0.06^{0.3})$$

$$= < 5 \text{ mins}$$

Therefore adopt a 5 minute time of concentration in line with the local Council's guidelines. Iterations will usually need to be repeated until $t_{sheet\ flow}$ matches 10 year ARI rainfall intensity on the IFD chart for that duration. However in this case the time of concentration is very low for all ARIs, and therefore a 5 minute time of concentration is adopted for all design events. Note that IFD data will need to be determined in accordance with the local Council's development guidelines.

<u>Design Runoff Coefficient</u>

Runoff Coefficients

C_{10} = 0.95 (commercial – refer to local Council development guidelines)

	C Runoff			
ARI	1	2	10	50
QUDM Factor	0.8	0.85	1	1.15
C_{ARI}	0.76	0.81	0.95	1.00

Catchment Area A = 3,500 m^2 (0.35ha)

Rainfall Intensities (IFD for Surfers Paradise)

t_c = 5 mins

I_1 = 122.8 mm/hr

I_{50} = 239.3 mm/hr

Rational Method

$Q = CIA/360$

$Q_{1yr\ ARI}$ = 0.091 m^3/s

$Q_{50yr\ ARI}$ = 0.223 m^3/s

Maximum Infiltration Rate

The maximum infiltration rate (Q_{max}) through the sand filter is computed using Equation 8.1:

$$Q_{max} = K_{sat} \cdot A \cdot \frac{h_{max} + d}{d} \qquad = 0.04\ \text{m}^3/\text{s}$$

where K is the hydraulic conductivity of coarse sand = 3600 m/s (Engineers Australia 2003)

A is the surface area of the sand filter = 30 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

Summary of Design Flows:

Q_1 = 0.091 m^3/s;

Q_{50} = 0.223 m^3/s

Maximum Infiltration Rate = 0.04 m^3/s

8.7.3 Step 3: Design Sedimentation Chamber

The sedimentation chamber is to be sized to remove the 125 µm particles for the peak 1 year ARI flow.

Pollutant removal is estimated using Equation 4.1 (see Chapter 4):

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

A notional aspect ratio of 1 (W) to 2 (L) is adopted. From Figure 4.4 in Chapter 4 (reproduced below as Figure 8-5), the hydraulic efficiency (λ) is estimated to be 0.3. The turbulence factor (n) is computed from Equation 4.2 to be 1.4.

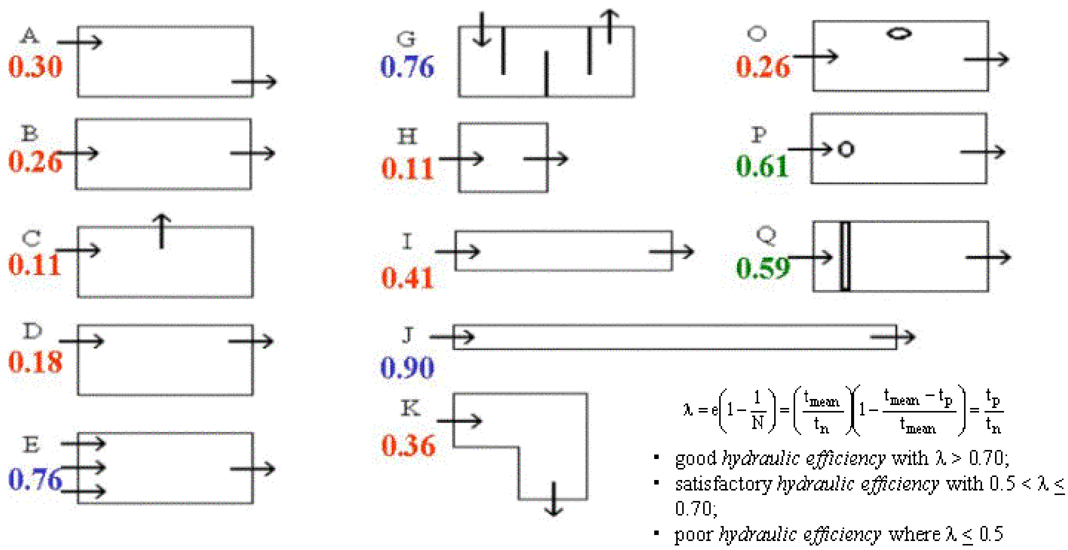


Figure 8-5: Hydraulic Efficiency (λ) Ranges (Figure 4.4 reproduced from Chapter 4)

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 8.6.1) and a notional permanent pool depth of 0.6 m (equal to the depth of the sand filter) has been adopted:

d_p = 0.6 m

d^* = 0.6 m

d_e = 0.20 m

v_s = 0.011 m/s for 125 µm particles (settling velocity)

Q = Design flow rate = 0.091 m^3/s

The required sedimentation basin area to achieve target sediment (125 µm) capture efficiency of 70% is 16 m^2. With a W to L ratio of 1:2, the notional dimensions of the basin are 3 m x 5.5 m = 16.5 m^2. Baffles could be incorporated into the sedimentation to improve its hydraulic efficiency and, subsequently, it's sediment removal efficiency. The proposed configuration, however, does still achieve the target sediment capture efficiency of 70%.

The available sediment storage (V_s) is 16.5 x 0.6 = 9.9 m^3. Cleanout is to be scheduled when the storage is half full. Using a sediment discharge rate (Q_{sed}) of 1.6 m^3/Ha/yr (Engineers Australia 2003) and a catchment area (A_c) of 0.35 ha, we have:

Frequency of basin de-silting $= \dfrac{50\% \times V_s}{A_c x\ Q_{sed} \times \text{capture efficiency}}$

$$= \frac{0.5 \times 9.9}{0.35 \times 1.6 \times 0.7} = 12.6 \text{ years} > 1 \text{ year} \rightarrow \text{OK}$$

During the 50 year ARI storm, peak discharge through the sedimentation chamber will be 0.223 m^3/s with flow depth of 0.8 m and a chamber width of 3 m. It is necessary to check that flow velocity does not re-suspend deposited sediment of 125 µm or larger (< 0.5 m/s).

The mean velocity in the chamber is calculated as follows:

v_{50} = 0.223/(3 x 0.8) = 0.1 m/s → OK

The weir connection between the sedimentation chamber and the sand filter chamber should have a discharge capacity greater than the Maximum Infiltration Rate (= 0.04 m^3/s) and can be calculated using Equation 8.4 as follows:

$$Q_{conn} = C_w \cdot L \cdot h^{3/2}$$

where Q_{conn} = flow rate through connection (m^3/s)

C_w = weir coefficient (assume = 1.7 for a broad crested weir)

h = depth of water above the weir = 0.2 m (extended detention in sedimentation chamber)

L = length of the weir (m)

The discharge capacity calculated from the above equation for a weir length of 0.3m is 0.045 m^3/s > 0.04 m^3/s → OK. (suggestion: creating 3 of 0.1m wide "slots" @ 1m spacings to assist even distribution of flows onto the surface of the sand filter media)

Final Sedimentation Chamber Specifications:

Sedimentation Chamber Plan Area= 16.5 m^2

Width = 3 m; Length = 5.5 m

Total weir length for connection to sand filter chamber (minimum) = 0.3 m (provided as three 0.1 m wide "slots")

Depth of chamber invert below weir crest = 0.6 m

Depth of Extended Detention (d_e) = 0.2m

8.7.3.1 Sand Filter Chamber

Dimensions

With the length of sedimentation chamber being 5.5 m, the dimension of the sand filter chamber is determined to be 5.5 m x 6.0 m, giving the required treatment area of approximately 30m^2 (i.e. matches the treatment area provided for in the concept design layout).

8.7.4 Step 4: Specify Filter Media Characteristics

Sand filter layer is to consist of sand/ coarse sand material with a typical particle size distribution as provided below:

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

It is expected that a sand filter media with this particle size distribution will have a saturated hydraulic conductivity in the order 3600mm/hr.

Drainage layer to be 200 mm deep and consist of 5 mm gravel.

8.7.5 Step 5: Under-drain Design

8.7.5.1 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 perforated pipe, the total number of slots = 2100/(1.5 x 7.5) = 186 per metre.

Discharge capacity of each slot can be calculated using the orifice flow equation (Equation 8.2):

$$Q_{perf} = C_d \cdot A \cdot \sqrt{2 \cdot g \cdot h}$$

where Q_{perf} = flow through perforations (2.67 x 10^{-5} m^3/s)

h = hydraulic head above the slotted pipe (0.80 m)

C_d = orifice discharge coefficient (~0.6)

The inflow capacity of the slotted pipe is thus 2.67 x 10^{-5} x 186 ~ 5 x 10^3 m^3/s/m-length.

Adopt a blockage factor of 0.5 giving the inlet capacity of each slotted pipe to be 2.5 x 10 m/s/m-length.
Maximum infiltration rate is 0.04m^3/s.
The minimum length of slotted pipe required is $L_{slotted\ pipe}$ = 0.04/2.5x10^{-3} = 16 m

With a maximum spacing of 1.5 m centre to centre, this equates to 4 lengths of 5.5 m at 1.5 m spacing (0.75 m from the edges). Therefore a total pipe length of 22m is used. The total flow through the perforations can now be calculated:

Q_{perf} = 22m x 2.5 x 10^{-3} m^3/s/m

= 0.055 m^3/s

Check total flow through perforations 0.055 m^3/s > max flow through filtration media 0.04 m^3/s ➔ OK

Four (4) 100 mm diameter slotted pipes (5.5 m lengths each) at 1.5 m spacing are required.

8.7.5.2 Perforated Pipe Capacity

The diameter of the slotted pipe is 100 mm. The discharge capacity of the collection pipe is calculated using an orifice flow equation (Equation 8.3):

$$Q_{pipe} = C_d \cdot A_{pipe} \sqrt{2 \cdot g \cdot h}$$

where Q_{pipe} = flow through pipe(s) = (0.019 m^3/s)

C_d = orifice discharge coefficient (~0.6)

A = area of the pipe(s) (4 pipes x 0.00785 m^2 per pipe)

g = gravity (9.81 m/s^2)

h = depth of water over the collection pipe (0.8 m)

Total discharge capacity (4 pipes) = 0.07 m^3/s > maximum infiltration rate of 0.04 m^3/s → OK

Combined slotted pipe discharge capacity = 0.07 m^3/s which exceeds the maximum infiltration rate.

8.7.6 Step 7: Size Overflow Weir

The width of the sedimentation chamber has been selected to be 3 m. An overflow 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 2.5 m length needs to convey flows up to the 50 year ARI peak discharging into the overflow chamber.

Calculate the depth of water above the weir resulting from conveying the 50 year ARI peak discharge through a 2.5 m length weir by rearranging Equation 8.4:

$$h = \left(\frac{Q_{weir}}{C_w \cdot L}\right)^{2/3} = 0.13 \text{ m, say } 0.15 \text{ m}$$

where Q_{weir} = design discharge = 0.233 m^3/s

C_w = weir coefficient (~1.7)

L = length of the weir (m) = 3m

h = depth of water above the weir (m)

With a depth above the weir of 0.15m, the discharge capacity of the overflow weir is 0.3 m^3/s > 50-year ARI peak flow of 0.23 m^3/s.

Crest of overflow weir = 0.2 m above surface of sand filter

Length of overflow weir = 3 m

50 year ARI weir flow depth = 0.15 m

Roof of facility to be at least 0.35 m above sand filter surface

8.7.7 Design Calculation Summary

The table below shows the calculation summary for the worked example.

SAND FILTER DESIGN CALCULATION SUMMARY

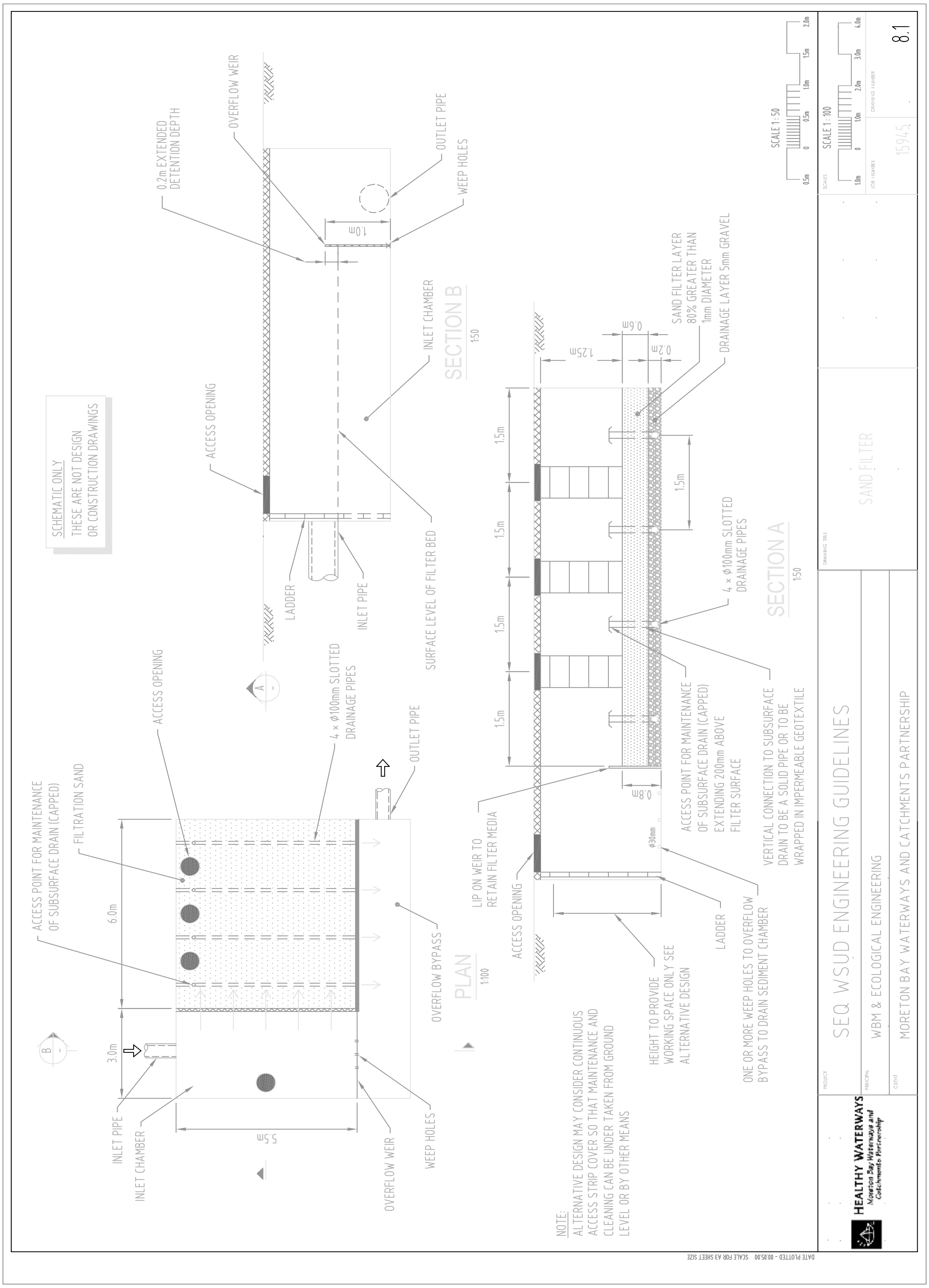
	Calculation Task	Outcome		Check
		CALCULATION SUMMARY		
	Catchment Characteristics			
	Catchment area	0.35	ha	✓
	Catchment landuse (i.e. residential, commercial etc.)	0.9 impervious		
	Storm event entering inlet	50	yr ARI	
	Conceptual Design			
	Sand filter area	30	m^2	✓
	Filter media saturated hydraulic conductivity	3600	mm/hr	
	Extended detention depth	0.2	mm	
1	**Confirm Treatment Performance of Concept Design**			
	Sand filter area to achieve water quality objectives			
	Total suspended solids (Figure 8-2)	80	% of catchment	✓
	Total phosphorus (Figure 8-3)	56	% of catchment	
	Total nitrogen (Figure 8-4)	26	% of catchment	
	Sand filter area	30	m^2	✓
	Extended detention depth	0.2	m	
2	**Determine design flows**			
	'Sedimentation chamber design flow' (1 year ARI)	1	year ARI	
	'Sedimentation chamber above design flow' (2 to 100 year ARI)	50	year ARI	
	Time of concentration	5	minutes	✓
	(Refer to local Council's Development Guidelines/ QUDM)			
	Identify rainfall intensities			
	'Sedimentation chamber design flow' - $I_{1\ year\ ARI}$	122.8	mm/hr	✓
	'Sedimentation chamber above design flow' - $I_{2\ year\ ARI}$ to $I_{100\ year\ ARI}$	239.3	mm/hr	
	Design runoff coefficient			
	'Sedimentation chamber design flow' - $C_{1\ year\ ARI}$	0.76		✓
	'Sedimentation chamber above design flow' - $I_{2\ year\ ARI}$ to $I_{100\ year\ ARI}$	1.00		
	Peak design flows			
	'Sedimentation chamber design flow' - 1 year ARI	0.091	m^3/s	✓
	'Sedimentation chamber above design flow' – 2 to 100 year ARI	0.233	m^3/s	
	$Q_{infiltration}$	0.04	m^3/s	
3	**Design sedimentation chamber**			
	Required surface area?	16	m^2	✓
	length x width	3.0 x 5.5	m	
	depth	0.6	m	
	Design particle size	0.125	mm	
	CHECK SCOUR VELOCITY (<0.5 m/s)?	0.1	m/s	
	CHECK OVERFLOW CAPACITY?	0.25	m^3/s	
4	**Specify sand filter media characteristics**			
	Filter media hydraulic conductivity	3600	mm/hr	✓
	Filter media depth	0.6	mm	
	Drainage layer depth	0.2	mm	
	Provided specification for sand media?	Yes		

	SAND FILTER DESIGN CALCULATION SUMMARY			
		CALCULATION SUMMARY		
	Calculation Task	Outcome		Check
5	Under-drain design and capacity checks			
	Flow capacity of filter media	0.04	m^3/s	✓
	Perforations inflow check			
	Pipe diameter	100	mm	✓
	Number of pipes	4		
	Capacity of perforations	0.055	m^3/s	
	CHECK PERFORATION CAPACITY > FILTER MEDIA CAPACITY	Yes		
	Perforated pipe capacity			
	Pipe capacity	0.07	m^3/s	✓
	CHECK PIPE CAPACITY > FILTER MEDIA CAPACITY	Yes		
6	Size overflow weir			
	Design storm for overflow (e.g. 2yr ARI)	50 Year		✓
	weir length	3	m	

8.7.8 Worked Example Drawings

Drawing 8.1 details the layout of the sand filter designed in the worked example.

Drawing 8.1 Sand Filter



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Engineers Australia 2003, *Australian Runoff Quality*, Engineers Australia, ACT,

http://www.arq.org.au/

Institution of Engineers Australia 1997, *Australian Rainfall and Runoff – A guide to flood estimation*, Institution of Engineers Australia, ACT

[1] At the time of preparation of these guidelines, QUDM was under review and a significantly revised edition is expected to be released in 2006. These guidelines refer to and use calculations specified in the existing QUDM document, however the revised version of QUDM should be used as the appropriate reference document. It should be noted by users of this guideline that the structure and content of QUDM will change, and as such, the references to calculations and/or specific sections of QUDM may no longer be correct. Users of this guideline should utilise and adopt the relevant sections and/or calculations of the revised QUDM guideline.

Chapter 9 Aquifer Storage and Recovery

9.1 Introduction **9-2**

9.2 Components of a Stormwater ASR System **9-2**

9.3 Aquifer Selection **9-3**

9.4 Treatment and Pollution Control **9-4**

9.4.1 Quality of Water for Injection and Recovery 9-4
9.4.2 Knowledge of Pollutant Sources in the Catchment Upstream 9-4
9.4.3 Pretreatment Prior to Injection 9-5
9.4.4 Injection Shutdown System 9-5
9.4.5 Maintenance and Contingency Plans 9-5
9.4.6 Recovered Water Post-Treatment 9-5
9.4.7 Construction of Injection Wells 9-5
9.4.8 Groundwater Attenuation Zones 9-6

9.5 Domestic Scale ASR **9-6**

9.6 Additional Information **9-6**

9.7 References **9-7**

9.1 Introduction

Aquifer storage and recovery (ASR) is a means of introducing recycled water into underground aquifers (via direct injection (i.e. pumping) or gravity) for subsequent extraction and reuse. It can be a low cost water storage alternative compared to surface storages and can minimise loss of water due to evaporation.

The overriding consideration for introducing recycled water to aquifers is to ensure there is no resulting deterioration of groundwater quality (EPA Qld 2004) and that the beneficial uses of an aquifer are protected. The level of treatment of recycled water prior to injection to the aquifer is dependant on the quality of the groundwater and its current uses.

Stormwater ASR systems operate by storing excess treated stormwater flows from urban catchments during wet periods and then subsequent extraction for reuse during drier periods. Urban stormwater must be treated before injection to an aquifer and in most instances, the treatment elements described in these Guidelines (configured into an appropriate 'treatment train') will provide sufficient treatment to protect an aquifer.

The viability of an ASR scheme is dependant on local hydrology, the underlying geology of an area and the presence and nature of aquifers. There is a range of aquifer types that can accommodate an ASR scheme, including fractured unconfined rock and confined sand, and gravel aquifers. In addition, it may be possible to construct an aquifer if the economics allow. Detailed geological investigations are required to establish the feasibility of any ASR scheme. This chapter provides an overview of the main elements of a system and directs readers to more specific guidance documents.

The broad requirements of ASR systems include:

- protecting or improving groundwater quality where ASR is practiced
- ensuring 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 and environmental flow requirements.

In addition to the broad requirements listed above, appropriate approval from the relevant local authority, the Environmental Protection Agency (EPA) and Department of Natural Resources and Mines (DNRM) may also be required to divert stormwater, install treatment measures and to inject and extract water from an aquifer. A thorough investigation of required permits should be undertaken as part of a conceptual design of an ASR system.

Where the aquifer may be used for extraction of potable water, recycled water must be of Class A+ (EPA Qld 2004). Where there is low risk of ingestion by humans, Class A standard would be appropriate. While the *Queensland Guidelines for the Safe Use of Recycled Water* (EPA Qld 2004) apply to recycled water from a number of sources (including wastewater), this chapter presents design considerations for stormwater ASR systems only.

The following information has been adapted from the *Code of Practice for Aquifer Storage and Recovery* (EPA SA 2004) with the permission of the author, to provide an overview of the main components of an ASR system.

9.2 Components of a Stormwater ASR System

An ASR scheme that harvests stormwater typically contains the following structural elements:

- 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 constructed 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 spillway or overflow structure incorporated into the wetland or detention storage for flows to bypass the injection system

- well(s) into which water is injected into an aquifer (may require extraction equipment for periodic purging (with scour valve))
- 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 of water injected and extracted
- systems to monitor the quality of injected water, groundwater and recovered water
- water quality sampling points on injection and recovery lines
- a control system to shut down injection in the event of unfavourable conditions.

Figure 9-1 presents a schematic of the major elements of an ASR scheme.

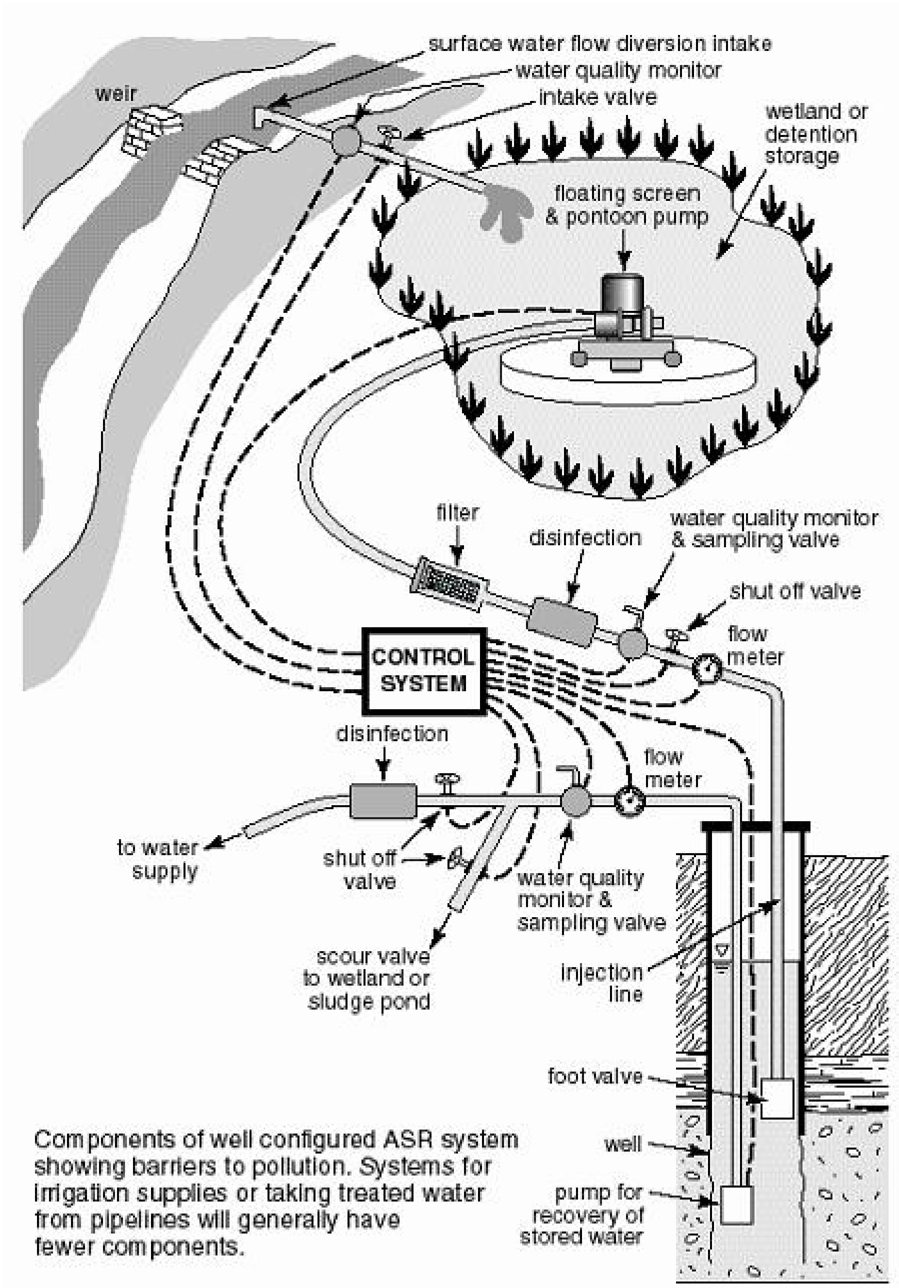

Figure 9-1: Components of a Well Configured ASR System

(Source: Dillon et al 2000 in DWLBC 2002)

9.3 Aquifer Selection

Factors to consider when choosing a suitable aquifer include:

- environmental values of an aquifer (e.g. high quality groundwater may exclude the use of an aquifer for ASR)
- an aquifer may already be providing beneficial uses to others and the quality and flow requirements of these users must not be compromised
- sufficient permeability of a receiving aquifer
- if the salinity of aquifer water is greater than injection water, then the salinity concentration will influence the viability of recovering water from the aquifer
- possible damage to confining layers due to pressure increases
- adverse effects of reduced pressure on other groundwater users
- aquifer mineral dissolution, if any, and potential for well aquitard collapse.

9.4 Treatment 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 existing quality of the groundwater and the beneficial uses associated with the groundwater. In accordance with the *Environmental Protection (Water) Policy 1997*, the primary considerations when introducing stormwater to an aquifer include:

- environmental values of the aquifer for other users
- existing water quality
- cumulative effect of the proposal with other known releases to the aquifer.

The following subsections provide a brief description of the issues to be considered when assessing the treatment and pollution control requirements of a stormwater ASR scheme.

9.4.1 Quality of Water for Injection and Recovery

The quality of water that can be injected into an aquifer should be determined through assessment of designated environmental values and beneficial uses of an aquifer, and subsequent discussion with the relevant local authority and relevant referral agencies (e.g. EPA and DNRM).

Designated environmental values of aquifer water, such as raw water for drinking, non-potable use, stock water, irrigation, ecosystem support and groundwater ecology, can be determined from:

- ambient groundwater quality, with reference to the National Water Quality Management Strategy (*Australian Drinking Water Guidelines* (NHMRC & NRMMC 2004)*; Australia & New Zealand Guidelines for Fresh and Marine Water Quality* (ANZECC & ARMCANZ 2000))

- local historical and continuing uses of those aquifers.

Once environmental values of the aquifer have been established, stormwater quality treatment requirements can be derived through discussion with the relevant local authority and relevant referral agencies (i.e. EPA and DNRM). These are intended to preserve or potentially improve existing groundwater quality.

9.4.2 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 treatment requirements for water detained for injection (see Section 10.4.1). An evaluation of the pollutants that may be present within injected water needs to be carried out on a catchment basis. Pollutants will vary according to whether the catchment drains residential, industrial, rural or a combination of any of these land use types.

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 (e.g. not diverting flow during large floods when treatment systems are often bypassed). Knowledge of the potential pollutant profile helps to define water quality sampling and analysis costs when determining the viability of an ASR project (for example, if there are any specific industrial activities upstream that contribute particular stormwater pollutants such as hydrocarbons).

9.4.3 Pretreatment Prior to Injection

Many of the treatment measures described in earlier chapters of these Guidelines are suitable as pretreatments for stormwater 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 stormwater treatment measures with large storages (e.g. wetlands) is that they offer a dilution effect. Should an isolated pollution event occur, this dilution effect reduces the risk of aquifer contamination.

9.4.4 Injection Shutdown System

Controls need to 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.

9.4.5 Maintenance and Contingency Plans

Protection of treatment and detention systems from contamination is a necessary part of designing an ASR system. 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 storage ponds.

Contingency plans should be developed to cater for the possibility of contaminated water being inadvertently injected into an aquifer. These include how to determine the duration of recovery pumping required (to extract contaminated water), sampling intervals required and how to manage recovered water.

9.4.6 Recovered Water Post-Treatment

Where recovered water is intended for drinking water supplies, further treatment standards (e.g. using ultraviolet disinfection) will be required to meet drinking water standards. For other forms of supply, such as irrigation via drippers, it may be necessary to insert a cartridge filter in the supply line to remove fine suspended solids. The extent of further treatment will depend on the intended end use and a fit-for-purpose approach should be adopted in accordance with EPA Queensland (2004).

9.4.7 Construction of Injection Wells

During and following construction, injection/ extraction wells must be purged for a sufficient period to remove poor quality water that may have been caused by the construction process. This water is usually high in fine sediment and will be unsuitable for disposal to a surface water body or a watercourse. It may potentially be used on site for irrigation, discharged to sewer (with the approval of the relevant authority), or returned to a treatment system.

9.4.8 Groundwater Attenuation Zones

In some cases, the impact of certain ground water pollutants can be diminished over time because of natural processes within an aquifer. Chemical, physical and microbiological processes can occur to ameliorate the harm or potential harm caused by these pollutants.

9.5 Domestic Scale ASR

It is possible to install a stormwater ASR scheme at 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 a number of additional design constraints exist.

Domestic scale ASR in shallow aquifers must not be undertaken in locations where the following apply:

- water tables are shallow (less than 5 m)
- 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.

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 particulate organics (i.e. leaves) and other gross pollutants. Runoff from paved areas must not be admitted unless it 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 injection well catchment and strategies devised to ensure these are excluded, or are treated and removed before water enters the well.

Aquifer pressure must at all times be below ground level. To achieve this, injection should be by gravity drainage, rather than by using a pressurised injection system, and there should be an overflow facility (e.g. to a garden area or to a stormwater drainage system) where excess water discharges to.

9.6 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). Some relevant guides and websites for further information are listed below.

EPA SA (Environment Protection Authority South Australia) 2004, *Code of Practice for Aquifer Storage and Recovery,* EPA SA, 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.

Aquifer Storage Recovery: http://www.asrforum.com/

International Association of Hydrogeologists — Managing Aquifer Recharge (IAH–MAR): www.iah.org/recharge/

Environmental Protection Agency (regarding water quality and licensing requirements): http://www.epa.qld.gov.au/

Department of Natural Resources and Mines (regarding water quantity and licensing requirements): www.nrm.qld.gov.au

9.7 References

ANZECC & ARMCANZ (Australian and New Zealand Environment and Conservation Council & Agriculture and Resource Management Council of Australia and New Zealand) 2000, *Australia & New Zealand Guidelines for Fresh and Marine Water Quality*, ANZECC & ARMCANZ, ACT

DWLBC (Department of Water, Land and Biodiversity Conservation – CSIRO Land and Water) 2002, *Aquifer Storage and Recovery: Future Directions for South Australia*, prepared by Russell Martin (DWLBC) and Peter Dillon (CSIRO) for DWLBC, SA

EPA Qld (Environmental Protection Agency Queensland) 2004, Queensland Guidelines for the Safe Use of Recycled Water - Public Consultation Draft, EPA Qld

EPA SA (South Australia) 2004, *Code of Practice for Aquifer Storage and Recovery*, EPA SA, 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, SA

NHMRC & NRMMC (National Health and Medical Research Council & Natural Resource Management Ministerial Council) 2004, *Australian Drinking Water Guidelines 2004*, NHMRC & NRMMC,

Appendix A Plant Selection for WSUD Systems

A.1 Introduction **A-2**

A.2 Swales (and Buffer Strips), Bioretention Swales and Bioretention Basins **A-2**

A.2.1 Required Plant Characteristics A-2

A.2.2 Plant Species Selection A-3

A.2.3 Vegetation Establishment and Maintenance A-4

A.3 Wetlands and Sedimentation Basins **A-8**

A.3.1 Required Plant Characteristics A-8

A.3.2 Plant Species Selection A-9

A.3.3 Vegetation Establishment and Maintenance A-9

A.4 References **A-13**

A.1 Introduction

This chapter provides guidance on selecting appropriate plant species for Water Sensitive Urban Design (WSUD) systems where the plants have a functional role in stormwater treatment and/ or erosion protection. Selecting suitable plant species is critical to the long term functional performance and structural integrity of WSUD systems. Maintenance costs can also be reduced by careful selection of plant species and by adopting suitably high planting densities. A list of recommended plant species for various WSUD systems, including appropriate planting densities, is provided in the following tables:

Table A-1 and Table A-2:

- Swales (incorporating Buffer Strips)
- Bioretention Swales
- Bioretention Basins

Table A-3 and Table A-4:

- Sedimentation Basins
- Wetlands

The plant species lists in Table A-1 and Table A-2 are not exhaustive and other plants may be used provided their physiological and structural characteristics match the characteristics of the plant species listed in the tables.

Non-indigenous natives and exotics should only be considered when there is a specific landscape need and the species has the appropriate growth form and habit. If non-indigenous natives and exotics are chosen, careful consideration should be given to the potential impacts on downstream receiving ecosystems. Species (including natives) that have the potential to become invasive weeds should be avoided.

A.2 Swales (and Buffer Strips), Bioretention Swales and Bioretention Basins

A.2.1 Required Plant Characteristics

Planting for bioretention basin elements may consist of up to three vegetation types:

- Groundcovers for stormwater treatment and erosion protection
- Shrubbery for screening, glare reduction and character
- Trees for shading, character and other landscape values.

For specific guidance on plant species the designer is initially directed to relevant guidelines provided by the local authority. In the absence of local guidance the designer can refer to the following sections and Table A-1 and Table A-2 which outlines plant species suitable for SEQ.

A.2.1.1 Groundcovers

The plant (groundcover) species listed in Table A-1 have been specifically selected, based on their life histories and physiological and structural characteristics, to meet the functional requirements of swales, buffer strips and bioretention systems (i.e. bioretention swales and bioretention basins). It should be noted that bioretention systems are designed to drain between events either through exfiltration to in-situ soils or through subsurface drainage placed in the base of the system. Some bioretention systems may be lined with geofabric and in this case efficient subsurface drainage is always provided within the lining to maintain free draining aerobic conditions in the filter media. Plant species selected for bioretention systems must therefore be able to tolerate these free draining conditions which result in long dry periods punctuated by very short periods of temporary inundation. Suitable plants species are listed in Table A-1. Other species can be used provided they are tolerant of the filter media conditions and have the required features to fulfill the functional roles of the WSUD element. In general, the plant species in Table A-1 have the following features:

- They are able to tolerate short periods of inundation punctuated by longer dry periods. For bioretention systems these dry periods may be reasonably severe due to the free draining nature (relatively low water holding capacity) of bioretention filter media
- They generally have spreading rather than clumped growth forms.
- They are perennial rather than annual.
- They have deep, fibrous root systems.
- Groundcover plants can be turf, prostrate or tufted.
- Prostrate species would typically be low mat forming stoloniferous or rhizomatous plants.
- Tufted species would typically be rhizomatous plants with simple vertical leaves

Most of the groundcover listed in Table A-1 are widespread, occurring throughout south-east Queensland. However, alternative locally occurring species that display the required features may be selected to tailor the species list to match the native vegetation associations of the area and to compliment surrounding vegetation communities. Please refer to the local authority for further guidance in this regard.

A.2.1.2 Shrubs and Trees

Shrubs and trees are not a functional requirement within swales, bioretention swales or bioretention basins, but can be integrated to provide amenity, character and landscape value. Planting trees and shrubs in bioretention systems requires the filter media to have a minimum depth of 800mm to avoid root interference with the perforated subsurface drainage pipes. They must also be accompanied by densely planted shade tolerant groundcover species with the characteristics outlined above. Trees and shrubs are to be managed so that the ground cover layer is not out-competed. To avoid over-shading, trees and shrubs should be planted at low densities. Periodic thinning of the upper vegetation layers may also be required. In general, tree and shrub species that can be incorporated into bioretention systems have the following general features:

- Trees 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
- They need to have relatively sparse canopies to allow light penetration to support dense groundcover vegetation
- Have shallow root systems and root systems that are not known the be adventurous ‘water seekers’ to reduce the risk of root intrusion into subsurface drainage pipes
- Trees must not be deciduous
- Preferably native and occur naturally in the local area

The shrubs and trees listed in Table A-2 are recommended as they display the above features.

Most of the shrub and tree species listed in Table A-2 are widespread, occurring throughout south-east Queensland. However, alternative locally occurring species that display the required features may be selected to tailor the species list to match the native vegetation associations of the area and to compliment surrounding vegetation communities. Please refer to the local authority for further guidance in this regard.

A.2.2 Plant Species Selection

Well established uniform groundcover vegetation is crucial to the successful operation of swale and bioretention system treatment elements. As a result, plant species selection needs to consider both the aesthetic and functional requirements.

When selecting plant species from Table A-1, consideration must be given to the following factors:

- other WSUD objectives such as landscape, aesthetics, biodiversity, conservation and ecological value
- region, climate, soil type and other abiotic factors
- roughness of the channel (Manning’s *n* roughness factor) (for swales)
- extended detention depth (for bioretention systems).

Typical heights of each plant species and comments relating to shade and salt tolerances and soil moisture requirements are provided in Table A-1 and will help with the selection process. The low growing and lawn species are suitable for swale elements that require a low hydraulic roughness. 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 depth should have vegetation that will grow to at least 300 mm high. Turf is not considered to be suitable vegetation for bioretention basins. The stem is not grow high enough and the root structure of turf is not suitably robust to ensure the surface of the bioretention filter media is continuously broken up to prevent clogging.

Included in Table A-1 is a recommended planting density for each plant species. The groundcover planting densities should ensure that 70-80 % cover is achieved after two growing seasons (2 years) given adequate irrigation and weed control. These high densities are required to ensure runoff does not establish preferential flow paths around the plants and erode the swale/ bioretention surface. High density planting is also required to ensure a uniform root zone, which is particularly important in bioretention systems, and reduces maintenance costs associated with weed control.

If prostrate shrubs that form scrambling thickets are used (in place of or in conjunction with the plant species in Table A-1) they should be planted at high densities (8-10 plants/m^2) and may require pruning to ensure even plant cover and to maintain an even root distribution below ground.

A.2.3 Vegetation Establishment and Maintenance

Swales, buffer strips and bioretention basins are living systems and require two years of establishment before the vegetation matures and reaches fully functional form. During this establishment period, regular site monitoring and maintenance is critical to the success of these systems. In addition, specific requirements for plant stock sourcing, topsoil selection and testing and vegetation establishment, as detailed in the relevant WSUD element chapters, are necessary to maximise successful vegetation establishment and system treatment performance. Particular reference is made to the sections titled 'Landscape Design Notes', 'Maintenance Requirements' and 'Construction and Establishment' for guidance on vegetation establishment and maintenance procedures. The 'Construction and Establishment' section also details a staged implementation approach by which the functional elements of the WSUD system are protected from building site runoff and associated sedimentation, weeds and litter during the building phase.

Table A-1: Groundcover plant species list for swales (incorporating buffer strips), bioretention swales and bioretention basins

Scientific Name	Common Name	Form	Height (mm)	Planting Density [1] (Qty/m^2)	Comments
Cynodon dactylon	Couch	Turf	50-150	Seeded or rolled	Mowing required to achieve smaller heights
Digitaria didactyla	Blue Couch	Turf	50-150	Seeded or rolled	Mowing required to achieve smaller heights
Paspalum distichum	Water Couch	Turf	To 500	Seeded or rolled	Not suitable for sandy soils with low water holding capacity
Paspalum vaginatum cv 'Saltene'	Salt Water Couch	Turf	To 500	Seeded or rolled	Salt tolerant
Sporobolus virginicus	Marine Couch	Turf	To 400	Seeded or rolled	Salt tolerant
Stenotaphrum secundatum	Buffalo	Turf	50-150	Seeded or rolled	Mowing required to achieve smaller heights
Bacopa monnieri	Bacopa	Prostrate	100	6-8	Not suitable for sandy soils with low water holding capacity
Myoporum parvifolium	Creeping Boobialla	Prostrate	150	4-6	
Baumea teretifolia		Tufted	300-1000	6-8	Not suitable for sandy soils with low water holding capacity
Carex appressa	Tall Sedge	Tufted	1000	6-8	Not suitable for sandy soils with low water holding capacity
Carex fascicularis	Tassel Sedge	Tufted	1000	6-8	Not suitable for sandy soils with low water holding capacity
Carex gaudichaudiana	Tufted Sedge	Tufted	600	6-8	Not suitable for sandy soils with low water holding capacity
Carex polyantha	Creek Sedge	Tufted	To 900	6-8	Not suitable for sandy soils with low water holding capacity
Carex pumila	Coastal Sedge	Tufted	250	8-10	Salt tolerant
Cymbopogon refractus	Barbed Wire Grass	Tufted	300	8-10	
Cyperus gunnii	Flecked Flat-sedge	Tufted	1000	6-8	Not suitable for sandy soils with low water holding capacity
Cyperus polystachyos	Bunchy Sedge	Tufted	600	6-8	
Dianella brevipendunculata	Flax Lily	Tufted	500	4-6	
Dianella caerulea cv 'Breeze'	Blue Flax-lily	Tufted	600	4-6	
Dianella caerulea cv 'Little Jess'	Blue Flax-lily	Tufted	400	4-6	Shade tolerant
Dianella longifolia var. longifolia	Pale Flax-lily	Tufted	300-800	6-8	Shade tolerant
Dianella tasmanica	Tasman Flax-lily	Tufted	1500	4-6	Shade tolerant
Dichelachne crinita	Long Haired Plume Grass	Tufted	200	6-8	
Dietes bicolor	Dietes	Tufted	1000	4-6	Exotic

1 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. 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.

Scientific Name	Common Name	Form	Height (mm)	Planting Density [1] (Qty/m²)	Comments
Dietes grandiflora	Dietes	Tufted	750	4-6	Exotic
Erograstis elongata cv 'Elvera'	Elvera	Tufted	300	6-8	
Gahnia aspera	Saw Sedge	Tufted	1000	4-6	Not suitable for sandy soils with low water holding capacity
Gahnia sieberiana	Red-fruited Sword Sedge	Tufted	1500-3000	4-6	
Imperata cylindrica	Blady Grass	Tufted	500	6-8	
Fincia nodosa (Syn. Isolepis nodosa)	Knobby Club Rush	Tufted	600	4-6	Salt tolerant, sandy conditions
Juncus kraussii	Sea Rush	Tufted	600-2300	8-10	Salt tolerant
Juncus usitatus	Common Rush	Tufted	500	8-10	
Lepidosperma laterale	Variable Sword Sedge	Tufted	500-1000	6-8	Shade tolerant
Liriope muscari cv 'Evergreen Giant'	Turf Lily	Tufted	500	4-6	Exotic, shade tolerant
Lomandra confertifolia subsp confertifolia	Matting Lomandra	Tufted	300	4-6	Shade tolerant
Lomandra confertifolia subsp pallida	Matt Rush	Tufted	400	4-6	Shade tolerant
Lomandra hystrix	Creek Matt Rush	Tufted	1000	4-6	Shade tolerant
Lomandra longifolia cv 'Katrinus'	Matt Rush	Tufted	1000	4-6	Shade tolerant
Lomandra longifolia cv 'Tanika'	Matt Rush	Tufted	500	4-6	Shade tolerant
*Pennisetum alopecuroides**	Swamp Foxtail	Tufted	1000	4-6	Shade tolerant
Pennisetum alopecuroides cv 'Nafray'*	Fountain Grass	Tufted	300-500	6-8	
Poa labillardiere cv 'Eskdale'	Eskdale	Tufted	450	6-8	
Themeda australis	Kangaroo Grass	Tufted	300-500	6-8	
Themeda australis cv 'Mingo'	Mingo	Tufted	200	8-10	

* *Pennisetum alopecuroides* is native to Australia and is not invasive as it has low seed viability and is grown by division. This species is not to be confused with *Pennisteum setcaceum* an introduced African variety that has become a weed in Australia.

Table A-2: Shrub and Tree plant species list for swales (incorporating buffer strips), bioretention swales and bioretention basins

Scientific Name	Common Name	Form	Height (mm)	Planting Density [2] (Qty/m^2)	Comments
Breynia oblongifolia	False Coffee Bush	Shrub	1.0-2.0	2-4	
Callistemon sieberi	River Bottlebrush	Shrub	3-10	1	Requires moist conditions during establishment but tolerates dry periods once established
Hardenbergia violacea	Purple Coral Pea	Shrub	1.0-3.0	2-4	Scrambling or prostrate, full sun to light shade
Jacksonia scoparia	Dogwood	Shrub	1.0-3.0	2-4	Sunny position
Kunzea ericoides	Burgan	Shrub	2-6	<1	
Leptospermum polygalifolium	Wild May	Shrub	1.0-4.0	2-4	Sunny position
Lomatia silaifolia	Crinkle Bush	Shrub	1.0-2.0	2-4	Partial sun or shade
Myoporum acuminatum	Coastal Boobialla	Shrub	0.5-6.0	2-4	Sun or semi-shade, salt tolerant
Callistemon salignus	White Bottlebrush	Tree	2.0-15.0	1	Full sun to semi-shade
Callistemon sieberi	River Bottlebrush	Shrub	3-10	1	Requires moist conditions during establishment but tolerates dry periods once established
Callistemon viminalis	Weeping Bottle Brush	Tree	5.0-10.0	<1	Requires moist soils during establishment but tolerates dry periods once established
Elaeocarpus obovatus	Hard Quandong	Tree	5.0-30.0	<1	
Eucalyptus camaldulensis	River Red Gum	Tree	12-50	<1	
Eucalyptus ovata	Swamp Gum	Tree	8-30	<1	
Lophostemon confertus	Brush Box	Tree	10-30	<1	
Lophostemon suaveolens	Swamp Box	Tree	5.0-25.0	<1	Sunny position
Melaleuca bracteata	River Tea Tree	Tree	5.0-15.0	<1	Sunny position
Melaleuca linariifolia	Flax-leaf Paperbark	Tree	5.0-10.0	<1	
Melaleuca nodosa	Prickly-leafed Paperbark	Tree	2.0-7.0	2-4	Sunny position
Melaleuca quinquenervia	Broad-leafed Paperbark	Tree	8.0-25.0	<1	
Melaleuca sieberi	Small-leaved Paperbark	Tree	2.0-10.0	<1	

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. 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.

A.3 Wetlands and Sedimentation Basins

A.3.1 Required Plant Characteristics

Planting for wetlands and sedimentation basins may consist of two vegetation types:

- Macrophytes and groundcovers for stormwater treatment and erosion protection. The macophytes are divided further into a range of different zones as outlines in Table A-3.
- Shrubbery and trees for screening, shading, character and other landscape values.

For specific guidance on plant species the designer is initially directed to relevant guidelines provided by the local authority. In the absence of local guidance the designer can refer to the following sections and Table A-3 and Table A-4 which outlines plant species suitable for SEQ.

A.3.1.1 Macrophytes and Groundcovers

The plant species listed in Table A-3 have been specifically selected based on their life histories and physiological and structural characteristics, to meet the functional requirements of wetland systems. Plant species suitable for wetlands will also be suitable for edge planting around sedimentation basins at corresponding depth ranges. The following sections address wetlands specifically as they have very defined vegetation requirements for stormwater treatment. This includes consideration of the wetland zone/ depth range, typical extended detention time (typically 48-72 hrs) and extended detention depth (typically 0.25-0.5 m).

Other species can be used to supplement the core species listed in Table A-3 provided they have the required features to fulfill the functional roles of the wetland zone. Careful consideration of the water depth range and wetland hydrological regime (water depth and inundation period) is also required to assess the suitability of alternate species for constructed wetlands.

In general, the species in Table A-3 have the following features:

- They grow in water as either submerged or emergent macrophytes, or they grow adjacent to water and tolerate periods of inundation (typically sedge, rush or reed species).
- They generally have spreading rather than clumped growth forms.
- They are perennial rather than annual.
- They generally have rhizomatous growth forms.
- They have fibrous root systems.
- They are generally erect species with simple vertical leaves (e.g. *Juncus spp, Baumea spp*).

A.3.1.2 Shrubs and Trees

Shrubs and trees are not a required element of wetlands or sedimentation basins but can be integrated to provide amenity, character and landscape value. Shrubs and trees (generally only planted in the littoral or ephemeral zones) should be accompanied by shade tolerant groundcover species with the above characteristics as an understorey as periodic inundation during extended detention may occur. Table A-4 provides a list of shrubs and trees that are natives to south-east Queensland and are suitable for planting in the littoral zone (i.e. on the batters) around wetlands and sedimentation basins.

Littoral zone vegetation (as opposed to ephemeral marsh vegetation) is primarily for batter stabilisation, aesthetics and to restrict public access, rather than for stormwater treatment. For this reason, species that do not have all of the above structural features, but fulfill the primary littoral zone requirements (e.g. erosion protection) and landscape objectives may still be acceptable for inclusion in this zone (refer to the 'Landscape Design Notes' section in the relevant WSUD chapter).

A.3.2 Plant Species Selection

Plant species listed in Table A-3 are recommended as core species for wetland planting. These plant species have been grouped into a wetland macrophyte zone according to their preferred water depth and the hydrologic conditions of the zone.

While individual plant species can have very specific water depth requirements other species can be quite adaptive to growing across various zones over time. It is however, recommended that the suggested zones and plant groups are adhered to for planting purposes. Plant species listed against the shallow marsh and ephemeral marsh wetland zones are equally suitable for edge planting (at equivalent depths) in sedimentation basins. Planting densities recommended should ensure that 70-80 % cover is achieved after two growing seasons (2 years).

Suitable plant species for the batters that surround wetlands and sedimentation basins have also been recommended in Table A-3. The batters relate to the berms or embankments around the systems that may extend from the permanent pool water level to (typically) 0.5 m above this design water level (i.e. within the extended detention depth). 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.

A.3.3 Vegetation Establishment and Maintenance

To maximise the success of plant establishment in wetland macrophyte zones specific procedures are required in site preparation, stock sourcing, vegetation establishment and maintenance. Reference is to be made to procedures detailed in 'Landscape Design Notes' Chapter 6 as follows:

- Sourcing plant stock (6.5.3)
  - Lead times for ordering plants
  - Recommended planting systems/ products
- Topsoil specification and preparation 6.5.4)
  - Sourcing, testing and amendment
  - Top soil treatments (e.g. gypsum, lime, fertiliser)
- Vegetation establishment (6.5.5)
  - Weed control
  - Watering
  - Water level manipulation

Constructed wetlands are living systems and they require two years of establishment before the vegetation matures and reaches fully functional form. During this establishment period, regular site monitoring and maintenance is critical to the success of these systems. Reference must also be made to the sections titled 'Maintenance Requirements' (Section 6.6) and 'Construction and Establishment' (Section 6.5) for guidance on maintenance procedures and vegetation establishment.

Similarly, the vegetation planted in sedimentation basins require an equivalent vegetation establishment period (i.e. 2 years) and level of attention to site preparation, stock sourcing, vegetation establishment and maintenance to ensure success. Reference must be made to the sections entitled 'Landscape Design Notes', 'Maintenance Requirements' and 'Construction and Establishment' in Chapter 4.

Table A-3: Macrophyte and Groundcover Plant Species List for Wetlands and Sedimentation Basins

Key to Table A-3:

Zone		Depth*(m)	Form	
P	Pool	1.5 – 0.5	S	Submerged macrophytes
DM	Deep Marsh	0.5 – 0.35	M	Emergent macrophytes
M	Marsh	0.35 – 0.2	G	Groundcover
SM	Shallow Marsh	0.2 - 0	T	Tufted
EM	Ephemeral Marsh	0 – +0.2**		
B	Batters	+0.2 - +0.5**		

* 'Depth' refers to depth below permanent pool water level

** '+' denotes levels above permanent pool water level

Scientific name	Common name	Zone	Form	Height (mm)	Planting Density [3] (Qty/m²)	Comments
Myriophyllum papillosum	Common Water-milfoil	P	S	To 200	1	
Myriophyllum verrucosum	Red Water-milfoil	P	S	100-1500	1	
Potamogeton crispus	Curly Pondweed	P	S	To 4500	1	Growth can be dense
Potamogeton ochreatus	Blunt Pondweed	P	S	To 4500	1	Rapid growth; aesthetic; seasonal; salt tolerant (2000 ppm)
Vallisneria gigantea	Ribbonweed	P	S	To 3000	1	Rapid growth; salt tolerant (1500 ppm)
Vallisneria spiralis	Eel Weed	P	S	150-300	1	
Baumea articulata	Jointed Twig-rush	DM	M	1000-2000	6-8	Slow growth, plant solo
Bolboschoenus fluviatalis	Marsh Club-rush	DM	M	1000-2000	4-6	Plant solo, flow resistant
Eleocharis sphacelata	Tall Spike-rush	DM	M	500-2000	6-8	Plant solo, rhizomes can restrict growth of other plants; slow establishment, flow resistant
Schoenoplectus litoralis	Shore Club-rush	DM	M	600-1500	4-6	
Schoenoplectus validus	River Club-rush	DM	M	600-1600	4-6	
Baumea arthrophylla	Fine Twig-rush	M	M	300-1300	6-8	Spreads quickly
Baumea rubiginosa	Soft Twig-rush	M	M	300-1000	6-8	Can be slow to establish
Bolboschoenus caldwellii	Sea Club-rush	M	M	300-900	4-6	Rapid establishment, salt tolerant
Lepironia articulata	Grey Rush	M	M	600-2300	4-6	
Schoenoplectus mucronatus	Star Club-rush	M	M	350-1000	4-6	Shade tolerant
Triglochin procera	Water-ribbon	M	M	200-500	4-6	Aesthetic; spreading
Baumea juncea	Bare Twig-rush	SM	T	300-1000	8-10	Slow establishment

3 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. 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.

Scientific name	Common name	Zone	Form	Height (mm)	Planting Density [3] (Qty/m^2)	Comments
Carex fascicularis	Tassel Sedge	SM	M	500-1000	6-8	Aesthetic
Carex gaudichadiana	Tufted sedge	SM	M	100-600	6-8	Aesthetic; tolerates drawdown
Cyperus exaltatus	Giant Sedge	SM	M	1000-2000	4-6	Short lived
Eleocharis acuta	Common Spike-rush	SM	M	300-900	6-8	High surface area
Eleocharis dulcis	Chinese Water Chestnut	SM	M	To 1500	6-8	
Eleocharis equisetina	Spike-rush	SM	M	500-1000	6-8	
Eleocharis pusilla	Small Spike-rush	SM	T	To 250	6-10	Readily grown
Ficnia nodosa (syn. Isolepis nodosa)	Knobby Club-rush	SM	M	500-1500	6-8	
Isolepis inundata	Swamp Club-rush	SM	M	To 300	6-8	High surface area; rapid growth
Juncus subsecundus	Finger Rush	SM	M	500-1000	8-10	
Juncus usitatus	Common Rush	SM	M	300-1200	8-10	Rapid growth
Phylidrium lanuginosum	Woolly Water Lily	SM	T	500-1000	6-8	Aesthetic
Restio pallens	Cord Rush	SM	M	500-1000	8-10	
Restio tetraphyllus	Tassel Cord-rush	SM	M	500-1500	6-8	
Carex appressa	Tall Sedge	EM	M	500-1200	4-8	High surface area
Carex inversa	Knob Sedge	EM	M	100-300	8-10	Rapid establishment
Carex polyantha	Creek Sedge	EM	M	To 900	6-8	
Cyperus gunnii	Flecked Flat Sedge	EM	M	600-1000	6-8	High surface area
Juncus flavidus	Yellow Rush	EM	M	400-1200	8-10	Aesthetic
Juncus pristmatocarpus	Branching Rush	EM	M	300-600	6-8	
Lepidosperma laterale var. laterale	Variable Sword-sedge	EM	M	400-900	6-8	Shade tolerant
Lepidosperma longitudinale	Common Sword-sedge	EM	M	600-1700	6-8	Aesthetic
Carex breviculmis	Short-stem sedge	B	T	To 150	6-8	Very adaptable
Carex pumila	Coastal Sedge	B	T	100-250	8-10	Salt tolerant, drought tolerant
Cyperus polystachyos	Bunchy Sedge	B	T	To 600	6-8	
Dianella longifolia var. longifolia	Pale Flax-lily	B	T	300-800	6-8	Aesthetic; shade tolerant
Gahnia clarkei	Tall Saw-sedge	B	T	1500-2500	4-6	Plant solo
Gahnia siberiana	Red-fruited Sword Sedge	B	T	1500-3000	4-6	Aesthetic
Lomandra filiformis spp. filiformis	Wattle Mat-rush	B	T	150-500	6-8	Shade tolerant when established
Lomandra longifolia var. longifolia	Spiny-headed Mat Rush	B	T	500-1000	4-6	Shade tolerant
Poa labillardieri	Tussock Grass	B	T	300-1200	6-8	
Schoenus apogon	Common Bog-rush	B	G	To 300	8-10	
Viola hederacea	Native Violet	B	G	To 150	2-4	Rapid growth; aesthetic; prolific growth once established

Table A-4: Shrub and Tree Plant Species List for Wetlands and Sedimentation Basins

Scientific name	Common name	Zone	Form	Height (mm)	Planting Density [4] (Qty/m^2)	Comments
Callistemon sieberi	River Bottlebrush	B	Shrub	3-10	1	Very wet to moist conditions in heavy clay soils, tolerates dry conditions once established
Banksia robur	Swamp Banksia	B	Shrub	1-1.5	2-4	Moist soils on coastal sand and peat soils
Leptospermum liversidgei		B	Shrub	1.0-3.0	2-4	Moist soil, sunny position
Myoporum acuminatum	Coastal Boobialla	B	Shrub	0.5-6.0	2-4	Sun or semi-shade, salt tolerant
Callistemon salignus	White Bottlebrush	B	Tree	2.0-15.0	1	Moist sandy and alluvial soils, full sun to semi-shade
Callistemon viminalis	Weeping Bottle Brush	B	Tree	5.0-10.0	<1	Moist, medium to heavy soils, tolerates dry periods once established
Casuarina cunninghamiana	River She-oak	B	Tree	10-35	<1	
Elaeocarpus obovatus	Hard Quandong	B	Tree	5.0-30.0	<1	Moist soils, tolerates water logged soils, hardy and fast growing
Eucalyptus camaldulensis	River Red Gum	B	Tree	12-50	<1	Damp alluvial soils, tolerates inundation and very dry periods once established
Eucalyptus ovata	Swamp Gum	B	Tree	8-30	<1	Moist soils, tolerates inundation and dry periods
Lophostemon confertus	Brush Box	B	Tree	10-30	<1	Moist deep alluvial clay soils or moist sandy soils
Lophostemon suaveolens	Swamp Box	B	Tree	5.0-25.0	<1	Moist sandy soils
Melaleuca bracteata	River Tea Tree	B	Tree	5.0-15.0	<1	Moist, free draining soils
Melaleuca linariifolia	Flax-leaf Paperbark	B	Tree	5.0-10.0	<1	Moist sandy soils and swampy areas
Melaleuca nodosa	Prickly-leafed Paperbark	B	Tree	2.0-7.0	2-4	Deep sands and moist sandy soils
Melaleuca quinquenervia	Broad-leafed Paperbark	B	Tree	8.0-25.0	<1	Very moist sands and alluvial soils, tolerates inundation
Melaleuca sieberi	Small-leaved Paperbark	B	Tree	2.0-10.0	<1	Moist sandy or poorly drained soil

4 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. 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.

A.4 References

BCC 2005, Growing Native Plants in Brisbane, BCC, Brisbane, accessed 25th July 2005, http://www.brisbane.qld.gov.au/BCC:STANDARD::pc=PC_1927

BCC, DMR & PRSC (Brisbane City Council, Queensland Department of Main Roads & Pine Rivers Shire Council) 2001, Preferred Species Manual: Green Routes Program, prepared for the Green Routes Program by BCC, DMR & PRSC, Brisbane