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Stormwater management manual for Western Australia

Chapter 9 Structural controls

Department of Water and Environmental Regulation

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Preface

A growing public awareness of environmental issues in recent times has elevated water issues to the forefront of public debate in Australia.

Stormwater is water flowing over ground or built-up surfaces and in natural streams and drains, as a direct result of rainfall over a catchment (ARMCANZ and ANZECC, 2000). Stormwater consists of rainfall runoff and any material (soluble or insoluble) mobilised in its path of flow. Stormwater management examines how the runoff quantity, and these pollutants can best be managed from source to the receiving water bodies using the range of management practices available.

In Western Australia (WA), where there is a superficial aquifer, drainage channels can commonly include both stormwater from surface runoff and groundwater that has been deliberately intercepted by drains installed to manage seasonal peak groundwater levels. Stormwater management is unique in WA as both stormwater and groundwater may need to be managed concurrently.

Rainwater has the potential to recharge the superficial aquifer, either prior to runoff commencing or throughout the runoff's journey in the catchment. Urban stormwater on the Swan Coastal Plain is an important source of recharge to shallow groundwater, which supports consumptive use and groundwater dependent ecosystems.

With urban, commercial or industrial development, the area of impervious surfaces within a catchment can increase dramatically. Densely developed inner urban areas are almost completely impervious, which means less infiltration, the potential for more local runoff and a greater risk of pollution. Loss of vegetation also reduces the amount of rainfall leaving the system through the evapo-transpiration process. Traditional drainage systems have been designed to minimise local flooding by providing quick conveyance for runoff to waterways or basins. However, this almost invariably has negative environmental effects.

This manual presents a new comprehensive approach to management of stormwater in WA, based on the principle that stormwater is a resource – with social, environmental and economic opportunities. The community's environmental awareness and water restrictions are influencing a change from stormwater being seen as a waste product with a cost, to a resource with a value. Stormwater management aims to build on the traditional objective of local flood protection by having multiple outcomes, including improved water quality management, protecting ecosystems and providing liveable and attractive communities.

This manual provides coordinated guidance to developers, environmental consultants, environmental/ community groups, industry, local government, water resource suppliers and state government departments and agencies on current best management principles for stormwater management.

Production of this manual is part of the State Government's response to the State Water Strategy for Western Australia.

It is intended that the manual will undergo continuous development and review. As part of this process, any feedback on the series is welcomed and may be directed to the Urban Water Branch of the Department of Water and Environment Regulation, at urbanwater.enquiry@dwer.wa.gov.au

Western Australian stormwater management objectives

Water quality

To maintain or improve the surface and groundwater quality within the development areas relative to pre-development conditions.

Water quantity

To maintain the total water cycle balance within development areas relative to the pre-development conditions.

Water conservation

To maximise the reuse of stormwater.

Ecosystem health

To retain natural drainage systems and protect ecosystem health.

Economic viability

To implement stormwater management systems that are economically viable in the long term.

Public health

To minimise the public risk, including risk of injury or loss of life, to the community.

Protection of property

To protect the built environment from flooding and waterlogging.

Social values

To ensure that social, aesthetic and cultural values are recognised and maintained when managing stormwater.

Development

To ensure the delivery of best practice stormwater management through planning and development of high quality developed areas in accordance with sustainability and precautionary principles.

Western Australian stormwater management principles

- Incorporate water resource issues as early as possible in the land use planning process.
- Address water resource issues at the catchment and sub-catchment level.
- Ensure stormwater management is part of total water cycle and natural resource management.
- Define stormwater quality management objectives in relation to the sustainability of the receiving environment.
- Determine stormwater management objectives through adequate and appropriate communityconsultati and involvement.
- Ensure stormwater management planning is precautionary, recognises inter-generational equity, conservation of biodiversity and ecological integrity.
- Recognise stormwater as a valuable resource and ensure its protection, conservation and reuse.
- Recognise the need for site-specific solutions and implement appropriate non-structural andstructural solutions.

Contents

Summary

The aims of this chapter are:

- describe structural controls, as well as provide an overview of their benefits, use, effectiveness and evaluation
- provide basic information on the selection of structural controls and the use of relevant technical guidelines
- provide technical guidelines on some of the most relevant structural controls that can be applied at regional, estate and allotment scales in WA.

Structural controls are engineered devices implemented to manage runoff quality and quantity to control, treat or prevent stormwater pollution and/or reduce the volume of stormwater requiring management. Structural controls may be located at-source, in-transit or at end-of-catchment. They are ideally installed at or near the source of stormwater runoff to protect receiving environments, including groundwater, waterways and wetlands. The implementation of structural stormwater best management practices into an urban landform has multiple benefits, including reducing storm flows, reducing pollutant export, maintaining and improving the urban landscape, protecting receiving environments and reducing irrigation and potable water supply requirements.

Structural controls can be designed for a new development on a greenfield or brownfield site, or for retrofitting within existing developments. They should be used in combination with non-structural controls (i.e. the ‘treatment train approach’) to achieve a balanced mix of stormwater management measures.

This chapter aids in the selection, location and design of the most appropriate structural controls. It is based on a current understanding of the performance of structural controls and assesses the controls in the context of WA’s local hydrology. It aims to ensure stormwater best management practices are implemented in a consistent manner and are achieving the objectives previously determined for a catchment through appropriate urban water management planning.

Table 1 shows the structural controls that are addressed in this chapter, and displays to which target audience each control applies. The structural control selection process is illustrated in Figure 1 and discussed in the Section 1.7. A summary of each structural control is provided in Section 1.10.

1 Introduction

1.1 Aims of the structural controls chapter

This chapter aims to ensure stormwater best management practices (BMPs) are implemented in a consistent manner and are achieving the objectives previously determined for a catchment through appropriate urban water management planning processes. It does not seek to address all possible structural controls for stormwater management, but focuses on those currently recommended for use and generally supported in WA by government agencies, and those which represent emerging technology considered suitable for application in WA.

1.2 Scope of the chapter

This chapter focuses on the most relevant structural stormwater controls that can be used in WA to manage the quantity and quality of stormwater runoff, prevent or treat stormwater pollution, and provide opportunities for water conservation through the use of stormwater as a resource.

This chapter aids in the selection, location and design of the most appropriate structural controls based on current understanding of the performance of structural controls and assessing the controls in the context of WA’s local hydrology.

Structural controls can be designed for a new development on a greenfield or brownfield site, as well as retrofitting within existing developed areas. Chapter 6 contains more information on retrofitting.

Non-structural controls are addressed in Chapter 7.

1.3 Stormwater management approach

This manual encourages a treatment train approach to stormwater management, where combinations of measures (structural and non-structural) are implemented in parallel or sequence to achieve best management of stormwater.

The implementation of structural stormwater BMPs into an urban landform has multiple benefits, including reducing storm flows, reducing pollutant export, maintaining and improving the urban landscape, protecting receiving environments and reducing irrigation and potable water supply requirements.

Refer to the *Decision process for stormwater management in Western Australia November 2017* (a component of Chapter 4) and *Understanding the context* (Chapter 2) of the manual for the current stormwater management approaches in WA.

1.4 Terminology and key definitions

Structural stormwater best management practices are engineered devices implemented to manage runoff quality and quantity to control, treat or prevent stormwater pollution and/or reduce the volume of stormwater requiring management. Structural controls may be located at-source, in-transit or at end-of-catchment. They are ideally installed at or near the source of stormwater runoff to protect receiving environments, including groundwater, waterways and wetlands.

Source controls are structural or non-structural BMPs designed to minimise the generation of excessive stormwater runoff and/or pollution of stormwater at or near the source and protect receiving environments, including groundwater, waterways and wetlands.

Non-structural stormwater BMPs are institutional and pollution-prevention practices designed to prevent or minimise pollutants from entering stormwater runoff and/or reduce the volume of stormwater requiring management (United States Environmental Protection Agency 1999). They do not involve fixed, permanent facilities and they usually work by changing behaviour through government regulation (e.g. planning and environmental laws), education and/or economic instruments (Taylor and Wong 2002).

Receiving environments are areas that receive stormwater runoff, including wetlands, waterways, coastal waters/dunes, groundwater and bushland areas.

Water bodies are waterways, wetlands, coastal marine areas and shallow groundwater aquifers.

Effective imperviousness is the combined effect of the proportion of constructed impervious surfaces in the catchment, and the connectivity of these impervious surfaces to receiving water bodies.

A detailed glossary at the end of the manual provides definitions of technical terminology used in this chapter.

1.5 The target audience

This chapter is primarily aimed at engineers and other urban water management professionals and local and state government approval officers.

Due to the range of multi-disciplinary professionals usually involved in urban development and catchment management, it is also an information source for planners, urban designers, environmental officers, landscape architects, environmental scientists, landcare and community groups, developers and individual landowners.

Table 1 provides a summary of the structural BMPs that are addressed in this chapter and highlights the relevance for application by the target audiences.

Table 1. Structural control BMP summary and target audiences

Target audiences					
Structural BMPs covered in this manual	**Government stormwater management agencies**	**Developers and consultants**	**Commercial or industrial landowners/ managers**	**Individuals, landowners or community groups**	**Section/ chapter reference**
Stormwater storage and use					
Rainwater storage systems	✔	✔	✔	✔	2.1
Managed aquifer recharge	✔	~	✘	✘	2.2
Infiltration systems					
Infiltration basins and trenches	✔	✔	✔	✔	3.1
Soakwells	✔	✔	✔	✔	3.2
Pervious pavement	✔	✔	✔	✔	3.3
Conveyance systems					
Swales and buffer strips	✔	✔	✔	✔	4.1
Bioretention systems	✔	✔	✔	~	4.2
Living streams	✔	✔	~	✔	4.3
Detention systems					
Dry/ephemeral detention areas	✔	✔	~	~	5.1
Constructed wetlands	✔	✔	~	~	5.2
Pollutant control					
Litter and sediment management	✔	✔	✔	✔	6.1
Hydrocarbon management	✔	✔	✔	✘	6.2

Key: □ = Highly relevant ~ = Some relevance × = Not relevant

1.6 How to use the BMP guidelines in this chapter

Structural controls should be selected according to the BMP selection process outlined in section 1.7.

The BMPs in Sections 2 to 6 of this chapter contain summarised background information, recommended practices, factors to consider, cost details, performance indicators, local application examples and references for a number of structural controls.

It is not necessary to read all of the information in Sections 2 to 6 in order to use this chapter. The detailed content should be selectively accessed as needed, to gather information on how to select and apply specific structural controls.

The dollar values quoted in this chapter relating to BMP costs are from around 2000 to 2007 and have not been adjusted for inflation or potential cost changes due to technological advances which may have occurred since this chapter was published in 2007. Costs presented in this document should therefore be considered in this context and users of the manual are encouraged to seek further specific industry advice on the current BMP costs as appropriate.

1.7 How to select structural BMPs

The selection of structural BMPs requires consideration of multiple factors, such as catchment management objectives, site characteristics, target pollutants, social values, and capital and operating costs to achieve a balance between quantity and quality management objectives and to create a sustainable outcome.

All BMPs, whether they are structural, non-structural, at-source, in-transit or end-of-catchment, have potential benefits and limitations. The key is finding the best combination of these measures to suit local circumstances.

Performance of structural BMPs largely depends on the pre-development (pre-implementation) site characteristics and scale in which the measures are to be implemented. Structural controls are designed to achieve pollutant removal (quality), volume management (quantity) and/or water conservation functions.

The approach adopted in the selection process recommends that these factors be examined before the assessment of BMP characteristics and functionalities.

A key decision for BMP selection is the life cycle cost (capital and maintenance costs). This will require a balance between outcomes sought and available funding.

The BMP selection process is illustrated in the flow chart in Figure 1 and discussed in the remainder of this section.

1.7.1 Setting objectives, outcomes and design criteria

Structural BMPs will be selected, planned, designed and implemented through the appropriate planning processes and stages in greenfield developments and developed catchments. Please refer to Chapter 2 of this manual (e.g., section 5.5 The planning framework and stormwater management), Chapter 5 Stormwater Management Plans, the decision process for stormwater management in WA 2017 (e.g. section 3.4 integrate stormwater management in the land and water planning process and Figure 1 – stormwater management and the land and water planning process) and State Planning Policy 2.9 Planning for Water Guidelines 2021 for further details on planning for stormwater management, including structural BMPs. The decision process for stormwater management in WA 2017 provides objectives, outcomes and design criteria for stormwater management.

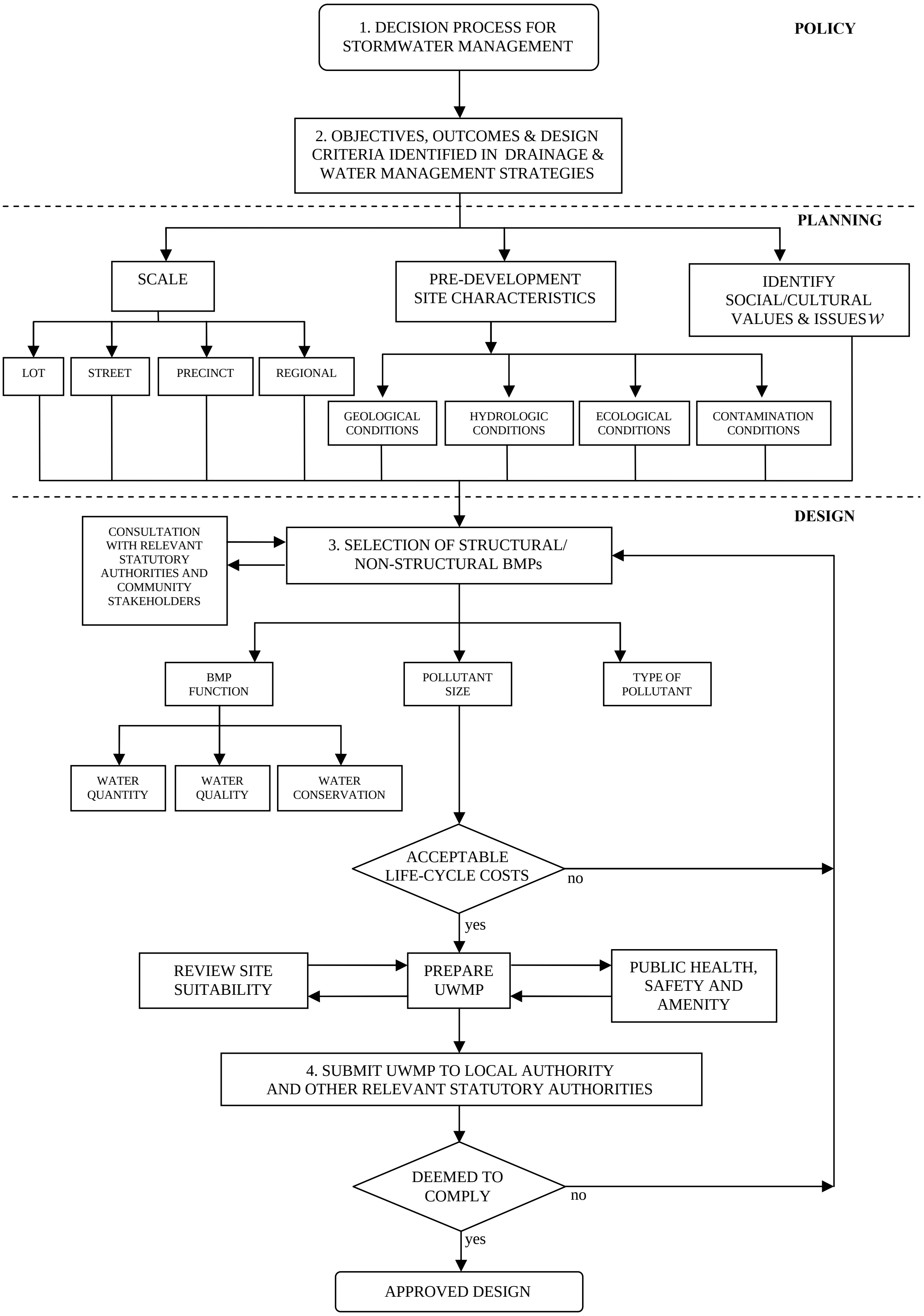


Figure 1. BMP selection flow chart (This flow chart to be read in conjunction with Figures 1 & 2 of Decision process for stormwater management in WA 2017)

In developed catchments, stormwater management plans are prepared to identify desired stormwater management outcomes for lot/neighbourhood, catchment or local government scales. Structural BMPs may be chosen as a retrofitting strategy to improve management of stormwater at-source (preferable), in-transit and end-of-catchment. Implementing numerous small-scale retrofit projects throughout the catchment can have significant beneficial impacts on the health of receiving water bodies and on the community amenity of the area. Chapter 5 addresses stormwater management plans that are prepared for a local government area or catchment area. See Chapter 6 for more information on retrofitting.

Objectives should be clearly defined at the beginning of a BMP selection process. It is a common mistake to poorly define the objectives of the BMPs, to allow these objectives to evolve as the project is implemented, or to define objectives that are impractical to measure. To demonstrate success or failure of the BMPs, the objectives should be specific, measurable, achievable, relevant and linked to a timeframe. See Chapter 10 for further information on performance monitoring and evaluation.

The BMP selection process discussed in section 1.7 is reliant on the establishment of water quantity and quality objectives in the higher planning processes for greenfield developments. In established areas, water quantity and quality objectives are defined through natural resource management strategies, catchment management plans and stormwater management plans.

The main water sensitive design approaches discussed in the remainder of this section must be factored into the selection of structural controls.

Natural drainage systems should be protected, and constructed stormwater systems should mimic natural drainage processes. Water sensitive urban design includes maintenance of the pre-development hydrologic regime; that is, maintenance of the pre-development stormwater quantity characteristics. This includes retaining/detaining small–moderate rainfall events throughout the catchment as close to the runoff source (i.e. the impervious surface) as possible.

Water sensitive urban design increases disconnection between impervious surfaces and receiving water bodies. As a general rule, stormwater should not be discharged directly into receiving water bodies and only moderate–large rainfall events should reach receiving water bodies via overland flow paths across vegetated surfaces.

Stormwater management systems should be incorporated throughout a catchment and integrated in the urban landscape, such as within road reserves and public open space. This will minimise the social and economic issues associated with allocating (and often fencing off) large areas of land for traditional devices such as steep sided trapezoidal open drains and large sumps. As shown in Figure 1, social/cultural values and issues should be identified during the planning stage of the BMP selection process and relevant statutory authorities and community stakeholders should be consulted during the BMP design stage of the selection process.

These approaches result in improved biodiversity and health of receiving water bodies and improved amenity and quality of urban areas.

1.7.2 Scale of BMPs

Scale refers to the intended location and ownership of structural BMPs. Four broad scales have been identified for the purpose of this manual. They include:

- lot level
- street level
- precinct level
- regional level

BMP selection is best achieved using an integrated approach that focuses on meeting the overall objectives as set in the Drainage and water management strategy (DWMS), catchment management plan or stormwater management plan. This typically requires the implementation of a treatment train approach across more than one scale. For example, a soakwell infiltration BMP may be proposed at a lot scale to complement the vegetated swale BMP at a street scale and the infiltration basin BMP at a precinct scale. This arrangement will satisfy the water quantity and quality objectives that might be unachievable if relying on a single BMP. Additionally, the impact from the failure of one device (e.g. flooding or water quality issues) will be reduced by the operation of the other devices in the treatment train.

The suitability of structural control BMPs applied to different scales can be assessed using the selection matrix in Table 2.

Table 2. Structural control BMP selection matrix

BMP	BMP function			Scale				Pollutant type							Pollutant size				
	Water quantity	Water quality	Water conservation	Lot	Street	Precinct	Regional	Litter and sediments	Particulate nutrients	Dissolved nutrients	Heavy metals	Oxygen demanding Materials	Microorganisms	Hydrocarbons	Gross solids	Coarse to medium solids	Fine particulates	Very fine colloidal particulates	Dissolved particulates
Stormwater storage and use																			
Rainwater storage systems	R		✓P	✓															
Managed aquifer recharge	R		✓P	✓		✓	✓												
Infiltration systems																			
Soakwells	RP	✓	~	✓	✓			**Pollutants not transported or concentrated**							**Pollutants not transported or concentrated**				
Pervious pavement	RP	✓	~	✓	✓														
Infiltration trenches	RP	✓	~			✓													
Infiltration basins	RP	✓	~			✓	~	✓	~		~	✓	✓	~		✓	✓	✓	✓
Conveyance systems																			
Swales and buffer strips	CP R D	✓	~	✓	✓	✓	✓	✓	✓	~	~		✓		✓	✓	✓	✓	
Bioretention systems	C R D	✓P	~		✓	✓		~	✓	~	✓		✓			✓	✓	✓	✓
Living streams	CP R D	✓	~			✓	✓	~	✓	~	~	~	✓			~	~	~	~
Detention systems																			
Dry/ephemeral detention areas	DP	✓				✓	~	✓	✓	✓		✓	✓	~		✓	✓	✓	~
Constructed wetlands	[1]	✓P				✓	~	✓	✓	✓		~	✓	~		✓	✓	✓	✓
Pollutant control																			
Litter and sediment management		✓P		✓	✓	✓		✓	✓			~			✓	✓			
Hydrocarbon management		✓P		✓	✓									✓		✓	✓	✓	✓

Key: ✓ BMP is applicable, ~ BMP is applicable to some extent, R = Retention, D= Detention and C= Conveyance. (P = Primary BMP Function)

1.7.3 Pre-development site characteristics

Detailed knowledge of the pre-development site characteristics is critical in the selection of BMPs, and the following characteristics can often dictate what structural control BMPs may or may not be effectively used at a particular site.

Geological conditions

The geotechnical and hydrogeological site assessment principally aims to determine the site constraints and the suitability of potential BMPs.

Soil permeability is a significant factor in selecting suitable devices. Onsite hydraulic conductivity tests should be conducted due to the differences in permeability through the vertical and horizontal soil profiles.

Hydrologic conditions

The hydrologic conditions of a catchment include the relationships between rainfall, runoff, infiltration and evaporation. Water sensitive urban design can maintain a catchment's hydrology by mimicking the natural hydrologic characteristics (volume, frequency, recharge and discharge). These characteristics are in balance with the unique soils, vegetation and topographic features of the catchment and should where possible be maintained to maximise the protection of receiving environments.

Natural surface to groundwater separation is another important issue to consider when selecting BMPs. Infiltration BMPs typically require some separation to deliver desired hydraulic performance and to allow treatment to be carried out as stormwater percolates through the soil. However, site modification using permeable fill may provide sufficient separation for the implementation of infiltration BMPs under certain circumstances. Also, due to the seasonal variability in groundwater levels in sandy soils, the separation distance may only be limited for part of the year and may not necessarily preclude the selection of a particular BMP. The water table is likely to be at the annual maximum groundwater level for only a short duration throughout the year. Performance may not need to be optimal year-round. For example, in the south-west of the state, a stormwater management system may perform as a retention/detention system for the majority of storm events, particularly in summer, autumn and early winter (when groundwater levels are at their lowest and pollutants carried by stormwater are usually at their highest), but act primarily as a conveyance system for the short duration that the groundwater is at its maximum level in late winter and spring each year, when pollutants carried by stormwater are usually at their lowest.

An indication of groundwater levels can be obtained from the Department of Water and Environmental Regulation's Water Information Network (WIN) database or the Department of Water and Environmental Regulation's Perth Groundwater Map. However, site-specific groundwater monitoring programs should be undertaken to determine the actual groundwater regime at a proposed development site.

Ecological conditions

Protection and enhancement of the natural site attributes should be maximised when selecting and locating BMPs. There are generally more opportunities for this in greenfield developments; however, protection of remnant environments is important in both greenfield and brownfield developments.

Landform and ecological surveys of local significant vegetation, waterways and wetlands should be conducted during pre-design work. A good understanding of the existing hydrology, water quality and ecological structure and interactions is required for setting objectives. Assessments should also consider the impacts of BMPs on the ecological system, such as permanently altering groundwater levels in natural wetlands.

Opportunities for retaining natural overland flow pathways should be identified as part of the assessment. The rehabilitation of degraded waterways can provide significant economic advantages in stormwater management, particularly due to their conveyance and water quality improvement functions and the improvement of aesthetic values within the development.

Contamination conditions

Historical land use

Historical land uses can cause soil and groundwater contamination. With urbanisation, it is important to manage stormwater quantity at-source so that there is less risk of these contaminants being mobilised. BMP selection offers an opportunity to target specific contaminants or hot spots for treatment. In extreme cases, site remediation may be required for the complete removal of these contaminant sources.

Acid sulfate soils

Acid sulfate soils (ASS) form when soils naturally containing sulphide minerals are oxidised, forming sulphuric acid. Oxidation can occur when soils are exposed to the air following excavation or draining, or lowering water tables. Large-scale drainage for flood mitigation, urban expansion and agriculture have exposed many areas of ASS in WA.

The acidic leachate and the metals consequently released from the exposed or drained soils cause significant environmental problems such as poor water quality and fish kills, as well as economic costs to communities through degradation of roads and corrosion of pipes and footings.

There are also public health risks associated with ASS via exposure to dissolved acids in water. These risks include potential for consumption, or skin and eye irritation from contact with acidic water.

ASS risk areas for parts of WA can be viewed on the Department of Water and Environmental Regulation’s website (www.der.wa.gov.au/your-environment/acid-sulfate-soils/65-ass-risk-maps).

Guidelines for managing ASS are contained in Department of Environment (2015).

Secondary salinity

Catchments with secondary salinity require urban stormwater management systems to be designed and managed to meet the outcomes of the local salinity management strategy. Issues such as exposing saline subsoils through cut and fill, increasing the regional groundwater level, changes to soil groundwater flow and disturbance to sensitive areas such as riparian corridors are some of the issues that will need to be considered when selecting structural BMPs.

Safeguarding Indigenous heritage

In addition to the physical site characteristics, it is recommended that stormwater managers and designers investigate other land issues that may impact on the implementation of BMPs.

Under the Aboriginal Heritage Act 1972, landowners have an obligation to determine if any Aboriginal heritage sites may be affected by any proposed development or constructed infrastructure. If there is possibility of affecting Aboriginal heritage sites, the proponent must abide by the provisions of the Aboriginal Heritage Act 1972, as administered by the Department of Planning, Lands and Heritage.

More detailed information on traditional owner matters can be found in Chapters 5 and 6. Contact the Reconciliation Action Plan Coordinator at Department of Water and Environmental Regulation to receive contact details for local Aboriginal stakeholders.

Protecting social values

It is important to ensure that social values (including cultural values) are taken into account. Social values embrace qualities for which a place has become a focus of spiritual, political, national or other cultural sentiment to a minority or majority group. Cultural significance includes aesthetic, historic, scientific or social values for past, present or future generations.

For example, a site (e.g. a park), natural feature (e.g. a water body, tree or rock formation) or structure (e.g. a weir) might have significant social/cultural values and will therefore require consideration in the selection and/or siting of a particular structural BMP.

As shown in Figure 1, consultation with the community to determine social/cultural values and issues is an essential component of BMP selection and design.

1.7.4 BMP function

Urban runoff has the potential to have a significant impact on the ecology of water bodies due to altered water regimes (volume, energy, frequency and timing of runoff) and poor water quality. Urban stormwater management BMPs can employ achieve key functions:

- stormwater quantity management
- stormwater quality management
- water conservation.

A structural control will have single or multiple functions that will help contribute to the overall objectives or outcomes established in the catchment management plan, stormwater management plan or urban water management plan for the area. Typically, a combination of structural and non-structural controls will be implemented in series or concurrently, forming a treatment train to help achieve an overall outcome (Chapter 4).

Stormwater quantity management

Stormwater quantity management recognises that urbanisation will typically lead to an increase in imperviousness and a corresponding increase in volume and rate of runoff.

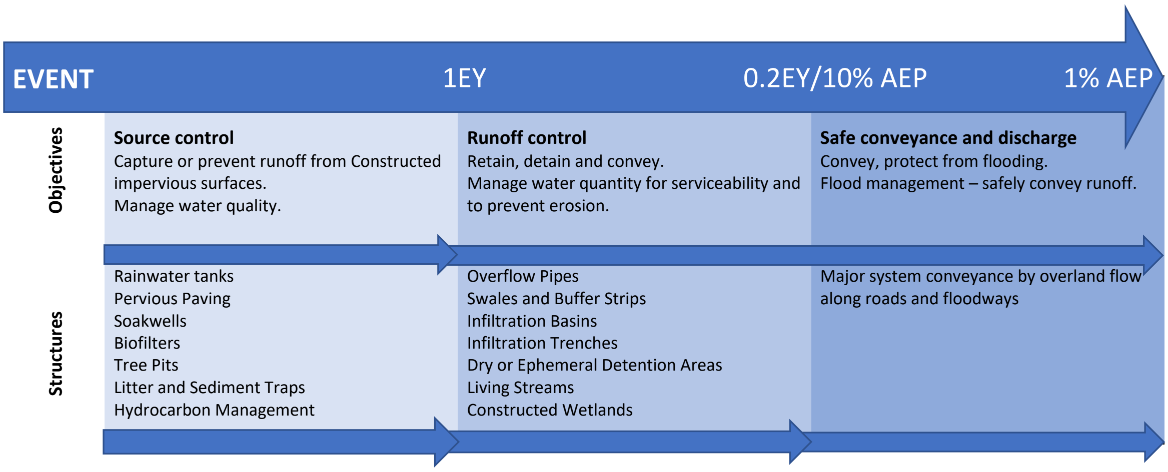
The sustainable approach to urban water management emphasises replicating post-development hydrology as close to pre-development conditions as possible. The Department of Water and Environmental Regulation’s decision process for stormwater management in WA 2017 illustrates this recommended approach in relation to managing runoff from various design storm events.

Techniques that can be incorporated to maintain the pre-development hydrology through effectively minimising the ‘effective imperviousness’ of a development area include:

- reducing the amount of constructed impervious areas; and
- disconnecting constructed impervious areas from receiving water bodies.

Reducing the amount of constructed imperviousness in a development area can be achieved through the application of alternative surfaces with lower runoff coefficients, such as permeable pavement, and through the retention of pervious areas, such as native vegetation, garden beds and parkland. This will reduce peak discharge, particularly for smaller storm events such as a 1 exceedance per year (EY) event.

Direct connection of impervious areas to receiving water bodies results in altered hydrologic regimes with associated erosion, loss of habitat, and the efficient delivery of pollutants (Walsh et al. 2004). Disconnecting impervious areas from receiving water bodies helps to maintain the pre-development hydrologic regime.



Objectives	**Source control** Capture or prevent runoff from Constructed impervious surfaces. Manage water quality.	**Runoff control** Retain, detain and convey. Manage water quantity for serviceability and to prevent erosion.	**Safe conveyance and discharge** Convey, protect from flooding. Flood management – safely convey runoff.
Structures	Rainwater tanks Pervious Paving Soakwells Biofilters Tree Pits Litter and Sediment Traps Hydrocarbon Management	Overflow Pipes Swales and Buffer Strips Infiltration Basins Infiltration Trenches Dry or Ephemeral Detention Areas Living Streams Constructed Wetlands	Major system conveyance by overland flow along roads and floodways

Figure 2. Approach to maintaining pre-development hydrology.

Runoff disconnection is typically designed to retain storm events up to the 1 EY event. This can be achieved through employing BMP retention and/or detention techniques.

Retention systems are designed to prevent off-site discharges of rainfall runoff, up to the design EY event. Stormwater may be infiltrated to groundwater, evapotranspirated or used as a water source. Retention and reuse devices include rainwater tanks, aquifer storage and recovery, soakwells and infiltration basins.

Detention systems reduce the rate of off-site stormwater discharge by temporarily holding rainfall runoff (up to the design EY event) and then releasing it slowly. Constructed wetlands and ephemeral detention basins can be used to detain stormwater.

Some structural controls have a conveyance function, such as swales, bioretention systems and living streams. Many of these conveyance systems also provide seasonal detention and retention functions.

Stormwater quality management

Urban surfaces collect contaminants, which are typically washed off during storm events. Typical contributors to pollutants in runoff include vehicular traffic, industries, garden maintenance and fertilisers, animal manure, eroded sediments and vegetative litter.

The major non-point source pollutants in urban development include litter and sediments, nutrients, heavy metals, oxygen-demanding materials (e.g. leaves), toxic materials (e.g. pesticides), microorganisms, surfactants (e.g. detergents) and hydrocarbons (e.g. asphalt and petrol). Structural stormwater quality management typically involves utilising a combination of physical, chemical and biological processes to achieve the desired objectives. The locations of the various BMPs in the treatment train are important considerations in ensuring the sustained effectiveness of the management approach. Generally, the siting of BMPs should take account of the pollutant treated by each of the treatment measures. For example, gross pollutants and sediment can reduce the performance of infiltration systems, constructed wetlands, pervious paving and swales. Pre-screening devices such as buffer strips, gross pollutant traps and sediment trapping areas can be installed before discharging stormwater runoff to downstream treatment systems. BMPs to remove types of pollutants are shown in Table 2. Sections 1.7.5 and 1.7.6 discuss pollutant removal further.

Designers are encouraged to manage (retain and/or detain, and treat (if required)) runoff generated by the small rainfall event at-source as much as practical to increase disconnection and therefore reduce the collection and transportation of pollutants to receiving water bodies. Site investigations should be undertaken to select appropriate infiltration BMPs (as shown in Figure 1). Development should not result in deterioration of water quality in receiving water bodies, including mobilisation of existing contaminants.

For example, it would usually be unsuitable to install an infiltration system up-gradient of a plume of contaminated groundwater. Infiltration systems are generally designed to maintain the pre-development site hydrologic regime, so there should be no increase in the amount of groundwater recharge compared to pre-development conditions. However, management of some contaminated sites might require that there be no groundwater recharge up-gradient of the contamination plume.

It should be noted that the most effective stormwater quality management programs use non-structural BMPs to complement the selected structural BMPs. For example, gross pollutants can be managed through implementing improved site management practices, litter bin provision, street sweeping, litter collection, vegetation selection and maintenance and regulation practices. See Chapter 7 for information on the selection and design of non-structural controls to reduce pollutant sources.

Water conservation

Water sensitive urban design and total water cycle management view stormwater as a resource, and options for collecting and using stormwater for irrigation and non-potable water supply are now being examined in WA.

The reduction in rainfall in the south-west of WA since 1975 and population growth resulting in increased demand throughout most of WA has necessitated the investigation of alternative water sources. A number of structural (such as stormwater harvesting and rainwater tanks) and non-structural (such as ‘fit for purpose’ use of water) initiatives are being examined.

In considering stormwater conservation and reuse opportunities, it should be noted that stormwater is also an important source of water for maintaining the condition and function of natural wetlands and waterways, and providing for ecological water requirements.

1.7.5 Type of pollutant

An urban catchment is usually made up of multiple land uses (current and historical), which largely determine the stormwater pollutant profile of the catchment. For example, gross pollutants are prevalent in commercial areas, whereas sediments and nutrients are typically more prevalent in developing urban areas. Therefore, the promotion of at-source treatment targeting specific pollutants within the subcatchment provides a far more efficient approach to stormwater management. Additionally, at-source use or infiltration of stormwater minimises the collection and downstream transportation of pollutants. See Chapter 2 (section 3.1) for information on pollutants and their environmental impacts.

To effectively manage stormwater, it is necessary to match the selected BMPs with the site characteristics, including target pollutants and their transport pathways, groundwater levels and water quality of the receiving water body. For this selection to be successful, the designer needs to know about the catchment (land use, current stormwater management practices, soil types, hydrology, and groundwater interactions) and its pollutants (typical components, dominant transport pathways). If one of the objectives of the project is to improve water quality, it is essential that the water quality of the stormwater is known (or estimated), as this will influence the choice of BMPs.

Different processes are required for removing different pollutants and their components. If litter is a large problem (i.e. from high traffic or commercial areas), then an at-source gross pollutant trap (GPT) may be useful. If high concentrations of hydrocarbons from street runoff are expected, then an oil and grit trap may be the best solution. Stormwater with a high amount of sediment or nutrients attached to sediment can be treated by using retention/detention areas. Stormwater with a high amount of dissolved nutrients can be treated with BMPs that encourage biofilm growth, such as bioretention systems and constructed wetlands.

BMPs for the treatment of pollutant types can be assessed using the selection matrix in Table 2.

1.7.6 Pollutant size

Treatment of stormwater pollutants usually requires the reduction of one or more of the following pollutant sizes:

- Gross solids: contaminants larger than 5 mm, such as litter and organic material.
- Coarse to medium solids: contaminant particles between 5 mm and 0.125 mm.
- Fine particulates: contaminant particles between 0.125 mm and 0.010 mm.
- Very fine colloidal particulates: contaminants between 0.010 mm and 0.00045 mm. These contaminants, specifically nutrients, heavy metals, toxicants and hydrocarbons, attach themselves to fine sediments.
- Dissolved particulates: contaminants less than 0.00045 mm. Dissolved contaminants include nutrients, metals and salts.

BMPs for the treatment of pollutants of various sizes can be assessed using the selection matrix in Table 2.

1.7.7 Public health and safety

Mosquito and midge management

Structural stormwater management systems should be carefully designed to minimise the risk of chironomid midge and mosquito breeding. These insects cause significant nuisance, affecting lifestyle and amenity and have direct and indirect economic impacts. Some mosquito species that breed in these environments can also be vectors of Ross River virus and other mosquito-borne diseases. Ideally, all components of a stormwater treatment train should be designed to ensure that they do not contribute to or create an environment that increases the opportunity for nuisance or disease vector species breeding onsite. At-source infiltration, ephemeral detention areas, overland flow paths over vegetated surfaces (swales) and living streams are preferred stormwater management options as they minimise the creation of areas of stagnant water.

An overall mosquito and chironomid midge risk and management program will need to be undertaken as part of the overall pre-implementation planning for a development area. There are three stages involved in developing a mosquito management program:

- Stage 1 – Establish a mosquito monitoring program to identify existing levels of mosquito activity, species diversity and density, and public health risks, prior to any ground disturbance. This needs to be conducted at appropriate times of the year and when environmental conditions are favourable for mosquito breeding. Ideally, such baseline surveys should include more than one ‘mosquito season’ to allow for substantial inter-annual variation in mosquito activity.
- Stage 2 – Design the stormwater management system to ensure that constructed waterway and wetland areas, multiple use corridors, road gullies (etc.) do not contribute to onsite mosquito breeding. For example, infiltration, evapotranspiration or drawing down of the water to prevent pooling for longer than four days will prevent completion of the aquatic (larval) stages of the mosquito life cycle. The four-day guideline applies during the warmest months (e.g. late spring, summer and early autumn) in the south-west of WA and throughout the year in the north, as larval mosquitoes develop more rapidly in warmer temperatures. Contact the relevant local government or Department of Health for information about mosquito breeding risk seasons in different regions in WA. Other more intensive and expensive management approaches will be necessary in areas where infiltration/evapotranspiration of stormwater cannot be achieved within four days during risk times, either due to an impermeable substrate, a high groundwater table, existence of

permanent water (e.g. in rainwater tanks and some constructed wetlands) or other factors. See individual BMPs for guidance on how to reduce mosquito breeding risk in these situations.

- Stage 3 – Ongoing inspection, maintenance and management of the stormwater system to ensure that it continues to operate as designed, thereby reducing the risk of conditions likely to promote onsite mosquito breeding.

For detailed advice on reducing the risk of mosquitoes, see the Department of Health (2019) *Mosquito Management Manual*, *Mosquito Management* website ww2.health.wa.gov.au/Articles/J_M/Mosquito-management and the Midge Research Group of Western Australia (2007) *Chironomid Midge and Mosquito Risk Assessment Guide for Constructed Water Bodies*.

Accident risk

Steep sided structures present a potential safety risk, particularly for children who play near or attempt to climb into the structure and may fall in. This risk is increased when the structure contains enough water to drown a child. Steep-sided structures also present a hazard for maintenance staff. For example, ride-on mowers may tip over on bank grades steeper than 1:6. Therefore, structural controls must be designed to reduce accident risks (e.g. use barrier vegetation or fencing, or design bank grades no steeper than 1:6 on open systems). The location of structural controls within the urban landscape (such as where to site them within public open space or considering their design and location when near schools) should also take account of accident risk.

Recreational water quality

Reduction in recreational water quality is also a public health issue. Stormwater discharged directly into receiving waters used for recreational activities, such as swimming, can reduce the water quality and increase the public health risk by introducing pathogens into bathing waters. Therefore, overflow of stormwater runoff towards receiving water bodies should be by overland flow paths across vegetated surfaces. If overland flow is not possible, then stormwater outlets that discharge directly into a water body should be situated a sufficient distance (e.g. greater than 200 m) from popular bathing beaches.

1.7.8 Site suitability review

Issues such as the area of land available and neighbouring land uses will be a factor in BMP selection. For example, some BMPs (e.g. constructed wetlands) require significant areas of land, so decisions about the best use of land will need to be made. There must also be adequate room to allow personnel access to clean and maintain a device. Neighbouring land uses will need to be considered, as some BMPs might be incompatible with certain land uses (such as schools).

1.7.9 Life cycle costs

Consideration of post-construction costs and differing life expectancies is necessary to compare alternative strategies. The concept of life cycle costing combines the capital and operating costs of devices over their operating life.

It is important that a holistic approach be adopted to adequately assess the economic viability of structural BMPs for urban stormwater management. Cost and benefit analysis should include social and environmental outcomes. Additionally, the assessment should take into consideration the implicit inter-relationship between the three key functions of BMPs: water quantity, water quality and water conservation management.

It is estimated that savings in the life cycle cost of structural BMPs can be achieved by eliminating the use of large ponds/sumps; reducing impervious areas; minimising the use of pipes for conveyance; and reducing the amount of grading and clearing earthworks via retention of the natural landform. Coffman (1997) estimates this approach can reduce stormwater and site development design, construction and maintenance costs by 25–30% compared to conventional approaches. Taylor (2003) describes how to estimate and document life cycle costs and Taylor (2005) provides literature review information on life cycle costing for various structural controls circa 2005.

Costs included in the 'Cost' section of each BMP are quoted directly from the source information and have not been adjusted to 2022 prices.

Capital costs

Capital costs primarily consist of expenditures incurred to construct or install the BMP. Capital costs include all land acquisition, labour, equipment and material costs, excavation and grading, control structures, landscaping and appurtenances. Capital costs should also include professional fees for the design and construction of the BMP.

The cost of constructing a BMP is variable and largely depends onsite conditions and the size of catchment that it services. For example, rock encountered during construction may significantly increase excavation costs. Land cost is a critical component as it can surpass all other costs. This is an area where the water sensitive approach to stormwater management has benefits over conventional stormwater management, as land is not excised from community use because stormwater management systems are integrated within streetscapes and public open space.

Operating and maintenance costs

Operating and maintenance costs are post-construction costs that ensure the continued effectiveness of a BMP during its design life. Annual operating and maintenance costs include labour, materials, energy sources (e.g. to operate pumps) and equipment. Tasks typically carried out in a maintenance program include landscape maintenance, revegetation, weed control, structural maintenance, infiltration maintenance and cleaning.

Operating and maintenance costs can be divided into either aesthetic or functional. Functional maintenance is important for device performance and public safety, while aesthetic maintenance is important for public acceptance of BMPs. Aesthetic appearance is particularly important for visible BMPs.

Operating and maintenance costs can be more difficult to estimate than the capital costs, but they are sometimes the most critical variable. Variations in maintenance techniques and the amount and contamination characteristics of the removed material (thus the disposal costs) all contribute towards maintenance costs. It is therefore important that operating costs are considered and budgeted for during the design phase. See Section 1.9.1 for more information about maintenance.

1.8 Urban water management plans (sub-division & development water management report)

Documentation of selected BMPs will need to be addressed in urban water management plans (or sub-division and development water management reports) prepared for a development area.

Documentation will need to include characteristics of selected BMPs (e.g. location, size and type), the timing of their implementation, who is responsible for their implementation and maintenance, and how they will be monitored and evaluated.

1.9 Implementation

1.9.1 Maintenance

Maintenance requirements, from the construction phase through to the expected lifetime of the BMP, need to be factored into the design. A maintenance plan and associated reporting processes must be developed during the design phase as part of the plan or report. Maintenance and asset managers will use these plans to ensure that the BMPs function as designed. The maintenance plan should be reviewed approximately every three years.

The plan should generally address the following topics:

- BMP design details
- costs
- responsibilities
- inspection frequency
- maintenance timing and frequency
- vegetation replacement and weed and nuisance/disease vector insect control
- performance monitoring data collection/storage requirements (i.e. during inspections)
- record keeping requirements
- detailed clean-out procedures (such as equipment, maintenance techniques, occupational safety and health, public safety, environmental management considerations, disposal requirements of removed material, access issues, stakeholder notification requirements and data collection requirements) to remove sediment and litter.

Some devices, such as GPTs, require regular inspection and monitoring to determine the optimal frequency and timing of cleaning to ensure they do not become a source of pollutants. For example, nutrients in an organic form can be converted to a bioavailable form in the anoxic environment in a poorly maintained trap. Remobilisation of trapped pollutants or bypassing due to a lack of storage volume in an unmaintained trap could result in the supply of pollutants to the stormwater system.

Constructed wetlands and infiltration systems require regular inspection for sediment build-up. Online vegetated systems (i.e. systems that are part of the main stormwater conveyance network) need to be periodically inspected to ensure that prolific plant growth does not block the channel. Branches or plants that are dislodged during high flows and transported downstream may need to be cleared if they become trapped and form a debris dam or block a culvert. Vegetation may also need to be periodically harvested to enhance nutrient removal or re-establish conveyance/storage capacity.

More information about maintenance is provided in each BMP section.

1.9.2 Monitoring and evaluation

Monitoring and evaluation should be conducted to determine if the BMP is performing as intended. Chapter 10 provides a process for how to monitor and evaluate structural and non-structural controls.

The pollutant removal effectiveness and performance of some structural controls is not well understood in WA, particularly on the Swan Coastal Plain. This is because most research has been conducted in the eastern states, where the climate and hydrogeology is very different to that of the Swan Coastal Plain. Therefore, where monitoring and evaluation has been undertaken, it would be appreciated if an electronic copy of the final report be sent to the Department of Water and Environmental Regulation, to help disseminate the knowledge of successes and failures to other stakeholders, as part of a continual improvement process for this manual.

1.10 Summary of structural controls addressed in this chapter

Stormwater storage and use

Demonstration domestic rainwater tank. (Photograph: Renee Romyn.)

Wastewater aquifer storage and recovery pump at Bolivar, South Australia. (Photograph: CSIRO Land and Water website © CSIRO 2005.)

Stormwater retention and use onsite is a part of integrated water cycle management in the urban setting. This best management practice is sometimes referred to as stormwater harvesting. Stormwater retention and use within an urban catchment has the potential to mitigate the impacts of development on flow regimes and provide an alternative non-potable water supply source. Capturing stormwater at-source and preventing runoff from small rainfall events also has the benefit of preventing the risk of this runoff picking up and transporting pollutants as it flows through the urban landscape.

Stormwater retention and use BMPs include:

- rainwater storage devices, including rainwater tanks
- below-ground rainwater/stormwater storage units and media filled storage tanks
- rain gardens, including roof gardens and small bioretention gardens
- stormwater sculptures and water features
- managed aquifer recharge (MAR).

MAR involves the storage of water in suitable aquifers through infiltration or well injection. This additional stored water can be recovered for use during periods of high demand. While formally managed MAR schemes at regional scales are relatively new in WA, use of local infiltration systems and irrigation bores have had widespread application in WA at domestic and local authority scales, operating as informal MAR schemes for non-potable use. Systems which directly infiltrate the collected stormwater onsite are discussed in Section 3 of Chapter 9 (Infiltration Systems).

This manual does not address the use of rainwater storage systems to supply a drinking water source. For information on using roof water for human consumption refer to the Department of Health and the Water Corporation.

Best management practices

BMP System		Applicable Scale	Primary and Secondary Functions	Status of Application in WA
2.1	Rainwater Storage Systems	Lot, Street	*Water Conservation* Water Quantity (Retention)	Traditionally applied in rural/ regional areas. Increased recent application as a non-potable supply for urban development.
2.2	Managed Aquifer Recharge	Lot, Street, Precinct, Regional	*Water Conservation* Water Quantity (Retention)	Lot (domestic) and precinct schemes (local authority public open space irrigation) have widespread use. Use of MAR schemes at the regional scale is being trialled and researched. Some limited application in Perth metropolitan area.

Infiltration systems

Infiltration through pervious paving. (Source: Washington Aggregates and Concrete Association 2006.)

Pervious surface under gutterless roof, Shire of Broome. (Photograph: Allan Ralph, Shire of Broome 2005.)

Infiltration BMPs consist of systems where the majority of the stormwater is infiltrated to the ground, rather than discharged to a receiving surface water body. Infiltration systems cover a wide range of application scales (lot to regional) and include infiltration basins and trenches, soakwells and pervious pavements. Infiltration can also be simply achieved through the provision of a soil surface or vegetated area allocated for this purpose, for example by directing roof runoff to a garden bed.

Infiltration systems are used at different scales and under different conditions to accomplish the same goals of reducing stormwater runoff peak flows and volumes; minimising pollution conveyance; reducing downstream flooding; managing the hydrologic regime entering receiving environments; and increasing groundwater recharge.

Sandy soils are ideal for infiltration systems. Even in areas where soils are less permeable, infiltration systems may still be an option for stormwater management if other engineering factors dictate the use of more permeable fill to raise the site level, or slow drainage infiltration systems are adopted. Infiltration at-source should be considered in preference to end-of-pipe or end-of-catchment systems where the stormwater has had the opportunity to pick up pollutants.

To prevent infiltration systems from being clogged with sediment/litter during road and housing/building construction, temporary bunding or sediment controls need to be installed. See section 2.1.1 'Land development and construction sites' of Chapter 7 for information about site management practices.

Best management practices

BMP system		**Applicable scale**	**Primary and secondary functions**	**Status of application in WA**
3.1	Infiltration Basins and Trenches	Street, Precinct, Regional	*Water Quantity (Retention)* Water Quality Water conservation	Widespread local application of infiltration basins. Infiltration trenches used to a lesser extent, in many cases as a retrofitting application to existing fenced sumps.
3.2	Soakwells	Lot, Street	*Water Quantity (Retention)* Water Quality Water conservation	Widespread local application at domestic, local authority and development scales.
3.3	Pervious Pavement	Lot, Street	*Water Quantity (Retention)* Water Quality Water conservation	Limited previous local application. Recent trials and increasing application in WA.

Conveyance systems

Bannister Creek drain to living stream project, Lynwood, WA. (Photograph: Department of Water 2007.)

Grassed swale in parkland, Daglish, WA. (Photograph: Department of Water 2006.)

Natural and rehabilitated living streams, bioretention systems and swales are increasingly playing a role in stormwater management, providing conveyance of runoff and an opportunity for water quality improvement and detention and retention of flows. These conveyance systems are being applied locally to new development areas and also retrofitted to existing development areas to replace existing steep-sided trapezoidal drains and to rehabilitate degraded waterways. In developed urban areas, these systems are also used to supplement or, where feasible, replace piped drainage.

If designed correctly, these conveyance systems can provide aesthetic, recreational and conservation values in the urban environment.

Best management practices

BMP System		**Applicable Scale**	**Primary and Secondary Functions**	**Status of Application in WA**
4.1	Swales and Buffer Strips	Lot, Street, Precinct	*Water Quantity (Conveyance, Retention and Detention)* Water conservation Water Quality	Widespread local application, particularly grassed swales. Used as both an infiltration system for frequently occurring events and conveyance of larger storms.
4.2	Bioretention Systems	Street, Precinct	*Water Quality* Water Quantity (Conveyance, Retention and Detention) Water Conservation	Wide application in eastern states, particularly areas of low infiltration. Limited local use to date with several trial applications in WA currently in progress.
4.3	Living Streams	Precinct, Regional	*Water Quantity (Conveyance Retention and Detention)* Water conservation Water Quality	Increased use recently, particularly for development of rural areas with steep-sided trapezoidal drains.

Detention systems

Liege Street Wetland, Cannington, during construction in May 2004 and immediately following completion of construction and initial planting in September 2004. (Photographs: Department of Water.)

Detention BMPs consist of a range of systems in which stormwater is primarily detained (rather than infiltrated) and water then discharged to a receiving environment. The primary detention system types include constructed wetlands, dry/ephemeral detention areas and onsite detention systems.

While the primary function of these systems in many cases is peak flow attenuation and flood protection of downstream environments, in the case of constructed wetlands detention is utilised together with biological processes for pollutant removal.

The Department of Water and Environmental Regulation is not including constructed ponds and lakes as a stormwater quality improvement BMP in this manual. Constructed lakes are defined as constructed, permanently inundated basins of open water, formed by simple dam walls or by excavation below-ground level. Constructed wetlands are vegetated detention areas that are designed and built specifically to remove pollutants from stormwater runoff. Constructed wetlands are designed to mimic natural wetlands in WA, which are often ephemeral, and avoid the problems often associated with constructed lakes. Constructed wetlands are designed to provide additional environmental benefits, such as valuable native flora and fauna habitat.

For information regarding the Department of Water and Environmental Regulation's current position on the construction of ponds and lakes, the reader is referred to the Interim Position Statement: Constructed Lakes (Department of Water 2007).

Best management practices

BMP System		Applicable Scale	Primary and Secondary Functions	Status of Application in WA
5.1	Dry/ephemeral detention areas	Precinct, regional	*Water quantity (Detention)* water quality	Widespread local application at subdivisional level, particularly as grassed multiple use areas.
5.2	Constructed wetlands	Precinct, regional	*Water quality* Water Quantity (Detention)	Limited previous application in WA due to design issues related to local environmental considerations (e.g. areas with high water tables and permeable sands). This chapter includes design guidelines for constructed wetlands on the Swan Coastal Plain.
	Onsite detention systems	Lot	*Water quantity (Detention)*	Refer to the Infiltration Systems and Stormwater Storage and Use BMPs for measures that can be used to retain/ detain stormwater onsite.

Pollutant control

Underground chamber device GPT, Backshall Place, Wanneroo. (Photograph: JDA Consultant Hydrologists 2004.)

GPT chamber following a storm event, near Lake Jualbup, Shenton Park. (Photograph: JDA Consultant Hydrologists 2005.)

This guideline summarises the range of pollutant control devices being applied in WA to new development areas and also retrofitted to existing development areas. The pollutant control devices presented are litter and sediment management systems (e.g. GPTs, trash racks, etc.) and hydrocarbon management systems (e.g. oil-water separators). These systems typically operate as one component of an overall stormwater management treatment train protecting the receiving environment. These pollutant control devices are often used where land constraints prohibit the use of other BMPs, or as pre-treatment to other BMPs, such as constructed wetlands.

Litter and sediment management (LSM) devices are primary treatment measures that retain gross pollutants by physical screening or rapid sedimentation techniques. Hydrocarbon management techniques are typically used in commercial, industrial and transportation land uses such as carparks and service stations, where impervious areas are expected to receive high hydrocarbon loadings.

Best management practices

BMP System		Applicable Scale	Primary and Secondary Functions	Status of Application in WA
6.1	Litter and sediment management	Lot, street, precinct	*Water quality*	Generally used as end-of-pipe solution to protect receiving environments; however, should usually be applied at- source to target areas with potential for high pollutant runoff. At-source application is usually more cost effective than LSM systems applied in-transit or end-of-pipe. There is a range of LSM devices available in WA designed to target various types of pollutants. Used in new developments and retrofitted applications. Requirement for these devices for new developments has been reduced due to the focus on retention of stormwater at-source and disconnecting pollutant transport pathways.
6.2	Hydrocarbon Management	Lot, Street	*Water Quality*	Some installations along major roads and intersections in WA to capture potential spills and treat road runoff. Increasingly used at-source in new developments and in retrofitting applications to treat runoff from hardstand areas likely to contain hydrocarbon contamination, foe example at service stations, carparks and industrial premises. Best used in combination with non-structural controls, such as good house-keeping practices on industrial sites, to minimise pollution of stormwater. Various hydrocarbon traps are available within WA.

1.11 Case studies

Bridgewater South Estate, Mandurah

Project description

Bridgewater South Stage 3 subdivision is located between the Indian Ocean and the Peel-Harvey Estuary and is about 3.5 km south of Mandurah. The development was completed in 2005. The area has a Mediterranean-type climate, with hot, dry summers and cool, wet winters. The annual rainfall pattern for Mandurah is shown in Figure 1. The subdivision is located predominantly on relatively free draining light grey sands. Despite the generally high permeability soils, some areas have low infiltration capacity due to their proximity to the estuary, which leaves little separation between surface levels and groundwater levels at certain times of the year.

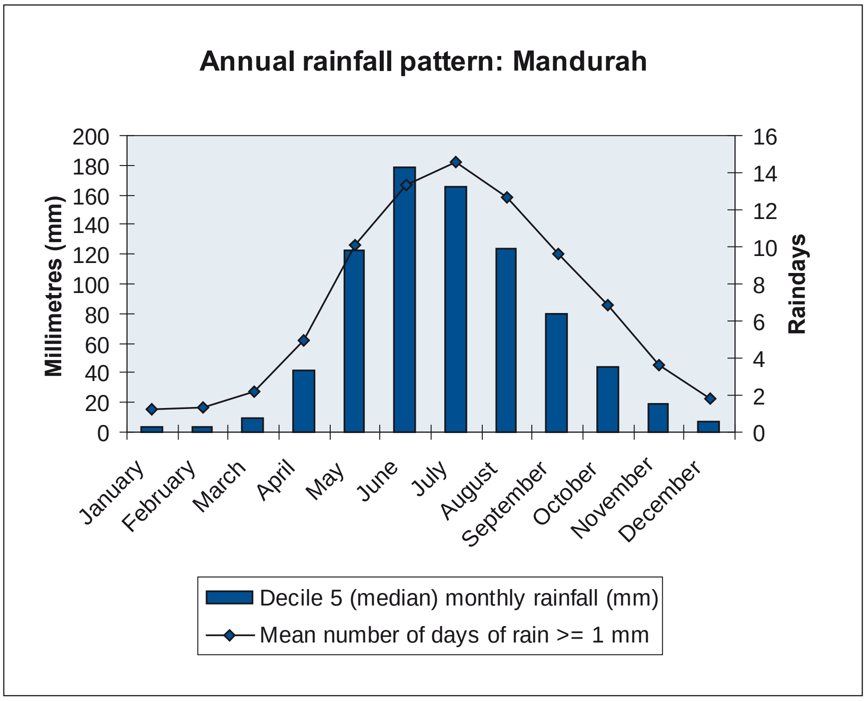


Figure 1. Mandurah annual rainfall pattern. (Source: Bureau of Meteorology 2007.)

The development is located adjacent to a Ramsar registered site (the Peel-Yalgorup system) and a conservation category wetland (a sumpland within Len Howard Reserve). Protection of these sensitive receiving environments influenced the stormwater management design.

The City of Mandurah is experiencing significant levels of growth. The city aims to achieve the following when undertaking development:

- protection of environmental assets for future generations
- continuous improvement in achieving best outcomes for the community
- ensuring environmental and economic wellbeing.

In order to achieve these aims within the Bridgewater South Stage 3 subdivision, the City helped to develop an innovative stormwater management system that utilised a combination of best management techniques that were not commonly applied in WA (Figure 2).

Approaches implemented

The following structural and non-structural stormwater management practices were implemented at the site:

Structural controls:

- flush and broken kerbing
- infiltration, retention and detention at-source
- dry infiltration basins
- infiltration swales (turfed or planted with native vegetation)
- disconnection from receiving water bodies
- overland flows through vegetated buffers
- roads to convey greater than 1% AEP events
- lot levels set to protect properties from flooding.

Non-structural controls:

- proactive guidance provided by the City of Mandurah and consultation with the developer about appropriate stormwater management approaches
- signage to promote water sensitive urban design (WSUD) and the protection of receiving environments
- ongoing commitment by the City of Mandurah to addressing community questions and concerns regarding pooling water and to explain the importance of this water retention in protecting wetlands and the Peel-Harvey Estuary.

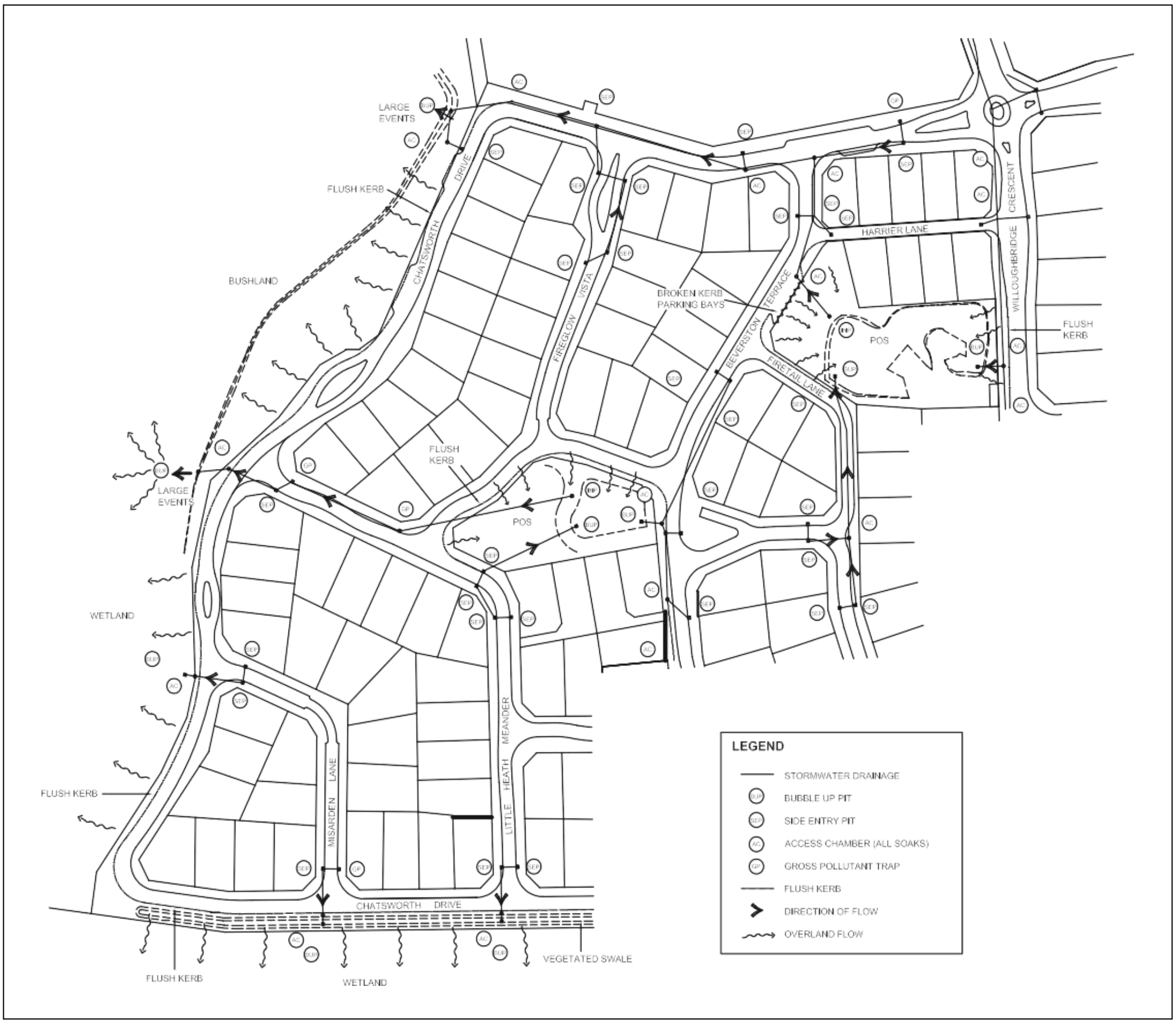


Figure 2. Drainage plan for Bridgewater South Estate inline storage and treatment system. (Source: Department of Water).

Piped system

The piped system includes leaky access chambers that act as soakwells (these are side entry pits with a permeable base such as blue metal) to capture and infiltrate flows close to source. These chambers intercept small storm events. In several of the subcatchments, higher flows are conveyed by the piped system to bubble-up pits located in parks (Figure 3).

Figure 3. Bubble-up pit located in a park. (Photograph: Department of Water 2006.)

Public open space (POS):

Grassed parks in the subdivision POS have been contoured to capture and detain flows (see Figures 4 and 5). The levels of the inlets and outlets to the detention areas have been designed to capture up to the 1 EY event and promote overland flow and infiltration. Levels have been designed to prevent water pooling for longer than 72 hours. Landholders are aware of the parks' role in stormwater management due to public education and signage installed by the City of Mandurah.

Figure 4. A multiple use park incorporating a landscaped stormwater detention area. (Photograph: Department of Water 2006.)

Figure 5.Grassed swale and bubble-up. (Photograph: Department of Water 2006.)

The POS areas have been developed to provide both recreation and stormwater management functions. Roads have been designed to convey the 1% AEP flood. At-source infiltration measures, in-transit detention areas and the use of the road system for conveyance of major flows have removed the need for pipes in some road sections and resulted in smaller pipes being required throughout the development. This provided cost savings to the developer in terms of reduced materials and installation outlays.

Overland flow

Where possible, overland flow has been utilised in preference to a piped system to slow flows and provide some treatment of stormwater. This approach has been implemented on the boundaries of parkland where runoff flows off the road, over flush kerbing and through grassed areas, which promotes deposition of particulate matter (see Figure 6). Overland flow at the downstream end of catchments has been achieved through the use of bubble-up pits from small piped catchments (see Figure 7). In the south-west corner of the subdivision, disconnection is achieved through the construction of a swale, which captures runoff from the adjacent road surface and piped areas. If the storage capacity is exceeded, the swale is designed to create a broad weir effect, encouraging lateral overland flow.

Disconnection

Disconnection in the stormwater system has been achieved through providing infiltration points in the piped system for small events, directing runoff into parks for medium events and using overland flow paths to convey large events. There are no pipes discharging into the adjacent water bodies. The development has been broken into five subcatchments. This decentralisation of the drainage network assists in managing stormwater by decreasing the concentration of flows and pollutants.

Results/achievements

The development of the Bridgewater South Estate catchment has avoided direct discharge of stormwater into the adjacent estuary and wetland, whereas a traditional drainage planning approach would have resulted in piped discharge to these sensitive water bodies. This has been achieved through a catchment approach to stormwater management, where numerous small management measures have been put in place throughout the catchment to manage stormwater close to source.

Figure 6. Flush kerbing to allow dispersion of road runoff into adjacent parkland. (Photograph: Department of Water 2006.)

Figure 7. Bubble-up pit located within a swale at the downstream end of the catchment. (Photograph: Department of Water 2007.)

The measures implemented individually could not have successfully managed the stormwater for the catchment. However, the cumulative impact of the combination of several measures has ensured that only large events reach receiving water bodies via overland flow.

The outcomes of the stormwater management approach implemented in this development demonstrate the potential environmental benefits, as well as cost savings that can be achieved. Distributing management measures throughout the catchment and utilising overland flow paths, instead of collecting and conveying flow in one centralised system, can result in significant reductions in piping and associated infrastructure requirements.

Located close to the high-value environments of the Ramsar-recognised Peel-Harvey Estuary and a conservation category wetland, the Bridgewater South Estate development required a sensitive approach to stormwater management. As the Peel-Harvey Estuary is an iconic landscape that is valued by the community, actions to protect its qualities were supported by residents. Community awareness about the importance of protecting the estuary had been raised by their experiences of the major adverse impacts caused by water quality degradation, such as fish kills and algal blooms.

The levels of bubble-up pits and overflows in the stormwater system required some minor adjustments to fine tune retention and pooling times in the detention areas. These adjustments have resulted in a system that performs well and is acceptable to the community.

Acknowledgments

Information and feedback provided by Mr Grahame Heal, Manager Infrastructure and Services, City of Mandurah.

Beachridge Estate, Jurien Bay

Project description

Beachridge Estate is the first stage of development of a 2000 hectare coastal bushland estate south of Jurien Bay in the Shire of Dandaragan. Jurien Bay experiences a Mediterranean-type climate, with hot, dry summers and cool, wet winters (Figure 1). The site is characterised by sandy soils and a shallow watertable. Groundwater flows in a westerly direction to the ocean into the Jurien Bay Marine Park. The Hill River bounds the development to the south. Pre-development, there were no surface drainage lines on the site because all stormwater infiltrated in the highly permeable sands, resulting in no runoff.

Developers have maximised the hydrologic benefit provided by the sandy soils and achieved a drainage system that avoids the use of pipes. This approach has resulted in stormwater from the majority of rainfall events infiltrating at or near its source. This departure from traditional drainage techniques has been combined with non-structural controls, such as incentives for appropriate landscaping and onsite water storage, further reducing the volumes and impacts of runoff from the site.

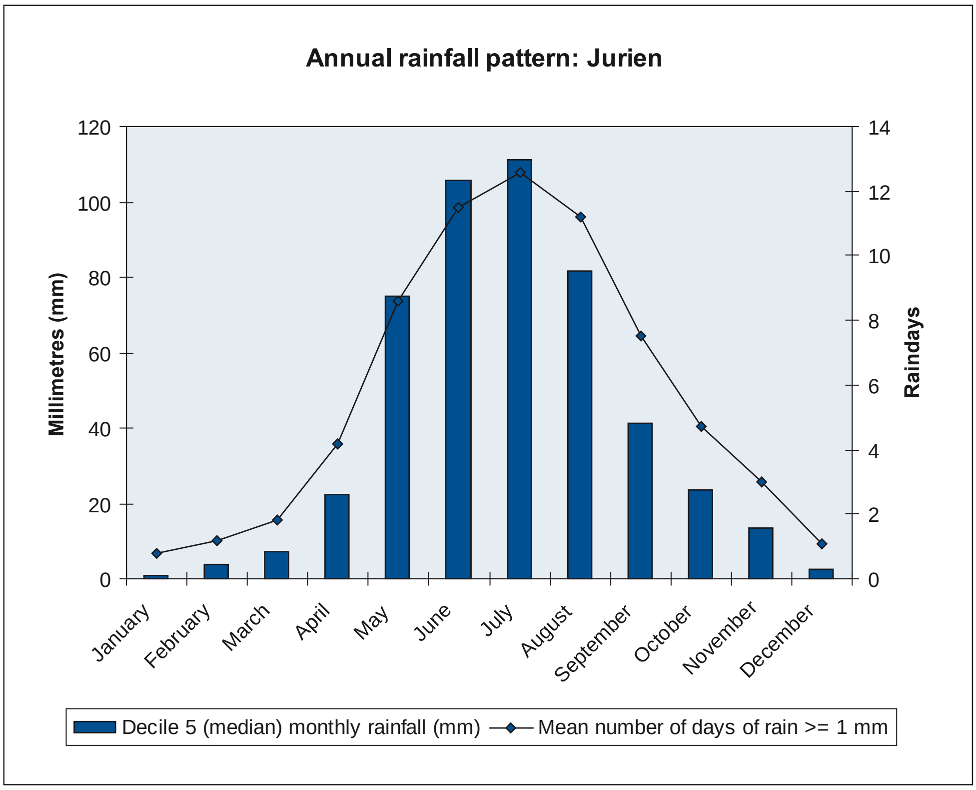


Figure 1. Jurien Bay annual rainfall pattern. (Source: Bureau of Meteorology 2007.)

Approaches implemented

Due to the high capacity of the soil to infiltrate stormwater, swale systems have been used as the primary structural control method to manage stormwater in this development. The following structural and non-structural controls have been used on the site:

Structural:

- swales planted with local native vegetation
- a disconnected drainage plan, where large events are directed to a series of decentralised ‘nodes’ (Figure 2)

- roads graded to direct flow to POS for greater than 10% AEP events.

Non-structural:

- rainwater tank rebates
- water conservation landscaping
- community education, including provision of local native plants species lists to landowners
- limited turf in POS
- reduced dwelling setback (3 m) to minimise front yard areas
- covenants in place to prevent planting of non-indigenous vegetation in front yards.

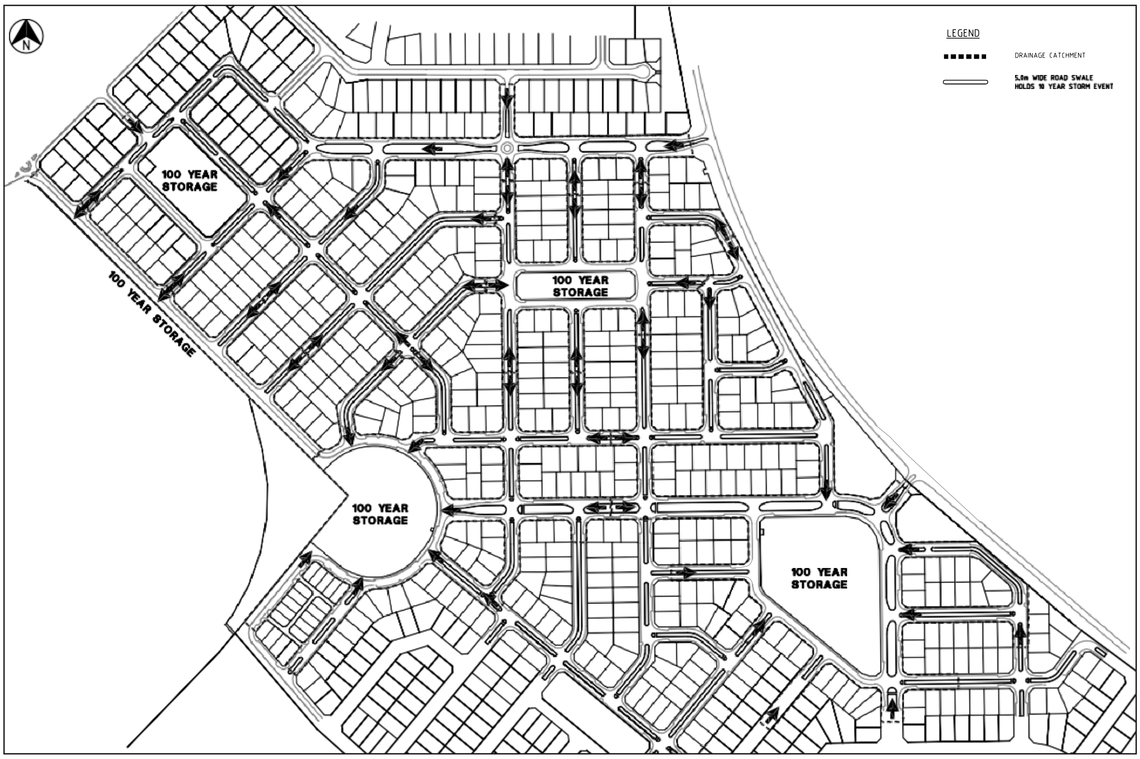


Figure 2. Overall drainage plan for Beachridge Estate. (Source: SKM 2007.)

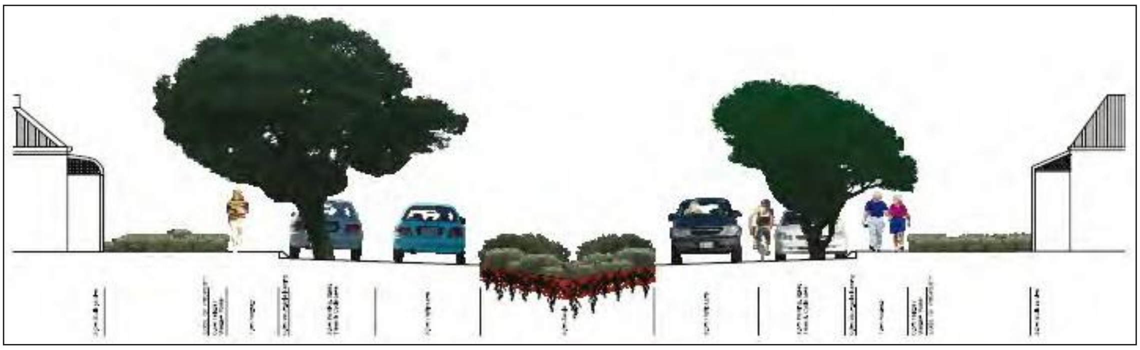


Figure 3. Swale concept plan from the Water Management Plan. (Source: MGA Town Planners 2003.)

Landscaped drainage swales built in road medians (Figures 3, 4 and 5) remove the need for traditional sumps and pipes by providing detention/retention opportunities high in the catchment, before flows and potentially associated pollutants are collected and concentrated. Slowing the movement of water promotes infiltration in the highly permeable underlying coastal sands. The aim of the design was to maintain the pre- development catchment hydrology by returning water to the environment as close to its source as possible, while preventing flooding of the urban area.

The swales have been designed to accommodate stormwater from up to the 10% AEP rainfall event. Flows in excess of this event are directed by overland flow paths to storage areas in POS for retention, detention and infiltration. The design of the swales has been based on calculations of runoff from hardstand areas, such as roads and pavements, as stormwater from residential lots and parkland areas is retained onsite and does not runoff to the swale network.

Swales are covered with coarse mulch and vegetated with indigenous plant species to avoid the need for irrigation after an establishment period of two years (Figures 4 and 5). This reduces the consumption of water resources traditionally associated with maintenance of turfed median strips. Additionally, maintenance of the native vegetation does not require the use of fertiliser, which consequently removes one of the sources of nutrient inputs that is commonly associated with urban environments.

Figure 4. Newly constructed and mulched swale. (Photograph: Ardross Developments 2004.)

Figure 5. Swale with established native vegetation. (Photograph: Ardross Developments 2007.)

Plant species were chosen based on their growth form (low shrub and groundcover) and water requirements. The indigenous plants have deep root systems that assist with maintaining the porosity of swale areas to promote infiltration through the root zone.

Temporary barriers erected around the swales during the construction phase protected the swales from sediment associated with greenfield development, such as from construction and wind erosion of disturbed topsoil. The physical barrier also protected the swales from vehicle access, which can damage the swales and compact the substrate, resulting in decreased infiltration rates.

Roads have been graded to direct flows in excess of the swale capacity to the designated storage node for infiltration (Figure 2).

Results/achievements

The stormwater management system at Beachridge Estate has been in place since 2003 and has achieved the design objectives for stormwater quantity management. All storm events to date, which have been small to moderate, have been successfully infiltrated within the swales.

In planning this project, the shire recognised an innovative approach was required for development so close to the highly valued coastal environment. Although there were concerns about the function of a system without pipes and potential impacts of stormwater infiltration, for example damage to infrastructure, the benefits offered by the proposed stormwater system outweighed the perceived risks.

The approach to stormwater management implemented at Beachridge Estate demonstrates the ability of non-traditional systems to perform flood protection, enhance aesthetics and more sustainably manage surface and groundwater resources. Vegetation has established well and is protected by temporary low barriers from wind erosion and vehicular access. The vegetation and mulch has successfully stabilised the slopes of the swales.

The construction of narrower roads has provided an additional benefit of encouraging reduced traffic speeds, and therefore improved safety and liveability of the development for pedestrians.

Challenges/lessons learnt

Stabilisation is required to prevent undermining of hardstand surfaces in areas that will experience higher flows, such as swale end points. Even though flow is dissipated, there has been some erosion at the ends of the swales, where the runoff is concentrated more than along the lateral edges of the swales. The developer is addressing this issue by further armouring (stone-pitching) and planting.

The alternative road layout of swales in the road median and street trees along roadside parking has reduced access to driveways and caused some problems with backing of boat-trailers and caravans, which is a significant issue in an area of high boat ownership.

Acknowledgments

Information and photos provided by David Kasehagen, Ecoscape, and Daniel Skerratt, Ardross Group of Companies. Diagrams provided by Vikki Wardley, SKM. Information provided by Craig Tuesley, former Department of Water, Geraldton.

Boronia Ridge Estate, Walpole

Project description

Boronia Ridge Stage 2A is a subdivision at Walpole in the Shire of Manjimup that was completed in 2002. The Walpole Inlet receives runoff from the development. Like most southern coastal towns in WA, Walpole has a climate where regular rainfall is expected even during the summer months (Figure 1). In southern coastal areas, the winter peak rainfall is less than in the south-western coastal areas and the summer rainfall is higher and more uniform. This is unlike at sites further to the north on the coastline, such as Perth.

Due to the more uniform year-round rainfall, a well-established perched groundwater pattern has developed over the site. Climatic and geological conditions have led to the formation of the palismont and palislope wetland types as identified by Semeniuk (1997). These landforms were investigated and found to be the result of water perching over an area of shallow laterite or 'coffee rock', which sustained a vegetation community with wetland properties. The high-perched water table was a dominant design consideration at the site. It was important that the stormwater management design did not artificially lower or alter the flow of the perched water table. Installation of services, such as sewers, was designed so that they did not act as subsoil drains across the site slope.

An assessment of the soil profile at the site found moderately to highly permeable sandy surface soils overlaying impermeable or lower permeability clayey subsoils at varying depths. This clay layer contributes to the formation of the perched water table. Shallow infiltration systems were considered suitable in areas with deeper, sandy surface soils. In areas where clayey soils were at the surface, then alternative stormwater management measures, such as rock armoured stilling pools to dissipate flows (Figure 10), were implemented.

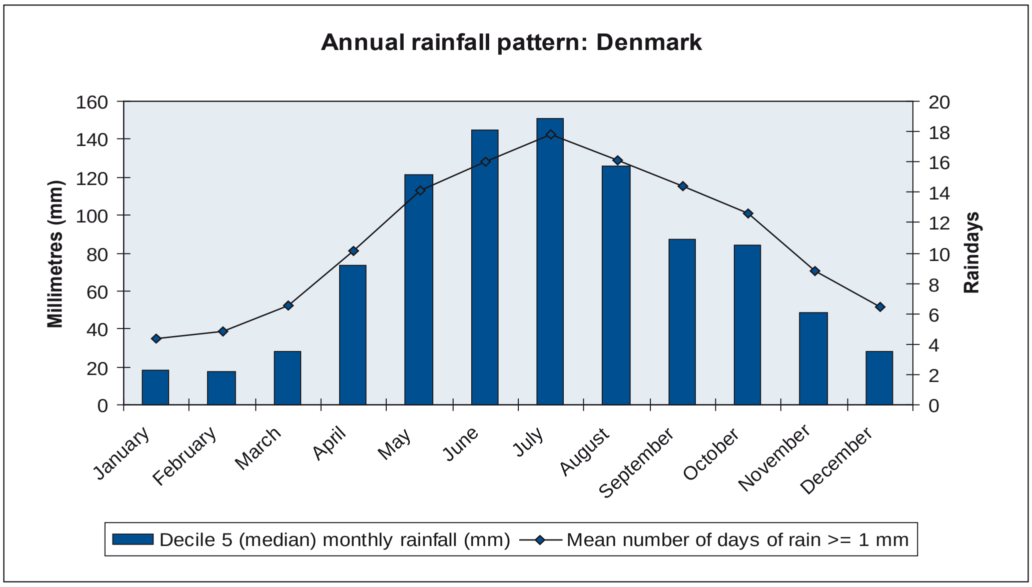


Figure 1. Denmark annual rainfall pattern. (Source: Bureau of Meteorology 2007.)

The year-round rainfall experienced in this region means that less emphasis needs to be placed on treating the first flush events that occur in the drier areas of the state. First flush rains generally carry higher pollutant loads that have built up in urban catchments over the dry season. In regions where year-round rainfall occurs, it is believed that these pollutants are flushed more regularly and so there is less of a concentrated load in the first flush rains. In the southern coastal region, pollutants mobilised by the first major rains of the season are still significant and must be addressed, but it is more important to develop stormwater management systems that will function effectively year-round.

Approaches implemented

The BMPs introduced in this stage of the subdivision were based on knowledge gathered from earlier stages, site investigations undertaken and the latest practices and principles of WSUD. The implemented stormwater management system consisted of the following structural (Figure 2) and non- structural techniques:

Structural:

- kerbed roads elevated above natural surface levels, to not interfere with subsurface flows
- side entry pit collection systems that bubble-up into adjacent detention areas (Figure 3)
- flow retention and detention in soakwells and swale systems capturing up to the 1 EY event
- piped conveyance system for flows exceeding the 0.2 EY event (Figure 4)
- extended detention basins and flow dissipation structures at the end of each piped system (Figures 9 and 10).

Non-structural:

- maintenance of the stormwater system to ensure effective operation of the BMPs.

The stormwater management approach that has been developed for this site relies heavily on infiltration of stormwater as close to its source as possible, to maintain the pre-development hydrology of the catchment. Infiltrating stormwater at or near its source has minimised the impact on the water balance of the perched water table system. The at-source treatment of stormwater also reduces the transfer of any pollutants associated with urban development to the final receiving water body, the Walpole Inlet. No single BMP could be applied to this site to solve all of the stormwater management issues; rather an integrated series of BMPs was required to suit the conditions.

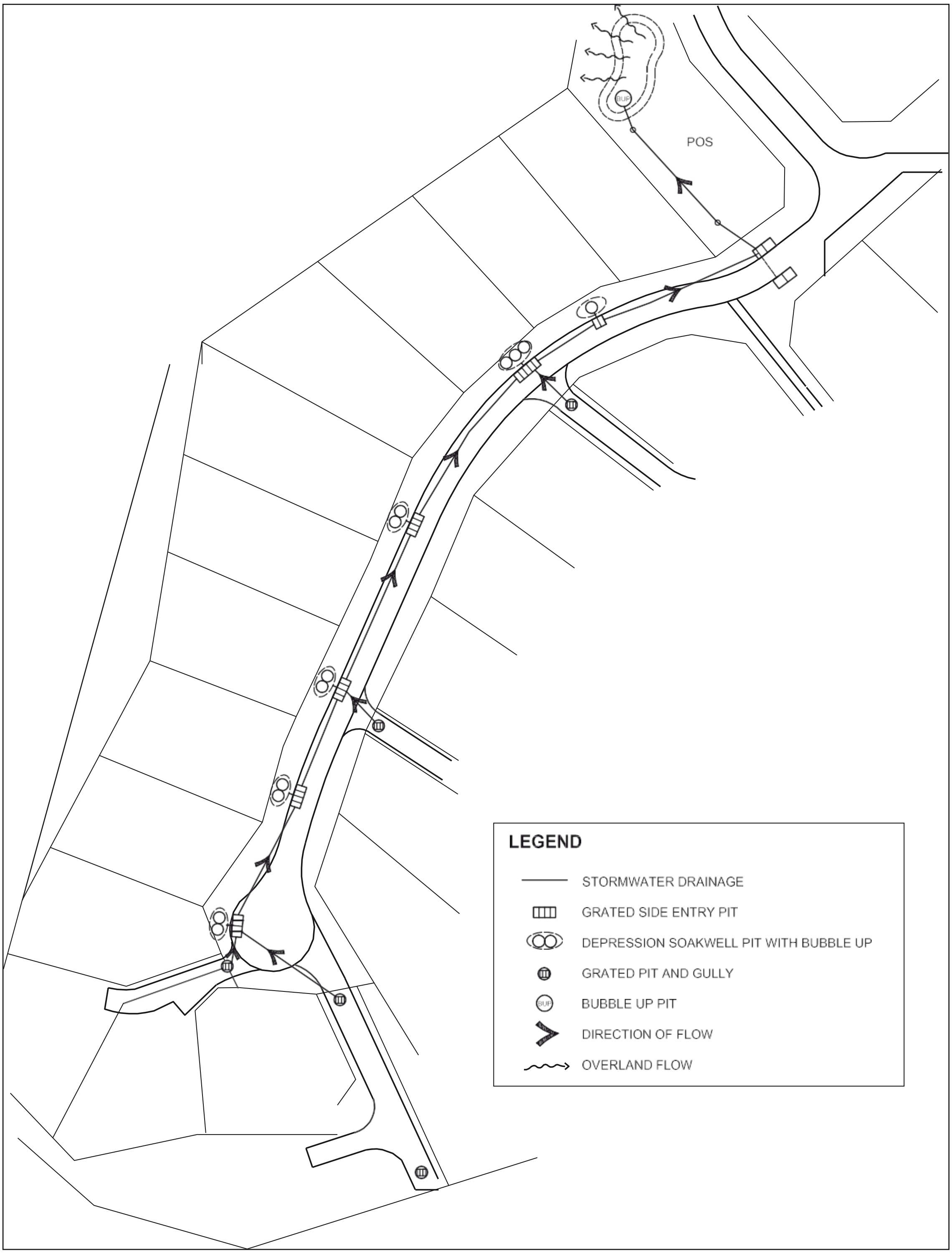


Figure 2. Boronia Ridge drainage plan. (Source: Department of Water.)

Lot-scale stormwater management

Impervious property crossovers were built with flush kerbing so that runoff sheets onsite. A series of soakwells, with linked high level overflows, were used when site constraints prevented overland flows. There is no direct flow into the foreshore reserve.

Figure 3. Side entry pit and bubble-up located within mulched detention area. (Photograph: TME 2003.)

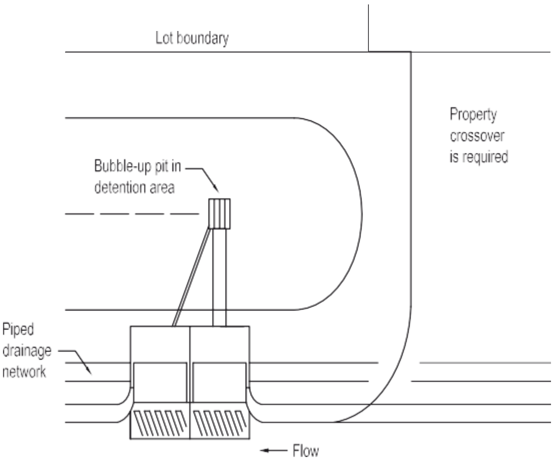


Figure 4. Plan view of the pipe network with bubble-up connection to the piped system. (Source: TME 2003.)

Figure 5. Soakwell construction. (Photograph: TME 2003.)

Figure 6. Construction of side entry pits to capture road runoff and divert it into soakwells that have bubble-up lids and are located within a detention area. (Photograph: TME 2003.)

Infiltration

In a conventional side entry pit system, stormwater is collected and transferred into a piped system for direct conveyance to the downstream stormwater network or receiving environment. At Boronia Ridge Estate, kerbed roads are used to collect and channel road and verge runoff to the side entry pit collection zones. These pits are used to capture flows and transfer them directly to a soakwell that is located in a detention area within the verge, as shown in Figures 3 and 4. The pit and soakwell systems are located at regular intervals to ensure capture and retention of minor events as close to source as practical (Figure 2).

The soakwells directly infiltrate stormwater into the surrounding permeable surface soils. The soakwells are sized to capture up to the 1 EY event. The stormwater management system is very simple but highly effective at trapping low flows and infiltrating them at-source, hence minimising any impacts on the subsurface hydrology of the site. Locating the soakwells within detention areas allows additional infiltration and storage capacity of stormwater. Construction of the side entry pit and soakwell system is shown in Figures 5 and 6. The detention areas also act as capture zones for runoff from the urban lots and help direct this runoff into the piped system (see Figure 7).

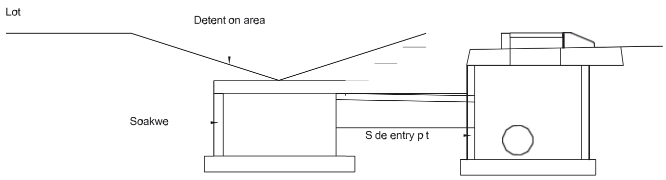


Figure 7. Central drainage linked to soakwells and detention area. (Source: TME 2003.)

Figure 8. Streetscape of constructed system. (Photograph: TME 2003.)

Flow that exceeds the capacity of the detention area overflows into a second side entry pit, where it is directly conveyed by a piped system into a downstream stilling pool prior to overland flow. The piped system is generally designed to convey up to the 0.2 EY event, and larger events are conveyed within the road reserve. To ensure that the detention area does not remain permanently inundated and create a maintenance problem for the shire, a small trickle pipe was installed just below the surface level of the detention area to slowly release any excess water back into the piped system connected to the second side entry pit.

The detention areas and road reserve areas have been covered with mulch (see Figure 8) produced from the vegetation cleared from the site. The mulch enhances capture of surface runoff and improves the ability of vegetation to re-establish on the verges. Mulching has also substantially reduced the initial impact of the cleared road verges on the site by protecting these areas from erosion and improving the aesthetics of the development area.

The detention areas were designed to be disconnected so that flow concentration and potential erosion by overland flow in the verges did not occur. Road pavement levels were designed so that excavation during construction was not required in areas that may influence the subsoil flow of water. The roads were designed and constructed to sit above the natural surface levels wherever possible.

Flood management

Flows in excess of the 0.2 EY event are conveyed along the kerbline of the road surface to the downstream end of each subcatchment. An extended detention basin is located at the downstream end of the eastern catchment (Figure 9) and a flow dissipater has been built at the end of the south-western catchment (Figure 10). The flow dissipation structure captures end-of-system flows, reduces their velocity and then discharges via sheet overland flow over a low weir into the existing adjacent wetland vegetation. The existing vegetation at the outlet further reduces flow velocities, prevents erosion and treats stormwater. This system has worked very well, with no scour or silt transfer observed at the outlet. There is no direct discharge to the foreshore reserve or wetland areas from any of the elements of the stormwater system.

The flow dissipation structure is an effective means of controlling end-of-system flows, requiring only a small land area and minimising any adverse impacts.

The detention basins were designed with a maximum 1 in 6 bank slope and a low profile to minimise the visual impact on the surrounds. However, the gentle bank slope and shallowness of the basin profile, which results in requiring a larger surface area, had to be balanced with the need to minimise clearing of vegetation.

Maintaining subsurface hydrology

Using the knowledge gathered from the previous stages of development, it was necessary to ensure that service trenches did not act as de-facto subsoil drains that would lower or alter the subsurface flow patterns.

Figure 9. Extended detention basin construction in Stage 2. (Photograph: TME 2003.)

Figure 10. Flow dissipation structure at outlet of subcatchment. (Photograph: TME 2003.)

Where the services were laid in permeable sandy soils they will have little impact on the subsurface flows, but where they were laid in low permeability or clayey soils then some modification to the flow patterns may occur. In these instances, impermeable clay plugs were installed in the service trenches to prevent longitudinal flow along the trenches. The spacing and location of these plugs were set depending on the soil conditions encountered in each trench, and were assessed as the services were being laid onsite. The clay plugs were required to extend to the depth of the low permeability material that was removed and replaced with sand backfill for services installation.

Maintenance

Ongoing maintenance is essential for most BMPs to continue to manage the quantity of flows and to effectively remove contaminants from stormwater. The stormwater management system proposed in this development will be readily maintainable by the shire in conjunction with their routine maintenance practices. This stormwater system will require cleaning out of the soakwells and sweeping of all roads at least once per year. Annual inspection of the extended detention basin, flow dissipation structure, soakwells and detention areas will also be required and any cleaning or repairs be undertaken as necessary.

Results/achievements

When Boronia Ridge was developed in 2002, it was unique in WA for pioneering the integration of an at-source infiltration system with a piped drainage system to convey larger events. The stormwater management system performed well during its first four years of operation. The experience gained in this stage of development extended to the design of the final stages that were due to be constructed in 2007. The success of this system has led to greater acceptance of similar stormwater management systems in subsequent developments by the shire. Contingency measures implemented in the Boronia Ridge Stage 2 development, such as a piped system to accommodate overflow from greater than 0.2 EY events, provided reassurance to the shire regarding their concerns about systems that were fully reliant upon infiltration onsite.

Good vegetation regrowth has been observed and there are no noticeable signs that the hydrology of the area and its associated vegetation have been impacted.

Challenges and opportunities for improvement

Similar projects in the future could be improved by designing a below-ground infiltration device that is cost efficient, but takes up less of the road verge than required for the installation of circular soakwells. Linear infiltration trenches may be more suitable where land space is a constraint.

Overflows onto private property have also been identified as an issue. While this has not caused a major problem or any property damage, residents who are not familiar with WSUD have voiced concerns about water that does not drain away immediately. Temporary pooling of water in some areas following greater than 1 EY events forms part of the designed detention function of the system. The shire has identified that education of potential residents is crucial to the effective management of the stormwater system adopted in the subdivision. Water sensitive building design and lot-scale water management must also be implemented as part of the catchment approach to stormwater management.

Grouted riprap was used around the banks of the dissipation pool. However, grouting does not allow vegetation and in-stream habitats to establish on the banks, and loose rock riprap would be recommended in the future. The design of the system could also have been enhanced by increasing the permeability of the base of the dissipation pool (designed and constructed with lined bases) to allow for revegetation and increased bioretention and treatment of stormwater prior to infiltration or overflow downstream.

Resources

No specific cost comparisons were undertaken at the design stage, but any additional costs of the system were considered minimal compared to a conventional fully piped system. This was because cost savings were achieved through the use of source controls, such as soakwells, resulting in smaller infrastructure requirements at the downstream ends of the subcatchments. The soakwells were the only significant additional cost to the stormwater management system constructed in this development.

Acknowledgments

Information and photos provided by Mr Wayne Edgeloe, former employee of Thompson McRobert Edgeloe Pty Ltd. Information also provided by Mr Graham Lantzke, engineer, former employee of Shire of Manjimup. The shire engineer at the time of the development was Mr Gavin Harris. The developer was Mr Graeme Robertson of RC Developments.

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Lake Goollelal, Joondalup, Swan Coastal Plain

Project description

The City of Joondalup is located in the northern suburbs of Perth. The average rainfall pattern for the closest weather station with suitable data is shown in Figure 1. On the eastern side of the City is the Yellagonga Regional Park, which is 1,400 hectares encompassing the wetlands of Lake Joondalup, Lake Goollelal and Beenyup and Walluburnup swamps. These wetlands are surface expressions of the Gnangara Mound, an important groundwater resource and water supply for the Perth metropolitan area. There are about 20 existing stormwater outfalls that discharge directly into Lake Joondalup and Lake Goollelal, allowing significant amounts of particulate matter and pollutants to enter the wetlands system. These pollutant inputs result in increased nutrient loading, algal blooms, litter and sedimentation problems and increased midge and mosquito populations.

Historically, the lands surrounding Yellagonga Regional Park were used for market gardens and horse agistment, which resulted in significant nutrient enrichment of the catchment. The area is now predominantly used for residential development. Urban development is a significant factor that has caused an increase in peak stormwater flows and runoff volumes, and the deterioration of stormwater quality and environmental amenity. These issues have been ongoing for several years and while the City has adopted a policy for new developments to prevent direct stormwater discharge into wetlands, the existing stormwater systems need to be addressed.

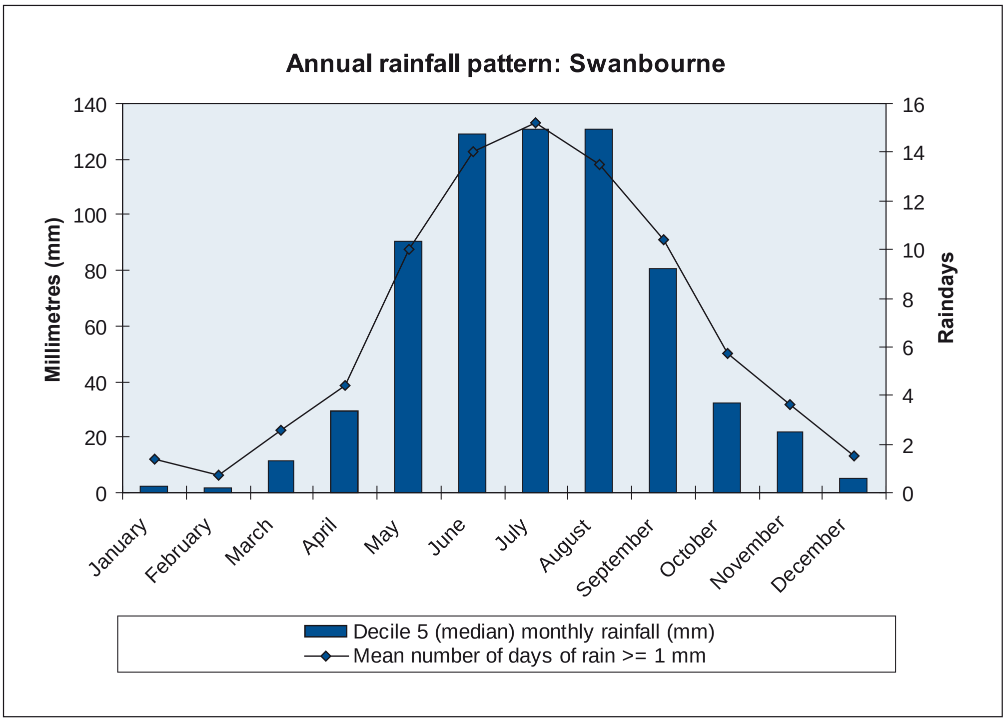

Figure 1. Annual rainfall pattern, Swanbourne. (Source: Bureau of Meteorology 2007.)

Lake Goollelal is part of Bush Forever Site and is a conservation category wetland. The groundwater flow is in a westerly direction and the soil type is highly permeable Spearwood Sands. Lake Goollelal is located within an area with a high risk of ASS disturbance.

A review of the environmental and engineering aspects of the various stormwater outfalls associated with the Yellagonga Regional Park was undertaken in 2003. From these investigations, an overall strategy was prepared with the community and other stakeholders. The strategy proposed retrofitting the various outfalls entering the Regional Park lake system with inline stormwater treatment measures.

Outfall Number 21 is located on the southern extreme of Lake Goollelal, near the arterial road of Hepburn Avenue. The catchment area of Outfall Number 21 is about 38 hectares with a mixture of land uses, including natural bushland, parks and reserves, commercial uses, a petrol station and residential development. Stormwater pollutant characteristics are largely determined by the land uses within the catchment. Due to its catchment characteristics, this outfall was selected to be studied in the development of a strategy to improve the water quality of the Yellagonga Regional Park lake system. A conventional stormwater pipe system, constructed when the subdivision was developed in the 1970s, discharged directly into the lake. The principal source of stormwater discharge and associated pollutants is the impervious surfaces, namely the road reserves and commercial developments, which account for 14% of the catchment area (as of 2007). About 86% of the catchment consists of residential housing and vegetated areas (as of 2007). The highly permeable sands allow the majority of lot stormwater runoff to be infiltrated. Residential roofed and paved areas drain into onsite soakwells.

Approaches implemented

In order to protect the receiving waters of Lake Goollelal, the following structural and non-structural stormwater management measures have been implemented in the catchment area of Outfall Number 21 (Figure 2):

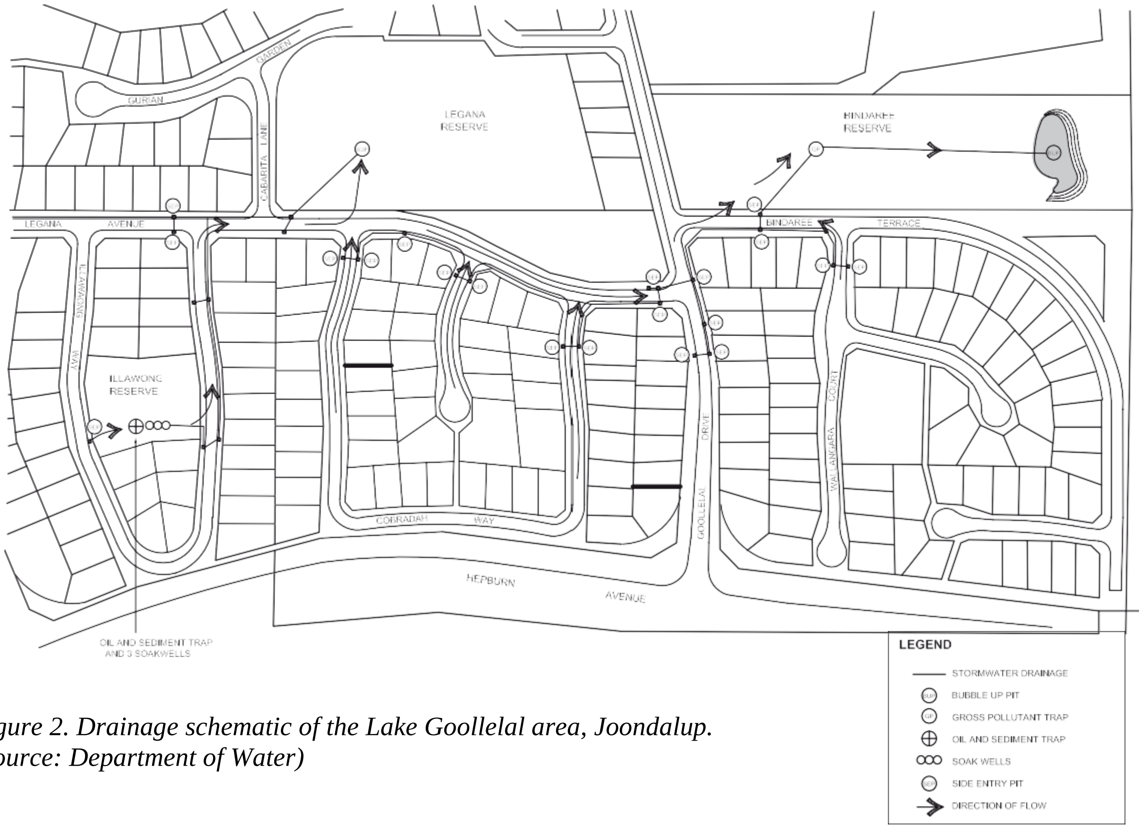

Figure 2. Drainage schematic of the Lake Goollelal area, Joondalup.
(Source: Department of Water)

Structural controls:

The structural treatment measures implemented in this project are:

- GPTs
- soakwells and infiltration areas
- bioretention basins
- disconnection of the stormwater system, i.e. isolation of receiving waters from sections of the catchment.

Non-structural source controls:

Non-structural controls are important to minimise the source of pollutants; however, it can be difficult to enforce these controls or to engage the community in their implementation. For this reason, a combination of structural and non-structural controls is highly recommended to achieve best practice stormwater management. The following three non-structural controls are considered essential to this project:

- community education and awareness for stormwater management (see BMPs 2.3.2, 2.3.3 and 2.3.5 of Chapter 7, and Chapter 8)
- best management practices on construction and land development sites (see BMP 2.1.1 in Chapter 7)
- best management practice municipal operations such as street sweeping, drainage maintenance, maintenance of parks and reserves, graffiti removal and cleaning activities (see BMPs in section 2.2 of Chapter 7).

These measures were considered suitable to reduce potential stormwater contaminants identified in the catchment, including:

- gross pollutants, such as wrappers, cigarette butts, containers and bottles
- hydrocarbon pollutants from the petrol station and road runoff
- nutrients and sediments from residential areas and parks and reserves
- heavy metals in road runoff due to vehicle brake and tyre wear and fuel combustion.

Sediment and litter control

GPTs are primary treatment measures that retain litter and coarse sediments by physically screening stormwater and encouraging sedimentation within a chamber or basin. GPTs are suitable for retrofitting existing piped drainage systems. GPTs are usually most cost effective when placed in the stormwater treatment train at-source to target areas with potential high pollutant runoff. However, in retrofitting situations this may not always be feasible due to site constraints, such as land availability or the layout of the stormwater network.

There were two locations in the catchment for Outfall Number 21 that were identified as being optimal for GPT installation. The first location is within the park at Illawong Way. The upstream catchment from this location is predominantly commercial areas along Moolanda Boulevard and includes a petrol station. A GPT with additional oil storage capacity capable of trapping gross pollutants, such as wrappers, cigarette butts, containers and bottles, as well as hydrocarbons, was selected as these pollutants were likely to be generated from this subcatchment.

Runoff is treated by this GPT before flowing to a series of soakwells (Figure 3). Any overflow continues downstream to an infiltration basin located in the lower catchment (see Figure 2). This GPT reduces the maintenance requirements of other BMPs, as well as forming the first stage in the treatment train process.

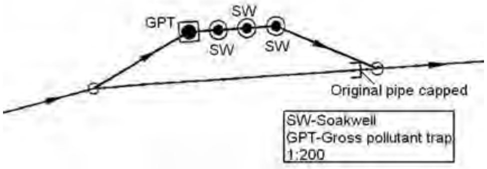

Figure 3. GPT and soakwell configuration at Illawong Way Reserve, Joondalup. (Source: Connell Wagner 2003.)

The second location selected for installation of a GPT is on Bindaree Terrace, where a 900 mm diameter stormwater pipe enters Bindaree Park. A basic GPT was adequate to trap coarse sediments and litter, which were the main potential pollutants being generated from the residential area and parks in this subcatchment. This GPT provides pre-treatment of stormwater prior to flowing into an infiltration/retention basin and ultimately the receiving water body downstream.

Soakwells

Soakwells enable stormwater storage and promote infiltration, hence reducing stormwater runoff volumes and velocities downstream and allowing entrained pollutants to be trapped. Offline soakwells were installed downstream of the GPT within Illawong Way Park. The three soakwells capture and infiltrate low flow events. A high level overflow pipe is connected from the soakwells to the existing stormwater system to provide flood protection for the catchment.

Disconnection

Much of the upper catchment has been ‘disconnected’ from Lake Goollelal. This means that stormwater that would have previously directly flowed untreated into Lake Goollelal is now diverted. Stormwater from the commercial areas receives treatment through the GPTs and soakwells. It then bubbles up into an overland flow channel, which further treats the stormwater and conveys it into a bushland area in the park on Legana Avenue (Figure 2).

Infiltration basins

Locating the GPT upstream of the infiltration basin reduces the impacts of sedimentation and clogging of the basin. Cleaning out trapped sediment and litter from the GPT is easier and cheaper than having to remove these pollutants from the basin. This is a benefit to the asset manager and contributes to the sustainability of the maintenance regime to ensure the stormwater system continues to function effectively.

Dry infiltration basins and swales were considered the most appropriate stormwater treatments for this catchment as they allow entrapment of nutrients within the substrate, prior to recharging the underlying groundwater. The highly permeable Spearwood Sands of the area are also well suited for infiltration.

The infiltration basin is designed to retain all storms up to the 0.2 EY event (Figure 4). If properly maintained, the basin will continue to reduce downstream runoff volumes and velocities and trap pollutants, thereby protecting Lake Goollelal (Figure 5). Bindaree Park basin has a high-flow bypass connection to the original Outfall Number 21 to Lake Goollelal. This protects the basin from erosion and re-suspension of settled materials in large events. However, the piped bypass connection to the lake could be replaced with a vegetated overland flow channel to treat and reduce the velocities of high flows and increase disconnection from the receiving waters.

Figure 4. Infiltration basin in Bindaree Reserve, upstream of Lake Goollelal, Joondalup. (Photograph: Department of Water 2006.)

Figure 5. Bubble-up pit located in Bindaree Reserve, Joondalup, being cleaned using a vacuum truck. (Photograph: Department of Water 2006.)

The shapes of the infiltration basins were gently integrated into the surrounding landform to maintain the conservation value and landscape amenity of the Lake Goollelal area. The basins were built with depths of 1.2 to 1.5 m and gently graded banks between 1:4 to 1:6. The basins were designed with a length to width ratio of 2:1 and a high surface area to volume ratio to maximise infiltration capacity. Landscaping was done in context with the surrounding bushland and parkland area. The basin fringes were planted with vegetation endemic to the area and further revegetation is planned to mimic natural ephemeral wetlands.

In the planning phase, a constructed wetland was not considered the best treatment option for Outfall Number 21 due to the constrained and small catchment size. A treatment train approach using GPTs, swales and infiltration systems could achieve better water quality outcomes than a constructed wetland, while minimising construction and maintenance costs.

Consultation

Consultation with the stakeholder groups that had an interest or were involved in managing the Yellagonga Regional Park was considered essential to the development and successful implementation of the strategy. Once the project had progressed to selecting specific treatments and developing preliminary designs, consultation was undertaken with the residents in the immediate vicinity of the area. The City of Joondalup received positive support from the community for the proposed stormwater treatments.

Results/achievements

The treatment train approach that has been implemented to retrofit the Lake Goollelal catchment has optimised opportunities to manage stormwater within the existing available space and community requirements. The investment in investigating the catchment, strategically planning the project and undertaking community consultation has resulted in cost effective techniques being selected to best manage the pollutants from the contributing catchments.

The implemented stormwater management measures have achieved:

- no direct discharge from Outfall 21 to Lake Goollelal of flows up to the 0.2 EY event without receiving treatment
- a reduction in the peaks and ‘flashiness’ of flows from the urban catchment
- integration of the stormwater management measures in POS.

Maintenance practices are being monitored and evaluated to assess the required maintenance frequency and to fine-tune the ongoing maintenance program. This is important to optimise the allocation of maintenance resources and to reduce the risk of BMP failure due to a lack of maintenance.

The treatment train approach trialled by the City of Joondalup to reduce the impacts of stormwater pollution from Outfall Number 21 on Lake Goollelal will be extended to address other outfalls discharging to this sensitive environment.

Challenges and lessons learnt

At one of the bubble-up pit sites, overland flow has caused erosion and requires repair. Where flows have been directed to areas with bubble-up pits, further 'soft engineering' is required, such as the application of organic matting and revegetation, to stabilise overland flow paths. Loose riprap should be applied around the bubble-up pits to dissipate flow energy and prevent erosion. A defined flow channel that is stabilised with organic matting, revegetation and a series of rock riffles (loose rock check structures) could be built to further reduce flow velocities, increase detention in the channel and prevent erosion. These stabilisation works will reduce maintenance requirements in the long-term.

Alternatives to the existing piped systems, such as leaky side entry pits and infiltration crates, could be explored to increase infiltration and disconnection in the catchment. Overland flow could also be implemented on a larger scale to increase detention, infiltration and treatment of stormwater. Due to the highly permeable sands, there are significant opportunities to extend these approaches within the catchment to further reduce pollutant inputs to Lake Goollelal.

Achieving a balance between the varying views, values and priorities of the community and the best solutions to achieve the water quality objectives of a project can be a challenge. The City of Joondalup incorporated the issues raised through the public consultation process into the stormwater management strategy for Lake Goollelal. For example, the shape and location of the detention basins were changed to address public concern about the potential impacts on local native vegetation. Designs can often be improved to achieve multiple benefits by incorporating other options or trade-offs identified through consultation with stakeholders.

The City of Joondalup has recognised that to implement the changes required to improve stormwater quality and the health of Lake Goollelal, collaboration between state and local government, as well as industry and community groups, is required. The development of an Integrated Catchment Management Plan incorporating stormwater improvement strategies is also an important tool in achieving best practice stormwater management.

Resources

The cost for implementation of the pilot treatment works for Outfall Number 21 was about $100,000 (cost circa 2004 to 2007). This included two GPTs, three soakwells, five bubble-up pits and an infiltration basin.

Acknowledgments

Kym Hockley, Principal, Connell Wagner, and Peter Pikor, Manager Infrastructure Management and Ranger Services (previously), City of Joondalup provided information and endorsed this case study. David Djulbic, Director, Infrastructure and Operations, City of Joondalup, also endorsed this case study. Dave Mather, Coordinator Subdivisions/Civil Projects, City of Joondalup, provided additional information on the project progress.

1.12 Acronyms

AEP	Annual Exceedance Probability
ARI	Average recurrence interval
ARQ	Australian Runoff Quality
ASS	Acid sulfate soils
ASR	Aquifer storage and recovery
BMP	Best management practice
DPLH	Department of Lands, Planning and Heritage
DWMS	Drainage and water management strategy
DWMP	District water management plan
DWMR	District water management report
DWER	Department of Water and Environmental Regulation
EPA	Environmental Protection Authority
EWR	Ecological water requirement
EY	Exceedances Year
GPT	Gross pollutant trap
LSM	Litter and sediment management
LWMR	Local water management report (formerly known as local water management strategy)
MAR	Managed aquifer recharge
POS	Public open space
PRI	Phosphorus retention index
SMP	Stormwater management plan
TDS	Total dissolved solids
TN	Total nitrogen
TP	Total phosphorus
TSS	Total suspended solids

Subdivision and Development WMR	Subdivision and development water management report (formerly known as urban water management plan)
WIN	Water Information Network
WSUD	Water sensitive urban design

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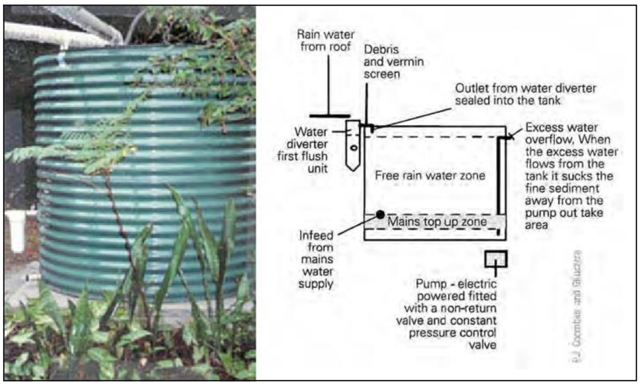
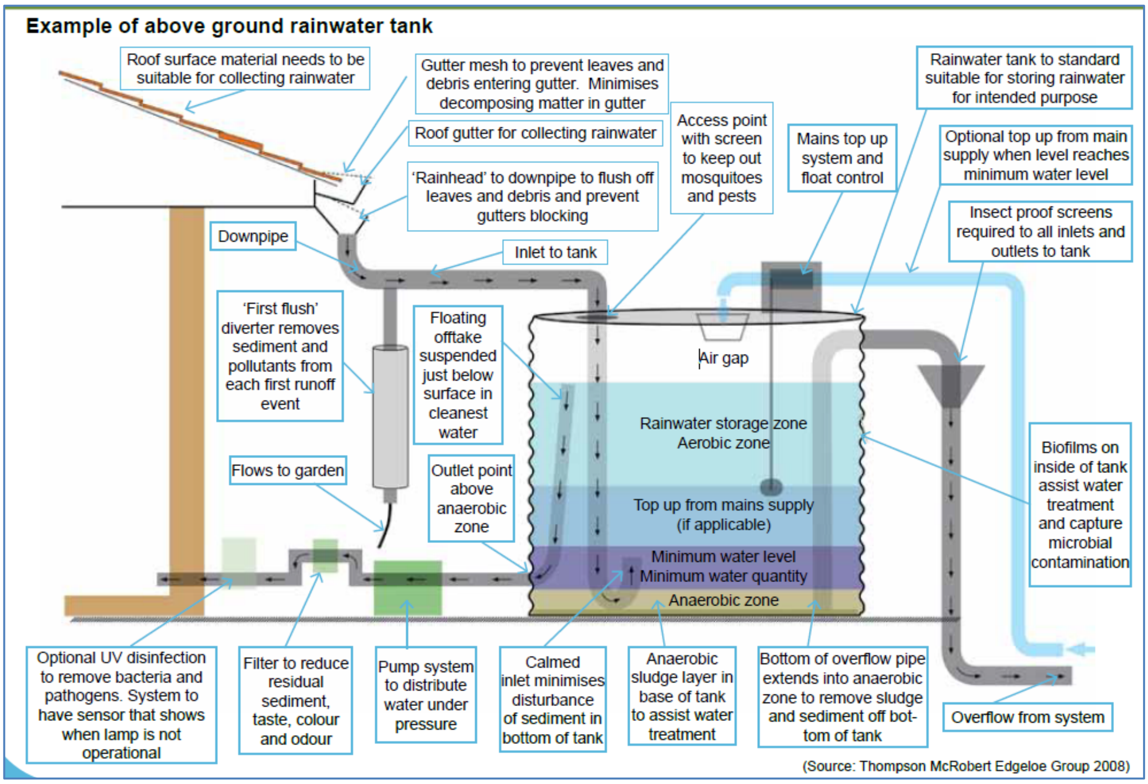
2 Stormwater storage and use

2.1 Rainwater storage systems




Concrete underground tank
Slimline domestic rainwater tank
Poly domestic rainwater tanks



(Source: Thompson McRobert Edgeloe Group 2008)

Figure 1. Rainwater tank components

Background

At the lot scale, rainwater storage tank systems are an effective way to capture stormwater for non-potable use (such as garden watering, toilet flushing, in washing machines and for car washing) and therefore help to conserve scheme drinking water supplies. There are increasingly innovative designs which hold rainwater in a range of forms including bags, fences, walls and roof gutters, with these systems designed for different uses, volumes and aesthetic requirements. Slimline plastic or metal tanks are increasingly popular in urban areas due to the unobtrusive form of the tanks adjacent to or beneath buildings.

Rainwater storage systems can also be applied at a larger scale, for example high volume underground storage systems applied to capture runoff from paved or parking areas, or the roofs of buildings. There are several new below-ground storage propriety devices that are designed for both roof water catchments and impervious surface catchments, such as hardstand areas. The storage systems are modular and their volumes can be anything from 5 kL to 5 ML. These devices have different filtration systems over the inlet for primary treatment and some have a secondary filter, depending on the quality of the runoff and the desired water use. A pump system returns the collected water to provide a non-potable supply for domestic, commercial, industrial and municipal buildings, such as sports centres and community halls. Collected stormwater could be used for irrigation, vehicle washing, art/water features, toilet flushing, cooling water systems and many other applications that currently use high grade potable water. Stormwater used for cooling water systems might have to be treated to a higher standard because nutrients and heavy metals in the water may cause slime formation and microbial growth, while suspended solids could cause blockages and fouling.

In high-density urban areas, there is limited space for outdoor gardens, so roof water and stormwater-fed roof gardens or courtyard gardens are an attractive option for outdoor living space. In WA, these systems are a great opportunity to provide a water sensitive garden in urban environments. Plant species that require irrigation throughout the dry season will not be suitable due to the need for top up water.

Urban space designers can also use harvested stormwater as a feature in the landscape. However, emphasis should be placed on the ephemeral nature of our environment, so these features should be aesthetically pleasing with and without water. Use of groundwater and scheme water to top up water features over summer is not considered water wise or beneficial to the environment.

Rainwater tanks have primarily been used to provide an alternative water supply source and reduce scheme water consumption. However, in areas of limited infiltration (high water table, clay soils) they provide additional benefits of reducing catchment runoff (peak flow and volumes) from smaller rain events, which assists the post-development catchment to replicate its pre-development hydrology.

This BMP has good potential in a retrofitting scenario and can be applied in highly impervious built environments that may preclude the installation of other BMPs.

The quantity of water that can be detained in these systems will depend on the rainfall (amount and annual and seasonal variability). For large-scale harvesting schemes, the ecological water requirements of the catchment need to be considered in determining the volume of water that can be collected.

Unless adequately treated, collected rainwater is not reliably safe to drink due to the possibility of contamination from air-borne chemicals and microorganisms. The Australian Government Department of Health has *Guidance on the use of rainwater tanks* (Department of Health 2011) that consolidates the most recent information and advice on the range of potential hazards and preventive measures to ensure water quality.

There may also be additional state and local government regulations and guidelines for the catchment that will guide the use of roof water and stormwater as a water source.

Performance efficiency

In urban environments in south-western areas of WA, the Mediterranean climate means that a domestic rainwater storage tank is likely to be dry in the summer months when garden watering demands are highest.

Using rainwater as an effective alternative source for garden watering is therefore considered limited.

Substantial reductions in the volume of stormwater runoff and the use of scheme water are achieved when stored rainwater is utilised for indoor use, such as washing machine use and toilet flushing (Figure 2). Use of rainwater for these purposes is supported by the Australian Government Department of Health (2011). By connecting the rainwater storage system to indoor uses, the water is consistently used, freeing up space in the tank to capture more runoff.

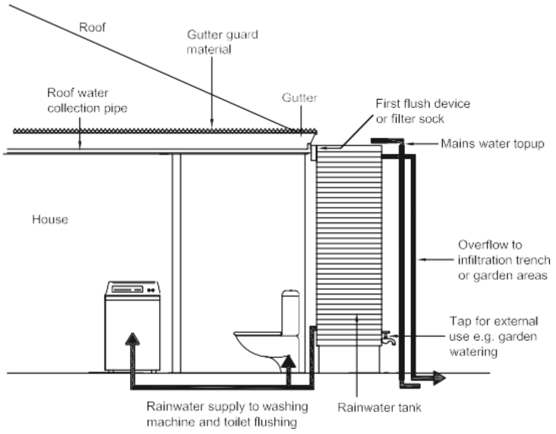


Figure 2. Elements of a domestic rainwater system (the rainwater treatment train).

Research by Fletcher et al. (2006) indicated that modelling stormwater harvesting in Brisbane and Melbourne for three land use scenarios (low, medium and high density) demonstrated that stormwater retention and use onsite could help restore stormwater flows and water quality towards their pre-development level. In a study of eastern states capital cities, Coombes and Kuczera (2003) found that domestic rainwater tanks with capacities of 1 kL to 5 kL provide considerable reductions in scheme water demand and stormwater runoff. The average retention volumes available in rainwater tanks prior to storm events ranged from 0.25 – 0.7 kL for 1 kL tanks and 2.3 – 8.4 kL for 10 kL tanks in a study for Brisbane, Sydney, Melbourne and Adelaide. Areas with lower annual rainfall had the largest retention volume available in the tank due to internal use emptying the tank and less rainfall to refill the tank. The same study (Coombes and Kuczera 2003) found the annual scheme water savings ranged from 18–55 kL/year for a 1 kL rainwater tank, increasing to 25–144 kL/yr for a 10 kL tank. However, these results should be applied with caution to WA conditions due to considerable differences in rainfall seasonality between the different regions of WA and eastern Australia.

Cost^

Costs of rainwater tanks can vary considerably depending on material, design, size and installation requirements. There are various suppliers of rainwater systems all over the state of WA and distances from distribution centres will dictate regional pricing, especially for larger tanks. Local suppliers usually provide free quotes or advertise their prices online.

^The costs quoted in this section are from around 2000 to 2007 and have not been adjusted for inflation or potential cost changes which may have occurred since this chapter was published in 2007. Therefore, it should be considered in that context and users of the manual are encouraged to seek further specific industry advice on the current costs as appropriate.

Design considerations

The design of a rainwater storage system is dependent on the intended uses of the water and the quality required.

There are several factors that contribute to water quality. At the lot, street and precinct scales, the quality of the stormwater will be highly dependent on the type of hardstand area and the use of the surfaces that the runoff flows over. In a lot-scale rainwater supply system, the quality of runoff from the roof depends on roofing materials, the types of materials deposited on the roof and the roof maintenance regime. Storage systems need to consider sediments and organic material as the major contaminants. Physical, chemical and biological processes can improve the quality of the roof water in the storage tank (Coombes and Mitchell 2006). Gutter guards, first flush devices and filter socks can limit the transfer of sediment and debris to rainwater storage systems. Mesh screens on inlets, outlets and overflow devices will exclude animals and mosquitoes and other insects from entering tanks, therefore minimising the risk of harmful microorganisms and disease-carrying mosquitoes entering the tanks. A mosquito-proof mesh should cover the inlet to prevent insects entering the tank. Biofilms and sludge in the systems remove organics, microorganisms and metals from rainwater. Further treatment, such as running the water through a hot water system, also improves water quality.

A rainwater storage tank should be fitted with a first flush device or filter sock to limit the transfer of contaminants into the rainwater tank that may have built up between storm events. If collecting roof water, roof gutters should be installed and well maintained. It is recommended that gutter guards or screen mesh be used on buildings to reduce the amount of debris entering the storage tank and minimise the need for maintenance. Leaf diverters are also an important feature in roof water systems. Inline filters or ultraviolet (UV) disinfection may be used depending on the use of the water. Insect screens on inlets, outlets and overflow pipes and insect proof lids and inspection ports are required to reduce the risk of insect breeding, particularly mosquitoes. Gutters and pipework should be self-draining or fitted with drainage points.

For street- and precinct-scale stormwater storage systems, the maximum amount of water available to be retained and used should be calculated by comparison to the pre-development hydrograph or by appropriate ecological water requirement studies so that the environment continues to receive suitable flows to maintain ecological functions. Further advice should be sought from the Department of Water and Environmental Regulation on any studies to determine ecological water requirements.

Across WA, average rainfall, patterns of rainfall and water usage vary. These factors will impact the optimal storage sizing and performance efficiency of the system.

The harvested rainwater should be used for the purpose identified at the planning stage of BMP selection. Changing the purpose may need a review of the existing storage and treatment systems to ensure that the water quality and quantity available is suitable for the new intended use.

The location of the storage infrastructure will be dependent on aesthetic and space requirements for the chosen device. If the storage system is below-ground, site soil characteristics will need to be considered, in particular if there are salinity or ASS concerns which would affect the integrity of the structure. Underground tanks will need to be maintained to ensure that surface runoff does not enter the tank. Refer to Department of Health *A Compilation of Australian Standards on Water Holding Tanks* for a list of Australian Standards for the Design, Manufacturing, Installation and Inspection of Water Tanks. More information is provided on the Department of Health (DoH) website at www.healthywa.wa.gov.au/Articles/U_Z/Water-tanks-on-your-property. There may also be local government policy requirements on pump noise.

The discharge of overflow water should be via overland flow or to an infiltration system.

Design guidelines

The required capacity of a rainwater storage system will depend on the water use of the premises, as well as the rainfall and roof area. In areas with a scheme water supply available, roof water tanks with capacities of 1–5 kL are generally sufficient for domestic use. Smaller tanks can also provide water conservation benefits.

Maximum collection volume

Maximum volumes of water that can be collected from a roof and annual rainfall are calculated using the formula (enHealth Council 2011):

$$Runoff(litres) = c_{eff} * (R_{ann} - l_i) * A_{roof}$$

Where:

$c_{eff} = the\ efficiency\ of\ collection.$ *Values of 0.8–0.85 have been used (enHealth Council 2011).*

$l_i = \ loss\ associated\ with\ absorption\ and\ wetting\ of\ surfaces\ (mm).$ *A value of 2 mm per month (24 mm per year) has been used (enHealth Council 2011).*

$R_{ann} = Annual\ rainfall\ (mm)$

$A_{roof} = roof\ area\ (m^2)$

The maximum volumes of rainwater that can be collected from various areas of roof at a range of average annual rainfalls are shown in Table 1.

Table 1. Maximum volume of water collected based on roof area and annual rainfall

Annual rainfall (mm)	Maximum volumes of rainwater per year (kL)*						
	Roof area (m^2)						
	100	150	200	250	300	400	500
150	10	15	20	25	30	40	50
200	13	21	27	35	42	53	70
250	18	27	36	45	54	72	90
300	22	33	44	55	66	88	110
400	30	45	60	75	90	120	150
500	38	57	76	95	114	152	191
600	46	69	92	115	138	184	230
800	62	93	124	155	186	248	310
1000	78	117	156	195	234	312	390
1200	94	141	188	235	282	377	470

* These volumes were calculated using a value of 0.8 for A and 24 mm for B (Source: enHealth Council 2011.)

Tank size and security of supply

Where a tank is to represent the sole source of supply, determining maximum volume is only the first step. The next step is to calculate the size of the tank needed to ensure that the volume of water collected and stored in the tank will be sufficient to meet demand throughout the year, including during the drier months, or through periods of low or no rainfall.

There are several mathematical models available for determining the size of tank needed to provide a defined security of supply. The simplest way of checking an estimated tank size required to provide water throughout an average year is to use monthly rainfall data and to assume that at the start of the wetter months the tank is empty. The following water balance formula should be used for each month:

$$V_t = V_{t-1} + (Runoff - Demand)$$

Where:

$t = number\ of\ months$

$V_t = volume\ of\ water\ remaining\ in\ the\ tank\ at\ the\ end\ of\ the\ month$

$V_{t-1} = volume\ of\ water\ left\ in\ the\ tank\ from\ the\ previous\ month$

Runoff should be calculated as discussed above. Starting with the tank empty, then $V_{t-1} = 0$. If, after any month, V_t exceeds the volume of the tank, then water will be lost to overflow. If V_t is a negative figure, then demand exceeds the available water. Providing the calculated annual runoff exceeds the annual water demand, V_t will only be negative if periodic overflows reduce the amount of water collected so that it is less than the demand.

If water demand is to be met throughout the year, the tank should be large enough so that V_t is never negative, so calculations should be repeated using various tank sizes until V_t is ≥ 0 at the end of every month. If this cannot be achieved, then the catchment area connected to the tank may need to be increased or demand reduced. The greater the values of V_t over the whole year, the greater the security of meeting water demand when rainfalls are below average or when dry periods are longer than normal. However, the larger tank size is associated with higher costs.

It may be necessary to have a dual water system to use both roof water and scheme water when the tank level is low due to dry weather or high usage. This ensures a reliable water supply that will still provide significant scheme water savings and stormwater management benefits .

Maintenance

Rainwater storage systems require very little maintenance provided they are correctly installed. Typical maintenance requirements include:

- cleaning of the first flush device every 3–6 months
- removing leaf debris from gutters and roofs every 3–6 months
- checking insect screens and other potential mosquito entry points at the onset of warm weather each year (e.g. spring in the south-west of WA) and whenever routine tank inspection and maintenance is undertaken
- ensuring that water is not pooling beneath overflow outlets or taps whenever routine tank inspection and maintenance is undertaken
- checking sediment levels every two years.

Below-ground stormwater storage devices will require some maintenance of the filters and pumps. Manufacturers' maintenance guidelines should be adhered to, but critically assessed once the system is installed, to ensure that site-specific conditions are taken into consideration in the maintenance regime.

Rain gardens require normal garden maintenance, including maintaining the plants in a healthy condition and checking that the garden is using up all of the rain and stormwater that it receives. Checking on the garden media to ensure that it is not clogged or waterlogged should occur before each wet season.

Stormwater sculptures and water features will require regular maintenance to ensure that storage capacity is at its maximum, pumps are operating and overflow devices are not blocked.

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2.2 Managed aquifer recharge

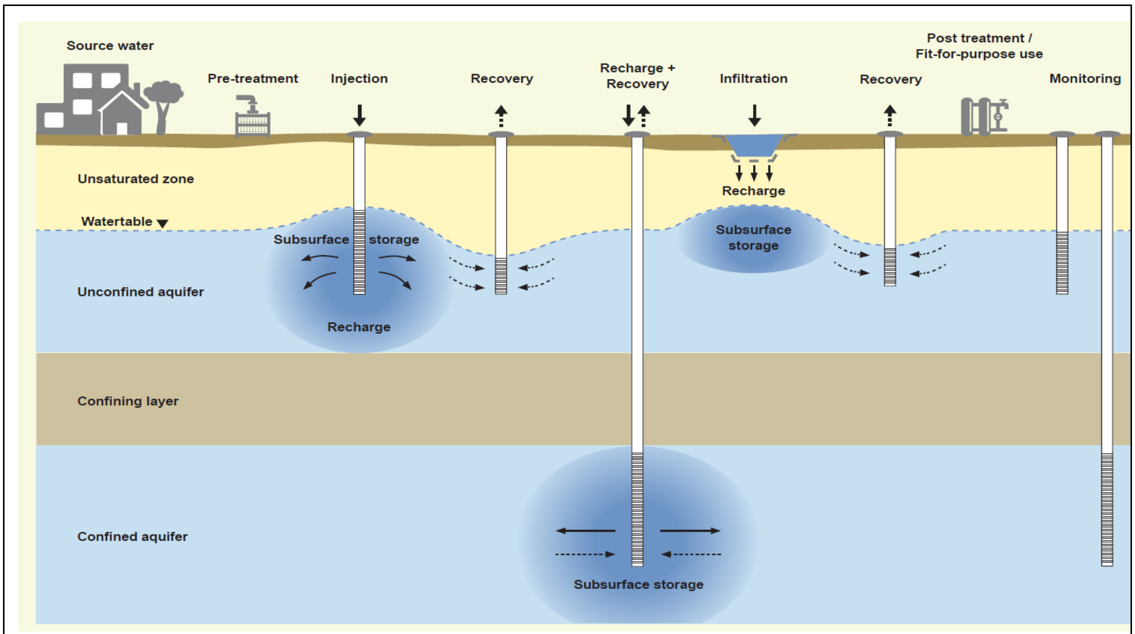

Figure 1. Conceptual Model of Aquifer Storage and Recovery Bore. (Source:DWER 2020.)

Background

Managed aquifer recharge (MAR), also known as artificial recharge, is the infiltration or injection of water into an aquifer. The water can be withdrawn at a later date, left in the aquifer for environmental benefits, such as maintaining water levels in wetlands, or used as a barrier to prevent saltwater or other contaminants from entering the aquifer. As the water infiltrates or is injected into the soil, natural biological, chemical and physical processes may assist in removing pathogens, chemicals and nutrients from the water, and thus improve water quality .

MAR may be used as a means of managing water from a number of sources, including stormwater and wastewater. A number of pilot studies of MAR schemes using treated wastewater have been conducted. For example, a Water Corporation MAR scheme in Halls Head, Mandurah, has demonstrated significant improvements in secondary treated wastewater following MAR (Toze et al. 2004).

The Department of Water and Environmental Regulation has updated its policy on MAR. The new policy – Managed aquifer recharge in Western Australia (2021) replaces Operational policy 1.01 – Managed aquifer recharge in Western Australia (Department of Water, 2011) and earlier practices for MAR operations adopted by the department. It should be used in conjunction with the department's new guideline – Water and environmental considerations in MAR operations in Western Australia (2021). The MAR policy and guideline provide a management framework for MAR operations under the current water and environmental legislation in WA. They include useful information to assist with establishing a MAR project, while ensuring the environment, water users and public health are protected.

This chapter considers only MAR using stormwater. The lot-scale infiltration of runoff via soakwells is not considered to be a form of MAR in this document. Soakwells are addressed in BMP 3.2 Soakwells.

Scope and viability of MAR

MAR schemes can range in complexity and scale from the precinct scale, through local authority infiltration systems for road runoff and POS irrigation bores, through to the regional scale, which involves infiltration or well injection of stormwater and provision of third pipe non-potable water supply for domestic use.

Formal MAR schemes at a regional scale are relatively new in WA. Examples of MAR at the precinct scale include stormwater infiltration and irrigation systems adopted by the City of Greater Geraldton and Town of Mosman Park. The Town of Cottesloe, supported by the Water Smart Australia program, is implementing MAR using stormwater to replenish the Cottesloe groundwater aquifer. A number of local governments are also currently investigating MAR using stormwater for the irrigation of POS.

A MAR scheme can be designed to incorporate BMPs such as vegetated swales, bioretention systems and constructed wetlands for pre-treatment purposes. At the regional scale, MAR can assist a post-development catchment to replicate its pre-development hydrology through reducing runoff to the receiving environment and by reducing the importation of scheme water. MAR can also contribute to reducing the size and hence capital cost of stormwater infrastructure. This is particularly the case where stormwater is infiltrated at-source, resulting in reduced design runoff rates.

The viability of a MAR scheme is firstly dependent upon the quality of water available to be used, or level of treatment required to achieve the necessary water quality. Stormwater can contain contaminants such as oil, grease, metals and pesticides, which build up on surfaces in urban areas. These come from sources such as pavement deterioration, tyre and brake-pad wear, vehicle emissions and spills. MAR may improve water quality for a number of contaminants as a result of filtration in the aquifer, and through biochemical processes in the soil or aquifer. It is however noted that there are a number of contaminants that may not be removed by MAR, and that there exists the potential for MAR to cause contamination of the aquifer if improperly designed or managed. The potential for contamination of the soil or aquifer through which the water moves also requires consideration.

The aquifer characteristics must also be well understood and mapped before implementation of a MAR scheme. Knowledge of the aquifer characteristics is required to predict the flow and fate of injected water. Understanding and monitoring of the aquifer and injected water is required so that recovery bores can be located to ensure that sensitive receptors, such as bores, wetlands and ASS, are not affected.

The quantity of water available for abstraction following MAR will depend upon a number of factors, including the potential for impacts to the regional groundwater system. At times, either due to recovery efficiencies or due to environmental water allocations, the volume of water available to be recovered will be less than the volume of water which has been recharged to the aquifer in the scheme.

Regulatory requirements

MAR systems may require approvals from a number of government agencies, including the Department of Water and Environmental Regulation, Department of Biodiversity, Conservation, and Attractions, Environmental Protection Authority, Department of Health and local government. In the case of large schemes or those with the potential for significant impacts, all relevant agencies must be consulted prior to proceeding with detailed design. Any MAR proposal that is likely, if implemented, to have a significant effect on the environment must also be referred to the Environmental Protection Authority under section 38 of the Environmental Protection Act 1986. Currently, MAR in public drinking water source areas requires consultation and may not be supported.

MAR proposals should be assessed using a risk management framework, as set out in the Australian Guidelines for Water Recycling – Managing Health and Environmental Risks (Phase 2) (NRMMC AND EPHC 2009).

A consultation and communication program should run in parallel with development of any MAR proposal. This is discussed in the Australian Guidelines for Water Recycling – Managing Health and Environmental Risks (NRMMC AND EPHC 2009).

Design considerations

At the MAR planning stage, it is necessary to compile an inventory of existing environmental values attributed to the groundwater system, such as drinking water, aquatic ecosystem values and primary industries. This inventory may be included within a subdivision or development water management report or a stormwater management plan (SMP). This should provide design objectives for planning the MAR system and identify the location of existing bores, their intended uses (e.g. monitoring, POS irrigation) and groundwater dependent ecosystems (phreatophytic vegetation, caves, wetlands and waterways). As the aquifer may already be providing beneficial uses to others, quality, quantity and flow requirements of these users need to be considered in the aquifer selection.

Stormwater quality

Quality of the stormwater is a primary design consideration. Water quality treatment may be required prior to infiltration or injection into groundwater. MAR that uses infiltration as the recharge method may need little or no pre-treatment prior to recharge. The level of treatment depends on factors including the quality of the water used for the recharge, the local groundwater conditions, the intended use of the recovered water and local regulation. One of the key issues is the variability of stormwater, through factors such as the timing between rainfall events, rainfall intensity and distribution, and variability in catchment land uses. Treatment also has the added benefit of removing sediment and reducing the risk of ‘clogging’ the infiltration or injection system.

Each MAR proposal must identify potential pollution sources within the catchment and plan risk management strategies, including pollution contingency plans. An evaluation of the pollutants that may be present within the injected water needs to be carried out on a catchment basis as pollutants vary with land use. The concentrations of pollutants typically have seasonal or within-event patterns, and heavy pollutant loadings can be avoided by being selective in the timing of diversions. Comparisons with the aquifer water quality and environmental values will indicate the requirements for treatment of water detained for injection. Knowledge of the potential pollutant profile helps to define water quality sampling and analysis costs when determining the viability of the MAR project.

The Beach Health Program 2004-06 (Department of Water 2007) conducted a baseline study of the types and concentrations of contaminants in and around 65 traditional coastal stormwater drains in the Swan Region. The study found that stormwater at Perth’s marine beaches is contaminated predominantly with microbes and heavy metals. Nutrients, petroleum hydrocarbons, organic chemical compounds and suspended solids are also present in stormwater but to a lesser extent. Proposals must evaluate the need for pre-treatment of stormwater prior to MAR to address these potential contaminants.

Many structural BMPs are suitable as pre-treatments for MAR schemes. In general, methods that have long detention times are advantageous to reduce pathogenic microorganisms in addition to other pollutants. An advantage of using treatment with large storages (e.g. constructed wetlands) is the dilution effect if an isolated pollution event occurs, thus reducing the risk of aquifer contamination. See the BMPs in Chapter 7 for non-structural controls to reduce pollution and treat stormwater quality.

Aquifer characterisation

The in-situ water quality of the aquifer also requires consideration. Groundwater salinity, acidity, total dissolved solids and hydrogen sulphide levels may limit the potential for MAR; conversely MAR may dilute problematic local groundwater qualities. Infiltration or injection of stormwater may not be suitable in areas with high groundwater levels. ASS should be investigated, as these may decrease the quality of recovered water. There is the potential for MAR to increase the concentration of some contaminants by leaching these from the aquifer; it is therefore crucial that both the stormwater and aquifer are fully characterised, physically, chemically and biologically, prior to approval or implementation of a MAR scheme.

Water quantity issues include the recoverable volume and the impact on the surrounding environment. Under pre-development conditions, groundwater entering or recharging an aquifer system is in equilibrium with the groundwater discharge from the system. Groundwater flows are generally discharged into waterways, wetlands, oceans or deeper aquifers. When groundwater withdrawal takes place, a hydraulic gradient due to pumping changes the base flow regime. Detailed hydrologic investigations must be carried out as part of the MAR design process, including identification of ecological water requirements (EWRs) sufficient to maintain and protect groundwater dependent ecosystems under drying climatic conditions.

Factors to consider in evaluating the suitability of an aquifer include:

- environmental values of the aquifer including ecosystem maintenance of caves, wetlands, phyreatophytic vegetation, surface water systems and human uses (irrigation, drinking water supply)
- adverse impacts on the environment and other aquifer users (e.g. reduced pumping pressure for nearby irrigators)
- an existing and/or future drinking water source area
- sufficient permeability and storage within the receiving aquifer
- depth of abstraction from the aquifer
- existing allocation of the aquifer and groundwater resource
- existing ambient groundwater quality and contaminant concentrations
- loss of aquifer permeability and/or infiltration due to precipitation of minerals or clogging
- possible damage to confining layers due to pressure increases
- higher recovery efficiencies of porous media aquifers
- aquifer mineral dissolution, if any
- potential for local aquitard collapse or distortion.

System controls and monitoring

Controls should be incorporated to shut down an injection pump or valve if any of the parameters determined for the project exceed the criteria for the environmental values of the aquifer. Examples of parameters to be measured include:

- 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 injected water that can be monitored in real time.

- Other ongoing monitoring should include monitoring water levels in valuable groundwater dependent ecosystems.

Protection of the treatment and detention system from contamination is a necessary part of the MAR system design. 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, minimising planting of deciduous vegetation, and preventing mosquitoes and other pests breeding in the storage pond. Contingency plans should be developed to cater for the possibility of contaminated water being inadvertently recharged into the aquifer. These include how to determine the duration of recovery pumping (to extract contaminated water), what sampling intervals are needed and how to manage recovered water.

A monitoring system should be designed to ensure that any treatment system is performing as expected, and that MAR is not causing any adverse impacts to the receiving aquifer. The scope and complexity of the required monitoring system will be dependent on the potential impacts of the proposal.

Components of a MAR system

The following material has been reproduced from WSUD Engineering Procedures – Stormwater (Melbourne Water 2005) with the permission of the author, to provide an overview of the main components of an MAR system.

As an example, a MAR scheme for infiltration of treated stormwater into a shallow aquifer contains the following structural elements (Figure 2):

- soakwells, swales or infiltration basins used to detain runoff and preferentially recharge the superficial aquifer with harvested stormwater
- an abstraction bore to recover water from the superficial aquifer for reuse
- a reticulation system (in the case of irrigation reuse) (will require physical separation from potable water supply)
- a water quality treatment system for recovered water depending on its intended use (e.g. removal of iron staining minerals)
- systems to monitor groundwater levels and abstraction volumes
- systems to monitor the quality of groundwater and recovered water.

- In addition to the above elements, an MAR system may also incorporate the following (Figure 2):

- a diversion structure from a 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 spill or overflow structure incorporated in the constructed wetland or detention storage
- well(s) into which the water is injected (may require extraction equipment for periodic purging)
- an equipped well to recover water from the aquifer (injection and recovery may occur in the same well)
- a treatment system for recovered water (depending on its intended use)
- sampling ports on injection and recovery lines
- a control system to shut down recharge in the event of unfavourable conditions.

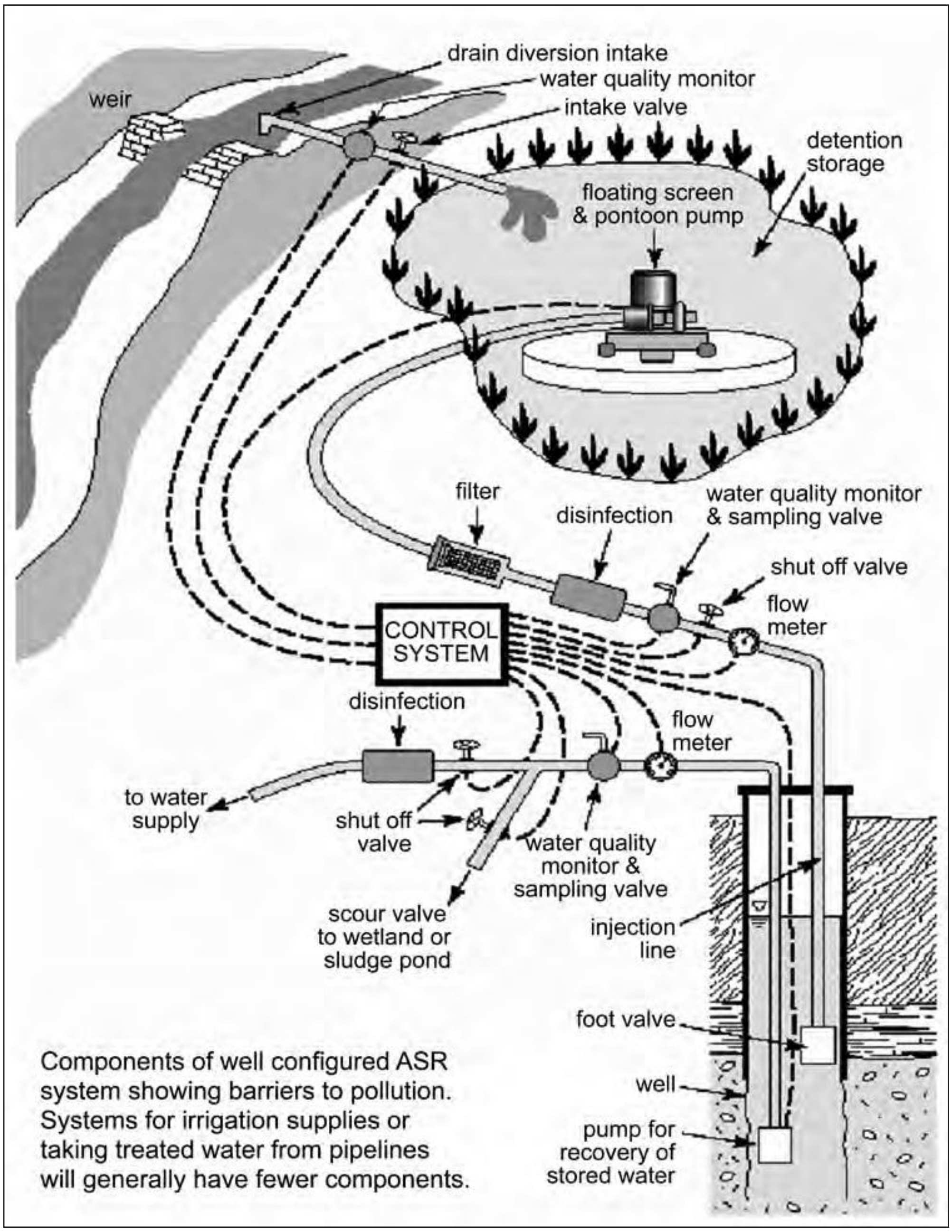

Figure 2. Components of a well-configured MAR system (DWER, 2020)

Refer to the Infiltration Systems, Conveyance Systems and Detention Systems BMPs for design guidelines for soakwells, swales, infiltration basins, dry/ephemeral detention areas and constructed wetlands. As this manual does not provide guidelines for ponds or constructed lakes, refer to the *Chironomid Midge and Mosquito Risk* Assessment *Guide for Constructed Water Bodies* (Midge Research Group of Western Australia 2007), *Mosquito Management Manual* (Department of Health 2019) and Mosquito management website ww2.health.wa.gov.au/Articles/J_M/Mosquito-management for pond design parameters to minimise mosquito breeding risk.

Cost

The cost of implementing MAR systems varies significantly, depending on the level of pre- and post- MAR treatment required, peak demand on the system (and therefore the capital infrastructure costs), size of the area to be serviced, and extent of recharge and recovery infrastructure requirements. For example, injection wells tend to be much more expensive to establish and maintain than infiltration basins.

An analysis of a 400-lot MAR system for a residential area at Forrestdale detailed operating cost estimates as shown in Table 1. The operating unit cost of the MAR system (for garden watering only) is comparable to the current price of scheme water. It should be noted, however, that this MAR cost estimate does not include any capital infrastructure costs.

Table 1. Operating unit cost of water from Forrestdale MAR system (400 lots)

Operations and Maintenance Items (Irrigation Use Only)	**Annual Cost ($)**
Energy cost – bores and transfer pumps	5,200
Operations and maintenance	50,000
Administration costs (50%)	27,600
TOTAL	82,800
Operating Unit Cost of Non-Potable Groundwater Supply	**$0.67/kL**

(Source: Parsons Brinckerhoff 2005 – cost circa 2005.)

GHD (2005) provide feasibility level cost estimates (cost circa 2005) for a 2.3 GL/yr MAR scheme injecting and recovering stormwater from the Leederville Aquifer for the Wungong Urban Water Project at Brookdale. Total unit costs are detailed as ranging between $0.94/kL to $1.41/kL inclusive of capital, energy, maintenance and administration costs (excludes distribution costs). Capital costs for the injection scheme were estimated as ranging between $1 – $1.4 million, with annual operating costs between $0.36–$0.6 million/yr. Recovery costs were estimated based on a separate series of bores distributed throughout the development as ranging between $0.75– $1.10/kL. However, as this proposal involves injecting water into a confined aquifer against a positive head, it would be more expensive than a scheme involving gravity feed and a smaller head (Toze, S. 2007, pers. comm.[1]).

Maintenance

The developer, local authority and service provider (typically Water Corporation) are three key stakeholders in the ownership and management of the MAR systems at precinct and regional scales.

In a conventional urban subdivision, the developer enters into an agreement with the service provider on fulfilling WA Planning Commission conditions for a designated area of subdivision. The developer provides water supply, sewerage and drainage infrastructure for the subdivision. The service provider

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[1] Personal communication with Simon Toze, Principal Research Scientist, CSIRO Land and Water, 2007.

assumes ownership of the assets upon completion of the works and incorporates them into the service provider's schemes. The service provider then operates and maintains these assets in line with their operating licence conditions.

Opportunities exist for local governments (or alternative water service providers) to undertake the management of non-potable MAR schemes as they usually manage the operation and maintenance of the POS within shire boundaries. There are numerous examples of successful management of reuse schemes by local shires throughout regional Australia.

Monitoring equipment should be recalibrated at manufacturer's specified intervals. Pumps and pre-treatment equipment need to be maintained (e.g. by replacing filter media at manufacturer specified intervals or volumes). Keeping maintenance records is a component of good management practice.

References and further reading

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3 Infiltration systems

3.1 Infiltration basins and trenches

Figure 1. Landscaped POS Infiltration Basin, Quandong Park, City of Mandurah. (Photograph: Department of Water 2007.)

Figure 2. Crate cell infiltration basin system below POS, City of Mandurah. (Photograph: Grahame Heal, City of Mandurah 2004.)

Background

Two primary infiltration systems used at larger scales are infiltration trenches and infiltration basins.

Infiltration basins are typically used in applications such as POS parklands (see Figure 1). They consist of a natural or constructed depression designed to capture and store the stormwater runoff on the surface prior to infiltrating into the soils. Basins are best suited to sandy soils and can be planted out with a range of vegetation to blend into the local landscape. The vegetation provides some water quality treatment and the root network assists in preventing the basin floor from clogging. Pre-treatment of inflows may be required in catchments with high sediment flows.

An infiltration trench is a trench filled with gravel or other aggregate (e.g. blue metal), lined with geotextile and covered with topsoil. Often a perforated pipe runs across the media to ensure effective distribution of the stormwater along the system. Crate systems are modular plastic open crates or cells which can be laid out in a trench or rectangular basin, typically around 0.5 to 1.5 m deep, surrounded by geotextile and covered with topsoil (see Figure 2). Piped stormwater enters the system, often via a pre-treatment system, depending on the catchment characteristics, and flows into the trench or crates where the water seeps into the surrounding soil. Systems usually have an overflow pipe for larger storm events. There are a range of products which have various weight-bearing capacities so that the surface of the system can be used for parkland or vehicle parking areas. These systems can be combined to treat a large area.

Performance efficiency

Data on the performance efficiency of individual types of infiltration systems is limited, particularly in WA.

Fletcher et al. (2004) reports pollutant removal efficiencies for infiltration systems, as reproduced in Table 1. It should be noted that the expected removal shown in Table 1 refers to changes as a result of in-situ pollutant reduction, and hence does not consider flow loss due to the proportion of mean annual flow that is infiltrated. Removal efficiencies viewed in the context of the receiving surface water bodies would therefore be greater than the estimates shown in Table 1, particularly for sandy soils with high infiltration capacity. The decrease in surface flow results in a decrease in potential pollutant transport to the receiving environment.

The effectiveness of infiltration systems for nutrient removal is dependent upon the vegetation used in landscaping the system and the phosphorus retention index (PRI) of the soil or infiltration medium. Soil amendment may be necessary to achieve a high rate of phosphorus removal, due to the low PRI of most naturally occurring sands in WA.

Table 1. Typical annual pollutant load removal efficiencies for infiltration systems

Pollutant	**Expected Removal** (mean, range) (%)	**Comments**
Litter & organic matter	100%	Expected to trap all gross pollutants, except during high-flow bypass.
Total suspended solids	85 (65 – 99)%	Pre-treatment required to reduce clogging risk.
Total nitrogen	64 (50 – 70)%^	Dependent on nitrogen speciation and state (soluble or particulate).
Total phosphorus	70 (40 – 80)%	Dependent on phosphorus speciation and state (soluble or particulate).
Coarse sediment	95 – 100%	May pose a clogging risk. These systems should have pre-treatment to remove coarse sediment prior to entry to the filter media.
Oil and grease	n/a	Inadequate data to provide reliable estimate, but expected to be >75%.
Faecal coliforms	n/a	Inadequate data
Heavy metals	85 (50 – 95)%	Dependent on state (soluble or particulate).

^ Occasional instances of 'negative removal' have been reported in the literature, but are not expected to represent typical performance. (Source: Fletcher et al. 2004.)

Cost^

Construction costs associated with these facilities can vary considerably. Cost variability factors include topography, whether installed as part of new construction or implemented as a retrofit, varying subsurface conditions, and the degree and extent of landscaping.

Local cost data for infiltration basins is limited. An alternative method of costing these systems is to examine the costs of similar systems, such as ponds and swales. Taylor (2005) reported costs for ponds (sourced from limited data in Australia) ranging from \$2,000/ha of catchment to \$30,000/ML of pond volume, and \$60,000/ha of pond area. Taylor (2005) also reported costs for vegetated swales of approximately \$4.50/m^2, which included earthworks, labour and hydro-mulching. For swales with rolled turf the cost was approximately \$9.50/m^2 and for a vegetated swale with indigenous species the cost was approximately \$15–20/m^2.

It would be expected that the above costs for both these systems would be comparable to the components of a landscaped infiltration basin.

With respect to infiltration trenches, cost estimates based on eastern states examples provide a construction cost range of \$46 – \$138 per linear metre (based on a 1 m wide, 1 m deep trench) (Taylor 2005).

It is important to consider the longevity of the infiltration system and budget for maintenance costs. Calculation of the 'lifespan' and the effect of sediment accumulation on permeability should be done at the design phase to help estimate these costs. As reported by the Center for Watershed Protection (1998) cited in Taylor (2005), annual maintenance costs would be expected to typically be in the range of ~5 – 20% of the construction cost.

^The costs quoted in this section are from around 2000 to 2007 and have not been adjusted for inflation or potential cost changes due to technological advances which may have occurred since this chapter was published in 2007. Therefore, it should be considered as indicative only and users of the manual are encouraged to seek further specific industry advice on the current costs as appropriate.

Design considerations

Soil types, surface geological conditions and groundwater levels determine the suitability of infiltration systems.

These devices should not be placed in loose Aeolian wind-blown sands. However, well-compacted sands are suitable. At the other extreme, infiltration devices should not be sited in rock or shale, although site-specific permeability should be investigated as some limestone and sandstone permeability can be comparable to medium clays. Care should also be taken at sites with shallow soil overlying impervious bedrock, as the water stored on the bedrock will provide a stream of flow along the soil/rock interface.

Soils must be sufficiently permeable to ensure that collected runoff can infiltrate quickly enough to reduce the potential for flooding and mosquito breeding (i.e. water ponding for no more than four days). See section 1.7.7 'Public health and safety' of the introduction section of this chapter for more information on mosquito management. Infiltration techniques can be implemented in a range of soil types, and are typically used in soils ranging from sands to clayey sands. Soils with lower hydraulic conductivities do not necessarily preclude the use of infiltration systems, but the size of the required system may typically become prohibitively large, or a more complex design approach may be required, such as including a slow drainage outlet system.

The presence of a high groundwater table limits the potential use of infiltration systems in some areas, but does not preclude them. There are many instances of the successful application of infiltration basins on the Swan Coastal Plain where the basin base is located within 0.5 m of the average annual maximum groundwater level. The seasonal nature of local rainfall and variability in groundwater level should also be considered. For example, the groundwater table may only be at its maximum for a short duration, and greater capacity for infiltration may be available throughout most of the year. However, infiltration in areas with rising groundwater tables should be avoided where infiltration may accelerate the development of problems such as waterlogging and rising salinity.

Infiltration basins and trenches typically take up a relatively small percentage (2–3%) of the contributing catchment. Additional space may be required for buffers, landscaping, access paths and fencing. Trenches have the advantage of being able to fit into thin, linear areas, such as road verges and medians. Due to their flexibility in shape, trenches can be located in a relatively unusable portion of the site. However, design will need to consider clearance distances from adjacent building footings or boundaries to protect against cracking of walls and footings.

Root barriers may need to be installed around sections of infiltration systems that incorporate perforated/slotted pipes or crate units where trees will be planted to prevent roots growing into the system and causing blockages.

Generally, infiltration is not recommended for stormwater collected at industrial and commercial sites that have the potential to be contaminated. Where infiltration BMPs are adopted in industrial sites, pre-treatment may be required. Stormwater collected at industrial and commercial sites that do not have the potential for contamination (e.g. roof runoff and runoff from staff carparks) can be infiltrated onsite.

Generally, stormwater runoff should not be conveyed directly into an infiltration system, but the requirement for pre-treatment will depend on the catchment. Treatment for the removal of debris and sediment is recommended to prevent clogging. It may also be necessary to achieve a prescribed water quality standard before stormwater can be discharged into groundwater. Pre-treatment measures include the provision of leaf and roof litter guards along roof gutters, vegetated strips or swales, litter and sediment traps, sand filters and bioretention systems. To prevent basins/trenches from being clogged with sediment/litter during road and housing/building construction, temporary bunding or sediment controls need to be installed. See section 2.1.1 'Land development and construction sites' of Chapter 7 for information about site management practices.

Design guidelines

The calculations contained in this section for sizing the storage volumes and determining emptying time are based on Engineers Australia (2006) and Argue (2004) and the assumed simplified hydrograph detailed in Figure 3. The calculations should be applied with caution to the sizing of infiltration systems where shallow groundwater is present. This approach does not consider the impacts of shallow groundwater in its calculation, which may reduce infiltration capacity. Detailed modelling of shallow groundwater situations is recommended. Designers should take into account the maximum groundwater level, and hence the minimum infiltration potential, in determining their flood detention design. However, designers should also consider maximum infiltration opportunities to achieve aquifer recharge when the groundwater table is below its maximum level (refer to Design Considerations section of this BMP for further discussion).

Hydrologic effectiveness

The hydrologic effectiveness of an infiltration system defines the proportion of the mean annual runoff volume that infiltrates. Hydrologic effectiveness is used for sizing infiltration systems in the eastern states and this method can to some extent be applied in WA. However, in most instances in WA, infiltration basins are designed for capturing and infiltrating flows up to a particular design event, and the Design Storm Method is used.

Field investigations

Field investigations must be undertaken to determine the soil type; hydraulic conductivity; presence of soil salinity, rock and other geological limitations; slope of the terrain; and groundwater level, depth and quality.

A combination of poor soil conditions (e.g. sodic and dispersive soils), steep terrain and shallow saline groundwater can render the use of infiltration systems inappropriate. Dryland salinity is caused by a combination of factors, including leaching of infiltrated water and salt at 'break-of-slope' terrain. Soil with high sodicity is generally not considered to be suited for infiltration as a means of managing urban stormwater. Sodic soils (soils with a relatively high proportion of exchangeable sodium) cause increased soil dispersion and swelling of clays, which adversely impacts the soil structure and results in reduced infiltration, reduced hydraulic conductivity and the formation of surface crusts.

Infiltration into steep terrain can result in the stormwater re-emerging as spring flow downstream. The likelihood of this occurring is dependent on the soil structure, for example where soils intersect a less permeable layer in the area of re-emergence. This situation does not necessarily preclude stormwater infiltration unless leaching of soil salt is associated with this process. This issue will need to be taken into consideration at the design stage.

Field hydraulic conductivity tests are essential to confirm the assumptions of soil hydraulic conductivity adopted during the concept design stage. Saturated hydraulic conductivities for various soil types based on Engineers Australia (2006) are shown in Table 2.

Table 2. Hydraulic conductivity for various soil types (Engineers Australia 2006)

Soil Type	Saturated Hydraulic Conductivity	
	mm/hr	m/s
Sand	> 180	$> 5 \times 10^{-5}$
Sandy Clay	36 – 180	$1 \times 10^{-5} – 5 \times 10^{-5}$
Medium Clay	3.6 to 36	$1 \times 10^{-6} – 1 \times 10^{-5}$
Heavy Clay	0.036 to 3.6	$1 \times 10^{-8} – 1 \times 10^{-6}$

Soils are inherently heterogeneous and field tests can often misrepresent the areal hydraulic conductivity of a soil. Field tests of point soil hydraulic conductivity often lead to underestimating the areal hydraulic conductivity of clayey soils and overestimating sandy soils. Engineers Australia (2006) recommends that a soil moderation factor be applied to field hydraulic conductivity values (Table 3).

Table 3. Soil moderation factors (Engineers Australia 2006)

Soil Type	**Soil Moderation Factor (U)** (to convert point k_h to areal k_h)
Sand	0.5
Sandy Clay	1.0
Medium and Heavy Clay	2.0

Estimating design flows and hydrographs

Infiltration systems can be subject to a range of performance criteria, including that of peak discharge attenuation and volumetric runoff reduction.

The *Decision process for stormwater management in Western Australia* (Department of Water and Environmental Regulation 2017) requires managing up to the small rainfall events (e.g. runoff generated by the first 15 mm of rainfall) at-source as much as practical. One of the main methods by which this can be achieved is through onsite infiltration (where site condition permits). Infiltration systems (such as retention basins) could be designed to accommodate larger events, depending on the site-specific conditions and catchment management objectives.

Two flows need to be considered in the design of infiltration systems:

the peak inflow rate to the infiltration system for design of the inlet structure; and

major flow rates for design of a submergence, conveyance or bypass system.

Design flows and hydrographs for particular storm events can be estimated using a range of hydrologic methods and models with varying complexity. The Rational Method is only suitable for simplistic small catchments or lot-scale catchments where flood routing is not critical (ARR 2019 Book 9). This method, with ‘limited’ runoff generation and surface routing capabilities, and is not likely to be suitable for a ‘precinct’ scale estimate of peak flow as it cannot adequately simulate the array of flood processes that are encountered, even in the simplest of catchments. For further information on the limitation of the Rational Method, please refer to Book 9 ‘Runoff in Urban Areas’ of Australian Rainfall and Runoff 2019.

Engineers Australia (2006) details a simplified alternative to hydrologic modelling to determine an inflow hydrograph that will provide a satisfactory design solution. It is based on assuming a simplified shape of the inflow hydrograph that can be used to estimate the temporary storage volume for an infiltration system, as shown in Figure 3, where:

$i = average\ rainfall\ intensity\ (mm/hr)$

$t = critical\ (design)\ storm\ duration\ (hr)$

$t_c = site\ time\ of\ concentration\ (hr)$

$\tau = time\ base\ of\ the\ design\ storm\ hydrograph\ (hr)$

$Q_{peak} = maximum\ flow\ rate\ in\ response\ to\ the\ rainfall\ event\ (m^3/s)$

$\forall = volume\ of\ stormwater\ runoff\ that\ enters\ the\ device\ (m^3)$

Engineers Australia (2006) indicates use of this simplified approach is likely to result in a conservative estimate of infiltration storage volume requirements in comparison to detailed mathematical modelling.

Determination of an appropriate t (critical design storm duration) is essential in this calculation. Engineers Australia (2006) defines a range of potential interpretations/definitions of this parameter, which may be used as a basis for design.

For further details regarding the implementation of this approach, the user is referred to Engineers Australia (2006).

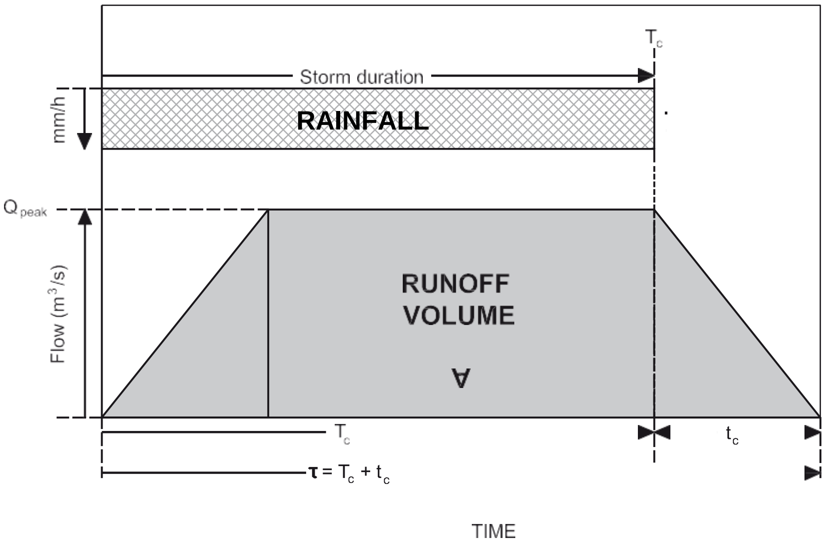

Figure 3. Simplified inflow hydrograph (for use in design without hydrologic modelling).

Siting of infiltration systems

Infiltration systems should not be placed near building footings as continually wet subsurface conditions or greatly varying soil moisture contents can impact on the structural integrity of these structures.

Engineers Australia (2006) recommends minimum distances from structures (and property boundaries to protect possible future buildings in neighbouring properties) as shown in Table 4 for various soil types.

Identification of suitable sites for infiltration systems should also include avoidance of steep terrain and areas of shallow soils overlying largely impervious rock (non-sedimentary rock and some sedimentary rock such as shale).

An understanding of the seasonal and inter-annual variation of the groundwater table is also an essential element in the design of these systems.

Table 4. Minimum setback distances (Engineers Australia 2006)

Soil type	Minimum distance from building footings for infiltration system
Sand	1.0 m
Sandy Clay	2.0 m
Weathered or Fractured Rock e.g. sandstone	2.0 m
Medium Clay	4.0 m
Heavy Clay	5.0 m

Sizing storage volume (design storm method)

The required storage volume of an infiltration system is defined by the difference in inflow and outflow volumes for the duration of a storm. The inflow volume is a product of the rainfall, runoff coefficient and contributing area connected to the infiltration system, i.e.:

$$Flow\ Volume = \frac{CiAD}{1000}$$

Where:

$C = runoff\ coefficient$

$i = probabilistic\ rainfall\ intensity\ (mm/hr)$

$A = contributing\ area\ connected\ to\ the\ infiltration\ system\ (m^2)$

$D = storm\ duration\ (hours)$

Outflow from the infiltration system is via the base and sides of the infiltration system and is dependent on the area and depth of the system. In computing the infiltration from the walls of an infiltration system, Engineers Australia (2006) suggests that pressure is hydrostatically distributed and thus equal to half the depth of water over the bed of the infiltration system, i.e.:

$$Outflow\ Volume = \frac{\left[A_{inf} + \left(\frac{Pd}{2}\right)\right] U k_h D}{1000}$$

Where:

$k_h = point\ saturated\ hydraulic\ conductivity\ (mm/hr)$

$A_{inf} = infiltration\ area\ (m^2)$

$P = perimeter\ length\ of\ the\ infiltration\ area\ (m)$

$d = depth\ of\ the\ infiltration\ system\ (m)$

$U = point\ soil\ hydraulic\ conductivity\ moderating\ factor$

$D = storm\ duration\ (hours)$

Approximation of the required storage volume of an infiltration system can be computed as follows:

$$Required\ Storage\ = Inflow\ Volume - Outflow$$

Computation of the required storage will need to be carried out for the full range of probabilistic storm durations, ranging from six minutes to 72 hours and this calculation is usually performed using spreadsheet analysis. The critical storm event is the one which results in the highest required storage.

Infiltration trench sizing

To determine the length (L) of a gravel-filled or crate-box trench:

$$L = \frac{\forall}{e_s bH + 60k_h\tau\left(b + \frac{H}{2}\right)U}$$

(refer Argue 2004 for derivation)

Where:

L = *length of the trench (m)*

$\forall$ = *Inflow volume* (m^3)

e_s=*void space*

b = *width of the trench (m)*

H = *depth of the trench (m)*

k_h = *soil saturated hydraulic conductivity (m/s)*

τ = *time base of the design storm runoff hydrograph (min)*

U = *soil moderation factor (Table 3)*

Typical values for e_S are 0.35 for gravel, 0.95 for plastic milk-crate units and 0.50–0.75 for trenches part-occupied by perforated pipes.

In low permeability soils, the above equation results in trenches of impractical lengths. In such cases, it is recommended to build the infiltration device as a ‘soakaway’, that is a trench with a relatively larger plan area where length (L) is approximately equal to width (b). To determine the plan area (a) of this arrangement, the above equation reduces to:

$$a = \frac{\forall}{(e_s H + 60k_h\tau U)}$$

(refer Argue 2004 for derivation)

Where:

a = *required infiltration plan area* (m^2)

$\forall$ = *Inflow volume* (m^3)

e_s=*void space*

b = *width of the trench (m)*

H = *height/thickness of the system (m)*

k_h = *soil saturated hydraulic conductivity (m/s)*

τ = *time base of the design storm runoff hydrograph (min)*

U = *soil moderation factor (Table 3)*

The above equations assume the device is empty at the commencement of flow. Application of these equations must be followed by a check on the emptying time of the system’s storage.

Emptying time

Emptying time is defined as the time taken to completely empty a storage associated with an infiltration system following the cessation of rainfall. This is an important design consideration as the computation procedures previously described assume that the storage is empty prior to the commencement of the design storm event. Continuous simulation modelling for a range of catchments is required to provide reliable emptying time criteria. In the absence of this modelling, Engineers Australia (2006) recommends the interim emptying time criteria outlined in Table 5.

Table 5. Interim criteria for emptying time of an infiltration system for different EY/AEP

EY / annual exceedance probability (AEP)	< = 1 EY	0.5 EY	0.20% EY	10% AEP	5% AEP	2% AEP	1% AEP
Maximum emptying time in days	0.5	1.0	1.5	2.0	2.5	3.0	3.5

Emptying time is computed simply as the ratio of the volume of water in temporary storage (dimension of storage × porosity) to the infiltration rate (hydraulic conductivity × infiltration area).

The following formulae calculate the emptying time of infiltration basins and trenches, assuming draining by infiltration or percolation only. If assisted drainage is incorporated into the system, for example by provision of a slow drainage outlet pipe, then this needs to be taken into account.

The calculated emptying time should be compared to the values provided in Table 5 for the appropriate ARI to determine whether the acceptable emptying time criterion is exceeded. If so, the design should be amended, for example by distributing the flow to a greater number of infiltration units or larger area, or by providing a slow drainage outlet.

For a gravel-filled (or similar) trench, the emptying time is:

$$T = -\frac{4.6Lbe_s}{2k_h(L+b)} log_{10}\left(\frac{Lb}{Lb + 2H(L+b)}\right)$$

(refer Argue 2004 for derivation)

Where:

T = emptying time (s)

L = trench length (m)

b = trench width (m)

H = trench depth (m)

e_s = void space ratio (volume of voids/total volume occupied)

k_h = soil saturated hydraulic conductivity (m/s)

Where infiltration trenches have length (L) approximately equal to width (b), this equation simplifies to:

$$T \approx \frac{2He_s}{k_h}$$

(refer Argue 2004 for derivation)

Where the parameters are defined as above.

This equation can also be used for an open infiltration basin, by setting $e_s = 1.0$.

Inlet hydraulic structure

The inlet hydraulic structure is required to perform two functions for infiltration systems: provision of energy dissipation and bypass of above-design discharges.

Bypass can be achieved in a number of ways, most commonly using a surcharge pit, an overflow pit or discharge into an overflow pipe connected to a stormwater system.

Maintenance

Regular maintenance is required for proper operation of infiltration systems.

Maintenance plans should identify owners and parties responsible for maintenance, along with an inspection schedule. The use and regular maintenance of pre-treatment BMPs will significantly minimise maintenance requirements for infiltration systems.

Depending on the specific system implemented, maintenance should include at least the following:

- inspect and clean pre-treatment devices biannually (i.e. before and after the wet season) and ideally after major storm events
- once the infiltration system is operational, inspections should occur after every major storm for the initial few months to ensure proper stabilisation and function. Attention should be paid to how long water remains standing after a storm; standing water within the system for more than 72 hours after a storm is an indication that soil permeability has been over-estimated
- after the first wet season, infiltration systems should be inspected at least biannually (i.e. before and after the wet season).

Important items to check and clean or repair if required include:

- accumulated sediment, leaves and debris in the pre-treatment device, signs of erosion, clogging of inlet and outlet pipes and surface ponding
- when ponding occurs, corrective maintenance is required immediately.

In the case of infiltration trenches, clogging occurs most frequently on the surface. Grass clippings, leaves and accumulated sediment should be removed routinely from the surface. If clogging appears to be only at the surface, it may be necessary to remove and replace the first layer of filter media and the geotextile filter.

The presence of ponded water inside the trench after an extended period indicates clogging at the base of the trench. Remediation includes removing all of the filter media and geotextile envelope, stripping accumulated sediment from the trench base, scarifying to promote infiltration and replacing new filter media and geotexile. Vegetation can assist in prevention of clogging as the root network breaks up the soil and thereby promotes infiltration.

In the case of infiltration basins, sediment should be removed when it is dry enough so the sedimentation layer can be readily separated from the basin floor. Refer to BMP 2.2.2 *Maintenance of the stormwater network* in Chapter 7 for further guidance on managing sediments removed from the stormwater system.

Worked example

Caution: The following worked examples use Rational Method as per the Australian Rainfall and Runoff (ARR) Book VIII (Institution of Engineers, Australia 2001). However, as per the updated ARR Book 9

'Runoff in Urban Areas', the Rational Method is only suitable for lot-scale scale catchments or simplistic small catchments where flood routing is not critical. This method is not suitable for a 'precinct' scale estimation of peak flows as it has 'limited' runoff generation and surface routing capabilities. If runoff volume management infrastructure forms part of a solution, or if an understanding of potential impacts on downstream flooding are required, then a 'strong' hydrologic estimation method such as a runoff-routing model should be used (ARR 2019). For further information on the limitations of Rational Method, please refer to Book 9 'Runoff in Urban Areas' of ARR 2019.

The following worked example is based on a WSUD workshop held by John Argue in Perth, November 2005.

An onsite stormwater retention system is to be designed for runoff from a roof located in Perth. The site is located in an elevated area with good clearance to groundwater, hence application of the formulae contained in the design guideline for this BMP is considered appropriate. Given the layout of the site, an infiltration trench with length (L) approximately equal to width (b) is required to be designed. Two styles of trench are compared in the design process to determine which is most suitable for the site.

The design parameters are listed below:

roof area, $A\ =\ 400\ m^2$

soil saturated hydraulic conductivity, $k_h\ =\ 1.6\ \times\ 10-4\ m/s\ (sandy)$

gravel-filled infiltration trench void space , $e_s\ =\ 0.35$

crate system infiltration trench void space, $e_s\ =\ 0.95$

gravel-filled infiltration trench depth, $H\ =\ 0.40\ m$

crate infiltration trench height, H = 0.40 m

Based on spreadsheet analysis, for a required design average recurrence interval (ARI) of two years (or 0.5 EY), refer Engineers Australia (2006) for methods of 't' calculation:

$site\ t_c\ =\ 15\ minutes\ (calculated\ site\ time\ of\ concentration)$

$site\ t\ =\ 30\ minutes\ (calculated\ site\ time\ of\ concentration)$

$$\tau\ =\ 15 + 30$$
$$= 45\ minutes\ (time\ base\ of\ the\ design\ storm\ runoff\ hydrograph\ in\ Figure\ 3)$$

Based on the above, the design rainfall intensity i_2 = 31.7 mm/hr (refer to Rainfall Intensity–Frequency–Duration curves for Perth, available from Bureau of Meteorology).

Runoff volume

$$Inflow\ Volume\ \forall = \frac{CiAD}{1000}$$

From ARR Book VIII (Institution of Engineers Australia 2001):

$$C_y\ =\ F_y.C_{10}$$

Where:

$C_y\ =\ runoff\ coefficient\ for\ a\ 'Y'\ year\ ARI\ (currently\ termed\ as\ EY\ or\ AEP)$

$F_y\ =\ frequency\ factor\ for\ rational\ method\ runoff\ coefficients$

$C_{10}\ =\ 10\ year\ ARI\ (or\ 10\%\ AEP)\ runoff\ coefficient\ (0.9\ where\ the\ fraction\ impervious\ is\ 1)$

Therefore, for ARI = 2 years or (0.5 EY):

$$C_2 = F_2 . C_{10}$$

$$C_2 = 0.85 * 0.90$$

$$C_2 = 0.765$$

$$Inflow\ Volume\ \forall = 0.765 * \frac{31.7}{1000}(mhr^{-1}) * 400(m^2) * \frac{30}{60(hr)}$$

$$\forall = 4.85\ (m^3)$$

Gravel-filled infiltration trench

Determine the plan area (a) for the gravel-filled infiltration trench:

$$a = \frac{\forall}{(e_s H + 60 k_h \tau U)}$$

$$a = \frac{4.85}{(0.35 * 0.4 + 60 * 1.6 * 10^{-4} * 45 * 0.5)}$$

$$a = 13.6 m^2$$

Determine the emptying time (T):

$$T = \frac{2He_s}{k_h}$$

$$T = \frac{2 * 0.4 * 0.35}{1.6 * 10^{-4}}$$

$$T = 1750\ seconds$$

$$T = 29\ minutes$$

The acceptable maximum emptying time for a two year ARI event or (0.5 EY) is one day (Table 5), therefore the gravel-filled infiltration trench design is suitable.

Crate infiltration trench

Determine plan area (a) for the crate infiltration trench:

$$a = \frac{\forall}{(e_s H + 60 k_h \tau U)}$$

$$a = \frac{4.85}{(0.95 * 0.4 + 60 * 1.6 * 10^{-4} * 45 * 0.5)}$$

$$a = 8.14 m^2$$

Determine the emptying time (T):

$$T = \frac{2He_s}{k_h}$$

$$T = \frac{2 * 0.4 * 0.95}{1.6 * 10^{-4}}$$

$$T = 4750\ seconds$$

$$T = 1\ hour\ 19\ minutes$$

The emptying time is less than the maximum acceptable emptying time of a two year ARI (or 0.5 EY) event, therefore the design of the crate infiltration trench is suitable.

Given that both the gravel-filled and crate system infiltration trenches emptied within an acceptable time, a crate system is selected for this site as it requires a smaller plan area.

References and further reading

Agriculture Western Australia 1998, *Soilguide: a handbook for understanding and managing agricultural soils*, Bulletin No 4343, Agriculture Western Australia, Perth, Western Australia.

Argue, J. R. (Editor) 2004, *Water Sensitive Urban Design*: *basic procedures for 'source control' of stormwater – a handbook for Australian practice*, Urban Water Resources Centre, University of South Australia, Adelaide, South Australia, in collaboration with Stormwater Industry Association and Australian Water Association.

Department of Water and Environmental Regulation, *Decision process for stormwater management in WA*, Department of Water and Environmental Regulation, Perth, Western Australia.

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Taylor, A.C. 2005, Structural Stormwater Quality BMP Cost/Size Relationship Information from the Literature (Version 3), Cooperative Research Centre for Catchment Hydrology, Melbourne, Victoria.

3.2 Soakwells

Figure 1. PVC Soakwell. (Source: Reln Pty Ltd 2006.).

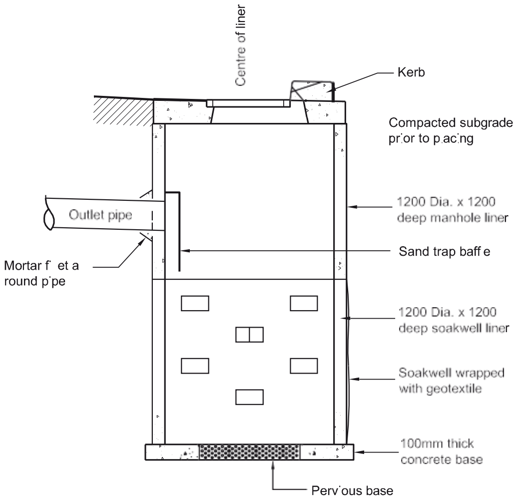

Figure 2. Standard Combination Gully/Soakwell. 2021.) (Source: Glover, City of Bayswater.)

Background

An alternative method for infiltration is using soakwells. These systems are used widely in WA as an at-source stormwater management control, typically in small-scale residential and commercial applications, or as road side entry pits at the beginning of a stormwater system. Soakwells can be applied in retrofitting scenarios and existing road side entry pits/gullies can be retrofitted to perform an infiltration function. See section 6.2.2 of Chapter 6: Retrofitting for further information.

Soakwells consist of a vertical perforated liner, with stormwater entering the system via an inlet pipe at the top of the device (Figure 3). The base of the soakwell is open or perforated and usually covered with a geotextile. Alternatively, pervious material, such as gravel or porous pavement, can be used to form the base of the soakwell.

Where source water may have a high sediment load, there should be pre-treatment, such as filtering, as soakwells are susceptible to clogging.

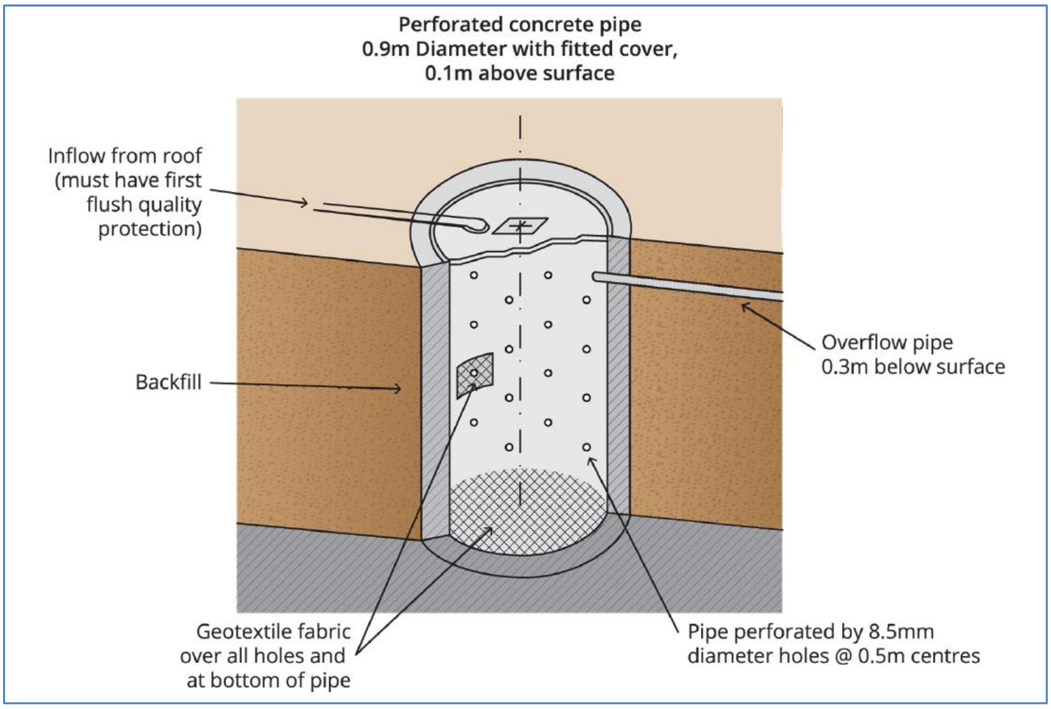

Figure 3. Leaky well infiltration system.

Performance efficiency

Data on the performance efficiency of infiltration systems is presented in the Performance Efficiency section and Table 1 of the Infiltration Basins and Trenches BMP, based on Fletcher et al. (2004).

Cost^

The cost for soakwell systems can vary considerably according to the type of soakwell to be installed, site-specific conditions (including soil type), configuration, location, storage volumes, and landscaping and restoration requirements.

See Chapter 6: Retrofitting, Case Study 7.1 'Town of Mosman Park – Total Water Cycle Project' for further information on the costs of a catchment-wide infiltration project. An example of the techniques used in the Town of Mosman Park to maximise infiltration in the catchment is the installation of combination gully/soakwells, as shown in Figure 2. The cost of installation and materials for each 2.4 m deep soakwell was approximately $1,300 per unit (note : 2004/05 prices) (Glover, M. 2007, pers. comm.[2]).

Installation and other associated works are a significant proportion of the cost of these systems. Soakwells are a relatively cheap stormwater management measure for lot-scale application.

^The costs quoted in this section are from around 2000 to 2007 and have not been adjusted for inflation or potential cost changes due to technological advances which may have occurred since this chapter was published in 2007. Therefore, it should be considered indicative only and users of the manual are encouraged to seek further specific industry advice on the current costs as appropriate.

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[2] Personal communication with Martyn Glover, City of Bayswater, 2007.

Design considerations

Design considerations for soakwells are similar to those for other infiltration systems.

Soil type and stability, topography, separation to groundwater, setback to buildings and pre-treatment to remove sediment, litter and other pollutants must all be considered. These issues are discussed in the Design Considerations section of the Infiltration Basins and Trenches BMP.

Design guidelines

The calculations contained in this section for sizing the storage volume of soakwells and determining emptying time are based on Engineers Australia (2006) and Argue (2004). The calculations should be applied with caution to the sizing of infiltration systems where shallow groundwater is present. This approach does not consider the impacts of shallow groundwater in its calculation, which may reduce infiltration capacity. Detailed modelling of shallow groundwater table situations is recommended. Designers should take into account the maximum groundwater level, and hence the minimum infiltration potential, in determining their flood detention design. However, designers should also consider maximum infiltration opportunities to achieve aquifer recharge when the groundwater table is below its maximum level (refer to the Design Considerations section of the Infiltration Basins and Trenches BMP for further discussion).

Inflow volume

The required storage volume is defined by the difference in inflow and outflow volumes for the duration of a storm. The inflow volume is a product of the rainfall, runoff coefficient and contributing area connected to the infiltration system, i.e.:

$$Flow\ Volume = \frac{CiAD}{1000}$$

Where:

$C = \textit{runoff coefficient}$

$i = \textit{probabilistic rainfall intensity (mm/hr)}$

$A = \textit{contributing area connected to the infiltration system (m}^{2}\textit{)}$

$D = \textit{storm duration (hours)}$

Soakwell sizing

Note that the following equation is based on an approximation where d ≈ H and may not be valid for other design situations.

Argue (2004) provides the following formula for sizing of a soakwell:

$$d = \sqrt{\frac{\forall}{\frac{\pi}{4}(H + 120k_h tU}}$$

(refer to Argue 2004 for derivation)

Where:

$d\ =\ well\ diameter\ (m)$

$\forall =\ Inflow\ volume\ (m^3)$

$H\ =\ well\ height\ (m)$

$$k_h = soil\ saturated\ hydraulic\ conductivity\ (ms^{-1})$$

$$\tau = time\ base\ of\ the\ design\ storm\ runoff\ hydrograph\ (min)$$

$$U = soil\ moderation\ factor\ (Table\ 3\ in\ the\ Infiltration\ Basins\ and\ Trenches\ BMP)$$

The above equation assumes the device is empty at the commencement of flow. Application of this equation must be followed by a check on the emptying time of the system's storage.

Emptying time

Emptying time is defined as the time taken to completely empty a storage associated with an infiltration system following the cessation of rainfall. This is an important design consideration as the computation procedures previously described assume that the storage is empty prior to the commencement of the design storm event.

Argue (2004) provides the following formula for calculating the emptying time for soakwells:

$$T = -\left(\frac{4.6d}{4k_h}\right) log_{10}\left(\frac{\frac{d}{4}}{H + \left(\frac{d}{4}\right)}\right)$$

(refer to Argue 2004 for derivation)

$$T = emptying\ time\ (s)$$

$$d = well\ diameter\ (m)$$

$$H = well\ height\ (m)$$

$$k_h = soil\ saturated\ hydraulic\ conductivity\ (ms^{-1})$$

Further discussion regarding emptying times is contained in the Design Guidelines section of the Infiltration Basins and Trenches BMP.

Maintenance

Soakwells require maintenance for efficient operation and to reduce the risk of mosquito breeding, including regular inspection and cleaning to prevent clogging by sediments and litter. Pre-treatment BMPs can significantly reduce the maintenance requirements by preventing sediments and litter from entering the system. To prevent road/carpark soakwells from being clogged with sediment/litter during road and housing/building construction (see Figure 4), temporary bunding or sediment controls need to be installed (see Figure 5 for an example of a sediment fence). See section 2.1.1 'Land development and construction sites' of Chapter 7 for information about site management practices.

A maintenance plan for infiltration systems is described in the Maintenance section of the Infiltration Basins and Trenches BMP.

Figure 4. Soil entering a side entry pit that has a soakwell at the base; located within a pipeless subdivision within the City of Mandurah, WA. (Photograph: Department of Water 2007.)

Figure 5. Silt fence for controlling sediment during land development. (Photograph: André Taylor, Ecological Engineering Pty Ltd.)

Worked example

Caution: The following worked examples use Rational Method as per the Australian Rainfall and Runoff (ARR) Book VIII (Institution of Engineers, Australia 2001). However, as per the updated ARR Book 9 'Runoff in Urban Areas', the Rational Method is only suitable for lot-scale scale catchments or simplistic small catchments where flood routing is not critical. This method is not suitable for a 'precinct' scale estimation of peak flows as it has 'limited' runoff generation and surface routing capabilities. If runoff volume management infrastructure forms part of a solution, or if an understanding of potential impacts on downstream flooding are required, then a 'strong' hydrologic estimation method such as a runoff-routing model should be used (ARR 2019). For further information on the limitations of Rational Method, please refer to Book 9 'Runoff in Urban Areas' of ARR 2019.

The following worked example is based on a WSUD Workshop held by John Argue in Perth, November 2005.

An onsite stormwater retention system is to be designed for runoff from a roof located in Perth. The site is located in an elevated area with good clearance to groundwater, hence application of the formulae contained in the design guideline for this BMP is considered appropriate.

The design parameters are listed below:

$$Roof\ area, A\ =\ 400\,m^2$$

$$Soil\ saturated\ hydraulic\ conductivity, k_h\ =\ 1.6\ \times\ 10-4\,ms^{-1}\ (sandy)$$

$$Standard\ soakwell\ effective\ height, H\ =\ 2.30\,m$$

Based on spreadsheet analysis, for a required design average recurrence interval (ARI) of two years (or 0.5 EY), refer Engineers Australia (2006) for methods of 't' calculation:

$$site\ t_c\ =\ 15\,minutes\ (calculated\ site\ time\ of\ concentration)$$

$$site\ t\ =\ 30\,minutes\ (calculated\ site\ time\ of\ concentration)$$

$$\tau = 15 + 30$$
$$= 45\, minutes\ (time\ base\ of\ the\ design\ storm\ runoff\ hydrograph\ in\ Figure\ 3)$$

Based on the above, the design rainfall intensity i_2 = 31.7 mm/hr (refer to Rainfall Intensity–Frequency–Duration curves for Perth, available from Bureau of Meteorology).

Runoff volume

$$Inflow\ Volume\ \forall = \frac{CiAD}{1000}$$

From ARR Book VIII (Institution of Engineers Australia 2001):

$$C_y = F_y . C_{10}$$

Where:

$$C_y = runoff\ coefficient\ for\ a\ 'Y'\ year\ ARI\ (currently\ termed\ as\ EY\ or\ AEP)$$

$$F_y = frequency\ factor\ for\ rational\ method\ runoff\ coefficients$$

$$C_{10} = 10\ year\ ARI\ (or\ 10\%\ AEP)\ runoff\ coefficient\ (0.9\ where\ the\ fraction\ impervious\ is\ 1)$$

Therefore, for ARI = 2 years or (0.5 EY):

$$C_2 = F_2 . C_{10}$$

$$C_2 = 0.85 * 0.90$$

$$C_2 = 0.765$$

$$Inflow\ Volume\ \forall = 0.765 * \frac{31.7}{1000}(mhr^{-1}) * 400(m^2) * \frac{30}{60(hr)}$$

$$\forall = 4.85\ (m^3)$$

Soakwell sizing

Argue (2004) provides the following formula for sizing of a soakwell:

$$d = \sqrt{\frac{\forall}{\frac{\pi}{4}(H + 120 k_h t U}}$$

(refer to Argue 2004 for derivation)

Where:

$$d = well\ diameter\ (m)$$

$$\forall = Inflow\ volume\ (m^3)$$

$$H = well\ height\ (m)$$

$$k_h = soil\ saturated\ hydraulic\ conductivity\ (ms^{-1})$$

$$\tau = time\ base\ of\ the\ design\ storm\ runoff\ hydrograph\ (min)$$

$$U = soil\ moderation\ factor\ (Table\ 3\ in\ the\ Infiltration\ Basins\ and\ Trenches\ BMP)$$

Where U = 0.5 for sandy soils.

$$d = \sqrt{\frac{4.85}{\frac{\pi}{4}(2.3 + 120 * 1.6 * 10^{-4} * 45 * 0.5)}}$$

$$d = 1.50\ m$$

Emptying time

Determine the emptying time (T):

$$T = -\left(\frac{4.6d}{4k_h}\right) log_{10}\left(\frac{\frac{d}{4}}{H + \left(\frac{d}{4}\right)}\right)$$

$$T = -10781 * -0.8533$$

$$T = 9199\ seconds$$

$$T = 2\ hours\ 33\ minutes$$

The acceptable maximum emptying time for a two year ARI event (0.5 EY event) is one day (Table 5) of the Infiltration Basins and Trenches BMP), therefore the soakwell design is suitable.

References and further reading

Argue, J. R. (Editor) 2004, *Water Sensitive Urban Design*: *basic procedures for 'source control' of stormwater – a handbook for Australian practice*, Urban Water Resources Centre, University of South Australia, Adelaide, South Australia, in collaboration with Stormwater Industry Association and Australian Water Association.

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Fletcher, T.D., Duncan, H.P., Poelsma, P. and Lloyd, S.D. 2004, *Stormwater flow and quality, and the effectiveness of non-proprietary stormwater treatment measures — a review and gap analysis*, Cooperative Research Centre for Catchment Hydrology, Melbourne, Victoria.

Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M and Testoni I (Editors) 2019, *Australian Rainfall and Runoff: A Guide to Flood Estimation*, Commonwealth of Australia, Barton, Australian Capital Territory.

Taylor, A.C. 2005, *Structural Stormwater Quality BMP Cost/Size Relationship Information from the Literature (Version 3)*, Cooperative Research Centre for Catchment Hydrology, Melbourne, Victoria.

3.3 Pervious pavement

Figure 1. Pervious paving in a commercial carpark, Burswood. (Photograph: Department of Water 2006.)

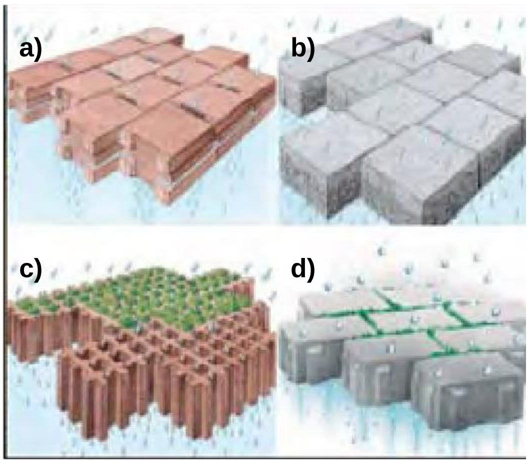


Figure 2. Types of permeable paving: a) pavers with canals b) porous pavers c) greened permeable pavers with small apertures d) greened permeable pavers with wide joints. (Dierkes et al. 2002.)

Background

Permeable/porous (collectively termed pervious) paving can be used as an alternative to traditional impervious hard surfaces, such as roads, carparks, footpaths and public squares. Bitumen, concrete and other hard surface areas (such as paving surrounding buildings) are typically impermeable and result in high runoff rates during a storm event. This runoff can be reduced by interspacing permeable material, such as lawn or pebbles, between widely spaced impermeable pavers, or by installing porous paving.

There are different types of porous pavements, including porous asphalt pavement, porous concrete pavement and modular interlocking concrete bricks with internal or external drainage cells. Porous pavement comprises a thick layer of highly porous material, for example an asphaltic layer of gap-graded coarse aggregate held together with bitumen, or a well-compacted mixture of graded sand and gravel (Argue 2004).

The porous pavement is typically laid on top of a high-void aggregate or gravel base layer, with a geotextile in between (Figure 3). The stormwater passes through the pore spaces of the pavement, through the geotextile and into the aggregate/gravel layer, which provides temporary storage as the water gradually infiltrates into the subsoil. Where the subsoil has low permeability, the water can be removed by providing a slow drainage outlet to the receiving stormwater system.

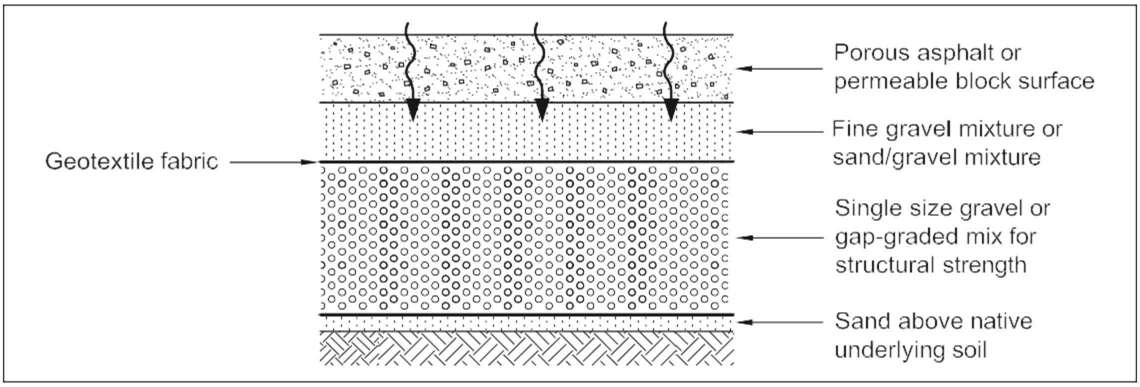


Figure 3. Schematic of a section through pervious pavement.

Performance efficiency

Pervious pavements can remove sediments and some nutrients, heavy metals and hydrocarbons from polluted stormwater via the processes of adsorption, filtering and biological decomposition.

A field study by Brattebo and Booth (2003) of four different types of porous paving installed in a parking area found no oil, fuel or lead in the water infiltrated through the paving, even though these pollutants were present in the direct surface runoff from the impermeable asphalt control sample.

Field studies have also shown pervious pavement to be very effective at retaining dissolved metals (Dierkes et al. 2002).

Rankin and Ball (2004) found that the impervious area on a road surface reduced from 45% to 5% when pervious pavements were used. Subsequent monitoring found that surface runoff water quality improved and there was no increase in groundwater contaminants.

Cost^

Summary costs for pervious paving are presented in Table 1. These costs are inclusive of excavation and profiling and installation of gravel, sand and geofabric liners.

Fletcher et al. (2004) reported that the typical annual maintenance costs of permeable paving in California (when converted from US dollars) were approximately $9,700/ha.

Table 1. Pervious paving installation costs (Boral 2003 cited in Taylor 2005)

Pervious Paving Method	Construction Cost
Porous paving allowing infiltration	\$111/m^2
Porous paving over sealed sub-grade allowing water collection	\$119/m^2
Augmentation with porous paving (i.e. mixing porous with normal pavers)	\$98/m^2
Porous paving with asphalt	\$67/m^2
Porous paving with concrete slab	\$90/m^2

^The costs quoted in this section are from around 2000 to 2005 and have not been adjusted for inflation or potential cost changes which may have occurred since this chapter was published in 2007. Therefore, it should be considered as indicative only and users of the manual are encouraged to seek further specific industry advice on the current costs as appropriate.

Design considerations

As with other infiltration systems, designing pervious pavement systems requires consideration of the site conditions and potential contamination of the receiving groundwater environment. A detailed discussion of these considerations is provided in the Design Considerations section of the Infiltration Basins and Trenches BMP.

There are some specific considerations for the design of pervious pavement. Some pervious pavement systems have a high failure rate that is attributed to poor design, clogging by fine sediment and excess traffic use (USEPA 1999).

Pervious pavement systems are not suitable for areas with slopes greater than 5% or high wind erosion rates (USEPA 1999). Soils that feature a rising water table, saline conditions, dispersive clay or low hydraulic conductivity are not suitable for pervious pavement.

Pervious pavement systems require regular vacuum sweeping to prevent clogging by fine sediment and maintain porosity. Alternatively, sediment traps and vegetation filter strips can be used to prevent sediment entering the system (Coombes 2003). Excessive vehicle traffic is also a common cause of failure. Pervious pavement should be used for low volume parking and roads with light vehicle use (USEPA 1999). To prevent pervious pavement from being clogged with sediment/ litter during road and housing/building construction, temporary bunding or sediment controls need to be installed. See section 2.1.1 ‘Land development and construction sites’ of Chapter 7 for information about site management practices.

Design guidelines

The following method for calculation is based on Argue (2004). The equations are applicable where the overall value of the hydraulic conductivity for the product and its underlying sub-structure is known. This method should be applied with caution to the sizing of infiltration systems where shallow groundwater is present. This approach does not consider the impacts of shallow groundwater in its calculation, which may reduce infiltration capacity. Detailed modelling of shallow water table situations is recommended. Designers should take into account the maximum groundwater level, and hence the minimum infiltration potential, in determining their flood detention design. However, designers should also consider maximum infiltration opportunities to achieve aquifer recharge when the groundwater table is below its maximum level (refer to the Design Considerations section of the Infiltration Basins and Trenches BMP for further discussion).

The required infiltration capacity of a soil surface, vegetated area or pervious pavement for a selected design storm event (with zero overflow) is calculated by:

$$Q_{peak} = k_h A_{inf}$$

Where:

$Q_{peak} = peak\ design\ runoff\ rate\ from\ the\ contributing\ catchment\ (m^3/s)$

$k_h = design\ hydraulic\ conductivity\ (m/s)$

$A_{inf} = surface\ area\ available\ for\ infiltration\ (m^2)$

Hence:

$$\frac{CiAD}{1000 * 60^2} = k_h A_{inf}$$

Where:

$C = runoff\ coefficient$

$i = probabilistic\ rainfall\ intensity\ (mmhr^{-1})$

$A = total\ defined\ catchment\ area\ (m^2)$

i.e. the area of the treatment surface plus the surrounding contributing catchment area

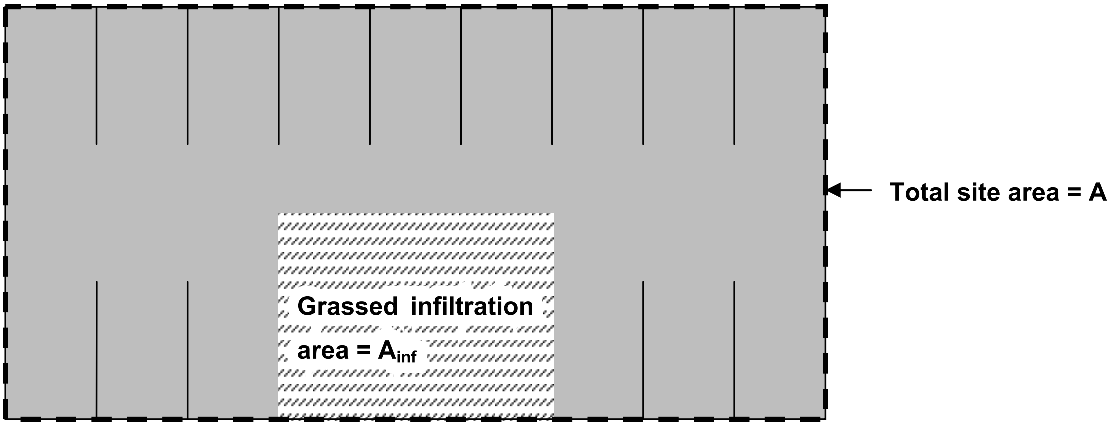
This equation applies where the infiltration surface is located within the total defined catchment area (A), as shown in Figure 4, the paving is uniformly porous and the overall value of the hydraulic conductivity for the product and its underlying sub-structure is known. However, for permeable paving where part of the pavement area is impervious (for example, area taken up by lattice work) and this has not been accounted for in the overall value of the hydraulic conductivity, a blockage factor must be applied. The blockage factor accounts for the surface area of the pavement that is not contributing to infiltration (as shown in Figure 5).

Hence:

$$\frac{CiAD}{1000 * 60^2} = k_h(1 - \Psi)A_{inf}$$

Where:

$$\Psi = infiltration\ surface\ blockage\ factor$$



Note: this equation applies where the infiltration surface is located within the total defined catchment area (A).

Figure 4. Example definition of a catchment area where the infiltration surface is located within the defined site area.

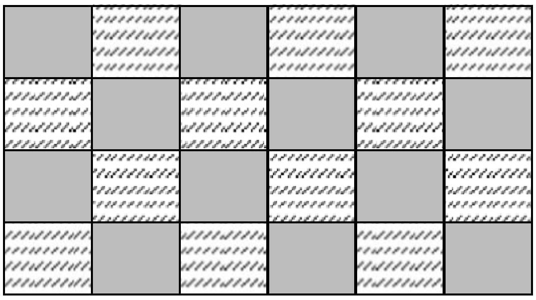


Figure 5. A blockage factor of 0.5 would need to be applied to account for the impervious concrete pavers interspaced with grass squares in this illustration of permeable paving.

Where the infiltration surface is external to the impervious area from which it is receiving runoff (as shown in Figure 6), Q_{peak} passing to the infiltration surface must also take into account the rainfall input to the surface itself.

Hence, total peak inflow:

$$Q_{peak} = \frac{CiA}{1000 * 60^2} + \frac{A_{inf}i}{1000 * 60^2}$$

The flow capacity of the pervious area:

$$Q_{peak} = k_h(1 - \Psi)A_{inf}$$

To determine the required area of the pervious surface:

$$\frac{CiA}{1000 * 60^2} + \frac{A_{inf}i}{1000 * 60^2} = k_h(1 - \Psi)A_{inf}$$

$$A_{inf} = \frac{CiA}{1000 * 60^2 * \left[k_h(1 - \Psi) - \frac{i}{1000 * 60^2}\right]} \ [m^2]$$

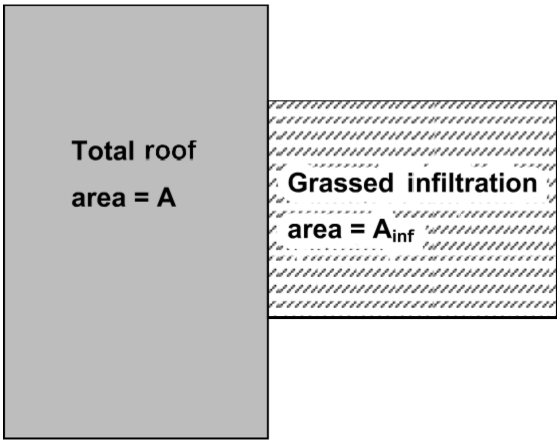


Figure 6. Example definition of a catchment area where the infiltration surface is located external to the defined site area.

Note that in the previous equations, if the soil hydraulic conductivity has been determined by small test pits and boreholes, k_h should be multiplied by the moderation factor U (see Table 3 in the Infiltration Basins and Trenches BMP). Where the long-term or life span hydraulic conductivity is used (as described below), U = 1 may be applied.

The design of pervious paving should consider the reduction in permeability of the pervious surface over time due to sediment accumulation and clogging. Laboratory testing found that the permeability decreased to around 30-50% of the original 'new' product value after a period of approximately 30 modelled years (Argue 2004). Over the lifespan of the paving, it is anticipated the permeability reduces to approximately 20% (Argue 2004). Therefore, the design of pervious infiltration systems should adopt a hydraulic conductivity equal to 20% of the 'new' value to ensure acceptable lifespan performance. The lifespan of a pervious paving system will depend on the ratio of impervious to pervious area of the contributing catchment surface, and the catchment characteristics, e.g. the amount of trees and sediment in the catchment. Partial blockage over time of a permeable paving system adjacent to an impervious catchment is illustrated in Figure 7.

The lifespan of vegetated porous surfaces is around five times the lifespan of pervious pavement. Further design information, including estimated lifespans of pervious paving systems under different conditions, is provided by Argue (2004).

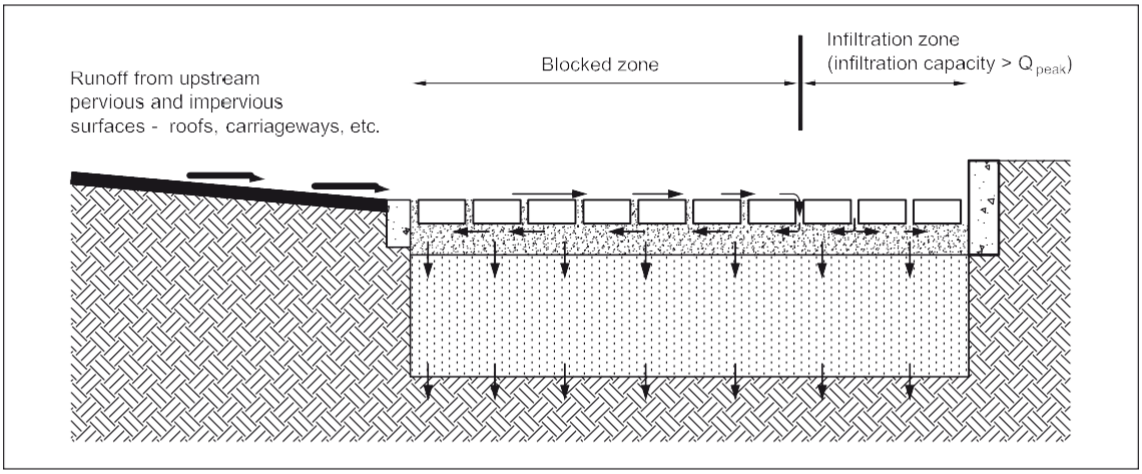


Figure 7. Partial blockage over time of a permeable paving system adjacent to an impervious catchment.

Maintenance

Maintenance of pervious pavement systems requires regular inspection and cleaning to maintain porosity, repair of potholes and cracks and replacement of clogged areas.

Regular vacuum sweeping can improve the efficiency of the system. It is recommended that cleaning be undertaken every three months (Coombes 2003). Overseas experience in the use of pervious paving has shown that complete clogging can occur between five and 10 years after installation, so cleaning of the paving is essential (Dierkes et al. 2002).

A maintenance schedule similar to conventional road surfaces, involving retaining the pavers and replacing part of the underlying sand to remove contaminants, is also recommended for concrete grid, ceramic and plastic modular blocks (Coombes 2003).

Worked example

Assess the use of a reinforced turf courtyard to infiltrate runoff from a 200 m^2 adjacent bitumen carpark.

$$carpark\ impervious\ surface\ area, A\ =\ 200\ m^2$$

$$blockage\ factor\ for\ the\ reinforced\ turf\ product\ selected, \Psi\ =\ 0.1$$

$$hydraulic\ conductivity\ of\ the\ `new'\ reinforced\ turf, k_h\ =\ 2.5\ \times\ 10 - 4\ ms^{-1}$$

$$hydraulic\ conductivity\ of\ the\ `old'reinforced\ turf, k_h\ =\ 5 * 10^{-5} - 5\ ms^{-1}$$

$$site\ time\ of\ concentration, t_c\ site\ =\ 5\ minutes$$

$$ARI\ =\ 2\ years$$

$$runoff\ coefficient\ for\ a\ two\ year\ ARI\ (0.5\ EY) event, C_2\ =\ 0.765$$

(see Worked Example section of the Infiltration Basins and Trenches BMP for calculation of C)

Based on t_c = 5 minutes and ARI = 2 years, the rainfall intensity i_2 = 78.0 mm/hr (from Rainfall Intensity – Frequency – Duration curves for Perth, available from the Bureau of Meteorology).

The required area of the courtyard is estimated as:

$$A_{inf} = \frac{CiA}{1000 * 60^2 * \left[k_h(1 - \Psi) - \frac{i}{1000 * 60^2}\right]}\ [m^2]$$

$$A_{inf} = \frac{0.765 * 78.0 * 200}{1000 * 60^2 * \left[(5 * 10^{-5}) * (1 - 0.1) - \frac{78.0}{1000 * 60^2}\right]}$$

$$A_{inf} = 142\ m^2$$

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4 Conveyance systems

4.1 Swales and buffer strips

Figure 1. Flush kerbing and broken kerbing used to allow flow from a carpark into a swale at Point Fraser, Perth. (Photograph: Department of Water 2006.)

Figure 2. Vegetated swale in Gosnells, making use of native species in a parkland setting. (Photograph: Department of Water 2004.)

Background

Swales are very important for disconnecting impervious areas from downstream surface water bodies and receiving environments. These systems convey stormwater, promoting infiltration and reducing stormwater runoff peak flow, velocity and volume, and remove coarse and medium sediments, including suspended solids and trace metals. Swales also assist in protecting surface water bodies from frequent storm events by reducing flow velocity compared to discharges from hydraulically efficient piped drainage systems.

A vegetated swale is a broad, shallow channel with vegetation covering the side slopes and base. Vegetation can range from grass to native sedges and shrubs, depending on hydraulic and landscape requirements.

Vegetated swales are used instead of the conventional piped system as part of stormwater conveyance. They are usually placed in POS (Figure 2), or within the median or along the shoulders of main roads, rather than within residential lots and verges. Typically combined with buffer strips and/ or bioretention systems, vegetated swales are reliant on hydraulic roughness and gentle slopes to retard flow velocities. Swales also have lower capital costs than traditional piped systems and enhance biological diversity and create beneficial habitat, as well as improve visual aesthetics within a community.

The treatment efficiency of swales is variable for different pollutants and swales may not provide sufficient treatment on their own to meet water quality objectives. However, when used as part of the overall stormwater management system, swales are a useful at-source and in-transit water quantity management tool, while providing initial treatment for water quality outcomes.

Buffer strips are areas of vegetation through which runoff passes while travelling to a discharge point and are therefore aligned perpendicular to the direction of flow. They reduce sediment loads by passing a sheet flow of shallow depth through vegetation. The vegetation acts to slow the flow and trap coarse sediments.

Buffer strips typically require uniformly distributed flow, such as sheet flow that comes off a road, carpark or other impervious area. Buffer strips also can be applied around other structural BMPs, such as living streams and constructed wetlands.

The processes which occur in vegetated swales and buffer strips are quite complex, and involve physical and biochemical components. Physical processes for particulate removal (and consequently particle-bound pollutants, such as phosphorus) include infiltration, deposition and filtration. Nitrogen removal is a function of denitrification, biostorage (plant and animal uptake) and changes in soil storage.

While providing water conveyance, vegetated swales and buffer strips will often retain and detain water at different times of the year, due to the seasonal nature of local rainfall and variability in groundwater levels. For example, in summer, autumn and early winter in the south-west of WA (when groundwater levels are at their lowest), a swale in sandy soils may perform as a retention/detention system, with the majority of storm events infiltrating and little or no flow occurring.

Performance efficiency

While essentially a conveyance based system, one of the major roles of swales is to provide disconnection from the receiving environment. Research and past experiences suggest that vegetated swales represent a practical and potentially effective technique for controlling urban runoff quantity and quality. While limited local WA performance data exists for vegetated swales, it is known that riffles, gentle slopes, permeable soil, dense vegetation cover and slow velocity all contribute to successful pollutant removal by the swale system.

Even vegetated strips adjacent to major roads that are not intended for treatment of stormwater runoff can play an important part in reducing the concentrations of pollutants and reducing the volume of stormwater discharged to surface waters as a result of retention/infiltration in to the soil. Removal of heavy metals appears to be directly related to removal of sediment.

Grass swales and filter strips are also effective means of removing sediment from urban stormwater runoff. The removal of total suspended solids (TSS) along the grass swale is a primarily physical process (sedimentation and filtration), reflecting the balance between flow and particle settling velocity. The higher the flow rate, the longer the distance (and therefore grass length) required to remove suspended solids. Removal of total nitrogen (TN) and total phosphorus (TP) also occurs in the form of exponential decay along the grass length. As the removal performance of grass swales and filter strips is a function of flow rate, grass density, particle size and density, the above conclusions therefore, may not apply in different field situations.

Australian Runoff Quality (Engineers Australia 2006) provides estimates of typical expected annual pollutant load removal efficiencies for vegetated swales, as shown in Table 1, based on research of eastern states catchments. Actual swale performance will vary depending on individual design parameters such as temporal variation in flow and pollutant input concentration, vegetation height, infiltration capacity, length of swale and detention (contact) time.

Swale performance in WA is often likely to vary from the efficiencies of swales in the eastern states shown in Table 1, particularly at sites with sandy soils and shallow groundwater. Additionally, infiltration is more likely to be a dominant process at sandy sites. Annual pollutant load removal efficiencies for swales on sandy soils would usually be expected to be higher than shown in Table 1 due to the increased infiltration rate reducing surface water discharge.

Table 1. Typical annual pollutant load removal efficiencies for vegetated swales

Pollutant	Expected removal	Comments
Litter	> 90%	Should be 100%, provided there is adequate vegetation cover and flow velocities below 0.5 m/s.
Total suspended solids	60–80%	Assumes low level of infiltration. Will vary with varying particle size distribution.
Total nitrogen	25–40%	Depends on speciation and detention time.
Total phosphorus	30–50%	Depends on speciation and particle size distribution.
Coarse sediment	> 90%	Assumes re-suspension and scouring prevented by controlling inflow velocity <0.8 m/s and maintaining dense vegetation.
Heavy metals	20–60%	Highly variable, depends on particle size distribution, ionic charge, detention time, etc.

(Source: Engineers Australia 2006)

Cost^

Standard cost data for construction of swales and buffer strips in WA is not readily available. As a guide, a range of costs for swale and buffer strip construction and maintenance for eastern states areas is presented in Table 2, based on data contained in Taylor (2005).

Table 2. Cost estimates for swales and buffer strips

Publication/ Data source	Construction ($/m^2)	Annual maintenance ($/m^2/yr)	Location	Description
Swales				
Lloyd et al. (2002)	-	$2.50	-	Grass swale
	-	$9.00		Vegetated swales (initial)
	-	$1.50		Vegetated swales (after 5 yrs)
Fletcher et al. (2004)	$4.50		Melbourne	Hydro-mulching, earthwork and labour
	$9.50			Rolled turf
	$15 – $20			Vegetated swale
URS (2003)	$10	-	Western Sydney	Grass swale (seeded)
	$18	-		Rolled turf
Buffer Strips				
Gary Walsh (2001), pers comm., as cited in Taylor (2005).	$3.50	-	Melbourne	Turf buffer strip
	$7.50	-		Sedge/mulch buffer strips
URS (2003)	$10 – $15	-	Sydney	Grass buffer strip
	$20 – $50	-		Native grasses and shrubs

^The costs quoted in this section are from around 2002 to 2005 and have not been adjusted for inflation or potential cost changes which may have occurred since this chapter was published in 2007. Therefore, it should be considered as indicative only and users of the manual are encouraged to seek further specific industry advice on the current costs as appropriate.

Design considerations

The most important design consideration for a swale drain is the longitudinal slope. It is important to ensure flow velocities along a swale are kept sufficiently low to avoid scouring of vegetation and collected pollutants. Typically, the slope is considered to be most efficient between 1% and 4% to ensure that velocities do not scour the channel or compromise public safety, while at the same time limit ponding at low flows.

Where the longitudinal slope exceeds 4%, riffles along swales can help to distribute flows evenly across the swale as well as reduce velocities. The riffles maximise the retention time within the swale, further decreasing the velocities and better promoting particulate settling.

Vegetated swales can be used for water quality treatment wherever the local climate and soils permit the establishment and maintenance of a dense vegetative cover. The principal selection criteria for swales should firstly address the function of conveyance and secondly ensure that the system has features that will maximise treatment objectives and habitat and aesthetic values.

Pre-treatment for swales may include litter traps at point source inlets and buffer strips parallel to the top of the banks to pre-treat sheet flows entering the swale.

Figure 3. Bollards used to prevent vehicular access onto a swale for conveying road runoff in Mandurah. (Photograph: Grahame Heal, City of Mandurah.)

The selection of vegetation can impact the overall performance of the swale. Vegetation should be designed to cover the entire width of the swale, be capable of withstanding peak flows and be sufficiently dense to provide good filtration. For best performance, the vegetation height should be above the treatment flow water level. To ensure that swales are both functional and aesthetically pleasing, they should be incorporated into landscaping features. Using local species, vegetated swales can be low maintenance and be hardy enough to withstand long periods without water once established. Types of suitable vegetation that can be used in a swale include grasses, sedges and tussock grasses and other ground covers (e.g. herb form plants).

Swales are most effective when located within POS or within the centre medians or verges of roads. Swales should not be located within residential verges if other options are available due to maintenance and safety issues, as well as the need to provide driveway crossings. To protect the vegetation and thus the integrity of the swale, it is imperative that traffic movements along the swales be prevented. Traffic (including parking) can ruin the vegetation, compact the swale, cause rutting and harden the surface to provide preferential flow paths that do not allow infiltration. Traffic controls can be achieved by selecting swale vegetation along the edges that discourage vehicular movements or by providing physical barriers such as bollards (Figure 3) and non-mountable kerbing.

Another key consideration is the provision of road (median) or driveway crossings. Where possible, the location of the swale should minimise the need for crossovers. ‘At grade’ crossings follow the profile of the swale. Crossings when constructed ‘at grade’ reduce the maximum allowable swale batter slopes to approximately 1:9 (vertical to horizontal) to ensure that vehicles can traverse the crossing.

Most crossings are elevated with a culvert system to alleviate low flows. The disadvantage with elevated crossings is cost, particularly in dense urban developments. In addition, safety concerns with traffic movement under potential flood conditions due to blockages or when flows exceed the culvert capacity need to be addressed. For swales located on steep grades, crossings can be designed as a form of riffle to control flows.

Another consideration when locating a swale is to ensure that it will not be in the line of other services, such as sewers and underground electricity. These services will need regular maintenance and as such should not be within swales. Temporary bunding or sediment controls should be installed to protect the swale during road and housing/building construction.

Standing water in poorly designed vegetated swales can result in potential safety, odour and mosquito problems. There is also some potential for unstable conditions and erosion in extreme events that exceed the design event for the system. Therefore, other structural controls within a catchment should be designed to manage stormwater quantity, so that excessively large flows are not conveyed into the swale.

Design guidelines

Swales can be designed for greenfield applications or in retrofitting scenarios to replace a proportion of the traditional piped network.

Design of vegetated swales needs to consider three types of storm events as discussed in the Decision process for stormwater management for WA 2017:

- Small rainfall events for ecological protection including managing water quality and maintaining form and hydrology of sensitive receiving environments (e.g. runoff from 15mm rainfall & up to 1 EY event).
- Minor rainfall events for serviceability, amenity and road safety (e.g. as per service providers' requirements).
- Major rainfall events for protection from flooding and inundation (e.g. up to 1% AEP event).

Design flows for particular storm events can be estimated using a range of hydrologic methods with varying complexity. For small simplistic catchments or lot-scale catchments where flood routing is not critical, the Rational Method is suitable for peak flow estimation, while for large more complex catchments, use of hydrologic/hydraulic models may be more appropriate for design.

A description of buffer strips is contained in the Inlet structures subsection.

Swale geometry

- The swale's geometrical design is an iterative process that needs to take into consideration the site's constraints including topography, development layout and density, how flow reaches the swale and available reserve width. Design considerations are outlined below:
- The longitudinal slope of a swale is typically controlled by catchment topography. To maintain conveyance and prevent ponding during low flows, the longitudinal slope should not be less than 1%, unless additional treatments such as subsoil drains are present or swales are located in soils providing infiltration opportunities. For more information about prevention of ponding (and therefore reducing mosquito breeding risks), see the Design Considerations and Design Guidelines sections of the Infiltration Basins and Trenches BMP. Where slopes are steeper than 4%, riffles should be constructed at regular intervals to prevent scouring and reduce flow velocities.
- Swale dimensions and contributing catchment area should be selected to ensure 1 EY flow

velocities for the swale are maintained at less than 0.5 m/s. Swales located within road reserves can be subjected to velocities associated with major flood flows being conveyed along the road corridor. The resultant velocities within the swale should be checked to ensure that the maximum velocity does not exceed 1.8 m/s to prevent scour.

- Riffles are typically low level (e.g. 100 mm) porous rock weirs that are constructed across the base of a swale. A rule of thumb for locating riffles is to ensure that the maximum grade taken from the toe of the upstream riffle to the crest of the downstream riffle does not exceed 4% (Figure 4). Further information about riffle design is provided in the Living Streams BMP in this chapter.

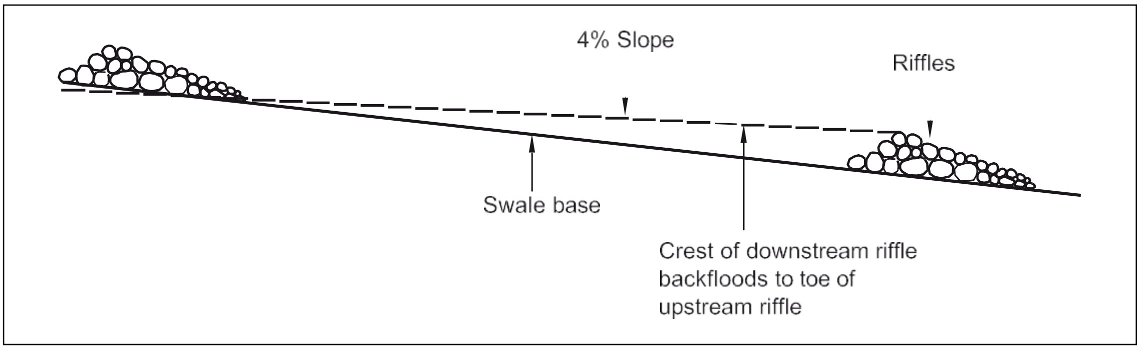

Figure 4. Location of riffles in a swale.

Side batters should be constructed at 1:6 where possible and should not be steeper than 1:3. The batter slope needs to be able to cater for the design flow, as well as providing a suitable grade for vegetation establishment, access for maintenance, crossovers for lot access and public safety. Typically, the side batter is limited by the available reserve width.

The required width of the swale is that which can adequately contain the design flow within the banks of the swale, given the above-design considerations.

Hydraulic capacity

The hydraulic capacity of a swale can be determined by use of hydraulic models or, for areas not subject to backwater effects, by application of Manning's equation for open channel flow:

$$Q = \frac{1}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}}$$

Where:

$Q = flow\ (m^3/s)$

$n = roughness\ coefficient$

$S = longitudinal\ slope\ (m/m)$

$A = Cross - sectional\ area\ of\ flow\ (m^2)$

$R = Hydraulic\ Radius\ (m), defined\ as\ A/P, where\ P\ is\ the\ wetted\ perimeter\ (m)$

Application of Manning's equation allows both the flow rate and depth to be determined for a range of geometric configuration and vegetation types. The discharge calculations from this equation are significantly influenced by the roughness coefficient, which varies with flow depth, channel dimensions and vegetation type. Typically, between 0.15 and 0.40 is considered reasonable for flow depths less than the vegetation height. The trade-off with planting taller, denser vegetation to increase water quality treatment is that greater setback areas for the swale are required. As flow depth extends beyond the full vegetation height, a sharp reduction in the roughness coefficient can be expected and a corresponding increase in velocity. Figure 5 shows the relationship between the roughness coefficient and the flow depth, with reference to a medium-length sod-forming grass tested in a swale with 5% bed slope. It can be reasonably expected that this relationship will remain consistent with other swale configurations, though there may be a marginal reduction in Manning's n for sheet flows. Manning's n values can also be estimated from tables (e.g. refer to Report No. 9 Stream Channel Analysis, Water and Rivers Commission 2000).

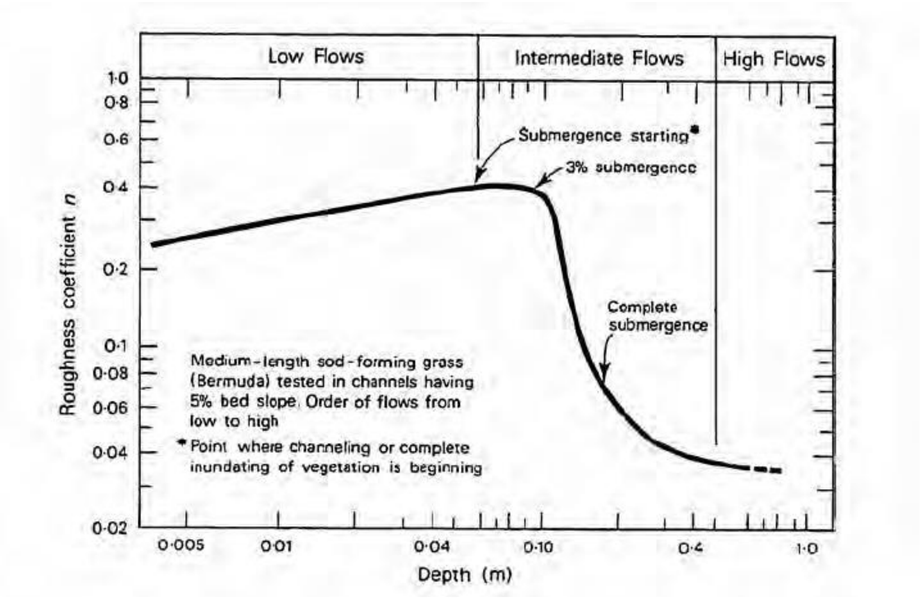

Figure 5. Impact of flow depth on hydraulic roughness. (Source: Engineers Australia 2006.)

Inlet structures (including buffer strips)

Inlets for swales can either be distributed (via buffer strips) or via point sources such as kerb breaks (Figure 1), pipes and bubble-up manholes.

For distributed flows such as buffer strips, it is essential to provide an area for coarse sediment to accumulate. Typically, the top of vegetation should be at least 40–50 mm below the flush kerb (Figure 6). This would require the top of ground surface (before turf is placed) to be between 80–100 mm below the flushed kerb.

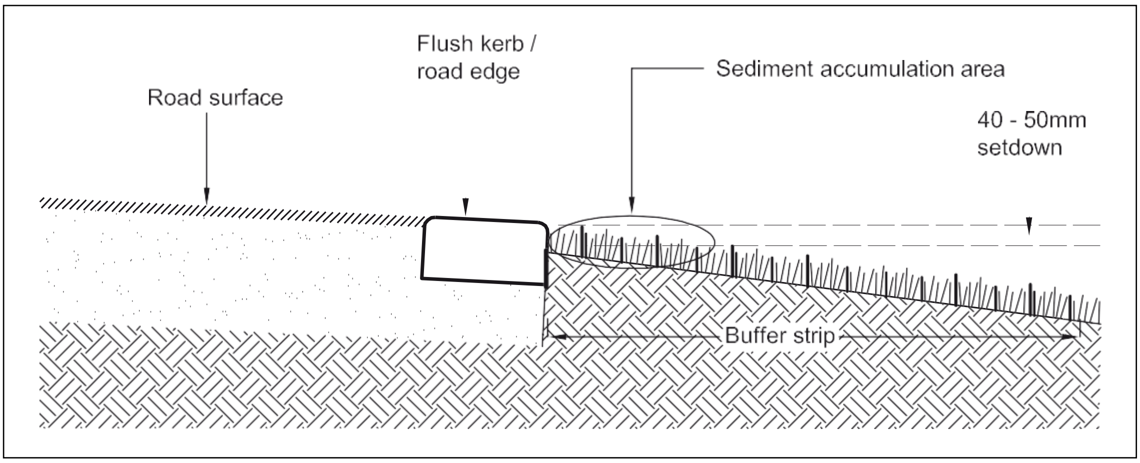

Figure 6. Edge setdown details for buffer strips.

Point source entry can either be from overland flow (e.g. kerb breaks) or from a pipe system. The main consideration for point source entrances into swales is the dissipation of energy at the inlet point to minimise erosion potential. This can usually be achieved with rock beaching and/or dense vegetation.

Bubble-up structures need to be made accessible for maintenance purposes so that any build-up of coarse sediment and debris can be monitored and removed if necessary (Figure 7). The use of bubble-up structures must ensure that residual runoff stored in the manhole can be dissipated, to reduce the risk of mosquito breeding. This can be achieved by making the base of the structure permeable, subject to the nature of the underlying soil permeability. If swales are installed within POS, it is preferable for them to be installed within a garden bed rather than in the middle of a grassed area to improve the recreational amenity and aesthetics of the swale.

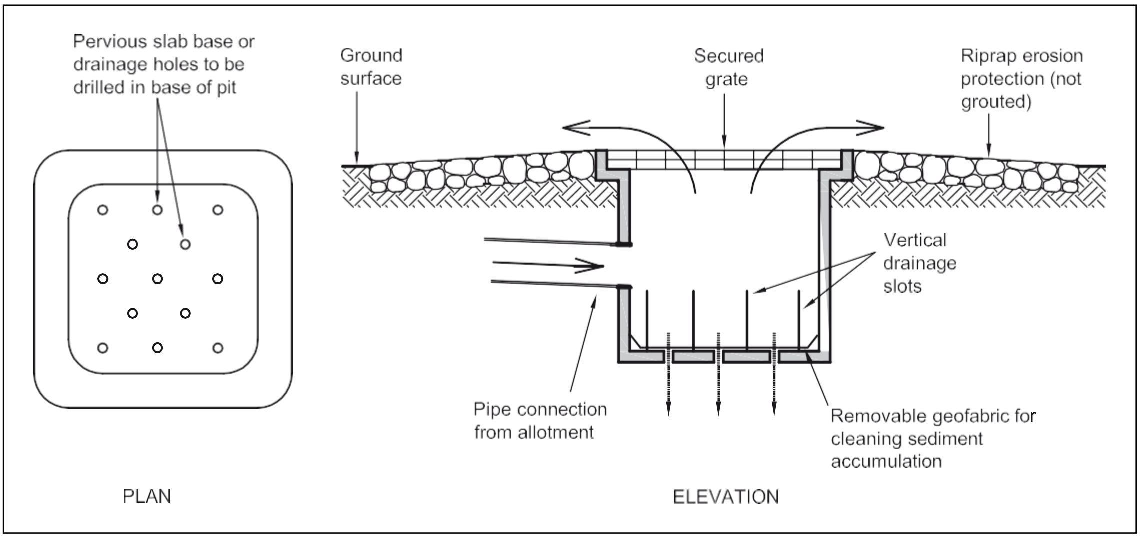

Figure 7. Example bubble-up structure for discharging to a swale.

Vegetation

Swales can use a variety of vegetation including turf, sedges and tufted grasses.

Vegetation is required to cover the whole width of a swale in order to have a water quality filtering function, rather than simply a conveyance and/or infiltration function. For a turf swale, a fine, close growing, water resistant grass should be selected to increase the surface area of the vegetation exposed to the runoff and thereby improve the effectiveness of the system. Turf swales (see Figure 3 for an example) are useful in residential areas but need to be mown and maintained regularly.

Swales vegetated with sedges and tufted grasses (see figures 1 and 2 for examples) have a higher hydraulic roughness and require a larger area and more frequent inlet pits to convey the flows compared to turf swales. The dense form and height of tuft grasses or sedges can provide an attractive landscape feature. Pollutant removal efficiency varies greatly depending on the specific plants involved. Selection should therefore emphasise pollution control, but must also ensure that vegetation will be able to thrive under local conditions. Sedges and tuft grasses should preferably be native and should not be weed species.

A description of common rushes, sedges, bulrushes and submergents of the south-west of WA is contained in Report No. 8 of the River Restoration Manual (Water and Rivers Commission 2000). The manual provides details of common species and those available commercially for rehabilitation projects, including details of appearance, location, soil type, water quality, water depth and propagation.

Maintenance

A monitoring and maintenance plan should be developed for the swale. The maintenance objectives for a vegetated swale system include retaining the hydraulic and pollutant removal efficiency of the channel, and maintaining a dense, healthy vegetation cover. A well-designed and maintained vegetated swale can have a long operating life.

Maintenance should include frequent inspection during the first few months to ensure vegetative cover is establishing well. If required, reseed or plant an alternative species. Once established, continue to inspect biannually for signs of erosion. Weed control and periodic mowing of grass swales (typically biannually), with grass never cut shorter than the design flow depth, are recommended. Cuttings should be removed from the channel and disposed in a local composting facility. Similarly, vegetated swales should be pruned and harvested in place of mowing. Information on maintenance of vegetation is provided in Report No. 4 of the River Restoration Manual (Water and Rivers Commission 1999) and BMP 2.2.7 of Chapter 7.

Before winter and after major storm events, debris and blockages should be cleared. Accumulated sediments should be removed to avoid the transportation of resuspended sediments during periods of high flow and to prevent a damming effect from sand bars. Repair of damaged areas within the channel should be undertaken as required. For example, if the channel develops ruts or holes, it should be repaired utilising a suitable soil that is properly tamped and seeded. The vegetation cover should be thick and reseeded as necessary. Swales should also be inspected regularly for ponding, as it can become a nuisance due to mosquitoes breeding in standing water if obstructions develop (e.g. debris accumulation, invasive vegetation) and/or slopes of swales are too flat and inadequately maintained, allowing water to pool for more than four days.

Appropriate traffic control solutions must also be maintained so that correct driving paths are taken and to prevent parking on swales.

Worked example

Caution: The following worked examples use Rational Method as per the Australian Rainfall and Runoff (ARR) Book VIII (Institution of Engineers, Australia 2001). However, as per the updated ARR Book 9 'Runoff in Urban Areas', the Rational Method is only suitable for lot-scale scale catchments or simplistic small catchments where flood routing is not critical. This method is not suitable for a 'precinct' scale estimation of peak flows as it has 'limited' runoff generation and surface routing capabilities. If runoff volume management infrastructure forms part of a solution, or if an understanding of potential impacts on downstream flooding are required, then a 'strong' hydrologic estimation method such as a runoff-routing model should be used (ARR 2019). For further information on the limitations of Rational Method, please refer to Book 9 'Runoff in Urban Areas' of ARR 2019

As part of a residential development, runoff from a street surface and footpath is to be collected and conveyed in a grassed swale system, located within the verge adjacent to parkland, to downstream treatments. An additional exercise in this worked example is to investigate the consequences on flow capacity of using a vegetated swale with vegetation height up to 300 mm.

The street and footpath will have a one-way crossfall with flush kerbs, to allow for distributed flows into the swale system across the side batter (buffer zone). The swale is to convey minor flood events, including all flows up to 0.2 EY. The width of the swale is fixed at 4.5 m. There will be a maximum catchment area the swale can accommodate, above which an underground pipe will be required to preserve the conveyance properties of the downstream swale. The maximum slope of the swale banks is 1:9 (11%) to allow for easy access for maintenance and safe access for pedestrians to the adjacent parkland.

The contributing catchment area includes a 7 m wide road pavement surface, a 1.5 m wide footpath and a 4.5 m wide swale easement (Figure 8). The area is 250 m long with the top 100 m having a 6% slope and the bottom 150 m having a 3% slope (Figure 9).

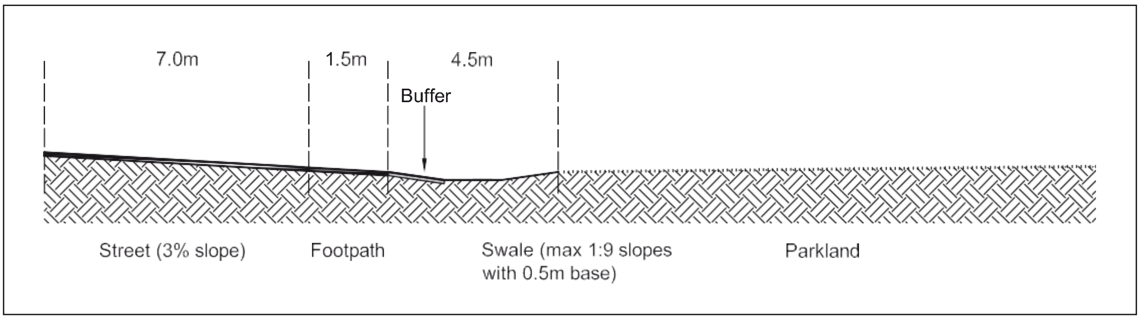

Figure 8. Cross section of proposed buffer/swale system.

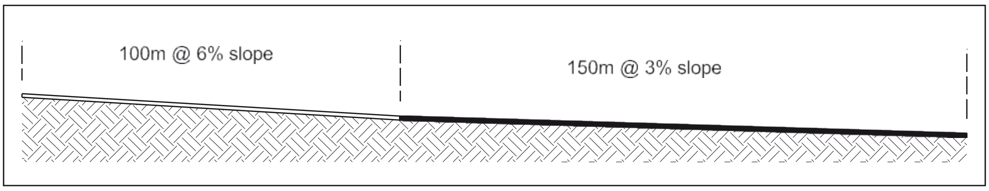

Figure 9. Long section of proposed buffer/swale system.

Design objectives

This worked example focuses on the design of conveyance properties for the buffer strip and vegetated swale. Analyses to be undertaken include the following:

- design the swale system, including riffles where required
- select vegetation such that the hydraulic capacity of the swale is sufficient
- determine the required capacity of the swale to convey five-year flows
- check velocities are maintained to acceptable levels
- design the overflow structure from the swale to an underground pipe (if required)
- configure the street kerb details so sheet flow is achieved through the buffer strip
- select suitable buffer strip vegetation

Site characteristics Catchment area:

$$Roads\ and\ concrete\ footpath: 250\ m * (7\ m + 1.5\ m) = 2\ 125\ m^2$$

$$Swale\ easement: 250\ m \times 4.5\ m = 1\ 125\ m^2$$

$$TOTAL = 3\ 250\ m^2\ (i.e, 0.325\ ha)$$

Overland flow slope:

$$Total\ main\ flowpath\ length = 250\ m$$

$$Upper\ section = 100\ m\ at\ 6\%\ slope$$

$$Lower\ section = 150\ m\ at\ 3\%\ slope$$

Soil type:

$$Clay$$

Fraction impervious:

$Roads/footpath = 1.00$

$Swale\ easement = 0.10$

Estimating design flows

The following example uses calculation methods from ARR (Institution of Engineers Australia 2001). Alternatively, this analysis could be performed using a hydrologic model.

The time of concentration (tc) is estimated assuming overland flow across the allotments and along the swale. From procedures in ARR (Institution of Engineers Australia 2001) Book VIII, tc is estimated to be 10 minutes.

Based on Intensity–Frequency–Duration calculations for Perth Airport, consistent with Institution of Engineers Australia (2001) Book II, rainfall shown in Table 3 is adopted for design purposes.

Table 3. Design rainfalls for calculated time of concentration

t_c	6 month ARI (2 EY)	5 year ARI (0.2 EY)
10 min	34 mm/hr	66 mm/hr

Based on Institution of Engineers Australia (2001) Book VIII, the overall runoff coefficient for the catchment is calculated as follows:

$$C_{10}^{1} = 0.1 + 0.0133({}^{10}I_1 - 25)$$

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

$$f_{av} = \frac{A_1 f_1 + A_2 f_2 + \cdots}{A_{total}}$$

Where:

$C_{10}^{1} = pervious\ runoff\ coefficient$

$C_{10} = 10\ year\ ARI\ runoff\ coefficient$

$f_{av} = fraction\ impervious$

${}^{10}I_1 = 10\ year\ ARI\ 1\ hour\ rainfall\ intensity$

$f_{av} = (2125 * 1 + 1125 * 0.1)/3250 = 0.69$

${}^{10}I_1 = 29.0\ mm/hr\ (Perth\ Airport)$

$C_{10}^{1} = 0.15$

$C_{10} = 0.67$

Runoff coefficients for various ARI are then calculated as $C_y = F_y C_{10}$, with the frequency factor F_y defined in Table 1.6 of Institution of Engineers Australia (2001) Book VIII.

$C_1 = 0.8 * 0.67 = 0.53$

$C_5 = 0.95 * 0.67 = 0.63$

As the minimum ARI considered for runoff coefficients is one year in Institution of Engineers Australia (2001) Book VIII, this is conservatively adopted for calculation of 2 EY (six month ARI) peak design flows.

Using the Rational Method, peak design flows for the catchment are calculated as:

$$Q = 0.00278CIA$$

Where :

$$I = rainfall\ intensity\ (mm/hr)$$

$$C = runoff\ coefficient$$

$$A = catchment\ area\ (ha)$$

$$Q_{6mth} = 0.00278 * 0.53 * 34 * 0.325 = 0.016\, m^3 s^{-1}$$

$$Q_{5yr} = 0.00278 * 0.63 * 66 * 0.325 = 0.038\, m^3 s^{-1}$$

Swale design

To facilitate access, the cross section shown in Figure 10 is proposed.

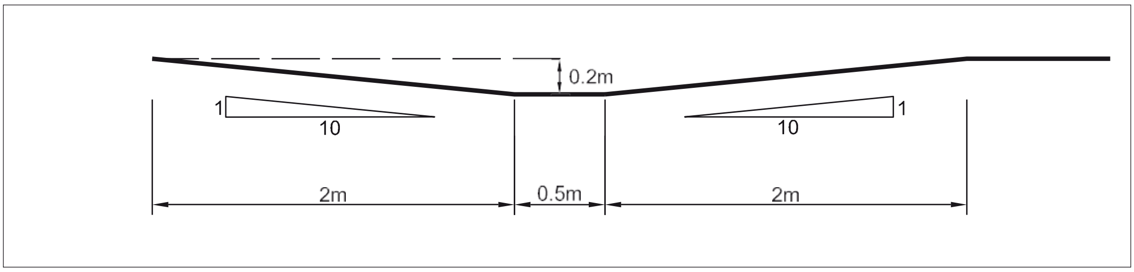

Figure 10. Proposed swale cross-section.

The capacity of the swale is then estimated at the most downstream point. This is considered to be the critical point in the swale as it has the largest catchment and has the mildest slope (it is assumed that the dimension of the swale will be the same for both the steep and gentle sloped areas for aesthetic reasons). Flow velocities will also need to be checked at the downstream end of the steep section of swale.

The worked example considers the swale capacity using a grass surface with a vegetation height of 50 mm. A range of roughness coefficients (Manning's n) are selected for different flow depths appropriate for grass (Table 4). The height for a flow at the channel capacity will be above the vegetation and therefore Manning's n is quite low and a figure of 0.04 is adopted (refer to Figure 5). Manning's n is varied according to the flow depth with reference to the vegetation height (as shown in Figure 5) and the corresponding discharge can be calculated simply in a spreadsheet application using the following procedure:

Flow rate at channel capacity:

$$Adopted\ slope = 3\%\ (minimum\ longitudinal\ slope)$$

$$Manning's\ n = 0.04\ (at\ 0.2\ m\ depth)$$

$$Side\ slopes\ 1{:}\,10$$

$$Area\ A = 0.5\, m^2$$

$$Wetted\ perimeter\ P = 4.52\, m$$

$$Hydraulic\ radius\ R = A/P = 0.111$$

Manning's equation:

$$Q = \frac{1}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}}$$

$$Q = 0.5\, m^3 s^{-1}$$

Table 4. Manning's *n* and flow capacity variation with flow depth – Turf

Flow Depth (m)	Manning's *n*	Flow Rate (m^3/s)
0.05	0.30	0.003
0.10	0.30	0.01
0.15	0.10	0.10
0.20	0.04	0.50

The capacity flow for the swale (Q = 0.50 m^3/s) is greater than the required peak flow rate (Q = 0.038 m^3/s). Therefore, the nominated swale has sufficient capacity without any requirement for an additional piped drainage system. From Table 4, it can be seen that both the six month and 0.2 EY (five year ARI) flow depths are above the vegetation height.

For the purposes of this worked example, the capacity of the swale 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 5 presents the adopted Manning's *n* values and the corresponding flow capacity of the swale for different flow depths.

Table 5 demonstrates that the swale with dimensions shown in Figure 10 is capable of conveying a 0.2 EY (5 year ARI) flow.

This worked example continues using grass for the remainder of its analysis.

Table 5. Manning's *n* and flow capacity variation with flow depth – Sedges

Flow Depth (m)	Manning's *n*	Flow Rate (m^3/s)
0.05	0.35	0.003
0.10	0.32	0.01
0.15	0.30	0.03
0.20	0.30	0.07

Inlet details

Flows reach the swale directly from the road and footpath surface.

Direct runoff from the pavement enters the swale via a buffer (the grass edge of the swale). The pavement surface is set 50 mm higher than the start of the swale and has a taper that will allow sediments to accumulate in the first section of the buffer off the pavement surface. Traffic control is achieved by using traffic bollards between the road and the footpath.

Velocity checks

Two velocity checks are performed to ensure vegetation is protected from erosion at high flow rates. Velocity is checked to be kept below 0.5 m/s for the 0.2 EY (five year ARI) flow event. Velocities are estimated using Manning's equation.

Firstly, velocities are checked at the most downstream location (slope = 3%). From Table 4, $d_{5\text{-year}} = 0.12$ m, i.e., the flow depth for the 0.2 EY (5 year ARI) flow event (Q = 0.038 m^3/s), and the corresponding Manning's *n* = 0.24.

Therefore, to calculate the velocity:

$$A = 0.204\, m^2$$

$$P = 2.91\, m$$

$$R = A/P = 0.070\, m$$

$$V_{5-year} = \frac{(0.07^{\frac{2}{3}} * 0.03^{\frac{1}{2}})}{0.24}$$

$$= 0.12\, m/s < 0.5\, m/s, therefore\ OK$$

Secondly, velocities are checked at the bottom of the steeper section (i.e. slope = 6% with reduced catchment area). $Q_5 = 0.015\ m^3s^{-1}$ for this section.

$$d_{5-year} = 0.10\, m$$

$$n = 0.30$$

$$A = 0.15\, m^2$$

$$P = 2.51\, m$$

$$R = 0.060\, m$$

$$V_{5-year} = 0.12\, m/s < 0.5\, m/s, therefore\ OK$$

For larger storm events, when the swale is flowing at full capacity, the maximum velocity will be 1.0 m/s. Some scour may occur that would require repair following these infrequent large flow events.

Vegetation specification

To complement the landscape design of the area, a turf species is to be used in the swale. For this application, a turf with a height of 50 mm has been assumed. Selection of a suitable species will be determined by the landscape architect, consistent with application requirements and design assumptions.

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4.2 Bioretention systems

Figure 1. Bioretention swale, soon after construction and planting in Dawesville. (Photograph: Grahame Heal, City of Mandurah, 2006.)

Figure 2. Bioretention area constructed in the Treendale development, Australind. (Photograph: Wayne Edgeloe, Thompson McRobert Edgeloe (TME) Consultants, 2006.)

Background

Bioretention systems consist of an excavated basin or trench that is filled with porous media and planted with vegetation. These systems provide water quality treatment by removing fine sediment, trace metals, nutrients, bacteria and organics (Davis et al. 2001). Bioretention systems are structural stormwater controls that capture and either retain or temporarily detain stormwater runoff before the water is released to the environment. These systems can reduce the volume of runoff from a drainage area, reducing the required size and cost of downstream stormwater management facilities, by promoting at-source treatment and infiltration. Bioretention swales operate by filtering stormwater runoff through the surface vegetation of a swale, followed by the stormwater percolating into filter media, where filtration, extended detention treatment, denitrification and some biological uptake occurs. Bioretention basins operate in a similar way; however, flows in excess of the design flow bypass the basin to prevent scour, rather than flowing over the surface as occurs in a swale.

Bioretention systems have numerous design applications. These include use as offline facilities adjacent to parking lots, along highway and road drainage swales, within larger landscaped pervious areas, and as rain gardens and landscaped islands in impervious or high-density environments. Layout of bioretention systems can be very flexible, including linear systems (Figure 1), basins (Figure 2) and planter boxes. The selection of plant species can provide for a wide variety of landscape designs. When properly designed and maintained, these systems are aesthetically pleasing due to the incorporation of plants.

A benefit of bioretention systems over some other structural controls is that they can be applied under a range of different climatic and geological conditions, as the design includes the replacement of the existing soil with an engineered filtration media.

Bioretention systems can be classified as either pervious or impervious. Pervious bioretention systems refer to systems that promote direct infiltration into highly permeable surrounding soils post-treatment. Impervious bioretention systems describe systems in low permeability soils where treated surface runoff cannot be effectively infiltrated and is therefore conveyed out of the system via a subsoil or base drain.

While formal use and recognition of bioretention systems as a BMP is relatively new in WA, various techniques and combinations of using infiltration and subsoil drainage systems (with the primary aim of limiting seasonal groundwater rise) have resulted in informal use of bioretention as a form of stormwater treatment over many years.

Caution: For further details and recent information on biofiltration systems, please refer to Adoption Guidelines for Stormwater Biofiltration Systems (CRC WSC 2015). The 'adoption guideline' provides information on preparing business case, planning, design, implementation, and maintenance of biofiltration systems in Australia. The information provided in section 4.2 of this chapter should be read in conjunction with the above mentioned 'adoption guideline'. For any potential conflicting information, the 'adoption guideline' supersedes the information provided in section 4.2.

Performance efficiency

The treatment performance of bioretention systems or biofilters can vary with characteristics of the design, site conditions, catchment, individual storm events, season and climatic variation. Optimal design will depend upon the objectives for the system, including the target pollutants, and contrasting required for the removal of different contaminants. As a result, no single design can be expected to achieve optimal removal of all stormwater pollutants. Refer to 'Table 3 – Pollutant removal capacity of biofilters, key design parameters and expected performance from systems that are optimally designed, constructed and maintained' of the *Adoption Guidelines for Stormwater Biofiltration Systems* (CRC WSC 2015).

Little data exists in WA regarding the performance of bioretention systems locally, particularly in areas with high water tables, where the system performance will vary seasonally with groundwater levels.

The following review of bioretention performance efficiency is provided as an indicative guide only, based on eastern states research with different hydrologic conditions to WA. Bioretention systems are generally considered highly effective in removing TSS in typical urban post-development runoff. When sized, designed, constructed and maintained in accordance with the recommended specifications, bioretention systems can expect to have 80% removal efficiency for TSS.

Typical pollutant removal rates for bioretention systems are shown in Table 1 as conservative average pollutant reduction percentages for design purposes derived from efficiencies detailed in Davis et al. (2001), and local Australian sampling data and research by the Cooperative Research Centre for Catchment Hydrology (eWater) based on eastern states conditions using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC) (Cooperative Research Centre for Catchment Hydrology 2003).

Table 1. Effectiveness of bioretention systems

Pollutant	Effectiveness	Mean % Removal
Litter	-	-
Coarse sediment	High	90%
Total suspended solids	High	80%
Total nitrogen	Medium	50%
Total phosphorus	Medium	60%
Heavy metals	High	80%

Figures 3 to 5 provide example pollutant removal performance efficiencies (TSS, TP, TN) for bioretention systems (either swales or basins) with varying depths of ponding (denoted extended detention in figures 3 to 5).

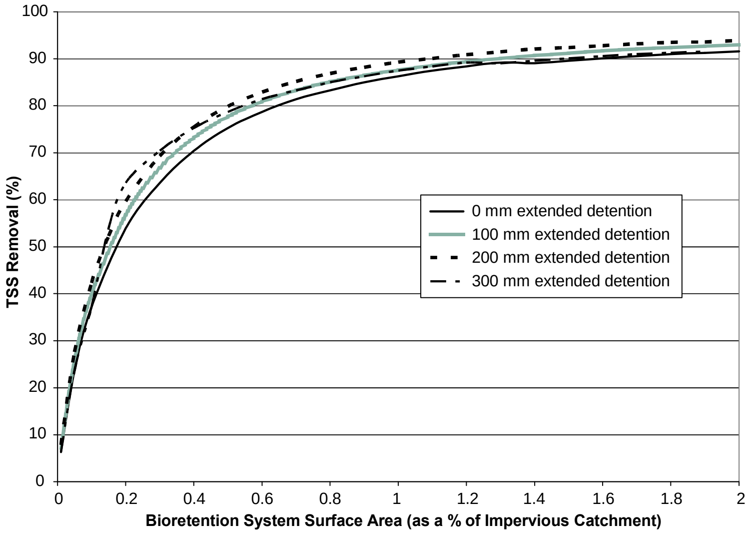

Figure 3. Bioretention system TSS removal performance.

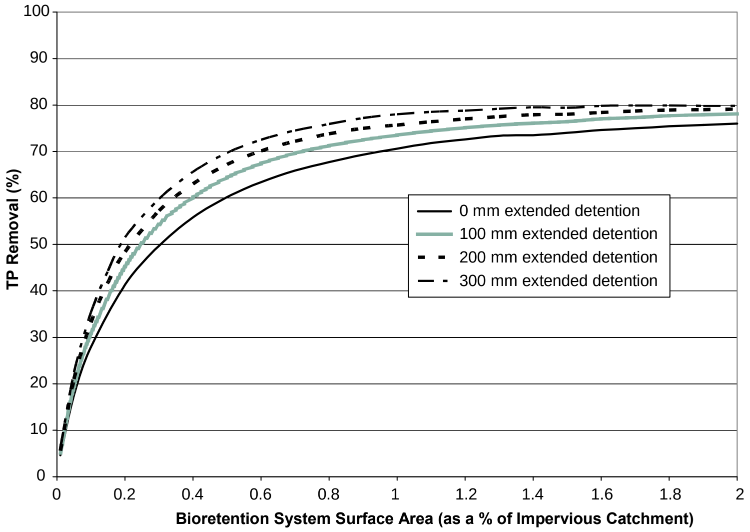

Figure 4. Bioretention system TP removal performance.

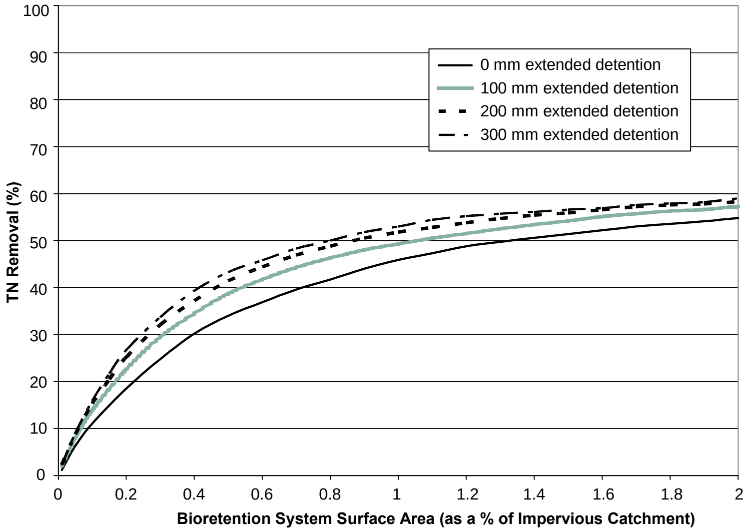

Figure 5. Bioretention system TN removal performance.

The above curves were derived using MUSIC modelling and assume a bioretention system receiving direct runoff without any pre-treatment. The following parameters were used to derive the curves:

- all standard MUSIC defaults were used
- impervious area assumed to be 100%
- filter area = surface area
- seepage = 0 mm/hr
- saturated hydraulic conductivity of 180 mm/hr (corresponding to a sandy loam)
- filtration media depth of 600 mm
- filter media particle size (d50) of 0.45 mm.

However, MUSIC has not been fully calibrated in WA for local hydrogeological conditions and BMP performance. Bioretention performance in WA is often likely to vary from the efficiencies of bioretention systems in the eastern states shown in Table 1 and figures 3 to 5, particularly at sites with sandy soils and shallow groundwater. Additionally, infiltration is likely to be a dominant process at sandy sites. Annual pollutant load removal efficiencies for bioretention systems on sandy soils would usually be expected to be higher than shown in Table 1 and figures 3 to 5 due to the increased infiltration rate reducing surface water discharge.

Since the publication of this *Stormwater Manual* chapter in 2007, the Department of Water and Environmental Regulation has produced the Urban Nutrient Decision Outcomes (UNDO) tool based off the MUSIC tool. It is a simple conceptual decision support tool with a flexible framework that evaluates nutrient reduction decisions for urban developments on the Swan Coastal Plain in south-west WA. It is specifically designed for ease of-use by urban development proponents and for assessment by local and State Government authorities.

The UNDO tool was calibrated to WA conditions using local soil geology and typical treatment configurations. The calibrated treatment effectiveness are presented below in Figure 1 a) and b). The typical geological and runoff conditions below are taken directly from the UNDO Technical Manual.

Sandy soils – Impervious runoff only: these are associated with structural treatments that treat only impervious runoff (or stormwater from road runoff that is routed through a pipe). They commonly include biofilters (although it is recommended that biofilters also treat subsoil drainage where present), swales and shallow ephemeral detention areas, but can also include constructed wetlands. When using this treatment type with embedded treatments, it is important that the treated area that is entered includes the entire catchment of the treatment, not just the impervious area).

Sandy soils – Impervious and subsoil drainage: this solution is only available for subregions that use subsoil drains. Treatments commonly include biofilters that have subsoil drainage directed to them, swales that drain subsoil drains, and lined wetlands (that do not interact with the groundwater). This includes end of pipe treatments from stormwater drainage systems that use a pit-and-pipe system in the road network that connects to a subsoil drainage system to control groundwater levels on urban lots.

Sandy soils – Impervious and all groundwater flow: These treatments will treat all nutrients mobilised by a catchment area – which includes the deeper groundwater flow that is not intersected by subsoil drains. This is limited to living streams and constructed wetlands, although it is recommended that constructed wetlands are separated from groundwater (DoW, 2008). Swales, detention/infiltration basins and biofilters do not have this option available, as it is necessary to design these treatments so they do not intersect the regional groundwater table.

Heavy soils – surface drainage: This option is for soils outside of the Swan Coastal Plain, which incorporate a piped or surface water drainage stormwater system. The soils have limited infiltration capacity, and subsoil drains are not used to control groundwater levels. This is similar to the hydrological conceptualisation used in MUSIC (eWater, 2009), and the amount of treatment will be identical to that in a standard MUSIC model (with parameters taken from the UNDO tool technical guide).

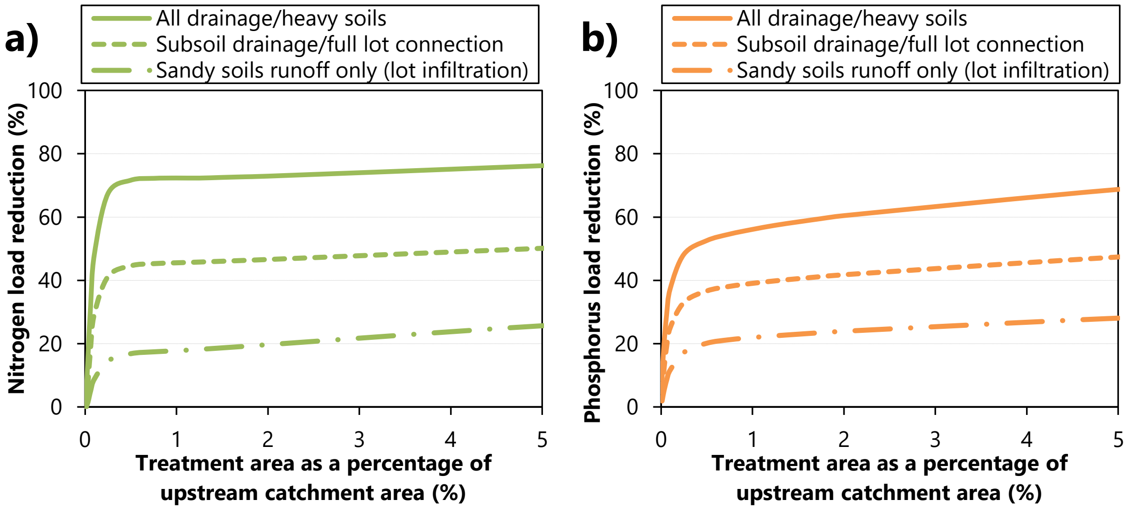

Figure 6. Nitrogen and phosphorus load removal effectiveness.

More information about UNDO can be found on the Department of Water and Environmental Regulation website via www.water.wa.gov.au/planning-for-the-future/water-and-land-use-planning/undo-tool.

Cost^

Bioretention systems are relatively expensive to implement compared to some other BMPs. However, the land take required is relatively small in comparison to some BMPs, such as constructed wetlands. A summary of bioretention system costs presented in Taylor (2005) are shown in Table 2.

In any bioretention system, the cost of plants can vary substantially and can account for a significant portion of the expenditure. Costs are likely to be higher than typical landscape treatments due to higher planting densities, additional soil excavation, backfill material, use of subsoil drains, etc.

The operation and maintenance costs for a bioretention system will be comparable to those of typical landscaping for a site. Costs beyond the normal landscaping fees will include the cost for testing the soils and may include costs for a sand bed and planting soil. Taylor (2005) estimated typical annual maintenance costs as 4.3% of the total acquisition cost.

An important consideration when evaluating the costs of bioretention is that it often replaces an area that would otherwise be landscaped. Therefore, the true cost of the bioretention system may be less than has been reported.

Table 2. Cost estimates for bioretention systems

Publication/ Data source	Construction ($)	Maintenance ($/m^2/yr)	Location	Description
Basins				
Leinster (2004)	\$125–\$150/m^2 \$225–\$275/m^2	-	South-east Queensland	> 100 m^2 area < 100 m^2 area
Swales				
Leinster (2004)	\$100–\$120/m	-	Southe-east Queensland	3-4 m top swale width
Fletcher et al. (2003)	\$135/m	\$2.50 \$1.50	South-east Melbourne	Grassed system Vegetated system (Natives)
Lane (2004)	\$350/m	-	NSW	-
URS (2003)	\$410/m	-	Western Sydney	3 m wide

Leinster, S. 2004, Ecological Engineering, pers comm., as cited in Taylor 2005.

^The costs quoted in this section are from around 2000 to 2005 and have not been adjusted for inflation or potential cost changes which may have occurred since this chapter was published in 2007. Therefore, it should be considered as indicative only and users of the manual are encouraged to seek further specific industry advice on the current costs as appropriate.

Design considerations

Considerations for selecting a bioretention system are the catchment area to be treated, the slope at the location of the system and of the catchment that drains to it, soil and subsurface conditions, and the depth of the annual maximum groundwater level.

Bioretention systems should ideally be used at or close to source to treat small catchments. When used to treat larger catchment areas, they tend to clog. In addition, it can be difficult to convey flow from a large catchment to the system. When designing for catchments with high sediment loads, pre-treatment devices may be required to capture sediment prior to flows entering the bioretention system.

Bioretention systems can be used in a greenfields development or retrofitting scenario. The advantage of bioretention systems over some other BMPs in a retrofitting scenario is that the relatively small land take and flexibility in shape of the systems enable them to be incorporated into existing road verges, median strips, parkland or landscaped areas. They are generally best applied to areas of flat terrain (< 2%) to allow uniform flow distribution so that water infiltrates the filter medium evenly (Engineers Australia 2006). These systems are therefore most typically applied to parking lots or residential landscaped areas, which generally have gentle slopes.

Bioretention systems can be applied in almost any soils, since they are designed with runoff percolating through a constructed bed of soil and then returning to the stormwater system. However, it is also possible to design a bioretention system to function like an infiltration system, where runoff percolates into the native soil below the system. This infiltration option is considered likely to have widespread application in WA; however, it should only be applied when soils and other site characteristics (such as existing groundwater quality and levels) are appropriately considered. In areas where significant infiltration is not intended and the hydraulic conductivity of the local soil is high (similar to the filter media), user of a liner should be considered.

A decision on permeable or impermeable bioretention systems will depend on factors such as potential interaction with groundwater, salinity, and the proximity and sensitivity to water of nearby infrastructure.

Designers need to consider conditions at the site and must incorporate design features to improve the longevity and performance of the system, while minimising the maintenance burden. Plants that are appropriate for the site, climatic and watering conditions should be selected. The appropriate selection of plants will aid in the effectiveness and maintenance of the bioretention system.

Traffic management measures should be put in place to protect the vegetation and prevent compaction of the bioretention system. If the system is being installed in a developing catchment, then measures such as temporarily covering the inlets to the system with filter cloth are recommended to prevent materials washing into and clogging the system. See section 2.1.1 'Land development and construction sites' of Chapter 7 for information about site management practices.

Design guidelines

Hydraulic calculations contained within the following design guidelines consider free discharge hydraulic conditions only, and do not consider any backwater effects of downstream hydraulics.

Use of the design guidelines for assessing hydraulic performance should therefore be applied with caution, and the use of hydraulic models is recommended where design of the proposed system and its hydraulic performance are likely to be impacted by a backwater effect of downstream hydraulic conditions.

Soil media specification

Between two to three types of soil media are required for the bioretention component of the system (Figure 6). It is important to check that the selected media meets the prescribed hydraulic conductivity and geotechnical requirements, and is free of rubbish and any other deleterious material.

A filter media layer provides the majority of the function through fine filtration, as well as supporting vegetation growth, keeping the filter media porous and providing some uptake of nutrients and other contaminants in stormwater. In order to support vegetation growth, the filter media layer needs to be typically between 300–1000 mm in depth. In construction, the material should be placed and lightly compacted to prevent subsidence or uneven drainage.

A drainage layer is used to convey treated flows into the subsoil pipes (if present). This layer is generally constructed using coarse sand or fine gravel (2 mm to 5 mm particle size). The layer should surround the subsoil pipe and is typically 150 mm thick.

If fine gravel is used, a transition layer of typically 100-150 mm thick and/or a suitable geotextile fabric should be included between the filter media and the drainage layer to prevent the filtration media from washing into the drainage layer and the subsoil pipes. The material size differential should be an order of magnitude between layers to avoid fine material being washed through the voids of a lower layer. The addition of a transition layer increases the overall depth of the bioretention system. This may be an important consideration for some sites and hence pipes with smaller perforations may be preferable.

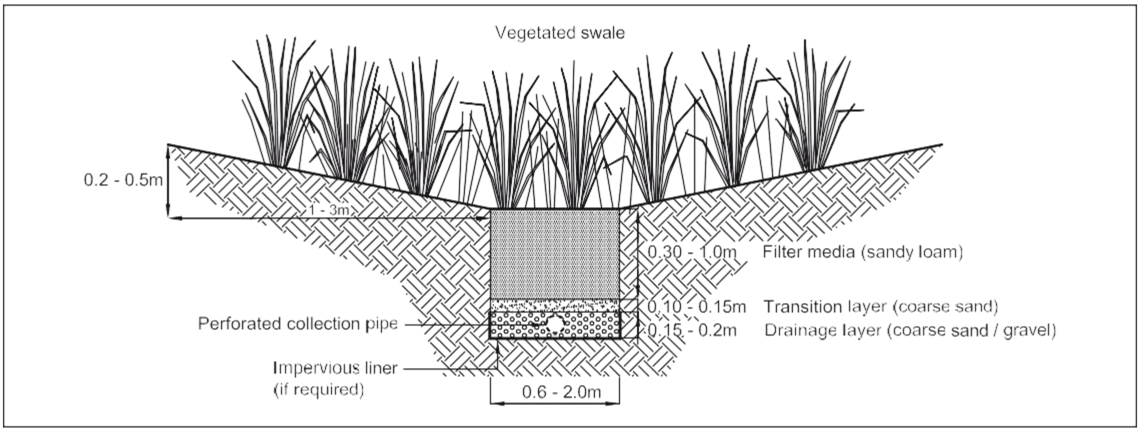

Figure 7. Typical liner arrangement for an impervious bioretention system.

The material for the filter media should be selected to suit infiltration and vegetation requirements. A lower infiltration rate (and higher detention time) may be appropriate where greater ponding above the filtration media is acceptable and can increase the volume of stormwater treated. Table 3, reproduced from Engineers Australia (2006), details typical saturated hydraulic conductivities for various soil types. Typically, filter media consists of a sandy loam with a saturated hydraulic conductivity between 50 and 300 mm/hr (Engineers Australia 2006). The use of a mulch layer or organic material mixed in the filter layer allows for sorption of nutrients (e.g. phosphorus and nitrate).

Table 3. Hydraulic conductivity for a range of media particle sizes (d50)

Soil type	**Particle size (mm)**	**Hydraulic conductivity (mm/hr)**	**Hydraulic conductivity (m/s)**
Gravel	2.0	36000	1×10^{-2}
Coarse Sand	1.0	3600	1×10^{-3}
Sand	0.7	360	1×10^{-4}
Sandy Loam	0.45	180	5×10^{-5}
Sandy Clay	0.01	36	1×10^{-5}

Reproduced from Engineers Australia (2006)

Materials such as by-products of industrial processes (e.g. red mud or blast furnace slag) and naturally occurring minerals (e.g. laterite or zeolite) are common media used to effectively adsorb and precipitate phosphorus and other inorganics (Douglas et al. 2004). Treatment media that are commonly used for the nitrogen removal are sawdust (Fahrner 2002; Schipper and Vojvodic-Vukovic 2001, 2000) and woodchips (Jaynes et al. 2004) mixed with sand. While sawdust has been reported to have significantly higher rates of denitrification than woodchips, there are some concerns about durability of finer sawdust media (Horn et al. 2006). Therefore, Horn and others recommended that woodchips should be considered as an alternative media in the long-term due to its durability compared to sawdust. Field trials conducted by Robertson et al. (2000) (seven years), Schipper and Vojvodic-Vukovic (2001) (five years) and Fahrner (2002) (16 months) recommended 20-30% volume sawdust/sand to be effective in removing nitrate from groundwater. When using laterite as a filtering media to remove phosphorus however, the recommended amount is 50% volume crushed laterite/sand mix (Douglas et al. 2004).

Use of impervious liners

For a bioretention system to treat stormwater runoff effectively, water lost to soils adjacent to the bioretention system should be limited.

In the predominantly sandy surface layer of the coastal plains of WA, the saturated hydraulic conductivity is typically more than one order of magnitude higher than that of the filtration media. As soil water flows tend to take the path of least resistance, filtration media can therefore be bypassed. To prevent this water loss from occurring, an impervious liner extending along the sides of the bioretention system (but not the base) is recommended (Figure 8).

Alternatively, where sites have a layer of adjacent lower permeable soil, a drainage layer is to be located at the base to create a head differential to drive flows through the filter layer. If the system is to be an impervious bioretention system, a liner will then typically be installed across the base of the bioretention system (Figure 7).

Impervious liners can have the added benefit of preventing export of water from the bioretention system into sensitive surroundings (e.g. sodic soils, shallow groundwater, or proximity to significant structures). Generally the greatest risk of this occurring is through the base of the bioretention trench. It is therefore recommended that if lining is required, particular attention should be given to the base and sides of the drainage layer. In addition, the base of the bioretention trench can be V-shaped to promote a more defined flow path of treated water towards the subsoil pipes.

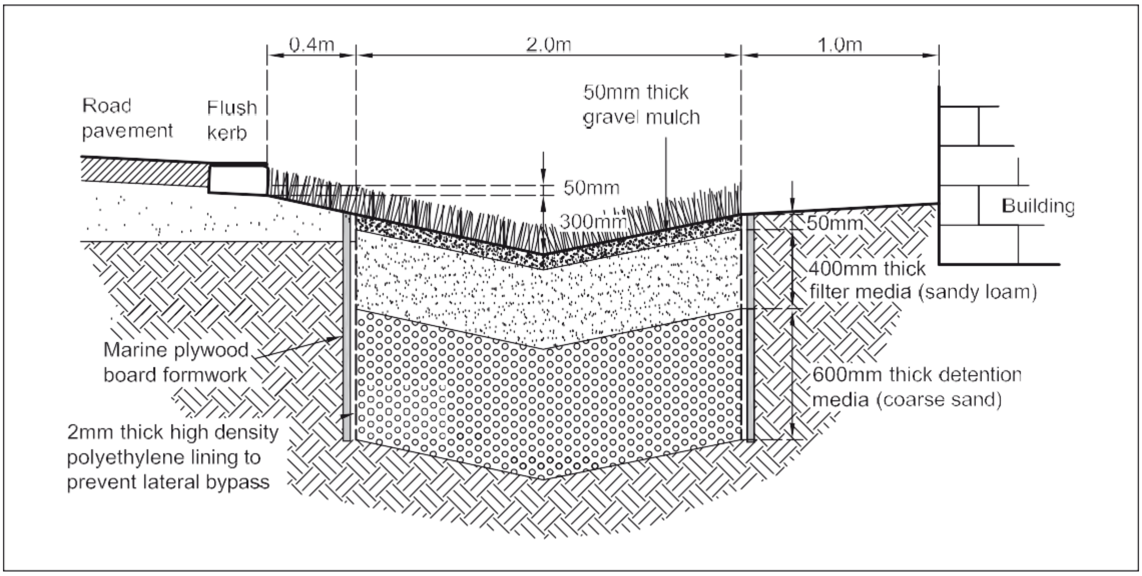

Figure 8. Typical liner arrangement for a pervious bioretention system.

Bioretention swale design

Design flow estimation, inlet structure details and swale design for a bioretention swale follow the same procedures as the design of a vegetated swale outlined in the Design Guidelines section of the Swales and Buffer Strips BMP, with some minor modifications. Additional guidelines for bioretention swales are described in this section.

The swale's geometrical design is an iterative process that needs to take into consideration site constraints including topography, development layout and density, how flow reaches the swale and the available reserve width.

The maximum infiltration rate through the filtration media must be considered to allow for the subsoil drain (if required) to be sized. The capacity of the subsoil drain, when installed, must exceed the maximum infiltration rate to ensure free draining conditions for the filter media.

The maximum infiltration rate (Q_{max} in m^3/s) through the filtration media can be estimated using Darcy's equation:

$$Q_{Max} = \frac{kLW_{Base}(H_{max} + d)}{d}$$

Where:

$k = \ hydraulic\ conductivity\ of\ the\ soil\ filter\ (ms^{-1})$

$Wbase\ =\ width\ of\ the\ infiltration\ area\ (m)$

$L = \ length\ of\ the\ bioretention\ zone\ (m)$

$h_{max}\ =\ depth\ of\ ponding\ above\ the\ soil\ filter\ (m)$

$d = \ depth\ of\ filter\ media\ (m)$

The suitability for using the above formula for design purposes will need to be assessed for each individual site, considering the influence of both the annual maximum groundwater level and infiltration capacity of surrounding natural soils on bioretention system infiltration, particularly for pervious bioretention systems.

Infiltration modelling software may be required to assess the maximum infiltration rate for design purposes.

Sizing of subsoil pipes

Subsoil pipes are perforated/slotted pipes located at the base of impervious bioretention systems to collect treated water for conveyance downstream.

These collection pipes are sized to allow free draining of the filtration layer and prevent 'choking' of the system. Typically, subsoil pipes should be limited to approximately 150 mm in diameter so that the thickness of the drainage layer does not become excessive. Where the maximum infiltration rate is greater than the capacity of the 150 mm diameter pipe, consideration should be given to using multiple pipes.

To ensure the subsoil pipes are of adequate size:

perforations must be adequate to pass the maximum infiltration rate into the pipe;

the pipe itself must have adequate hydraulic capacity to convey the required design flow; and

the material in the drainage layer must not be washed into the perforated pipes.

These requirements can be assessed using the equations outlined in this section, or alternatively manufacturers' design charts or hydraulic models can be adopted to select appropriately sized pipes.

To estimate the capacity of flows through the perforations, orifice flow conditions are assumed and a sharp edged orifice equation can be used. The number and size of perforations need to be determined (typically from manufacturers' specifications) and used to estimate the total flow rate into the pipe. Secondly, it is conservative but reasonable to use a blockage factor to account for partial blockage of the perforations by the drainage layer media. A factor of two is recommended.

Flow per perforation is therefore defined as:

$$Q_{perf} = \frac{CA\sqrt{2gh}}{\psi}$$

where:

$Q_{perf} = \ flow\ per\ perforation\ (m^3s^{-1})$

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

$h = \ maximum\ head\ of\ water\ above\ the\ pipe\ (m)\ (filtration\ media\ and\ ponding\ depth)$

$C = orifice\ coefficient$

$\Psi = blockage\ factor\ (\Psi\ =\ 2\ is\ recommended)$

$g = gravity\ constant\ (9.81\ m^2 s^{-1})$

The Colebrook-White equation can then be applied to estimate the flow rate in the perforated pipe. Note the capacity of this pipe needs to exceed the maximum infiltration rate.

$$Q_{pipe} = [-2\sqrt{2gDS_1}.log_{10}\left(\frac{k}{3.7D} + \frac{2.5\upsilon}{D\sqrt{2gDS_1}}\right)].A$$

where:

$D\ =\ pipe\ diameter\ (m)$

$A\ =\ area\ of\ the\ pipe\ (m^2)$

$k\ =\ wall\ roughness\ (m)$

$\nu\ =\ kinematic\ viscosity\ (m^2 s^{-1})$

$S_1\ =\ pipe\ slope\ (m/m)$

$g\ =\ gravity\ constant\ (9.81\ m^2 s^{-1})$

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

Grated overflow pit design

Flows greater than the bioretention swale’s design flow are either conveyed by the road reserve and/or by connection to an underground drainage system. To size a grated overflow pit (for discharge/conveyance of larger events above the maximum infiltration rate), two checks should be made for either drowned or free flowing conditions:

the broad crested weir equation to determine the length of weir required (assuming free overfall conditions); and

the orifice equation to estimate the area of opening required (assuming drowned outlet conditions).

The larger of the two pit configurations should then be adopted for design purposes. The weir equation for free overfall conditions is:

$$Q_{oflow} = CLH^{\frac{3}{2}}$$

Where:

$Q_{oflow}\ =\ overflow\ (weir)\ discharge\ (m^3 s^{-1})$

$C =\ 1.7$

$H =\ head\ above\ weir\ crest\ (m)$

$L =\ length\ of\ weir\ crest\ (m)$

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. It is considered likely that standard pit sizes will accommodate flows for most situations.

The orifice equation for drowned outlet conditions is:

$$Q_{oflow} = CA\sqrt{2gh}$$

Where:

$Q_{oflow} = \ overflow\ (orifice)\ discharge\ (m^3s^{-1})$

$C = \ 0.6$

$h = \ available\ head\ above\ weir\ crest\ (m)$

$A = \ orifice\ area\ (m^2)$

$g = \ gravity\ constant\ (9.81\ m^2s^{-1})$

Vegetation

Bioretention systems can use a variety of vegetation types, including turf, sedges and tuft grasses. Vegetation is required to cover the whole width of the swale or basin and the bioretention media surface to retard and distribute flows and protect the surface of the system. The vegetation should be able to withstand design flows and be of sufficient density to prevent preferred flow paths and scour of deposited sediments. Sedges and tuft grass are preferred to turf for surfacing bioretention systems due to the potential compaction of the media when mowing turf. Denser and taller vegetation also provides better treatment, especially during extended detention time. The vegetation will provide a surface for biofilm growth in the upper layer of the media, which is particularly useful for the transformation of pollutants such as nitrogen. Densely vegetated swales can provide good sediment trapping, withstand high flows, maintain the porosity of the filtration media and also provide attractive landscaping features. Root barriers may need to be installed around sections of bioretention systems that incorporate perforated/slotted pipes where trees will be planted to prevent roots growing into the pipes (Lloyd et al. 2002).

The ‘Vegetation guidelines for stormwater biofilters in the South West of Western Australia’ (Monash Water for Liveability Centre et al., 2014) form a comprehensive guide for biofilter plant selection, incorporating practical considerations, extensive planting lists and explanation of the background science. Readers are referred to these guidelines for more extensive guidance on plant selection for stormwater biofilters (CRC WSC 2015).

A description of common rushes, sedges and submergents of the south-west of WA are also contained in Chapter 8 ‘Using rushes and sedges in revegetation of wetland areas in the south west of WA’ of the *River Restoration Manual – a guide to the nature, protection, rehabilitation and long-term management of waterways in Western Australia* (Water and Rivers Commission 2000). The manual provides details of common species and those available commercially for rehabilitation projects, including details of appearance, location, soil type, water quality, water depth and propagation.

Maintenance

One of the primary maintenance requirements for bioretention systems is to inspect and repair or replace the treatment system components. Generally this involves periodic maintenance of the landscaped area.

Pesticide and fertiliser application to the plants should be limited during the establishment phase and avoided during the operation phase of the system. Regular watering of the vegetation may be required in the establishment phase. Bioretention system components should blend over time through plant and root growth, organic decomposition and the development of a natural soil horizon. These biological and physical processes will lengthen the facility’s life span and reduce the need for extensive maintenance.

A critical maintenance consideration is the monitoring of sediment accumulation at the inlet points. Depending on the catchment activities, the deposition of sediment can smother plants and reduce the available ponding volume. Should excessive sediment build-up occur, it may impact on plant health and lead to a reduction in their capacity to maintain the infiltration rate of the filter media.

Regular sediment removal and inspection and repair of any scour and erosion areas should be undertaken, including assessment after large storm events. Rubbish and other debris should also be removed from the surface components, including inlet structures, culverts and overflow pits.

Routine maintenance should include health evaluation of the trees and shrubs and the subsequent removal of any dead or diseased vegetation. Diseased vegetation should be removed by hand. Diseased plants imply inappropriate species selection, and the choice of plants should be reconsidered under these circumstances. In addition, bioretention systems can be susceptible to invasion by aggressive weeds, which can reduce infiltration and conveyance capacity if not routinely maintained. Vegetation may need to be pruned to maintain conveyance and the appearance of the system.

Highly organic and often heavily vegetated areas in standing shallow water can create a breeding ground for mosquitoes. Routine inspection for areas of standing water and corrective measures to restore proper infiltration rates are necessary to prevent water ponding for more than four days, to eliminate these breeding environments.

Mulch replacement is recommended when erosion is evident or when the site begins to look unattractive.

Worked example

Caution: The following worked examples use Rational Method as per the Australian Rainfall and Runoff (ARR) Book VIII (Institution of Engineers, Australia 2001). However, as per the updated ARR Book 9 'Runoff in Urban Areas', the Rational Method is only suitable for lot-scale scale catchments or simplistic small catchments where flood routing is not critical. This method is not suitable for a 'precinct' scale estimation of peak flows as it has 'limited' runoff generation and surface routing capabilities. If runoff volume management infrastructure forms part of a solution, or if an understanding of potential impacts on downstream flooding are required, then a 'strong' hydrologic estimation method such as a runoff-routing model should be used (ARR 2019). For further information on the limitations of Rational Method, please refer to Book 9 'Runoff in Urban Areas' of ARR 2019.

A site in Perth consists of a collector road and a service road separated by a 7.5 m wide median. The median area offers the opportunity for a local treatment measure. The area available is relatively large in relation to the catchment; however, is elongated in shape. The catchment area for the swale and bioretention area includes the road reserve and the adjoining gravel parking area (of approximately 35 m depth and with a fraction impervious of 0.6). The layout of the catchment and bioretention swale is shown in Figure 9.

Three median crossings are required. The raised access crossings separate the bioretention treatment system into a two-cell system (referred to in this example as Cell A and Cell B).

Each bioretention swale cell will treat its individual catchment area. Runoff from the collector 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 kerbless service road can enter the swale as distributed inflow (sheet flow) along the length of the swale.

The proposed system will not be subject to any backwater effects and will freely discharge to the receiving downstream pipe network.

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 at the downstream end of each bioretention cell.

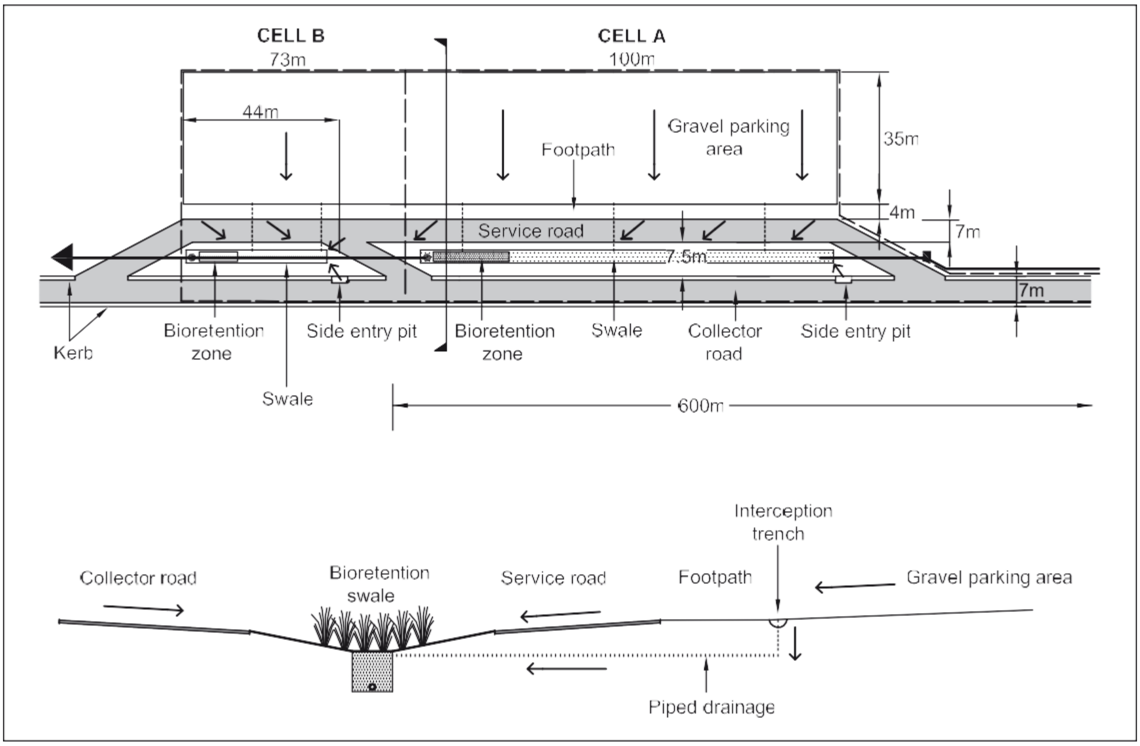

Figure 9. Catchment area layout and section for worked example.

Design objectives

- Treatment to meet water quality targets established for this site as 80%, 45% and 45% reductions of TSS, TP and TN respectively (compared to development without any water sensitive urban design applied).
- Subsoil drainage pipe to be designed to ensure that the capacity of the pipe exceeds the saturated infiltration capacity of the filtration media (both inlet and flow capacity).
- Design flows up to five year ARI (0.2 EY) range are to be safely conveyed into a piped drainage system without any inundation of the adjacent road.
- The hydraulics for the swale need to be checked to confirm flow capacity for the five year ARI (0.2 EY) peak flow.
- Acceptable safety and scouring behaviour for the five year ARI (0.2 EY) peak flow.

Design criteria and constraints

- The combined depth of the bioretention filter layer and transition layer to be a maximum of 600 mm.
- Maximum ponding depth allowable is 200 mm.
- Width of median available for siting the system is 6 m of its total 7.5 m width.
- The filtration media available is a sandy loam with a saturated hydraulic conductivity of 4.0 m/day (4.6×10^{-5} m/s).

Site characteristics

- The site features the following characteristics:
- Overland flow slopes of 1.3% for both Cell A and B
- Clayey soil, with a water table sufficiently below natural surface that it does not impact the design
- Fraction impervious: 0.60 (gravel parking area); 0.90 (roads); 0.50 (footpaths); 0.0 (median)
- Catchment areas as shown in Table 4.

Table 4. Catchment areas

Catchment	Parking area	Collector road	Service road	Footpath	Median	Catchment area
Cell A	100 m × 35 m	600 m × 7 m	100 m × 7 m	100 m × 4 m	100 m × 7.5 m	9550 m^2 (total)
	3500 m^2 × 0.6	4200 m^2 × 0.9	700 m^2 × 0.9	400 m^2 × 0.5	750 m^2 × 0.0	6710 m^2 (impervious)
Cell B	73 m × 35 m	73 m × 7 m	73 m × 7 m	73 m × 4 m	44 m × 7.5 m	4199 m^2 (total)
	2555 m^2 × 0.6	511 m^2 × 0.9	511 m^2 × 0.9	292 m^2 × 0.5	330 m^2 × 0.0	2599 m^2 (impervious)

Surface area of bioretention system

Figures 3 to 5 were used with the following parameters to estimate the size of the bioretention system to achieve the required target pollutant reductions specified in the design objectives:

- 200 mm extended detention (ponding)
- Impervious catchment area for Cell A 6710 m^2
- Impervious catchment area for Cell B 2599 m^2

Using Figures 3 to 5, the following bioretention system surface areas for each cell are selected:

- Cell A : 61 m^2 = 6 m × 10.2 m (0.89% of impervious catchment area)
- Cell B : 22 m^2 = 6 m × 3.7 m (0.85% of impervious catchment area)

These areas provide expected pollutant reductions of 85%, 68% and 48% for TSS, TP and TN respectively, which exceed the design requirements of 80%, 45% and 45%.

Estimating design flows

With a small catchment, the Rational Method is considered an appropriate approach to estimate the six month (2 EY) and five year ARI (0.2 EY) peak flow rates.

Time of concentration (t_c)

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

- Cell A: t_c calculations include consideration of runoff from the parking area, as well as from gutter flow from the upstream collector road. Comparison of these travel times concluded that the flow along the collector road was the longest and was adopted for t_c.
- Cell B: t_c calculations include overland flow across the parking area and road, as well as the swale/ bioretention flow time.

Following procedures in ARR (Institution of Engineers Australia 2001) Book VIII, the following t_c values are estimated:

- t_c Cell A : 10 mins
- t_c Cell B: 8 mins

Design rainfall intensities

Design rainfall intensities (Table 5) were derived for the example area consistent with Institution of Engineers Australia (2001) Book II.

Table 5. Design rainfalls for calculated time of concentration (Perth Airport rainfall data)

Catchment	t_c	1 year ARI (01 EY) (mm/hr)	5 year ARI (0.2 EY) (mm/hr)
Cell A	10 min	34	61
Cell B	8 min	36	66

Design runoff coefficient

Apply method outlined in *ARR* (2001), Book VIII:

$$C_{10}^{1} = 0.1 + 0.0133({}^{10}I_1 - 25)$$

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

Where:

$C_{10}^{1} = pervious\ runoff\ coefficient$

$C_{10} = 10\ year\ ARI\ runoff\ coefficient$

$f = fraction\ impervious$

${}^{10}I_1 = 10\ year\ ARI\ 1\ hour\ rainfall\ intensity$

Overall fraction impervious (based on impervious fractions for individual land use types):

$For\ Cell\ A\ (area\ weighted)\ f = 6710\ /\ 9550 = 0.70$

$For\ Cell\ B\ (area\ weighted)\ f = 2599\ /\ 4199 = 0.62$

${}^{10}I_1 = 29.0\ mmhr^{-1}$

$C_{10}^{1} = 0.15$

$C_{10}\ for\ Cell\ A = 0.68$

$C_{10}\ for\ Cell\ B = 0.62$

Runoff coefficients for various ARI events, as shown in Table 6, are then calculated as $C_y = F_y C_{10}$, with the frequency factor F_y defined in Institution of Engineers Australia (2001) Book VIII, Table 1.6.

Table 6. Calculated runoff coefficients for various ARI events

Catchment	C_1	C_5
Cell A	0.54	0.64
Cell B	0.49	0.58

Peak design flows

Using the Rational Method, peak design flows (m^3/s) for the catchment are shown in Table 7, as:

$$Q = 0.00278CIA$$

Where:

$I = rainfall\ intensity\ (mmhr^{-1})$

$C = runoff\ coefficient$

$A = catchment\ area\ (ha)$

Table 7. Calculated peak design flows for various ARI events

Catchment	$C_{1\ year}$ (m^{3}s^{-1})	$C_{5\ year}$(m^{3}s^{-1})
Cell A	0.051	0.11
Cell B	0.021	0.045

Maximum infiltration rate

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

$$Q_{max} = \frac{kLW_{base}(h_{max} + d)}{d}$$

$$Q_{max} = \frac{4.6 * 10^{-5} * L * 6.0 * (0.2 + 0.6)}{0.6}$$

Where:

$k = hydraulic\ conductivity\ of\ the\ soil\ filter\ (ms^{-1})$

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

$L = length\ of\ the\ bioretention\ zone\ (m)$

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

$d = depth\ of\ filter\ media\ (m)$

Maximum infiltration rate Cell A (L = 10.2 m) = 0.0038 m^3/s

Maximum infiltration rate Cell B (L = 3.7 m) = 0.0014 m^3/s

Swale design

The swales need to be sized to convey five year ARI (0.2 EY) flows to the underground pipe network without water encroaching on the adjacent road. Manning's equation is used with the following parameters:

- base width of 1 m with 1:3 side slopes (max depth of 0.76 m)
- grass vegetation (assume 5 year ARI (0.2 EY) flows above grass height)
- 1.3% slope

Note the depth of the swale and side slopes are determined by the requirement of discharging the parking area runoff onto the surface of the bioretention system. Given the cover requirements of the parking area drainage pipes as they flow under the service road (550 mm minimum cover), the base of the surface of the bioretention system is set at 0.76 m below road surface.

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

The maximum capacity of the swale is estimated adopting a 150 mm freeboard (i.e. maximum depth 0.76-0.15 = 0.61 m), as shown in Figure 10. Using Figure 5 of the Swales and Buffer Strips BMP, Manning's n = 0.038 for a depth of 0.61 m.

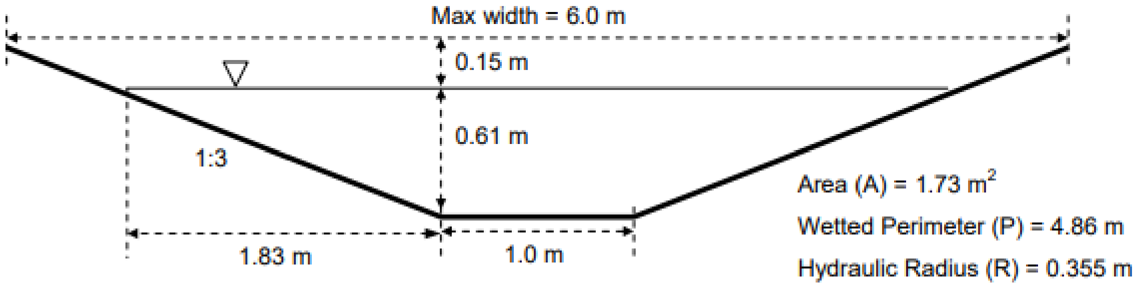

Figure 10. Swale dimensions for the worked example.

Applying Manning's equation, the maximum capacity of the swale is 2.6 m^3/s. Therefore, there is adequate capacity, given the relatively large dimensions of the swale, to accommodate the catchment runoff connection (Table 7).

Inlet details

There are two mechanisms for flows to enter the system: firstly underground pipes (either from the upstream collector road into Cell 1 or from the parking area runoff) and secondly direct sheet runoff from the service road and footpath. Flush kerbs with a 50 mm set down are intended to be used to allow for sediment accumulation from the service road and footpath surfaces. Riprap 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.

Vegetation scour velocity

To prevent scouring of the vegetation, velocities must be kept below 0.5 m/s during Q_{5year}. Using Manning's equation to solve the depth for Q_{5year}:

Q_{5year} = 0.11 m^3/s, depth = 0.18m (with n = 0.07 from Figure 5 of Swales and Buffer Strips BMP)

Velocity = 0.42 m/s < 0.5 m/s, hence OK

Hence, the swale and bioretention system can satisfactorily convey the peak five year ARI (0.2 EY) flood, with minimal risk of vegetation scour.

Sizing of perforated collection pipes

To estimate the inlet capacity of the subsurface drainage system (perforated pipe), it is assumed that 50% of the holes are blocked. To estimate the flow rate, an orifice equation is applied using the following parameters:

Head = 0.85 m (0.5 m filter layer depth + 0.1 m transition layer thickness + 0.2 m max pond level + 0.05 half of pipe diameter)

$$Pipe\ opening\ (100\%\ open) = 2100\ mm^2/m$$

$$Pipe\ opening\ (50\%\ blocked) = 2100\ mm^2/m$$

$$Slot\ width = 1.5\ mm$$

$$Slot\ length = 7.5\ mm$$

$$No.of\ rows = 6$$

$$Diameter = 100\ mm$$

$$Number\ of\ slots\ (per\ m) = (1050)\ /\ (1.5x7.5) = 93.3$$

Assume orifice flow conditions:

$$Q_{oflow} = CA\sqrt{2gh}$$

$$C = 0.6\ (Assume\ slot\ width\ acts\ as\ a\ sharp\ edged\ orifice)$$

$$Inlet\ Capacity\ (per\ m) = (0.6 * (0.0015 * 0.0075) * 2 * 9.81 * 0.85) * 93.3$$

$$= 0.0026\ m^3s^{-1}$$

Inlet capacity/m × total length:

Cell A = 0.0026 m^3/s/m × 10.2 m = 0.026 m^3/s > 0.0038 m^3/s (max infiltration rate)

Cell B = 0.0026 m^3/s/m × 3.7 m = 0.0095 m^3/s > 0.0014 m^3/s (max infiltration rate)

Hence a single pipe for each cell has sufficient perforation capacity to pass flows into the pipe.

The Colebrook-White equation is applied to estimate the flow rate in the perforated pipe. A slope of 0.5% is assumed and a 100 mm diameter perforated pipe (as above) was used. The capacity of this pipe needs to exceed the maximum infiltration rate.

Estimated flow using the Colebrook-White equation:

$$Q_{pipe} = (-2\sqrt{2gDS_1}Log_{10}\left(\frac{k}{3.7D} + \frac{2.5v}{D\sqrt{2gDS_l}}\right).A$$

where:

$$D = pipe\ diameter = 0.10\ m$$

$$A = area\ of\ the\ pipe = 0.0079\ m^2$$

$$k = wall\ roughness = 0.007\ m$$

$$v = kinematic\ viscosity = 1.007 \times 10 - 6\ m^2s^{-1}$$

$$S1 = pipe\ slope = 0.005\ m/m$$

$$g = gravity\ constant = 9.81\ m^2 s^{-1}$$

$$Q_{pipe} = 0.0027\ m^3 s^{-1}\ (for\ one\ pipe)$$

This is less than the maximum infiltration rate for Cell A of 0.0038 m^3/s, hence the pipe diameter will need to be increased (to a maximum of 150 mm) or two pipes installed to convey the maximum infiltration rate.

Overflow design

Overflow pits are required to convey flows in excess of the 200 mm maximum ponding depth from above the bioretention system to an underground pipe network. Hence, the inlet of each pit is set at 200 mm above the base of the swale. Grated pits are to be used at the downstream end of each bioretention system.

The maximum head for the pits is equal to the maximum allowable height of flows (i.e. the road surface less the 150 mm freeboard), minus the height of the pits (which is set at the 200 mm maximum ponding depth), i.e. (0.76 – 0.15) – 0.20 = 0.41m (see Figure 11)

First check using a broad crested weir equation:

$$Q_{oflow} = CLH^{\frac{3}{2}}$$

Where:

$$C = 1.7$$

$$H = 0.41$$

$$Q_{5\,year} = 0.11\ m^3 s^{-1}$$

Solving for L results in a required weir length L = 0.25 m (which would be provided by a 62 mm square pit, although standard pits are not available in this size and a larger pit would need to be used).

Now check for drowned outlet conditions:

$$Q_{oflow} = CA\sqrt{2gh}\ with\ C = 0.6$$

$$A = \frac{Q}{C\sqrt{2gh}}$$

$$A = \frac{0.11}{0.6\sqrt{2 * 9.81 * 0.41}}$$

$$A = 0.065\ m^2$$

The discharge area required is A= 0.065 m2 (a 300 mm × 300 mm pit would more than provide this area).

Hence, drowned outlet flow conditions dominate the overflow design. A pit size of 450 × 450 mm for both Cell A and Cell B is adopted as this is the minimum pit size acceptable by the local council to accommodate underground pipe connections.

Soil media specification

Three layers of soil media are to be used: a sandy loam filtration media (500 mm) to support the vegetation, a coarse transition layer (100 mm) and a fine gravel drainage layer (200 mm).

The design of the system is illustrated in Figure 11.

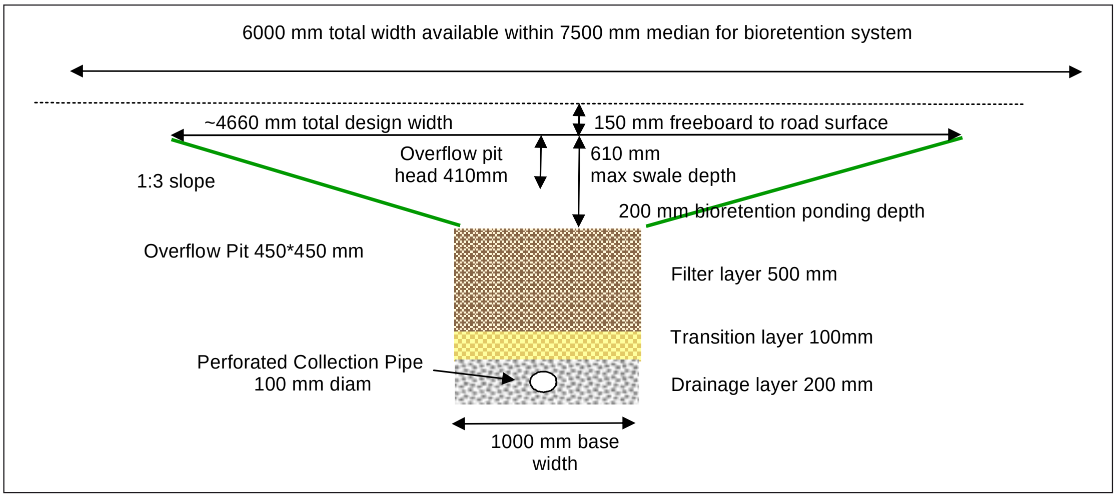

Figure 11. The completed bioretention system design for the worked example.

Drainage layer hydraulic conductivity

Typically, flexible perforated pipes are installed, surrounded by fine gravel media. In this case study, 5 mm gravel is specified for a 200 mm thick drainage layer. This media is much coarser than the filtration media (sandy loam). Therefore, a 100 mm thick transition layer of coarse sand is used to reduce the risk of washing the filtration layer into the perforated pipe.

Impervious liner requirement

In this catchment, the surrounding soils are clayey soils with an estimated 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 166 mm/hr (4 m/day).

Therefore, the conductivity of the filter media is > 10 times the conductivity of the surrounding soils and an impervious liner is not required.

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4.3 Living streams

Figure 1. Geegelup Brook drain to living stream project, Bridgetown. (Photograph: Cheryl Hamence, Blackwood Valley Landcare 2006.)

Figure 2. Paterson Street Drain to Living Stream Project, Bayswater. (Photograph: Bayswater Integrated Catchment Management Group 2004.)

Background

A healthy waterway or living stream is a complex ecosystem supporting a wide range of plants and animals. Living streams feature stabilised vegetated banks and a more natural morphology (compared to straight drains) that provide diverse habitats for animals such as frogs, fish and waterbirds. The protection of existing waterways and the rehabilitation of degraded waterways into living streams in urban areas are important techniques for improving stormwater management.

Changes to the catchment due to urbanisation can impact the health of waterways in a number of ways:

- increases in flow result in changes to the planform, size and shape of the channel
- erosion of the channel to accommodate increased flows results in vegetation loss, smothering of habitat, loss of river pools and increased turbidity
- changes to water quality, due to contaminants delivered by poorly managed stormwater, such as metals that are toxic to aquatic fauna and nutrients that can fuel excessive algal growth that depletes the water column of dissolved oxygen.

Protection and enhancement of existing natural waterways and the design of living streams need to consider the catchment management practices required to establish healthy ecosystems. In spite of the complexity of the physical, chemical and biological interactions, managing the increase in discharge from urbanisation is fundamental to facilitating waterway and ecosystem protection. The management of flood events to protect stream health is also consistent with drainage and flood protection objectives. Minimising changes to the hydrology should be done with application of WSUD measures across the catchment and disconnection of impervious surfaces from streams (Ladson et al. 2006).

Urban waterways are increasingly being recognised for their potential value as multiple use corridors that provide additional benefits to the traditional channel hydraulic objectives. A living stream achieves multiple outcomes, including creating a healthy ecosystem, improving water quality, conveying floodwaters and creating an attractive landscape feature for the residential community. The enhanced open space promotes recreational use, and improves environmental and landscape values. Healthy fringing vegetation provides wildlife habitat, ecological corridors, erosion control and bio-filtering of pollutants. Community understanding through visible linkage of stormwater and environment is also promoted.

This management practice is typically appropriate in areas with degraded natural streams and where there is opportunity to modify existing trapezoidal open drains with significant flows in areas of proposed development. Living streams have also been applied to replace sections of piped drainage, particularly as pre-treatment to a receiving water body. Construction of living streams is suitable for ephemeral, as well as permanent, water regimes.

Caution: The information provided in section 4.3 of this chapter should be read in conjunction with the brochure 'Water sensitive urban design brochure: Stormwater design considerations' (Department of Water 2011), River Restoration Manual (former Water and Rivers Commission, 1999 – 2003) and the Fact Sheet – Living Streams in Water Corporation assets (former Department of Water & Water Corporation 2016).

Performance efficiency

The major role of living streams in stormwater management is the conveyance of runoff and the provision of stormwater quality improvement, as healthy fringing and aquatic vegetation act as a biological filter.

While primarily providing a conveyancing function, living streams in many instances will also act to retain and detain water at different times of the year, due to the seasonal nature of local rainfall and shallow groundwater levels. The broad floodplain of living streams reduces flow velocities and provides flood storage, reducing the post-development peak flows.

Living streams filter both organic and inorganic material carried in the runoff and will assimilate a portion of the nutrients flushed from the catchment (Pen 1999). The healthier the vegetation and the wider the vegetation buffer, the better the water quality improvement performance of the living stream. Wide buffers allow the interception of overland flow across a greater extent and the trapping of suspended sediment including organic material.

Research on grass strip and other vegetative buffers indicate they can achieve phosphorus and nitrogen filtration rates of the order of 50–100% (Barling et al. 1994; Daniels et al. 1996; Haycock et al. 1996).

Studies in the south-west of WA have demonstrated that total suspended sediment exports can decrease by an order of magnitude from over 100 to less than 10 kg/ha/yr for nitrogen and phosphorus exports; however, this may be less in sandy low phosphorus sorption soils due to a change in the form of the nutrients (McKergow et al. 2003).

Cost^

The cost of urban waterways restoration is largely dependent on the extent of improvements required and the site conditions. Existing streams in a natural condition will require minimal costs, often associated with protecting the current conditions (such as using fencing).

More degraded systems may require erosion protection, vegetation management and waterway realignment, which significantly increase costs. River restoration projects are often completed on a voluntary basis by environmental organisations and local community groups, which can reduce costs.

Case studies for river revegetation projects in south-west WA and detailed cost estimates for these projects are contained in Water and Rivers Commission (1999). A summary of costs and indicative works undertaken is shown in Table 1, highlighting the wide range of costs depending on volunteer contributions. Costs for Paterson Street Drain in Bayswater (Figure 2) are considered the most representative in terms of modification of an existing urban drain to a living stream, as costs include (to some extent) professional fees and earthwork costs. Based on this cost, an indicative range of \$20–\$30/m^2 (2006 dollars) is provided as a guide for the conversion of drains to living streams for new development.

Table 1. Indicative costs for waterway restoration projects in WA

Project	Scope of Work	Total Capital Cost ($)	Unit Cost ($/m^2)
Baigup Reserve, City of Bayswater	Weed control, revegetation, signage and boardwalks for an area of approx. 20 ha adjacent to Swan River.	500,000	2.50
Paterson St Drain City of Bayswater	Restoration of 100 m section of Water Corporation steep sided trapezoidal drain. Works included weed control, revegetation and earthworks to lessen bank slope and introduce meanders. Cost includes professional fees of landscape design and survey. Approx. total area : 0.15 ha	26,000	17.30
Jane Brook Shire of Mundaring	Weed control and revegetation of Falls Park and Brookside Park Parkerville. Streambed erosion occurring due to flows from upstream urban development. Approx. total area of project : 5 ha	106,000	2.12
Avon River Shire of Toodyay	Weed control and revegetation of 1 ha area at Duidgee Park Toodyay. Approx. area of project : 0.1 ha	4,600	4.60
Mahogany Creek Shire of Mundaring	Weed control and revegetation of 0.06 ha area of Mahogany Creek in Hovea.	2,400	4.00

Source: Water and Rivers Commission (1999)

^The costs quoted in this section are from around 2000 to 2007 and have not been adjusted for inflation or potential cost changes which may have occurred since this chapter was published in 2007. Therefore, it should be considered as indicative only and users of the manual are encouraged to seek further specific industry advice on the current costs as appropriate.

Design considerations

Selecting the location and alignment for living streams is largely dictated by the natural terrain and established flow paths, and in some instances by other planning considerations (e.g. infrastructure requirements).

In considering waterways, priority should be given to maintaining and enhancing natural creeks where possible for their amenity and local significance. Waterway restoration may not be feasible in urbanised areas where space is limited. Where the floodplain has been infilled or developed, it may be difficult to retrofit existing waterways.

Modifications to existing waterways will generally be required where living streams are created from open drains. Channel cross sections, profiles and alignment may need to be modified to manage flow volume and velocity, as well as to provide storage capacity.

Reinforcement of the channel through soft restoration and stabilisation may be required to prevent erosion of the channel banks and bed. Factors such as soil types and stratigraphy and hydrogeological conditions will need to be examined. Highly erodible and deep soils will require careful attention to cross section design and stabilisation of surfaces. Protection measures such as detention areas, grade control structures (e.g. riffles), organic matting, brushing bank protection and use of intensive planting to increase hydraulic roughness can be implemented to manage high velocity zones.

To prevent erosion and improve water quality, vegetation around the stream is required. A vegetative buffer should preferably be of indigenous species; however, grass strips are also effective. Deciduous species should not be planted as they contribute a significant amount of leaf matter to the organic load over one season and also provide less ecological value. The leaves of deciduous species are also softer and degrade faster than the leaves of native plants, releasing nutrients into the waterway.

The invert of living streams should be designed to not intersect the groundwater table. In areas where this is not feasible and the living stream channel intersects the regional groundwater table, groundwater level and water quality investigations should be undertaken to better define the interaction between the waterway (flow, quality) and groundwater, and provide guidance to the design process. The constructed stream should be ephemeral, i.e. the invert of the channel should be dry during the dry season. This is to avoid the creation of warm, stagnant water during periods of little or no flow, as these conditions may result in algal blooms and increased midge and mosquito breeding. See section 1.7.7 'Public health and safety' of the Introduction section of this chapter for more information on mosquito management. Also see information regarding mosquito management in the Constructed Wetlands, Swales and Buffer Strips and Infiltration Basins and Trenches BMPs.

In the case of retrofitting existing open drains in urban areas, any changes to existing drain inverts, meandering and channel configurations should be hydraulically assessed (modelled) to ensure the existing flood capacity of the drain is maintained and flood levels are not adversely affected by the works.

Design guidelines

In natural streams, the shape and size of the channel and extent of vegetative growth in the channel are in balance with the discharge characteristics of the catchment. Constructed channels, in the 'living streams' approach, are designed to mimic natural streams with high flows accommodated along the vegetated streamline and its floodway. Infiltration, detention and treatment of the stormwater through contact with vegetation are maximised at base flow and during low intensity rainfall events. During high rainfall events, flood protection is maintained by conveyance in the floodway. Flow velocities can be reduced and flood storage maximised for high flows by providing a broad vegetated floodway. Therefore, designing a living stream requires design of a two-phased system, with a channel for frequent low flows and a floodway for rare larger events.

Guidelines on stream channel analysis, stabilisation and rehabilitation design are provided in River Restoration – a guide to the nature, protection, rehabilitation and long-term management of waterways in Western Australia (Water and Rivers Commission/Department of Environment 1999–2003).

Channel design

Construction or retrofitting of a watercourse should aim to create a natural channel form that replicates the ecological as well as hydrologic functions of a healthy waterway.

Channel variability, including large woody debris, meanders, pools and riffles, is important to create diverse habitat conditions. If available, a reference reach (either a good condition section of the same waterway or a good condition reach of a nearby waterway with similar sized catchment) should be surveyed to determine the natural channel characteristics, including cross-sectional size, channel shape, bed form (bed paving materials, snags, pools, riffles, etc.) and stream alignment, to be used as a reference for channel construction or restoration.

If a reference reach is unavailable, then theoretical equations exist to estimate appropriate channel morphology based on the catchment size and discharge. Oversized channels can lead to problems such as in-stream meandering, sediment deposition, vegetation congestion and subsequent flooding and erosion during large flow events. Undersized channels can lead to flooding, erosion and flows breaking out of the channel. Waterway capacities are sized to provide free flowing conditions that limit the accumulation of sediments and localised ponding. The design of this free-flowing condition must not permit supercritical flow, which may compromise the surface armouring (grass, shrubs, geotextile systems, etc.) or public safety. Typically, flow velocities over vegetated waterways should not exceed 1.2 m/s. Where localised supercritical flow occurs, it should be controlled over hard bed armouring, such as a rock riffle structure.

Manning’s equation can be used to estimate the channel capacity and velocity. Hydraulic modelling packages such as HEC-RAS (the US Army Corps of Engineers Hydrologic Engineering Center’s River Analysis System) can also be used to design a river channel and analyse parameters such as flow velocities and stage heights.

The stability of a channel can be established by selecting a suitable channel width to accommodate the dominant flow, known as the bankfull discharge. Typically, the bankfull discharge is the average peak flow for a 1.5 year ARI (~50% AEP) event. The bankfull width is the width of the channel at water level during a bankfull discharge. Flows greater than the bankfull discharge overtop the main channel and flow across the floodway (Figure 3). This two-staged channel approach assists in mimicking natural stream form, increasing storage and controlling flow velocities. Confining flood flows to a deep, narrow channel will increase the potential for erosion and deliver flows faster, increasing the risk of flooding downstream. The floodplain cross section should be evenly sloped on a slight grade towards the low flow channel to avoid waterlogging and water ponding (i.e. to avoid mosquito breeding). Bunds or levees should not be constructed along the banks of the channel as they confine flows to the channel and prevent floodwaters from re-entering the channel from the floodplain as floods recede.

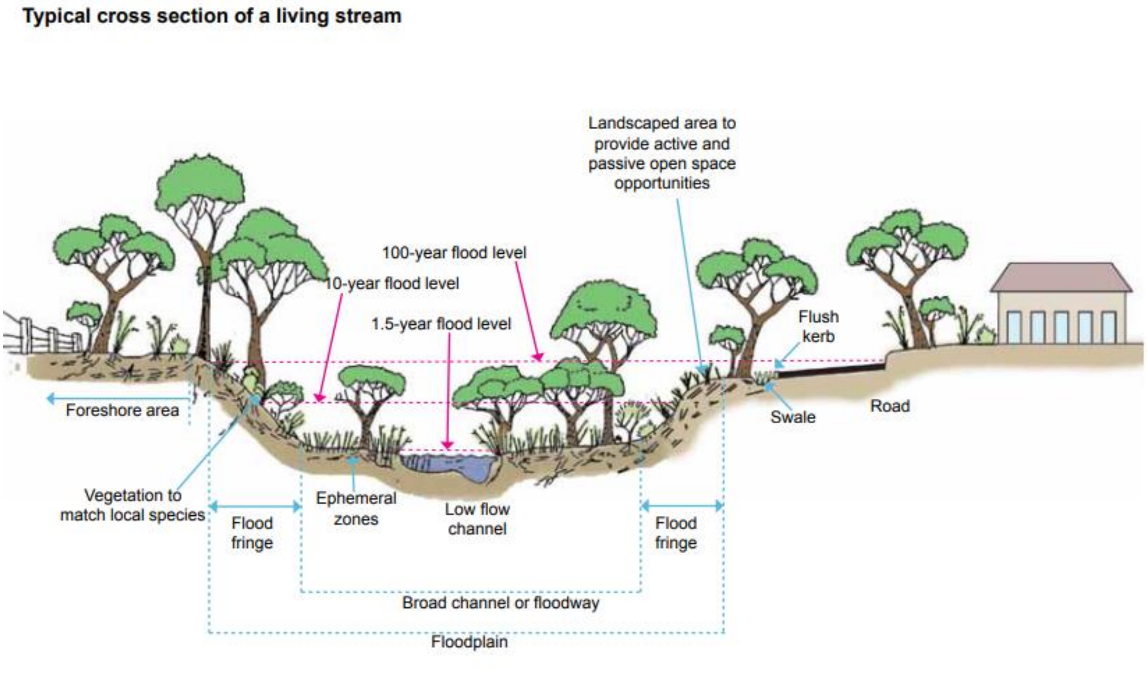


Figure 3. Stages of a natural river channel cross section. (Adapted from Department of Water 2011.)

Channels should be constructed to give a natural, broad U-shape, rather than a fixed artificial shape such as a trapezoidal or rectangular cross section. Newbury and Gaboury (1993) found the ratio of depth to width for natural channels to fall within the range of 1:10 to 1:15. However, broad, shallow channels are not the most efficient shape for conveying flow and may result in large areas of land being required to meet flood control objectives. If land availability is a constraint, then the conveyance efficiency of the waterway can be increased by designing a narrower, deeper channel; however, this will increase the tractive force of flows and may result in additional bed protection being required to prevent erosion (Newbury and Gaboury 1993).

Slope construction must be designed with a consideration of structural stability, free draining conditions, maintenance activities and public safety. For vegetated waterways, slopes should be limited to a maximum grade of 1:4 to facilitate vegetation establishment and ensure structural stability. A gentler slope of no steeper than 1:6 is required to facilitate ease of maintenance and to provide a safe transition between the bank and the drain invert.

A design aim of constructed waterways is to retain as much of the sinuosity of the natural drainage as possible so that velocities can be minimised, storage capacity can be increased and the ecological and landscape values of the drainage corridor can be enhanced. Studies of watercourse behaviour have shown that this meandering pattern generally provides the greatest stability in channel flow. Straightened drainage lines are often observed attempting to rebuild a natural meander pattern over time and require ongoing maintenance to retain an artificial alignment. Leopold, Wolman and Miller (2020) observed that river patterns consisted of the following characteristics (Figure 4):

- a full meander wavelength is found to occur between 7 to 15 times the bankfull width
- the average distance between the ends of riffles is half the meander wavelength
- typically, the radius of the sinusoidal curves ranges between 2.3 to 2.7 times the bankfull width.

The longitudinal slope of the constructed or retrofitted waterway should be the same as the natural (pre-interference) grade of the waterway or reference waterways in the region. Slopes steeper than 1:100 are feasible by using engineered bed armouring or a series of grade controls such as a riffle sequence. Gentler slopes may be required in open earth channels or where vegetation alone armours the bed. Very flat longitudinal slopes may result in waterlogging or stagnant ponds, however this may not be an issue in well-draining soils (e.g. coastal plain sands).

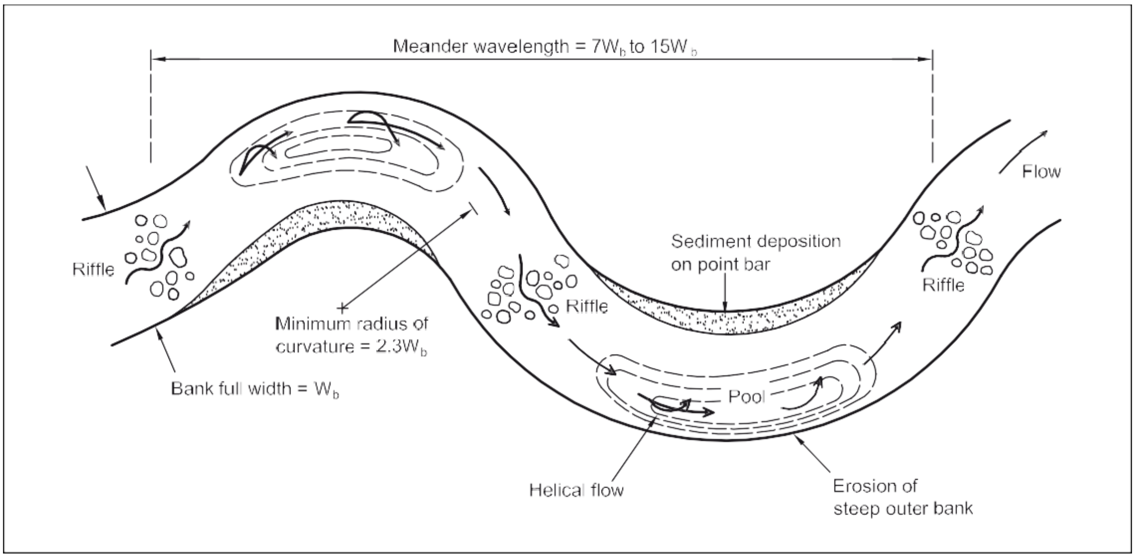


Figure 4. Theoretical meandering stream channel form. (Water and Rivers Commission/Department of Environment 1999–2003.)

Erosion prevention

Protection against erosion is a key factor that needs to be considered as part of channel design. Materials ranging from stone pitching and gabions to vegetation and geotextiles can be used to bind the soil and reduce velocities. Vegetation is particularly effective in preventing erosion and is discussed in a separate section. Design guidelines for erosion prevention at confluences, discharge points for stormwater and channel crossings, and techniques to prevent channel erosion are outlined in this section.

Erosion often occurs in the vicinity of crossings on waterways. Bridges or culvert crossings should be designed so that they do not obstruct flow or inhibit the migration of aquatic fauna. The design should ensure that the local hydrology and stream characteristics (e.g. meanders) remain essentially unchanged. Open span bridge/arch structures or low level floodway crossings are preferred where feasible to minimize interference with the natural flows and aquatic habitat of a river channel. If culverts are being used, then multi-celled box culverts that replicate the cross section shape and size of the channel are recommended so that flows are not concentrated or flooding increased and to provide faunal passage. At least one of the box culverts (located nearest to the bank) should be recessed below bed level to allow fish passage. Rocks can be adhered to the base of the culverts to reduce velocity and provide suitable conditions for fish movement (Witheridge 2002). Crossings with pipe culverts are not recommended on waterways due to problems associated with jetting effects, erosion, blocking with debris and creating barriers to faunal movement. Bridge pylons and culverts should be aligned perpendicular to the main flow channel. Crossings should be located along a straight reach of the river or meander inflection point (where the flow crosses from one bend into the next). Crossings should not be located at or near bends due to the potential to cause bank erosion. Erosion protection of the bed and banks in the vicinity of the crossing should be provided.

Channel instability can occur at the confluence of watercourses. The constructed channel should enter the receiving waterway at a gentle angle (rather than at right angles) to direct flows in the same direction as the main waterway and minimise the risk of erosion. The channel should also enter at the same bed level so that head-cutting does not occur.

Direct discharge of piped stormwater to a receiving waterway should be avoided where possible. Ideally, piped flow should cease at the boundary of the riparian area of the receiving living stream and flow made to spread out and filter through buffer vegetation prior to entering the waterway channel. Where piped flow enters waterways, the invert level, size, slope and alignment should be designed to minimise potential impacts on the waterway. Adequate wing and cut-off walls at the exit of the culvert should be provided to control the flow, prevent erosion and avoid failure of the structure. Armouring of the bed and banks around the culvert outlet is often required to prevent erosion of the waterway.

Different bed materials and channel conditions will have different velocity thresholds and resistance to erosion. Recommended maximum design velocities to prevent erosion of various channel materials are detailed in Table 2.

Table 2. Recommended flow velocities for various bed materials

Type of waterway bed material	Recommended maximum flow velocity
Rock lined channels (100–150 mm)	2.5–3.0 m/s
Grassed covered surfaces	1.8 m/s
Stiff, sandy clay	1.3–1.5 m/s
Coarse gravel	1.3–1.8 m/s
Coarse sand	0.5–0.7 m/s
Fine sand	0.2–0.5 m/s

Source: Concrete Pipe Association of Australasia (2012)

Log structures can also be used to increase channel roughness, enhance stream habitat and stabilise the streambed. Large woody debris can be placed across the channel or aligned close to the banks to assist stream stability and create habitat (Figure 5). In-stream large woody debris (logs and branches) are a feature of natural waterways and important for providing stable habitats, food sources and shelter for aquatic fauna. Further information on the importance and installation of large woody debris in waterways is provided in Water and Rivers Commission/Department of Environment (1999–2003); Water and Rivers Commission (2000a, 2000b and 2000c).

Figure 5. a) Construction of a log and rock riffle structure in the Canning River, Pioneer Park, Gosnells. (Photograph: Department of Water 2004.) b) Log riffle built on the South Dandalup River, Pinjarra, to stabilise the sandy riverbed, induce a scour pool and enhance habitat diversity. (Photograph: Water and Rivers Commission 1999.)

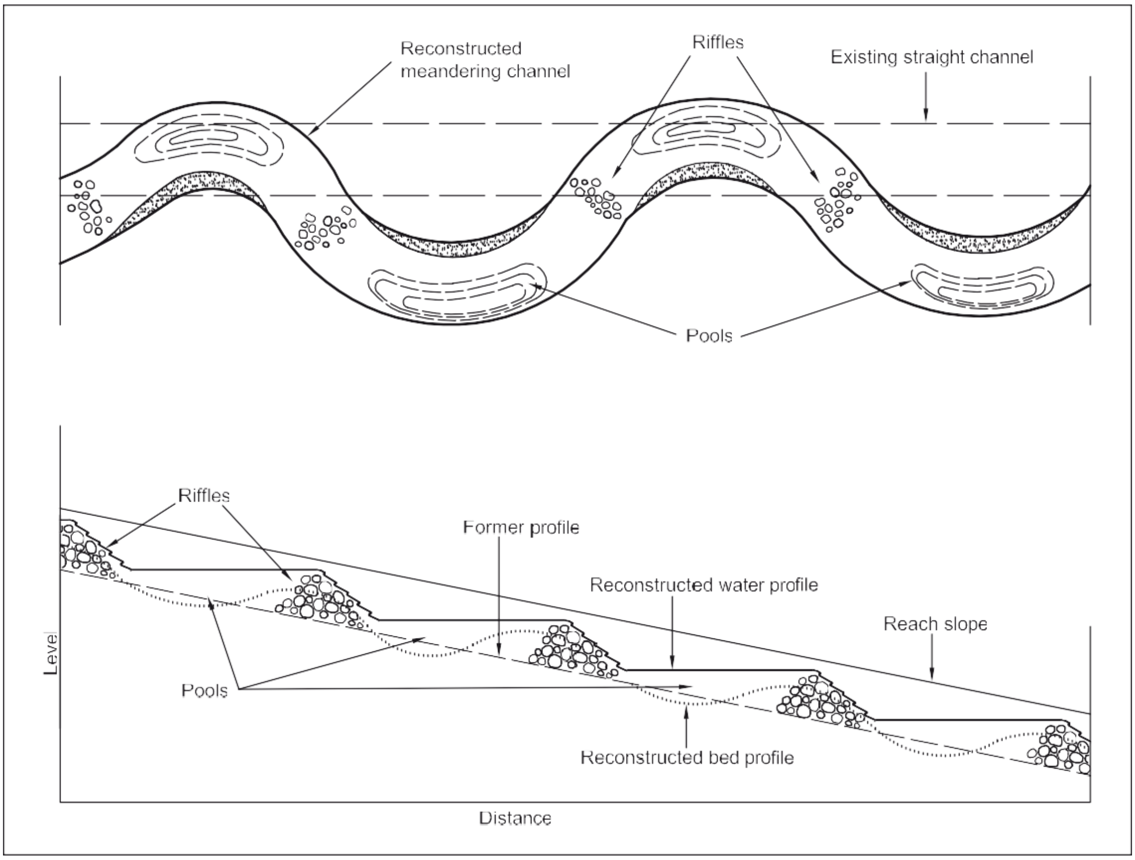
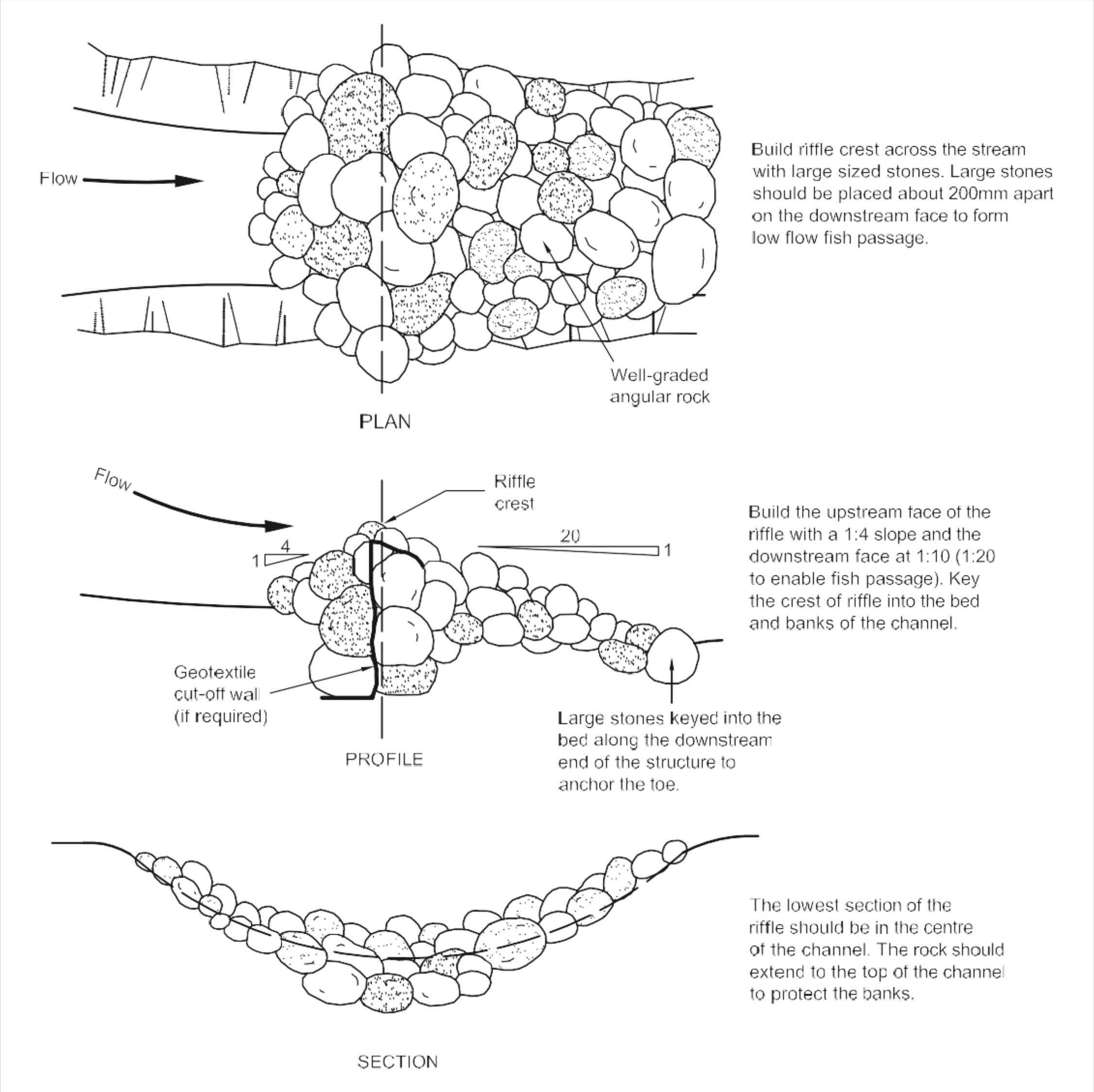


Figure 6. Ideal reconstructed channel alignment and pool-riffle sequence. (Water and Rivers Commission/Department of Environment 1999–2003.)

The degradation of channels caused by erosion can be managed by rebuilding the pool-riffle sequence for streams in steeper areas. This technique involves using riffles to increase the bed level and control the slope of the reach to achieve stability. Flows over unstable reaches are controlled using a series of step pools (Figure 6). A pool-riffle series was used to control the steep 0.7% slope of Geegelup Brook and restore habitat (Figure 1). The banks of the trapezoidal channel were regraded to a gentler slope and revegetated. Pool-riffle sequences also have the added benefit of promoting favourable habitats by increasing the holding capacity of water within the reach, creating pools that are a focus for fish and are typically a refuge for aquatic fauna during the dry season. Unlike traditional weirs and drop structures, riffles do not block the migration of fish and other aquatic fauna, but do enhance the habitat diversity of the waterway, encourage the growth of biofilms and aerate flows. If fish are present in the receiving waterway, then the channel should be designed so that there are no barriers to the migration of fish into the constructed channel. Fish require access along waterways and to floodplains and tributaries for breeding and other life cycle processes. Guidelines for riffle design can be found in Water and Rivers Commission/Department of Environment (1999–2003).



Build riffle crest across the stream with large sized stones. Large stones should be placed about 200mm apart on the downstream face to form low flow fish passage.

Build the upstream face of the riffle with a 1:4 slope and the downstream face at 1:10 (1:20 to enable fish passage). Key the crest of riffle into the bed and banks of the channel.

Large stones keyed into the bed along the downstream end of the structure to anchor the toe.

The lowest section of the riffle should be in the centre of the channel. The rock should extend to the top of the channel to protect the banks.

Figure 7. Design details for riffle construction.

The following considerations should be included in the design of riffle sequences:

1. Riffles should be located along straight reaches of the waterway or at meander inflection points (Figure 4). Riffles should not be located on bends as they may direct flows into the downstream bank, causing erosion.

2. The heights of the riffles should be set to follow the average slope of the reach. If using riffles to control erosion, the height of the riffle should backflood to the base of the next riffle upstream (Figure 6).
3. Riffles should typically be designed to block no more than 10% of the channel cross-sectional area and the height be kept under 500 mm to minimise disruption to high flows.
4. The crest of the riffle should be keyed into the bed and banks and the rocks extended to the bankfull level in the batters (Figure 7).
5. Use a range of rock sizes to allow for interlocking and to minimise voids in the structure. The minimum rock sizing can be estimated by determining the maximum tractive force during critical flows. The CHUTE design package can be used to assist analysis of critical flow conditions and guide appropriate rock sizing. CHUTE is available on the eWater Cooperative Research Centre Catchment Modelling Toolkit website toolkit.ewater.org.au/Tools/CHUTE.
6. The downstream rock apron should typically be constructed with a grade of 1:10. Where fish are present in the waterway, a grade of 1:20 is required to allow for fish passage.
7. In dispersive or non-cohesive soils, filter cloth may be required to line the channel beneath the rock. If using filter cloth on the banks, ensure the bank angle is well below the angle of repose of the rock to minimise the risk of failure caused by the rock slipping on the cloth.
8. To prevent flows piping between the rocks and washing out materials, filter cloth or a clay liner can be used to form a vertical cut-off wall through the crest of the riffle to below bed level.

Vegetation management

Managing vegetation in living streams is primarily aimed at addressing four key issues: stabilising natural surfaces against erosion by providing the necessary armouring; attenuating and treating stormwater flows; improving aesthetic value of multiple use corridors; and improving the ecological value of the catchment. The modification of existing vegetation should only be undertaken if a clear net gain to the overall waterway health can be demonstrated.

Native plants that provide shade and have hard leaves that decompose slowly are essential elements of healthy stream ecosystems. The type of species used in living stream revegetation should reflect those that are native to the botanic region within which the waterway is located, and preferably within the same local provenance. This is because such species are better adapted to local conditions; it avoids contaminating and possibly degrading the gene pool; and avoids the possibility of generating new weedy species. Even when the specific objective of the revegetation is nutrient management, there is often no need to import non-local species.

In choosing appropriate species, the soil limits of the species should be considered. The hydroperiod and flow velocity is also important to plant species selection. Given the cyclical nature of the local rainfall pattern in WA, plant species selection needs to consider the dry seasonal periods that typically extend five months each year. The wetland species can be defined as permanent or seasonal inundation tolerant. Their ability to survive inundation and waterlogging will be based on special physiological attributes such as the presence of air cells within the roots and stems. Runner type native grasses (rather than clumping varieties) and sedges and rushes that can bend in high flows and protect the channel from erosion are recommended in frequently flowing areas. A range of species should be selected to increase biodiversity and resilience of the community, allowing for varying success in survival, and should include sedges, submergents, shrubs and trees. Direct seeding, planting, transplanting, application of pre-seeded matting and brushing with branches harvested from plants bearing mature seeds are all techniques available to revegetate the living stream. Revegetated areas may require protection from erosion, grazing and trampling. Tree guards, fencing, organic matting and deflectors can be used to increase the success rate of plant establishment.

A vegetation plan is a useful tool for determining the area to be revegetated and/or the area of vegetation to be retained and enhanced, the range of soil types and riparian zones present, the number of plants/seeds required and timing schedule. A basic plan may include the following requirements:

- define floodplain, embankment and channel bed area (m2) requiring revegetation;
- characterise water quality – salinity, nutrients and turbidity;
- identify existing soil, vegetation community species and existing weed characteristics and extent in the waterway;
- map the morphology (plan and cross section) and indicate annual flood line and points of erosion and deposition;
- ongoing review of the success of the plantings and weed management.

Recommended buffer widths for waterways are determined using biophysical criteria, as outlined in Water and Rivers Commission (2001). The minimum recommended buffer of native vegetation adjacent to constructed waterways in urban areas is 10 to 20 m from the top of the bankfull channel. The channel banks and a minimum 10 m wide strip along the top of the banks should be vegetated with native vegetation. For natural waterways, a 'foreshore reserve' width of 30 and 50 m is recommended (Western Australian Planning Commission 2021). However, a flexible approach for determining a waterway reserve setback is recommended to account for site conditions such as topography, waterway form, vegetation complexes, soils and extent of the floodway. Consideration of these site conditions when determining the buffer for a constructed waterway is also applicable and might result in a buffer greater than 10 m. It is recommended that the biophysical criteria approach be used, especially where significant ecological, social or economic values are present (Western Australian Planning Commission 2006). This will minimise the potential for loss of valuable habitat and the degradation of foreshore and waterway values. Activities likely to degrade the buffer's protective function are not considered compatible in foreshore areas.

Rarely inundated floodplain areas are often managed as parkland or floodplain paddocks (in rural/semi-rural areas). Ideally, the channel should be fenced off (if grazing is an issue) to exclude livestock and the area revegetated. Where livestock have restricted access to the floodplain, pasture should not be overgrazed but managed to ensure complete groundcover at all times of the year so that in the event of a large flood, erosion of pasture land is minimised. Livestock should be kept off inundated foreshore/parkland to avoid pocketing the foreshore and creating numerous depressions that may hold standing water and allow mosquito breeding. Where possible, fences should be aligned parallel to the direction of flood flows to minimise the potential for debris accumulation and damage of fences. Materials such as lawn clippings, soil and waste stockpiles should not be stored where they can wash into the channel. Fertiliser use in or near waterways to enhance plant growth is generally not recommended. See section 2.2.7 of Chapter 7: Non-structural controls for more information.

If possible, sufficient time should be allowed for vegetation establishment to stabilise the reconstructed channel prior to diverting flows down the channel. There is some risk that during the period that the reconstructed channel is stabilising (where full vegetative cover has not been achieved and soil is exposed) flows may cause some erosion of the channel. If the works are likely to mobilise sediment during the early stages of disturbance, then techniques to trap sediment can be implemented, such as filter strips (vegetation planted perpendicular to flow) or temporary settlement ponds to prevent sedimentation of the waterway or receiving water bodies.

Refer to Water and Rivers Commission/Department of Environment 1999-2003, particularly report numbers RR 4: Revegetation – Revegetating riparian zones in south-west Western Australia, RR 5: Revegetation – Case studies from south-west Western Australia and RR 8: Using rushes and sedges in revegetation of wetland areas in the south west of WA. Also refer to the Water Note series of publications.

In areas outside the south-west, refer to local revegetation guidelines. For example, for the Avon catchment, refer to *Riparian Plants of the Avon Catchment – a field guide* (Department of Environment 2004).

Maintenance

The successful rehabilitation of a healthy, ecologically functioning waterway is a long-term process. The stability of vegetated waterways is largely dependent on the success of plant establishment to protect the channel from erosion.

By designing a stable meander alignment that mimics a natural waterway and is in balance with the catchment hydrology, long-term maintenance of the channel can be reduced. Waterway engineering works should be inspected at least annually and, if possible, after each heavy rain. If problems develop, maintenance should be performed promptly to prevent additional, costly damage. Abuse and neglect are the most common causes of waterway failure. Common maintenance problems include weeds, eroded or bare areas, sediment deposits, litter accumulation and inadequate plant establishment. Pre-treatment methods, such as filter strips or litter and sediment traps, upstream of the living stream will assist in managing maintenance by providing a designated area to remove these pollutants. This will minimise disturbance of the rehabilitation area for maintenance purposes, which may be difficult to access once vegetation establishes. Maintenance activities may be needed more frequently during the initial establishment phase, or when the waterway conveys large volumes of water or is on a steep slope. Structures such as riffles and large woody debris installations should be inspected after the first major flow event. Some minor settling or movement should be anticipated and may require repair. These types of structures usually stabilise after the first year or high-flow period.

It is recommended that a newly revegetated site be checked every two weeks for the first six months to allow early detection of germinating weed species, assessment of the success of plantings and maintenance of tree guards if these are used (Water and Rivers Commission 2002a). Once plantings are well-established and good weed control is achieved, the resources needed to maintain the site will decline. Weed control is usually most demanding in the first two to three years when native vegetation is establishing, and if conducted correctly during this period, future maintenance should be minimised. Weed control should be undertaken in an appropriate manner, such as by staging the works and gradually replacing weeds with native plants, so that the beneficial functions that the exotic vegetation may be providing are maintained. Poorly managed removal of exotic vegetation may lead to channel destabilisation, higher velocity flows and the loss of shade and faunal habitat. Common riparian weed species in the south-west of WA are Bridal Creeper (*Asparagus asparagoides*), Watsonia (*Watsonia* spp.), Victorian Coast Teatree (*Leptospermum laevigatum*), Willow (*Salix babylonica*), Common Fig (*Ficus carica*) and Castor Oil Plant (*Ricinus communis*) amongst many other species. Isolated plants should be removed before they mature and spread. To find out which plants are weeds in WA, go to the Department of Biodiversity, Conservation and Attractions Florabase website florabase.dpaw.wa.gov.au and the Department of Primary Industries and Regional Development's Western Australia Organism List website www.agric.wa.gov.au/organisms. Control options include physical removal, solarisation and herbicides. Due to the sensitivity of water environments, herbicides should be used cautiously. Clearing weeds from around native seedlings during the first two to three years will dramatically improve native plant growth and survival rates. However, there will always be a need to monitor the area and it is recommended this be undertaken once every season. Unexpected events such as fire, flood or increased human use can degrade the site and allow increased weed invasion. Refer to Water and Rivers Commission/Department of Environment (1999-2003) and associated Water Note series for further guidance on weed management in riparian zones.

Some selective thinning of vegetation may be required to restore the hydraulic capacity of the stream if overgrowth in the channel is causing a flooding problem or risk of an avulsion. However, a cautious approach is essential to prevent destabilising the stream. Specialist advice and required approvals to clear native vegetation should be sought. Clearing of significant debris dams and culvert blockages should be undertaken to maintain the capacity of the channel. Information on maintenance of riparian vegetation is provided in Water and Rivers Commission (2002a). Maintenance activities may include weed and feral animal control, infill planting, mulching and watering over the first summer.

Further information on monitoring of waterway works is available in Water Note 28: *Monitoring and Evaluating River Restoration Works* (Water and Rivers Commission 2002b).

Local examples

There are a number of successful local examples of living stream projects. Chapter 6: Retrofitting of this manual provides detailed examples of the following projects where drains were converted to living streams: Bayswater Main Drain at Paterson Street in the City of Bayswater; Bannister Creek Project in the City of Canning; Coolgardie Street Drain in the City of Belmont; and Geegelup Brook project in Bridgetown.

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5 Detention systems

5.1 Dry/ephemeral detention areas

Figure 1. Detention basin retrofit, Bridgewater, City of Mandurah. (Photograph: Department of Water 2007.)

Figure 2. Darkan Street Detention Basin with riser outlet, Shire of Mundaring. (Photograph: JDA 1997.)

Background

Dry/ephemeral detention areas are landscaped areas formed by simple dam walls, by excavation below-ground level or by utilisation or enhancement of natural swales or depressions. These areas primarily serve to capture and store stormwater to prevent excessive runoff and channel erosion in receiving environments, and as areas to remove particulate-based contaminants and sediment.

These areas are termed dry/ephemeral as they have a base level located at or above the regional groundwater level (typically defined as the long-term maximum groundwater level), with inundation of the area occurring as a result of intermittent stormwater inundation, rather than as a result of groundwater exposure.

The Department of Water and Environmental Regulation does not include constructed ponds and lakes as a stormwater quality improvement BMP in this manual. This applies to designs that involve artificial exposure of groundwater (e.g. through excavation, or lined lakes that require groundwater to maintain water levels in dry seasons) or the modification of a wetland type (e.g. converting a dampland into a lake) due to water conservation, environmental and health concerns.

For information regarding the Department of Water and Environment's current position on the construction of ponds and lakes, the reader is referred to the *Interim Drainage and Water Management Position Statement: Constructed Lakes* (Department of Water 2007).

So that detention areas can perform their design function of detaining flows, the storage volume needs to be available for the next storm event, thus maintaining a permanent pool is not considered best practice.

Performance efficiency

Dry/ephemeral detention areas are effective at removing particulate-based contaminants and sediment but less effective for treatment of soluble pollutants where biological uptake of nutrients is required. Pollutant removal through sedimentation relies on strong affinity for sorption of metals, nutrients and hydrocarbon contaminants with particulates. Pollutant removal efficiency increases with increasing hydraulic residence times.

There is little local data to assess the performance efficiency of dry/ephemeral detention areas. These systems operate with a similar principle to sedimentation basins, which have been assessed. Fletcher et al. (2004) examined the performance of sedimentation basins in removing pollutants, such as TSS, TP, TN and heavy metals, and the results are presented in Table 1.

Due to the infiltration capacity of sandy coastal plain soils, it would be expected that the performance efficiency of dry/ephemeral detention areas for many parts of WA would be at the higher end (or in excess) of the expected removal percentages shown in Table 1.

Table 1. Typical annual pollutant load removal efficiencies for sedimentation basins

Pollutant	Expected removal	Comments
Litter	> 95%	Subject to appropriate hydrologic control. Litter and coarse organic matter should ideally be removed in an aerobic environment prior to a basin, to reduce potential impacts on biological oxygen demand.
Total suspended solids	50–80%	Depends on particle size distribution.
Total nitrogen	20–60%	Depends on speciation and detention time.
Total phosphorus	50–75%	Depends on speciation and particle size distribution. Will be greater where a high proportion of phosphorus is particulate.
Coarse sediment	> 95%	Subject to appropriate hydrologic control.
Oil and Grease	n/a	Inadequate data to provide reliable estimate, but expected to be >75%.
Faecal Coliforms	n/a	Inconsistent data
Heavy metals	40–70%	Quite variable, dependent on particle size distribution, ionic charge, attachment to sediment (vs % soluble), detention time, etc.

Source: Fletcher et al. (2004)

Cost^

Construction costs associated with these facilities can vary considerably.

The variability can be attributed to whether the existing topography will support the function of a dry ephemeral detention area, the complexity of the outlet structure, and whether it is installed as part of new construction or implemented as a retrofit of an existing drainage system. Varying subsurface conditions and labour rates can also contribute to the inconsistent costs.

Local cost data for dry/ephemeral detention systems is limited. An alternative method of costing these systems is to examine the costs of similar systems, such as ponds and swales. Center for Watershed Protection (1998) cited in United States Environmental Protection Agency (2001); Fletcher et al. (2004) cited in Taylor (2005) and Walsh (2001); Weber (2001) cited in Taylor and Wong (2002) reported costs for ponds (sourced from limited data in Australia) ranging from $2,000/ha of catchment to $30,000/ML of pond volume, and $60,000/ha of pond area.

Taylor (2005) also reported costs for vegetated swales of approximately \$4.50/m^2, which included earthworks, labour and hydro-mulching. For swales with rolled turf the cost was approximately \$9.50/m^2 and for a vegetated swale with indigenous species the cost was approximately \$15–20/m^2. It would be expected that the above costs for both these systems would be comparable to the components of a landscaped dry/ephemeral detention area.

Center for Watershed Protection (1998), cited in United States Environmental Protection Agency (2001), estimated the annual cost of routine maintenance for ponds at typically about 3-6% of the construction costs. However, there is almost no actual maintenance cost data available in published literature and studies carried out have yet to experience the full maintenance cycle.

Maintenance costs may vary considerably depending on the aggressiveness of the vegetation management required at the site and the frequency of litter removal.

^The costs quoted in this section are from around 2000 to 2007 and have not been adjusted for inflation or potential cost changes which may have occurred since this chapter was published in 2007. Therefore, it should be considered as indicative only and users of the manual are encouraged to seek further specific industry advice on the current costs as appropriate.

Design considerations

The design approach should be selected based on the target pollutants as well as site and economic constraints.

As with other BMPs, pre-treatment can extend the functional life and increase the pollutant removal capability of ephemeral detention areas. Pre-treatment can reduce incoming velocities and capture coarser sediments, which will reduce the maintenance requirements and extend the life of the detention system. This is usually accomplished through means such as buffer strips and/or GPTs.

Forebays (or inlet zones) at the inflow points to the detention area can capture coarse sediment, litter and debris, which will simplify and reduce the frequency of maintenance. Forebays can be sized to hold either the expected sediment volume between clean-outs, and/or designed to have sufficient capacity to detain or infiltrate (where possible) frequently occurring storm events (typically < 1 EY) without discharge to the main ephemeral detention area.

Construction of dry/ephemeral detention areas has lower ASS risks compared to risks associated with construction of ponds and lakes, as dry/ephemeral detention areas should be designed to not alter groundwater levels, which could result in flooding or exposing ASS. However, ASS must still be considered when designing in areas that have a high risk of forming ASS, or where dewatering during construction in medium and low risk areas could affect soils in high risk areas. These areas are defined in ASS risk mapping available from the Department of Water and Environmental Regulation and using the freely available Maps Viewer on the DATA WA website catalogue.data.wa.gov.au/dataset/acid-sulfate-soil-risk-map-100k-dwer-048. Land development proposals within these areas will typically be required to undertake site-specific soil investigations and prepare ASS management plans and, where relevant, dewatering management plans. For more information about planning for ASS see the Department of Planning, Lands and Heritage website www.dplh.wa.gov.au/policy-and-legislation/state-planning-framework/fact-sheets,-manuals-and-guidelines/acid-sulfate-soils-planning-guidelines.

Dry/ephemeral detention areas must be designed to minimise the risk of mosquito breeding. See Section 1.7.7 ‘Public health and safety’ of the Introduction section of this chapter for more information on mosquito management. Some of the mosquito management guidelines in the Constructed Wetlands BMP may also be applicable for dry/ephemeral detention areas.

Design guidelines

Design flows

The Decision process for stormwater management in WA (Department of Water and Environmental Regulation 2017) provides general design flow criteria guidance for the use of landscaped dry/ephemeral detention areas in POS areas or linear multiple use corridors.

To protect receiving environments from flooding and erosion and to maintain EWRs, the generally adopted approach for design is to maintain pre-development discharge rates for storm events up to the 1% AEP, with events up to the 1 EY retained or detained onsite or as high in the catchment as possible. As discussed in the Design Considerations section of this BMP, this may result in the use of forebay/inlet zone areas to maintain frequently occurring storms separate from the main dry/ephemeral detention area.

A range of hydrologic methods can be applied to estimate design flows. If the catchment is relatively small, the Rational Design Method (Institution of Engineers Australia 2001) may be used for sizing of inlet hydraulic structures.

It is recommended, however, that the detention area sizing, design configuration and design of outlet structures (pipes, spillways, etc.) be undertaken using a comprehensive flood routing method. If required, the Department of Water and Environmental Regulation can provide advice on suitable hydrologic/hydraulic models to undertake this design.

The typical approach to estimate the design flow is to establish a pre-development model of the catchment area. The model is used to estimate the pre-development flow rates from the contributing catchment under its current land use. Where possible, modelled pre-development flow rate estimates should be verified against any existing historical data or anecdotal information.

Note that in some areas the pre-development flow rate may exceed the capacity of the downstream receiving environment, and the design flow estimate may need to be reduced accordingly to protect the receiving environment.

Basin layout

To optimise hydraulic efficiencies and thereby reduce the potential for short-circuiting and dead zones, it is desirable to adopt a high length to width ratio. The ratio of length to width varies depending on the size of the system and the site characteristics. To minimise earthworks, smaller systems have typically been built with low length to width ratios, which has often led to poor hydrodynamic conditions.

The term ‘hydraulic efficiency’ was used by Persson et al. (1999) to define the expected hydrodynamic conditions of stormwater detention systems. Engineers Australia (2006) presented a range of expected hydraulic efficiencies for detention systems for a series of notional shapes, aspect ratios and inlet/outlet placements. It was recommended that such systems should achieve a minimum hydraulic efficiency of 0.5, but ideally should be designed to promote values greater than 0.7 (Figure 3).

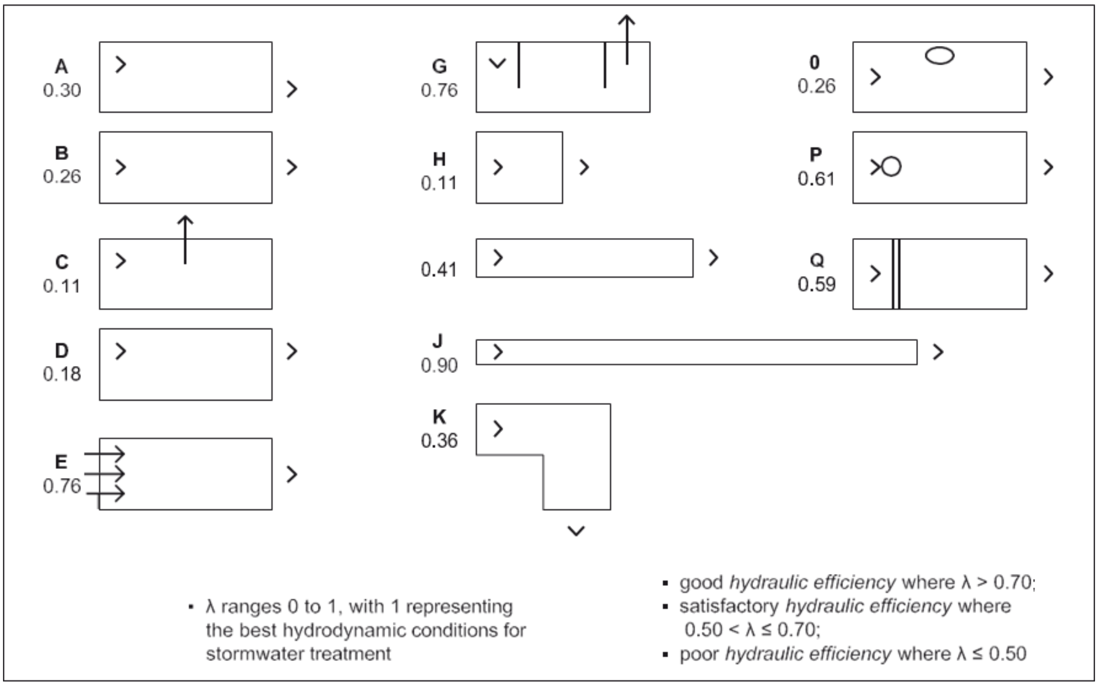

Figure 3. Hydraulic efficiency (λ) is a quantitative measure of flow hydrodynamic conditions in constructed wetlands and basins. λ ranges from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment. (Source: Engineers Australia 2006.)

Note that in Figure 3, the circles in diagrams O and P represent islands in a basin and the double line in diagram Q represents a structure to distribute flows evenly.

There can often be multiple inlets to the basin and the locations of these inlets relative to the outlet structure can influence the hydraulic efficiency of the system. Inlet structure designs should aim to reduce localised water eddies and promote good mixing of water within the immediate vicinity of the inlet.

Forebay/inlet zone

The forebay/inlet zone is a transitional zone between the stormwater outfall and the main ephemeral detention area. The function of the inlet forebay ranges from providing a sedimentation area to providing a small ephemeral wetland area that stores frequently occurring storm events.

A notional required forebay/inlet zone area can be computed by the use of sedimentation theory, targeting the 125 µm sediment (settling velocity of 11 mm/s) operating at the 1 EY peak discharge. The specification of the required area (A) for sedimentation is detailed in Engineers Australia (2006) (based on Fair and Geyer 1954) for systems with no permanent water pool:

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

Where:

$R\ =\ fraction\ of\ target\ sediment\ removed$

$V_s\ =\ settling\ velocity\ of\ target\ sediment$

$Q\ =\ rate\ of\ applied\ flow$

$A\ =\ basin\ surface\ area$

$n\ =\ turbulence\ or\ short - circuiting\ parameter$

Typical settling velocities of sediments can be estimated using the values listed in Table 2. The above expression for sedimentation is applied with ‘n’ being a turbulence parameter. Figure 3 provides guidance on selecting an appropriate ‘n’ value (according to the configuration of the basin). ‘n’ is selected using the following relationship:

$$n = \frac{1}{(1 - l)}$$

Where:

n = *turbulence or short − circuiting parameter*

λ = *hydraulic efficiency, ranging from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment.*

Table 2. Settling velocities for various particle sizes under ideal conditions

Classification of particle size	Particle diameter (µm)	Settling velocities (mm/s)
Very coarse sand	2 000	200
Coarse sand	1 000	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: Engineers Australia 2006.)

Hydraulic structures

Hydraulic structures are required at the inlet and outlet of the detention area. Their function is essentially one of conveyance of flow, with provisions for energy dissipation at the inlet structure and extended detention at the outlet.

Discharge of stormwater into the dry/ephemeral detention area may be via a forebay/inlet zone or direct input. It is essential that inflow energy is adequately dissipated to prevent localised scour in the vicinity of pipe outfalls. Design of stormwater pipe outfall structures is common hydraulic engineering practice. Litter control is normally required at the inlet structure. It is generally recommended that some form of GPT be installed as part of the inlet structure. Conveyance of flow to the detention area may be via an overland flow system, such as a swale or living stream, which will provide some pre-treatment.

Configuration of the outlet structure is largely dependent on the required operation of the system during periods of high inflows. The outlet structure typically consists of two components: the outlet pit and outlet culvert. In areas of low topographic relief, the outlet structure may consist of a single outlet culvert without an outlet pit.

The computation of the required outlet culvert is an essential element of the retarding basin design and will be based on flood routing computations, as outlined in ARR (Engineers Australia 2019).

The main function of the outlet pit is to connect the detention area to the outlet culvert. Design considerations of the outlet pit include the following:

- ensure that the crest of the pit is set at the invert of the detention area to allow the area to drain completely following a storm event;
- ensure that the dimension of the pit provides a discharge capacity that is greater than the discharge capacity of the outlet culvert;
- provide protection against clogging by flood debris.

In computing the dimension of the pit, two flow conditions need to be considered. Firstly, the weir flow condition when free outfall conditions occur over the pit (usually when the extended detention storage of the retarding basin is only partially used):

$$P = \frac{Q_d}{C_w H^{1.5}}$$

Where:

$P = Perimeter\ of\ the\ outlet\ pit\ (m)$

$H = Depth\ of\ water\ above\ the\ crest\ of\ the\ outlet\ pit\ (m)$

$Q_d = Design\ discharge\ (m^3 s^{-1})$

The second flow condition for consideration is the orifice flow condition, when the outlet pit is completely submerged (corresponding to conditions associated with larger flood events):

$$A = \frac{Q_d}{C_d \sqrt{2gH}}$$

Where:

$C_d = Orifice\ Discharge\ Coefficient\ (0.6)$

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

$A_o = Orifice\ area\ (m^2)$

$Q_d = Design\ discharge\ (m^3 s^{-1})$

The orifice flow condition provides the critical condition in terms of design capacity.

Note the above equations assume the outlet pit is freely discharging and operating under inlet control. Should outlet control conditions (i.e. backwater effects) be likely for the proposed outlet structure design, then flood routing computations are recommended.

The additional provision of an overflow route for extreme events is standard design practice to ensure that overflow from the detention area can be safely conveyed either by the use of a spillway or ensuring that any embankments are designed to withstand overtopping. This issue requires specialist design input on a case by case basis and is therefore not discussed further in this document.

Vegetation specification

Plant species for the forebay/inlet zone area will typically be predominantly ephemeral wetland species. Suitable indigenous plant species will vary depending on the location of the site. Local revegetation expertise should be sought. Suggested plant species suitable for the forebay area of detention systems on the Swan Coastal Plain and their recommended planting density are detailed in the Constructed Wetland BMP.

Vegetation within the main dry/ephemeral basin area will vary and may range from existing remnant vegetation, to grassed POS, to ephemeral wetland species (or a combination).

Maintenance

The maintenance plan should include removal of accumulated litter and debris in the detention area at the middle and end of the wet season. The frequency of this activity may be altered to meet specific site conditions and aesthetic considerations.

Biannual inspections for sediment accumulation, pest burrows, structural integrity of the outlet, and litter accumulation are typical. In parkland settings, maintenance plans should also address irrigation, nutrient and pest management issues. Accumulated sediment in the forebay should be removed about every 5-7 years or when the accumulated sediment volume exceeds 10% of the basin volume. Sediment removal may not be required in the main detention area for as long as 20 years. Refer to BMP 2.2.2 'Maintenance of the stormwater network' in Chapter 7 for further guidance on managing sediments removed from the stormwater system.

Vegetation harvesting should be timed so that it has minimal impact on factors such as bird breeding and there is time for regrowth for runoff treatment purposes before the wet season.

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5.2 Constructed wetlands

Figure 1. Liege Street Wetland, Cannington, intercepts and treats stormwater prior to it reaching the Canning River. (Photograph: City of Canning 2006.)

Figure 2. Tom Bateman constructed wetland in the City of Gosnells. (Photograph: South East Regional Centre for Urban Landcare 2006.)

Background

Constructed wetlands are vegetated detention areas that are designed and built specifically to remove pollutants from stormwater runoff. Constructed wetlands differ from constructed lakes, which are defined as constructed, permanently inundated basins of open water, formed by simple dam walls or by excavation below-ground level. For information on constructed lakes, refer to the Department of Water's *Interim Drainage and Water Management Position Statement: Constructed Lakes* (2007).

Constructed wetlands are particularly useful where stormwater contains high concentrations of soluble material that is difficult to remove with other treatment methods. Depending on their design, constructed wetlands can also serve to attenuate larger storm events and reduce peak flows, offsetting the changes to flow frequency relationships caused by increased catchment imperviousness, and protecting downstream environments from erosion and flooding. Constructed wetlands also increase flora and fauna habitat in already urbanised catchments where many natural wetlands have been cleared, drained or filled. They also provide passive recreation opportunities and can provide opportunities for educational and scientific studies.

New constructed wetlands should be designed specifically for local conditions. Deep permeable sands and a high groundwater table are common on the Swan Coastal Plain, and have typically made traditional wetland designs unsuitable. However, well-designed and well-vegetated constructed wetlands that mimic the ephemeral character of natural wetlands on the Swan Coastal Plan will provide effective water pollution filters. The more traditional constructed wetlands designs may be suitable in other parts of WA, for example, on the clay soils of the south coast region.

Constructed wetlands that expose contaminated or nutrient-rich groundwater or surface water can result in water quality problems in the wetland, reduce the treatment effectiveness of the system and result in the net export of pollutants. Poorly designed wetlands can create ideal habitats for algal blooms and midge and mosquito breeding.

Wetland vegetation provides an ideal structure for the growth of biofilms, which assimilate dissolved nutrients. Wetland plants can improve water quality by encouraging sedimentation, filtering nutrients and pollutants (through roots, stems and leaves), oxygenating their root zone, providing shade and, to some extent, by using nutrients when in the growth phase.

For more detailed information on constructed wetlands on the Swan Coastal Plain, see (the former) Department of Water and Swan River Trust’s River Science Issues 26: Constructed ephemeral wetlands on the Swan Coastal Plain – the design process (2007).

Performance efficiency

Changes in environmental conditions can greatly influence wetland processes. These include diurnal changes in water temperature and dissolved oxygen, and seasonal changes in daylight hours, water temperature, water depth, wetland vegetation growth, microbiological activity and chemical reactions. In areas with significant seasonal variation in water temperature, the treatment efficiency for a particular contaminant may vary markedly at different times of the year.

Alternating deep and shallow zones in the wetland, perpendicular to the water flow, can promote various chemical reactions to transform and remove nitrogen from the system. Shallow and ephemeral zones are generally well oxygenated, promoting mineralisation (breakdown of organic nitrogen to ammonium) and nitrification (breakdown of ammonium to nitrate). The deeper zones promote denitrification, a process occurring in the absence of oxygen, converting nitrate to gaseous nitrogen, which is then released to the atmosphere (Department of Water and Swan River Trust, 2007). However, the deeper zones should not cut into the groundwater table to remain wet year-round because this can create stagnant ponds and result in water quality problems, as discussed in the Background section of this BMP.

Phosphorus can be removed through sedimentation, filtration, biological uptake and sorption. Suspended material can be removed from the water column by sedimentation and filtration. Organic matter can be removed through sedimentation/filtration and degradation and microbial uptake. Pathogens can be destroyed by exposure to ultra violet light in open waters, adsorption and predation. Even some heavy metals can be removed from the water column through sedimentation, adsorption and plant uptake. However, high levels can be toxic to plants and animals and may have an adverse impact on the wetland (Department of Water and Swan River Trust, 2007).

It is also clear that treatment efficiency for some contaminants is influenced by the maturity of the wetland, with new wetland soils sometimes having a higher assimilation capacity for phosphorus and nitrogen than older wetland soils. The accumulation of organic matter from dead plant material also influences the soil pollutant interactions. Higher wetland vegetation density is likely to achieve greater treatment efficiency than lower density because of the increased contact between contaminants and plant surfaces that support microorganisms, which mediate most removal processes. Fringing wetland vegetation supports the growth of epiphytic biofilms, which are a matrix of bacteria, fungi and algae that assimilate dissolved nutrients. However, high-density vegetation may create suitable mosquito and midge breeding conditions if not designed appropriately. The Midge Research Group’s (2007) *Chironomid Midge and Mosquito Risk Assessment Guide for Constructed Water Bodies* provides guidance on how to plant vegetation to reduce the risk of mosquitoes and midges.

Indicative estimates of treatment efficiency for constructed wetlands based on Fletcher et al. (2004) are shown in Table 1. Actual treatment efficiencies will depend on the hydraulic efficiency and the design of the wetland.

The hydraulic effectiveness of a wetland reflects the interaction of three factors: detention period, inflow characteristics and storage volume. It defines the overall percentage of catchment runoff introduced to the wetland for treatment. As a general rule of thumb, the area of a constructed wetland should typically be approximately 1–2% of the total catchment area in order to be effective, otherwise excessive hydraulic loading and short-circuiting is likely to reduce its biofiltration effectiveness. However, this approximate figure should only be used for preliminary wetland sizing. Catchment characteristics (e.g. land use and water quality) will determine the wetland size.

Table 1. Typical annual pollutant load removal efficiencies for constructed wetland

Pollutant	Expected removal	Comments
Litter	> 95%	Subject to appropriate hydrologic control. Litter and coarse organic matter should ideally be removed in an aerobic environment prior to the wetland, to reduce potential impacts on biological oxygen demand.
Total suspended solids	65–95%	Depends on particle size distribution.
Total nitrogen	40–80%	Depends on speciation and detention time.
Total phosphorus	60–85%	Depends on speciation and particle size distribution. Will be greater where a high proportion of phosphorus is particulate.
Coarse sediment	> 95%	Subject to appropriate hydrologic control.
Oil and Grease	n/a	Inadequate data to provide reliable estimate, but expected to be >75%.
Faecal Coliforms	n/a	Inconsistent data.
Heavy metals	55–95%	Quite variable, dependent on particle size distribution, ionic charge, attachment to sediment (vs % soluble), detention time, etc.

Source: Fletcher et al. (2004)

Cost^

Costs for constructing wetlands can vary greatly depending on the configuration, location, site-specific condition (including hydrogeology, temporal patterns and seasonal temperature variations), volumes, flow rate and pollutant removal targets.

There is little available cost data for constructed wetlands in WA. Typical construction costs presented in Weber (2002), cited in Taylor and Wong (2002) and based on eastern states examples, range from approximately $500,000 to $750,000 per wetland hectare. The two key variables underpinning the construction costs are the extent of earthworks required and the types and extent of vegetation.

The Center for Watershed Protection (1998), Weber (2001) and United States Environmental Protection Agency (2001), cited in Taylor (2005), reported annual maintenance costs of approximately 2% of construction costs.

^The costs quoted in this section are from around 2000 to 2005 and have not been adjusted for inflation or potential cost changes which may have occurred since this chapter was published in 2007. Therefore, it should be considered as indicative only and users of the manual are encouraged to seek further specific industry advice on the current costs as appropriate.

Design considerations

Before the commencement of site investigations or the design process, the objectives for the constructed wetland must be established. Objectives include environmental benefits (such as water quality improvement, detention and erosion control), habitat value (enhancing biodiversity and conservation) or aesthetic and recreational values (Department of Water and Swan River Trust, 2007).

Constraints for the wetland must be identified and considered. On the Swan Coastal Plain, these constraints are likely to include (Department of Water and Swan River Trust, 2007):

- land availability, including future land use plans
- types and form of pollutants (e.g. dissolved nutrients, gross pollutants, toxicants and sediment)
- pollutant delivery (e.g. mostly diffuse; baseflows; first flush events; and timing of pollutant arrival)
- geology (e.g. very sandy soils or presence of bedrock)
- hydrology (e.g. frequently high groundwater table)
- topography (e.g. very flat or steep site)
- site-specific constraints (e.g. environmental, conservation and heritage issues, neighbouring land uses)
- location of service infrastructure (e.g. roads, sewerage, scheme water and gas pipelines, and telephone and power lines)
- end use of the treated water (e.g. delivery into downstream waterways or reuse as irrigation water).

Subject to the outcomes of constraints analysis, a site investigation will be required to determine whether a constructed wetland is the appropriate technique for the site. A site investigation typically includes the following (summarised from Department of Water and Swan River Trust, 2007):

- Topographical Survey: Preliminary information onsite topography can be extrapolated from aerial photographs and topography maps, however a detailed topography survey is required at intervals of 0.1 m to 0.5 m. The aim of this survey is to identify any constraints that may impact the wetland design.
- Groundwater Monitoring: Regular monitoring of shallow groundwater bores (less than 5 m below ground surface) provides a good indication of groundwater levels and quality where shallow groundwater is present. The bores should be monitored at least quarterly for physical parameters (dissolved oxygen, redox potential, conductivity, pH and temperature), nutrients and groundwater elevation for at least one year. This will help to establish water table fluctuations and seasonal changes in groundwater quality. Groundwater elevation data can then be collated to determine groundwater contours across the site and general groundwater flow direction, which may not follow topography. Analysis of groundwater quality will identify any hot spots caused by a contaminated groundwater plume or historical land use activities. See the Water Quality Protection Note: *Groundwater Monitoring Bores* (Department of Water 2006) for more information on bore installation.
- Geotechnical Survey: Drilling of bores can provide further information on soil horizons. Physical properties of the soil impact the success of plant establishment.
- Acid Sulfate Soils: Areas that have a high risk of forming ASS, or where dewatering in medium and low risk areas could affect soils in high risk areas, must be considered in designing constructed wetlands. These areas are defined in ASS risk mapping available from the Department of Water and Environmental Regulation and using the free Maps Viewer on the DATA WA website: catalogue.data.wa.gov.au/dataset/acid-sulfate-soil-risk-map-100k-dwer-048. Land development proposals within these areas should undertake site-specific soil investigations and prepare ASS management plans and, where relevant, dewatering management plans. For more information about planning for ASS see the Department of Planning, Lands and Heritage website: www.dplh.wa.gov.au/policy-and-legislation/state-planning-framework/fact-sheets,-manuals-and-guidelines/acid-sulfate-soils-planning-guidelines

- Surface Water Hydrology Monitoring: If existing surface water flows enter the wetland, then monitoring of these flows and water quality is recommended.
- Vegetation Survey: The existing native vegetation species onsite and in similar wetland types in the surrounding region should be identified and incorporated into the wetland design.

Typically, constructed wetlands are most appropriate on sites that meet or exceed the following criteria:

- impervious catchment area should be greater than 1 hectare
- soils are relatively impermeable or have sufficient base flow passing through them to sustain vegetation (unless selected vegetation can sustain long dry periods)
- must be situated on mild slopes or where slope stability is not an issue
- land availability is not significantly restricted to accommodate the detention volume.

The most important criterion for determining the success of a constructed wetland system is the local hydrology (Figure 3). It is therefore imperative that these systems be located in areas that have suitable hydrologic characteristics to ensure the long-term viability of wetland processes.

The hydrologic regime of the constructed wetland has a significant impact on its ability to assimilate nutrients. Ideally, the wet and dry season flows and when the pollutants are delivered to the wetland and their concentrations should be known. In addition, it is important that the water quality of the inflow (surface and groundwater) is known, as this determines the size of the wetland and influences the design. Large wetlands are more successful at removing sediment and the nutrients attached to sediment, while wetlands with alternating bed depths are more suitable for removing dissolved nutrients by the nitrification/denitrification process and biofilm growth.

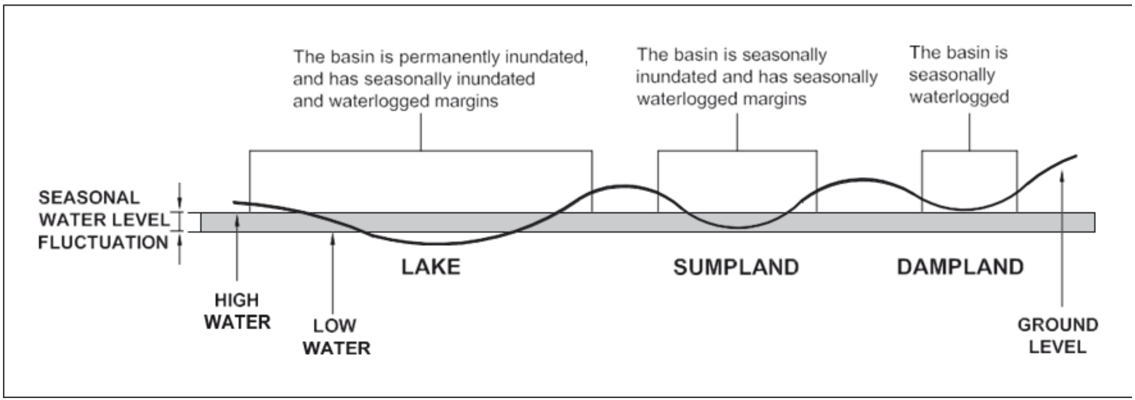

Figure 3. Natural wetlands are classified according to their landform and water permanence (Semeniuk and Semeniuk 1995). Constructed wetlands should mimic the natural hydrology and landforms suitable for the catchment.

Soils at the proposed site for a constructed wetland must have sufficient water retention characteristics and be able to promote wetland plant growth, particularly during the dry season. Wetland vegetation requires a suitable soil profile from the ground surface to below the static water level. It may be necessary to stockpile topsoil during construction and re-spread this soil along the base and side slopes of the wetland.

The utilisation of the existing site contours can reduce potentially costly earthworks involved in construction.

The proximity of the wetland to residential areas needs to be considered in the selection and design of this BMP. Neighbouring communities will need to be consulted on the appearance, functionality and role of the constructed wetland. There are also safety concerns where the wetland is built in a publicly accessible area.

Mosquito and midge breeding can be a problem if the wetland is poorly designed. According to the Midge Research Group of Western Australia's (2007) *Chironomid Midge and Mosquito Risk Assessment Guide for Constructed Water Bodies*, ephemeral water bodies (i.e. where the water level fluctuates and the water body dries out) generally present the lowest risk for mosquitoes and midges. Refer to the Mosquito and Midge Management guidelines section of this BMP for more information.

Design guidelines

The following design guidelines are based on Department of Water and Swan River Trust's *Constructed Ephemeral Wetlands on the Swan Coastal Plain – the design process* (2007). Unless constructed wetlands are appropriately located, designed and managed, they can produce significan populations of mosquitoes and chironomid midges, with subsequent impacts on surrounding residents. Therefore, where possible, the water body characteristics with a lower risk rating detailed in the Midge Research Group Western Australia's (2007) *Chironomid Midge and Mosquito Risk Assessment Guide for Constructed Water Bodies* should also be incorporated.

Key design principles

For wetlands to be a successful part of the treatment train, the following key design principles are recommended (adapted from Department of Water and Swan River Trust, 2007):

- **Design to minimise mosquito and midge risk.** For example, the *Chironomid Midge and Mosquito Risk Assessment Guide for Constructed Water Bodies* (Midge Research Group of Western Australia 2007) states that a buffer of more than 200 m provides the lowest risk from mosquitoes and midges. Some of these lower risk design features include seasonal drying out of the wetland, buffer zones, construction of smooth edges to prevent formation of stagnant, shallow pools and alignment of the long axis of the wetland parallel to the prevailing wind direction.
- **Integrate the wetland with local conditions and design for the inputs.** It is beneficial for the wetland to be incorporated into the hydrology, natural surface contours, existing vegetation and drainage lines of the site. The size of the wetland and its design is determined from the inflow water quality and it is essential that the types and loads of pollutants are identified.
- **Correctly size the wetland.** The sizing of wetlands must be done correctly to account for the volumes of water entering the system, while reducing the risk of stagnant water. The hydroperiod (time of inundation) should determine the dimensions of the system.
- **Ensure flow velocities remain low.** High flows entering a wetland can re-suspend the accumulated sediment and nutrients, resulting in detrimental impacts downstream. The system should be designed to include controls to reduce the inflow velocity or to bypass high flows.
- **Incorporate deep inlet zones.** A deep inlet zone providing a settling area can reduce large amounts of sediment from entering the wetland, which prevents clogging of vegetation and reduction of water depth.
- **Design wetland bathymetry and vegetation layout for variations in hydrology, including promoting shallow water areas and wetting and drying cycles**. Alternating deep and shallow zones perpendicular to the flow in the wetland can promote various chemical reactions, such as nitrogen removal (Figure 4). Seasonally dry zones promote aeration of the sediments and reduce mosquito breeding risks.

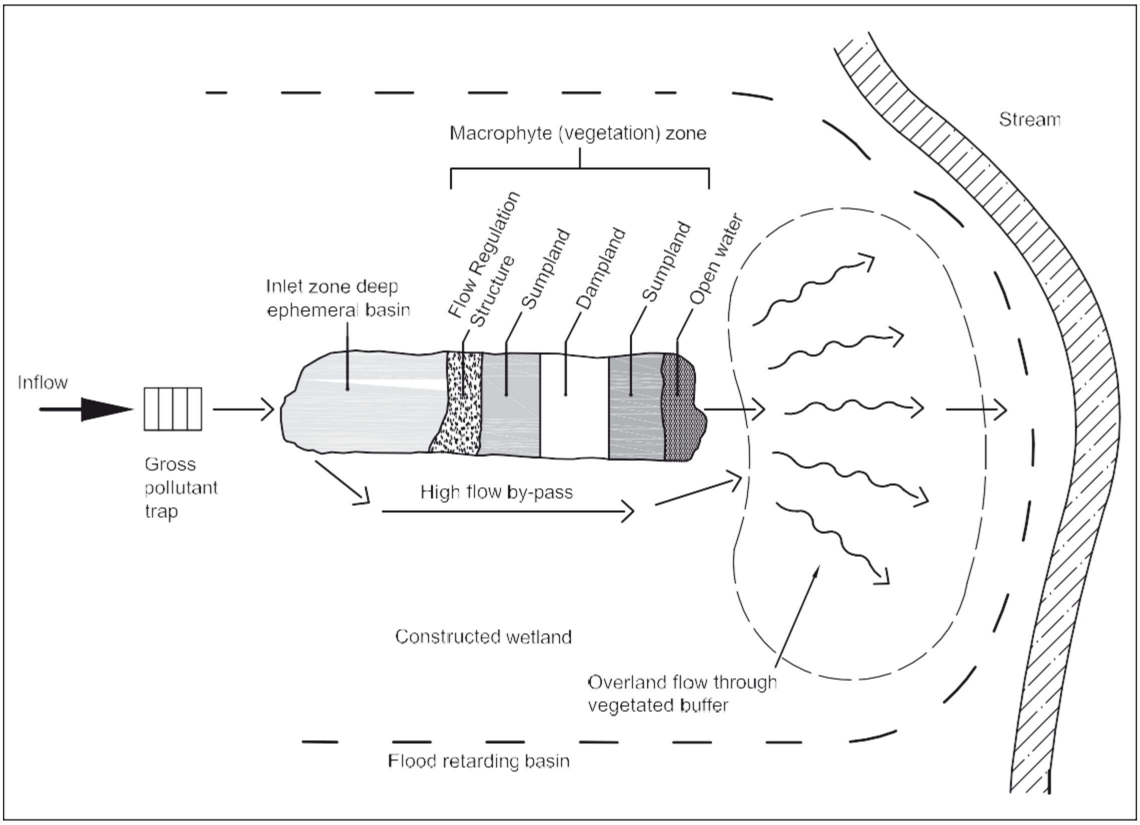

Figure 4. Bathymetry of a constructed wetland.

- **Create gentle sloping shorelines.** Banks should be designed with slopes of 1:6 to 1:8 to allow for public safety and create wider ranges of zones for plant growth. The banks should not be flat or contain depressions that can inhibit drainage, creating stagnant water and mosquito breeding areas.
- **Maximise vegetation-water contact by creating dense vegetation stands within the wetland.** Wetlands should be designed to incorporate large sections of densely vegetated zones, as vegetation aids water treatment by slowing flows and promoting sedimentation. Maximising water contact with biofilm growth on plant surfaces aids the removal of nutrients and other pollutants. Specialist advice on selecting suitable in-stream species is recommended.
- **Vegetate around the wetland's edge.** Fringing vegetation serves as a buffer for the wetland, capturing nutrients and pollutants in overland flow, preventing erosion and limiting weed and algal growth. Fringing vegetation provides shade and water temperatures more conducive to nutrient processing.
- **Limit the use of lawn and avoid using fertilisers and pesticides**. It is essential that wetlands are not surrounded by highly fertilised lawns, as this may result in the direct application or runoff of nutrients to the wetland. Native plants are a more suitable alternative because they can be cost effective, require little or no fertiliser, little watering (except in establishment phases) and provide habitat for fauna.
- **Design as part of a treatment train and use of source controls.** Constructed wetlands should form part of a treatment train for water quality improvement. Gross pollutants can clog wetlands, and heavy metals and other chemicals can impact on the growth of wetland plants and their associated biofilms. Pre-treatment systems and implementation of non-structural controls throughout the wetland's catchment are required to prevent these pollutants entering the wetland.

Key components

The key components of the constructed wetland system include the inlet, channel, basin, floodplain and outlet, as shown in Figure 5.

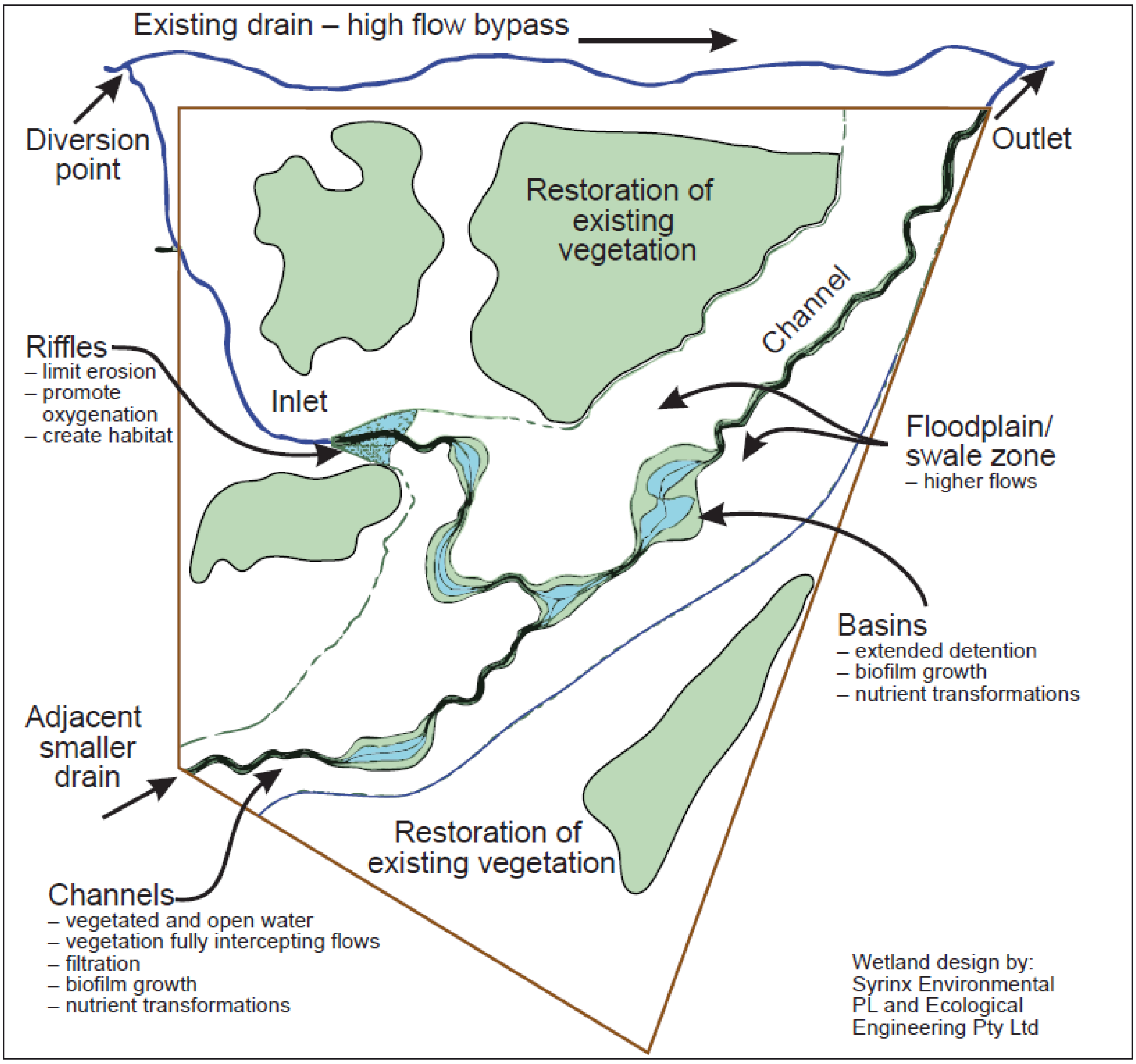

Figure 5. Example of a constructed wetland design, showing key components of an ephemeral wetland. (Department of Water and Swan River Trust,2007)

Inlet

Flow velocities in the wetland need to be managed by careful consideration of all stage heights and flows when designing the geometry. A deep inlet or similar system should be used to attenuate high flows. A diversion structure may be required to ensure that potentially damaging above-design high flows bypass the wetland. However, it is important that this structure is designed to allow normal storms and first flush events to enter the wetland for treatment.

Channels/creeklines and basins (low flow areas)

Creeklines (channels) and basins form the main treatment area for low flows. Channel widths should vary in an effort to move away from a linear-type drainage line. Shallow, wide, meandering streams with a series of basins (both vegetated and open water) are often very effective constructed wetlands as they increase detention times and create a more diverse range of habitats, both of which can promote nutrient removal processes. Average channel cross-sectional areas should be designed to provide sufficient volume to account for increases in hydraulic roughness (or surface friction) over time due to the establishment and growth of in-stream vegetation.

Channels and basins may need to be lined with a less permeable (e.g. clay) layer to reduce groundwater interactions and to achieve the water quality treatment objectives. The invert or base of the channel or basin should be above maximum groundwater levels; however, capillary action will allow soils to stay wet almost year-round in shallow groundwater areas and thus maintain vegetation during dry seasons.

Channels can incorporate three main components:

- Open sections are best located in flat areas of the wetland.
- Vegetated sections – about 70% of total channel area should be vegetated in-stream (i.e. less than 30% of the channel should be open water), fully intercepting flows to maximise settling, biofilm and plant uptake and microbial assisted nutrient transformations.
- Riffles (loose rock structures built in-stream) – can be constructed in sections of the channel that are steeper to reduce the risk of erosion. Riffles promote oxygenation of water (necessary in the breakdown of ammonium) and create additional macroinvertebrate habitat (such as habitat for filter feeders).

Floodplain (high flow areas)

The floodplain area attenuates higher flows (i.e. less frequent, large storm events), controlling flow velocities. Swales and floodplains can treat both overflow from baseflow channels and rising groundwater. Water inundating the floodplain area can either infiltrate into the groundwater or flow back into the channel as flows recede. A slope and/or soil permeability that allows for the high-flow waters to either recede back into the channel or infiltrate into the groundwater is necessary. To reduce the risk of mosquito breeding, there should be no surface water within floodplain areas within four days of being inundated (within high risk areas or mosquito breeding risk times of the year).

See the Swales and Buffer Strips BMP for more design guidelines on swales. The function and structure of the wetland components are outlined in Table 2.

Outlet structure

The outlet structure acts to control discharges from extended detention areas to ensure that sufficient detention time for biological processes has been achieved.

Outlet structures will typically consist of weir and/or riser type arrangements, designed to provide uniform detention time over the full range of the extended detention depth. The placement of riser outlet orifices and their diameters is designed using an iterative process by varying outlet orifice diameters and levels at discrete depths over the length of the riser up to the maximum detention depth.

Table 2. Components of a constructed wetland (former Department of Water and Swan River Trust, 2007)

Zone	Functions	Structure
Inlet	Buffer the wetland from high flows Encourage sedimentation	Piped inlets Diversion channel into wetland
Channel	Encourage filtration and sedimentation Evenly distribute flow and reduce flow velocity to promote further filtration and sedimentation Promote biological transformations and uptake Provide habitat for invertebrates	>100 mm clay layer (if required) 200 mm topsoil over clay layer Gently sloping embankments to reduce erosion Emergent plants with extensive shallow roots, planted perpendicular to flow path in low flow section Riffles in high flow/steep sections
Basin	Provide additional detention volume Promote pollutant transformation Promote biofilm growth Reduce flow velocity Treat localised groundwater, if groundwater interception already occurs; however, constructed wetlands at new sites should not artificially expose groundwater Provide habitat and refuge for fauna, especially invertebrates Provide diverse habitat for flora	>100 mm clay layer (if required) 200 mm topsoil Gently sloped embankments Offset inputs and outputs of flow path to avoid short-circuiting and increase hydraulic retention time Various depths between and within basins to create habitat
Swale/ Floodplain	Detain wet season flows Optimise water treatment using dense vegetation with dense shallow root systems for biofilm creation Increase adsorption, sedimentation and permeability and infiltration to groundwater through use of clay-sand semi-permeable topsoil cover and dense vegetation Promote biofilm growth for bacterial transformations Create habitat for bacteria, fungi and fauna	200 mm topsoil Dense vegetation cover to encourage sedimentation and reduce erosion Gentle swale gradients to reduce scouring and increase vegetation diversity
Outlet	Control stormwater detention time/water levels Can allow for effective gauging	Riser pit or V-notched weir

The target maximum discharge is computed as the ratio of the volume of the extended detention to the required detention time:

$$Target\ Max\ Discharge\ (m^{-3}s^{-1})\ =\ Extended\ Storage\ Volume\ (m^{3})/Detention\ Time\ (s)$$

The orifice areas and placement required to achieve the target maximum discharge rate can then be calculated using the orifice discharge equation as follows (if the outlet system will operate under inlet control conditions):

$$A = \frac{Q_d}{C_d\sqrt{2gH}}$$

Where:

$C_d\ =\ Orifice\ discharge\ coefficient\ (0.6)$

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

$A_o\ =\ Orifice\ area\ (m^2)$

$Q\ =\ Flow\ rate\ to\ drain\ out\ the\ extended\ detention\ area$

The weir equation can then be used to define the required perimeter (and thus dimension) of the riser outlet for discharge of larger events in excess of the riser orifice capacity:

$$P = \frac{Q_t}{C_w H^{1.5}}$$

Where:

$P\ =\ Perimeter\ of\ the\ riser\ outlet\ pit\ (m)$

$H\ =\ Maximum\ required\ design\ depth\ of\ water\ above\ the\ crest\ of\ the\ outlet\ pit\ (m)$

$Q_d\ =\ Design\ discharge\ (m^3s^{-1})$

$C_w\ =\ Sharp\ crested\ weir\ coefficient\ (1.7)$

In order to ensure the riser outlet will operate efficiently, it is important that the orifices are prevented from clogging up. Debris guard examples are shown in Figure 6.

Figure 6. Debris guard examples for riser outlets. (Source: Morton Bay Waterways and Catchments Partnership 2006.)

Any areas of the wetland requiring regular drainage for maintenance should contain a manually operated drain or bypass structure, allowing water to temporarily bypass the area of the wetland being maintained. Depending on maintenance requirements, this may require a separate outlet of different capacity to the outlet structure described above.

Vegetation design

Remnant vegetation areas should be retained and/or restored in keeping with the objectives of the constructed wetland. Choice of vegetation species for each zone depends on the expected hydroperiod and the substrate. The recommended vegetation types for each hydrologic zone for wetlands on the Swan Coastal Plain are outlined in Table 3. For some more specific information on vegetation types, the Perth Biodiversity Project has established reference sites for different vegetation communities, including wetland types, on the Swan Coastal Plain (go to walga.asn.au/). In accordance with the Decision process for stormwater management in WA (Department of Water and Environmental Regulation 2017), constructed wetlands should not artificially expose groundwater. Therefore if necessary, ephemeral plants should be chosen over species that require permanent inundation. Plant structures should be fairly open to allow passage of water and optimise sunlight penetration and biofilm growth, yet have dense surface roots.

Planting densities depend on lead time before stormwater enters the system, planting season and weed risk. However, a general density of 4 plants/m2 is recommended for channel and basin areas. Planting densities need to be high to reduce weed competition and minimise ongoing maintenance costs. Planting densities can be lower for floodplain areas, where an average 3 plants/m2 is recommended. Plants should be planted in rows perpendicular to the flow path, with each row offset from the previous to minimise short-circuiting and the creation of preferential flow paths.

In WA's south-west, the ideal time to plant low flow channel and basin areas is in early to mid spring when plant growth is at its optimum. Floodplain areas should be planted in winter unless the area will be irrigated. The success of planting is critical for the establishment phase. Water levels in the wetland may need to be manipulated to ensure that soils are saturated for at least eight weeks after planting or until seedlings exceed 200 mm in height. It is critical to allow time for plants to establish themselves before the wetland becomes fully operational.

Table 3. Vegetation types for wetlands on the Swan Coastal Plain (adapted from Department of Water and Swan River Trust, 2007)

Vegetation Zone	Vegetation Types	Examples
Channel and shallow permanent basins	Closed sedgeland and rushes	*Eleocharis acuta, Baumea juncea, Baumea articulata, Juncus kraussii*
Near-permanent basins	Scattered sedgeland with submergents	*Triglochin huegelii, Villarsia* spp, *Schoenoplectus validus*
Lower swale/floodplain	Melaleuca woodlands	*Melaleuca rhaphiophylla, Melaleuca preissiana*
Upper swale (dryland)	Closed rushland, sedgeland and heathland	*Melaleuca preissiana, Kunzea ericifolia, Baumea juncea*

Mosquito and midge management

If a constructed wetland scores a 'low risk' according to the *Chironomid Midge and Mosquito Risk Assessment Guide for Constructed Water Bodies* (Midge Research Group of Western Australia 2007), then it is likely that minimal monitoring and maintenance for chironomid midges and mosquitoes would be required. In other types of constructed wetlands, regular monitoring, including larval and adult trapping, should be undertaken to determine if control/treatment options are necessary.

Mosquito and midge control are achieved by adopting a composite methodology, known as integrated control, involving various complementary techniques that are designed to reduce the mosquito habitat or make it unsuitable, as well as encourage biological regulation of the mosquitoes, and thus limit or even eliminate the use of pesticides.

Water levels within the constructed wetland will vary through the natural fluctuation of the groundwater level and surface inflow. Drying out the constructed wetland will achieve mosquito control by interrupting the breeding cycle of mosquito larvae in the sedge bed zone.

Deep-water zones within the constructed wetland will generally be free from surface vegetation. Such areas do not support large populations of mosquitoes, at least in the long-term; mosquito populations that colonise the deep-water areas are eventually controlled (but not necessarily eliminated) by predation, physical disturbance, or depletion of food resources.

The long axis of the wetland should be parallel to the prevailing wind direction, which is the direction most common during spring/summer (Midge Research Group of Western Australia 2007). The construction of smooth rather than irregular edges and surfaces to the constructed wetland will help prevent the formation of stagnant pools. Stagnant pools may form within the marsh and ephemeral zones of the wetland that will create an environment conducive to mosquito breeding. Creation of these pools is minimised by creating a slope or installing permeable soil that prevents water ponding. For these areas, monitoring of mosquito populations should be undertaken and other management measures, such as chemical treatments, may need to be considered.

Mosquito control at breeding sites may be achieved by applying chemical larvicides and by introducing biological agents into the habitat. Some control agents can be toxic to other species, in particular frogs, turtles, fish, birds and invertebrates. Prolonged use can also lead to the development of resistance in the mosquito population. Chemical control should therefore not be viewed as a long-term strategy, but rather as a short-term response to episodes of heavy breeding. See the Department of Health (2019) *Mosquito Management Manual* for more guidance.

The constructed wetland should be regularly monitored to assess mosquito production and to assess the action to be taken if monitoring indicates an increase in mosquito populations. Both adult and larval mosquitoes should be monitored.

For detailed advice on how to reduce the risk of mosquitoes, see the Department of Health (2019) *Mosquito Management Manual* and the Midge Research Group of Western Australia (2007) *Chironomid Midge and Mosquito Risk Assessment Guide for Constructed Water Bodies*.

Maintenance

To determine whether the wetland is performing as expected, a monitoring program detailing hydrology and the water quality of inflow and outflow is recommended. At a minimum, the following monitoring should be undertaken (Department of Water and Swan River Trust, 2007):

- Monitoring of surface water levels and flow pathways and groundwater levels in the wetland is necessary to ascertain whether the actual wetland hydrology matches that of the design intent
- Monitoring of the inflow and outflows for TSS and nutrients should be undertaken in low flow and high-flow periods.

A detailed maintenance plan must be developed that specifies short and long-term maintenance of the wetland. For simple wetlands, the plan may only need to specify how often to maintain and inspect the banks, when to inspect inlet and outlet structures for signs of clogging and when to remove sediment. More complex wetland designs with mechanical devices, such as valves or pumps, may require much more detailed maintenance plans, including manufacturers' maintenance recommendations.

The maintenance plan should include the following:

- Schedule biannual inspections and conduct inspections after major storm events. Initially, determine if the constructed wetland is working according to design by looking for signs of bank erosion, excessive sediment deposits or plant deterioration. Routine inspection should include checking for clogged or damaged structures and inspecting and testing any mechanical structures such as gates, valves or pumps. Inspections should also include looking for the formation of any isolated pools on the wetland profile. These inspections should also include monitoring for mosquito larvae and undertaking mosquito control if and when required.
- Clear overgrown vegetation from access roads to ensure accessibility to the constructed wetlands for maintenance purposes.
- Remove environmental weeds, particularly those that are invasive.
- Remove accumulated sediment in the inlet zone and regrade approximately every 5–7 years or when the accumulated sediment volume exceeds 10% of the available volume. Sediment removal may not be required in the main pool area for as long as 20 years. Accumulated sediment removed from constructed wetlands should be assessed to determine the risk associated with contamination (e.g. from heavy metals, nutrients and hydrocarbons) so that appropriate steps can be undertaken to treat and dispose of the contaminated materials. Refer to BMP 2.2.2 ‘Maintenance of the stormwater network’ in Chapter 7 for further guidance on managing sediments removed from the stormwater system.
- Remove accumulated litter and debris at the middle and end of the wet season. The frequency of this activity may be altered to meet specific site condition and aesthetic considerations.
- Harvesting (periodic annual or semi-annual cutting and removal of wetland vegetation) may be necessary to maintain the wetland’s soluble nutrients and pollutants removal capacity. Annual vegetation harvesting best occurs in the dry season, as it is generally after the bird breeding season, and there is time for regrowth for runoff treatment purposes before the wet season.
- Revegetation to keep density as prescribed in the vegetation plan, including replacement of dead wetland plants with plants of equivalent size and species.
- Monitoring of the surface water hydrology, groundwater levels, and TSS and nutrients of inflow and outflow is recommended to determine whether the wetland is performing as it was designed.
- Records should be kept of monitoring and maintenance activities.

Case studies

There are a number of successful local examples of constructed wetland projects. Chapter 6: Retrofitting of this manual provides a detailed example of a constructed wetland project where existing linear drains were recreated as a wetland. The Liege Street Wetland in Cannington (section 7.6, Chapter 6) aims to treat nutrient-enriched stormwater and groundwater from three drains prior to discharging to the Canning River (figures 1 and 2).

Figure 7. Black Creek trapezoidal drain converted to a constructed wetland, City of Canning. (Photographs: (left) JDA 1999, (right) Water and Rivers Commission 2003.)

Another retrofitting example is the Black Creek Wetland, which replaced an existing trapezoidal drain in Cannington (Figure 7). The aim of the constructed wetland is to provide an area for sedimentation of particulate material from the industrial Black Creek catchment, as well as provide improved habitat to cater for the needs of various waterbirds.

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6 Pollutant control

6.1 Litter and sediment management

Figure 1. GPT upstream of Liege Street Wetland, Cannington. (Photograph: Department of Water 2004.)

Figure 2. Maintenance of a litter and sediment trap, upstream of Lake Goollelal, Kingsley. (Photograph: Department of Water 2006.)

Background

Litter and sediment management (LSM) systems are primary treatment measures that retain gross pollutants by physical screening or rapid sedimentation techniques. Gross pollutants generally consist of litter, debris and coarse sediments.

Litter includes human derived rubbish, such as paper, plastic, styrofoam, metal and glass. Debris consists of organic material, including leaves, branches, seeds, twigs and grass clippings. Coarse sediments are typically inorganic breakdown products from sources such as soils, pavement or building materials. Gross pollutants are defined as debris items larger than 5 mm (Allison et al. 1997) and coarse sediments are defined as grain sizes greater than 0.5 mm diameter. Some of these pollutants are a threat to wildlife, degrade aquatic habitats, reduce aesthetic qualities, leach harmful pollutants and attract vermin.

Through the implementation of WSUD in stormwater management, the requirement for LSM devices has been significantly reduced, particularly due to the focus on retention of stormwater at-source and ‘disconnecting’ pollutant transport pathways. Non-structural control methods, such as litter collection programs, strategic bin design and placement, sediment controls on construction sites, street sweeping and minimising the use of deciduous plants in streetscape landscaping, also have significant potential to reduce litter and sediment inputs to the stormwater system. Refer to Chapter 7 for guidelines on these non-structural management measures.

Sediment can also be trapped using filtration techniques, such as buffer strips and swales, and infiltration and detention systems, such as infiltration basins and constructed wetlands, as described in BMPs 4.1, 3.1 and 5.2 in this chapter (refer to Table 2 in the Chapter 9 Introduction). These methods can also retain fine particles. Where implemented at-source, filtration and infiltration methods have the advantage of separating pollutants prior to being carried by flows into the stormwater system, thereby avoiding the difficulties associated with separating pollutants entrained in the flow.

Nevertheless, LSM systems still have a role to play in stormwater management to complement non-structural controls and as pre-treatment to other measures, such as constructed wetlands and bioretention systems, where upstream characteristics warrant their use.

LSM systems can be aesthetically unobtrusive as they require a relatively small footprint and can be situated below ground. They are suited for retrofitting to an existing piped drainage system, particularly in highly urbanised areas, and targeting specific problem areas with high loads of gross pollutants. Use of these systems assists in preventing blockages of drains and other conveyance based systems.

There are six commonly used LSM systems in Australia. These range from at-source treatment for the upper reaches of the catchment (e.g. side entry pit traps) to those intended for slow-moving waterways (e.g. litter booms) further down the catchment (Allison et al. 1997):

- **Side entry pit traps** are baskets fitted below the entrances to stormwater systems from road and carpark gutters. When stormwater passes through the basket into the side entry pit, material larger than the basket mesh (typically 5–20 mm) is retained. This material remains in the basket until it is cleaned out during required regular maintenance.
- **Litter control devices** are baskets sitting below the entry point of the inlet pipe. Water entering the baskets flows out through the openings, while debris larger than the pore size is retained. As debris builds up, it reduces the pore sizes, allowing smaller material to be caught.
- **Trash racks** consist of vertical or horizontal steel bars, typically 40–100 mm apart, fitted across stormwater channels or inlet and outlet pipes to receiving water bodies. When water passes through the trash rack, it retains material larger than the bar spacing. As material builds up behind the rack, finer material may also be collected.
- **Gross pollutant traps (GPTs)** typically consist of a sediment trap with a weir and trash rack at the downstream end. Flows enter a large typically concrete lined basin and are detained in the basin by a weir, decreasing flow velocities and encouraging sedimentation. The trash rack collects debris from flows overtopping the weir. GPTs servicing small catchments can be located below ground. These devices typically use a series of underground chambers, weirs, screens or baffles to control flows and trap sediments. An alternative below-ground system is a continuous deflective separation (CDS) device, which operates by diverting the incoming flow of stormwater and pollutants into a chamber that has a circular screen that induces a vortex to keep pollutants in continuous motion, preventing solids from ‘blocking’ the screen. The secondary flows induced by the vortex concentrate sediment in the bottom of the unit. Water passes through the screen and flows downstream.
- **Floating debris traps**, or litter booms, are made by placing partly submerged floating booms across waterways to trap highly buoyant and visible pollutants such as plastic bottles. The booms collect floating objects as they collide with it. Newer designs use floating polyethylene boom arms with fitted skirts to deflect floating debris through a flap gate into a storage compartment. Floating booms are not suited to fast moving waters. Additionally, the traps miss most of the gross pollutant load because only a small fraction of gross pollution remains buoyant for a significant length of time.
- **Sediment basins** may be concrete lined, or built as more natural ponds excavated from the site soils and stabilised with fringing vegetation. The basins consist of a widening and/or deepening of the channel so that flow velocities are reduced and sediment particles settle out of the water column. Macrophytes planted in and around the basin will assist in minimising the risk of sediment re-suspension. A pervious rock riffle or weir at the outlet may also assist filtering the water and preventing the conveyance of sediment downstream. Sediment basins are often used as pre-treatment to remove coarse sediment prior to flow entering a constructed wetland system.

Other types of litter and sediment traps include (Victoria Stormwater Committee 1999):

- **Grate and side entrance screens**, which consist of metal screens that cover the inlet to the drainage network and prevent large litter items from entering and blocking the drain.

- **Baffled pits**, where a series of baffles are installed in a stormwater pit to trap floating debris and encourage sediments to settle.
- **Circular settling tanks**, consisting of a cylindrical concrete tank installed below ground that is divided into an upper diversion chamber and a lower retention chamber. A diversion weir at the inlet directs stormwater into the lower retention chamber where sediment settles to the base of the chamber. Flow exits the chamber through a riser pipe. The inlet and outlet pipes are set at the same level, trapping some oil in the retention chamber.
- **Boom diversion systems**, where a floating boom diverts all low to medium flows into a screened offline pollutant collection chamber, such as a baffled unit. Under high flow conditions, the boom floats and only deflects floating debris into the chamber, while the majority of flow passes under the boom and bypasses the trap.
- **Release nets** consist of a cylindrical sock made of netting that is secured over the outlet of a drainage pipe and captures all material larger than the pore size of the net. If the net becomes blocked or full, a mechanism is triggered to release the net from the pipe. The net moves downstream until it reaches the end of a short tether attached to the side of the drain that constricts the net opening and prevents the trapped pollutants from being released.

Performance efficiency

Manufacturers have developed a range of proprietary products designed to trap and separate litter and sediment from stormwater runoff. Most of these products have not been extensively independently tested in the field.

Removal efficiencies are often based on tests of scaled models in the laboratory or limited field testing. In addition, most gross pollutants cannot be sampled by traditional automatic samplers and have not been included in studies evaluating the impact of stormwater runoff on receiving waters.

Fletcher et al. (2004) reports the performance of litter and sediment management systems along with the rationale for these estimates and considerations for their application. Performance estimates for a range of pollutants are shown in Table 1.

While there is a lack of information regarding performance efficiencies locally, the failure to remove nitrogen and phosphorus, as shown in Table 1, is consistent with local studies reported in Martens et al. (2005), based on monitoring in Perth’s western suburbs.

Table 1. Pollutant removal effectiveness (Fletcher et al. 2004)

Pollutant	Expected removal	Comments
Litter	10 –30%	Depends on effective maintenance and specific design. 10% where trap width is equal to channel width, 30% where width is three or more times channel width.
Total suspended solids	0 –10%	Depends on hydraulic characteristics; will be higher during low flow.
Total nitrogen	0%	Transformation processes make prediction difficult.
Total phosphorus	0%	TP trapped during storm flows may be re-released during inter-event periods due to anoxic conditions.
Coarse sediment	10 –25%	Depends on hydraulic characteristics; will be higher during low flow.
Oil and grease	0-10%	Majority of trapped material will be that attached to organic matter and coarse sediment.
Faecal coliforms	unknown	
Heavy metals	0%	

Stormwater designers are recommended to check the claimed performance efficiency results of specific devices, examine the conditions the results were obtained under, ensure testing is independent and refer to guidelines, such as the Victoria Stormwater Committee (1999), for removal rate estimates in the absence of available data. For example, monitoring of a GPT capturing runoff from a 50 ha catchment in Coburg, Melbourne, over a period of three months found the unit trapped practically all gross pollutants (Allison et al. 1997). The device was also found to have minimal impact on flows in the drain. However, it should be noted that for the purposes of the study, the trap was cleaned after every storm, and the CDS unit was used as a downstream control in combination with side entry pit traps installed at all road drain entrances in the catchment.

Cost^

A life cycle cost method is recommended in assessing the true costs of LSM systems. This approach takes into consideration the capital costs as well as maintenance, servicing and spoil disposal costs over the life of the system.

Taylor (2005) details a range of costs for various LSM systems, but notes a very high degree of variability in most cost elements and urges caution in the use of the information provided.

Due to the high variability and lack of standardisation of available cost data, it is recommended that capital costs for individual LSM systems be assessed on a case by case basis. Annual maintenance costs presented in Taylor (2005) (provided by Tony Weber, Senior Waterways Program Officer of the Brisbane City Council, 2001 and 2002) typically range in the order of 7 to 30% of the capital cost.

^The costs quoted in this section are from around 2000 to 2005 and have not been adjusted for inflation or potential cost changes which may have occurred since this chapter was published in 2007. Therefore, it should be considered as indicative only and users of the manual are encouraged to seek further specific industry advice on the current costs as appropriate.

Design considerations

The principal design objective of LSM systems is to achieve a balance between the impact on the discharge capacity of the drainage system, the trapping efficiency of the unit and the capital and maintenance costs.

The expected gross pollutant loading and trapping efficiency have a significant impact on the dimensions of the LSM system and its maintenance requirements.

The selection and positioning of LSM systems need to be strategic as these devices can be expensive to install and maintain. LSM systems should be used to target high litter generation areas, such as commercial areas. In areas with low litter generation rates, such as typically found in low density residential areas, source controls and non-structural methods are likely to be more cost effective. The recommended four steps to optimise the location of LSM systems are (adapted from Victoria Stormwater Committee 1999):

- Identify high litter generation areas from field inspections, examination of land use maps and consultation with council officers and community catchment groups.
- Determine the drainage pathways for each of the high litter generation areas from examination of drainage plans and field verification.
- Determine whether an at-source, in-transit or end-of-pipe LSM system would be most suitable for each area.
- Identify the most suitable and optimal locations for installing the LSM systems in order to achieve the maximum load of gross pollutants trapped per dollar spent on the project.

A guide to determining whether an at-source, in-transit or end-of-pipe LSM system is most suitable is provided below:

- **At-source LSM systems**, such as entrance litter baskets, are likely to be most suited to treat runoff from small sized high litter generation areas, for example a local commercial strip with up to ten shops. At-source systems should also be considered in medium-sized high litter generation areas if runoff from the pollutant source areas flows to different drainage networks, or if the runoff combines with significant volumes of runoff from low litter generation areas downstream. In-transit LSM systems may not be cost effective in these cases as numerous units or a very large unit to treat high volumes of flow that carries low concentrations of litter may be required. The advantage of at-source systems is that the inlets that receive the most litter can be targeted. The disadvantage of distributed at-source systems is that the number of sites requiring maintenance is increased.
- **In-transit LSM systems** are generally most suitable to capture flows from medium to large sized high litter generation catchment areas (e.g. large shopping centres and light industrial areas with fast food outlets). Inline systems are most effective where the majority of the source area flows through one outlet and that outlet does not receive significant runoff from other low litter generation areas.
- **End-of-pipe LSM systems** are suited to medium to major high litter generation areas or where a number of smaller source areas are connected along the same drainage pathway.

The design of LSM systems must also consider that previously trapped material may be remobilised when high inflows causing turbulence or overflows occur.

There are also potential health risks to maintenance workers when handling litter and rubbish, particularly if contaminants have been left in an oxygen limiting environment (i.e. enclosed underground system) for an extended period. Retained water can become anaerobic due to decomposition of settled organics, possibly causing attached nutrients and heavy metals to become dissolved. Safety precautions in handling litter also need to consider potential needles and other sharp objects that may be in the trapped material. Due to the various potential health risks associated with handling litter, appropriate personal protective equipment should be used. Nuisance problems such as odour and mosquito breeding can also occur, particularly if the system is not operating correctly or if maintenance is required (e.g. removing accumulated litter). See section 1.7.7 ‘Public health and safety’ of the Introduction section of this chapter for more information on mosquito management.

Floating debris traps can also be visually unattractive.

Safety barriers may be required around LSM systems if they have steep sides or deep pools (children can drown in only 4 cm of water). To enable access and for public safety purposes, the bank slope of the trap or basin should typically be between 1:6 and 1:8 (refer to requirements of individual local authorities). Where the banks are too steep, railings, signage or vegetation can be used to discourage public access. If located within the floodway, railings should be aligned parallel to the main direction of flow so that they do not trap debris and contribute to flooding.

Vegetation can be used to disguise the LSM system, as well as prevent easy access to the structure. The appearance of a trap can be greatly improved by landscaping and the selection of construction materials. The use of local rock or coloured concrete in construction may also assist in minimising the visual impact.

Design guidelines

This section outlines important considerations when choosing a LSM system for site-specific purposes. The guidelines are based on *Bringing Order to the Pollution Control Industry – issues in assessing the performance of GPTs* (Wong et al. 1999).

Location and layout

The factors to consider when assessing the suitability of a location for installing a LSM system are (adapted from Victoria Stormwater Committee 1999):

- the location of the LSM system in the treatment train and the presence of any other existing or proposed stormwater controls;
- the location in the drainage network relative to the identified high litter generation areas;
- stormwater system details (e.g. pipe sizes and gradients);
- space constraints such as the presence of underground services;
- vehicle access to the site for maintenance; and
- potential impacts on the community (e.g. disturbance during construction and maintenance, visual impacts, odours, or breeding of nuisance and disease vector insects).

The dimensions of a LSM system, the flow paths of stormwater through the system and the mechanisms used to intercept and retain gross pollutants are factors that determine its suitability for installation at a chosen site. Systems designed to capture both gross pollutants and sediment invariably require a larger area than systems designed to trap gross pollutants alone. This is due to the fact that sediment loads are often significantly higher than gross pollutant loads and the trapping mechanism involves flow retardation (i.e. expansion of the waterway to reduce flow velocity) to facilitate settlement of sediment.

The layout of the LSM system and the overflow path are important design considerations in ensuring adequate hydraulic performance of the trap under above-design and non-ideal flow conditions. The available space onsite will need to be compatible with the selected design discharge and the provisions for above-design and non-ideal flow operation.

Design flow

The appropriate design flow for a LSM system varies from one application to another.

The selection of the design discharge is primarily used to define the minimum height of the flow diversion or flow bypass mechanism such that all flow at or below this design discharge will pass through the solids separation section of the LSM system. Flows in excess of the design discharge will either over pass or be diverted around the solid separation mechanism.

In LSM systems involving capture of sediment, principally by sedimentation, the minimum dimension of the sediment basin/chamber is set by matching the settling velocity of the targeted sediment size to the ratio of the design flow rate to the surface area of the basin. Remobilisation of settled sediments is an issue that may be addressed by setting the maximum flow velocity below that which is likely to cause re-entrainment.

The majority of storm events with the potential to mobilise and transport urban pollutants to receiving waters are events of relatively low rainfall intensity. This is demonstrated in Figure 3 which presents the overall percentage of the expected volume of the annual stormwater runoff treated by a LSM system, against the design standard of the system for urban catchments using a time of concentration of 1 hour (Wong et al. 1999).

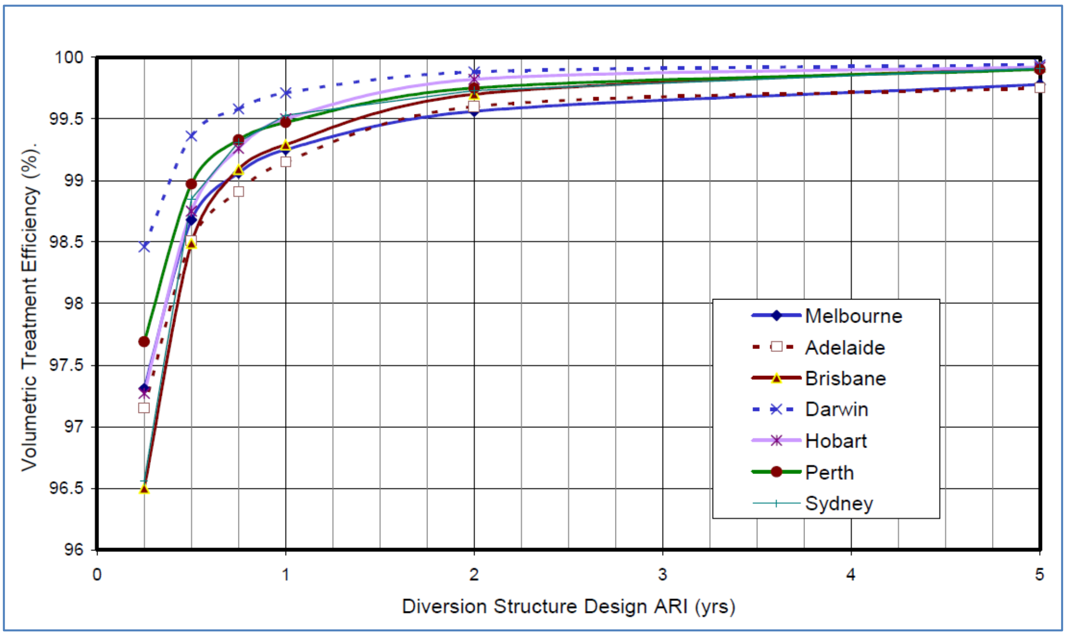

Figure 3. Treatment efficiency for various design ARIs. (Source: Wong et al. 1999, Cited in ARQ 2006.)

The volumetric treatment efficiency listed on the vertical axis of Figure 3 defines the percentage of the expected annual volume of runoff which can be expected to flow through the LSM system at a rate which is lower than the design discharge of the system. Analyses were carried out for catchments of different sizes with critical storm durations of 0.5, one, three and six hours. The results for each of these cases were found to be similar in that most devices can be expected to treat over 95% of the expected annual runoff volume when designed for a 0.25 year ARI peak discharge. The corresponding volumetric treatment efficiency for a device designed for a one year ARI (1 EY) peak discharge is approximately 99%.

These results are applicable to any type of hydraulic structure and clearly demonstrate that the design standard of structures need not be set excessively high to gain significant benefits in the overall proportion of stormwater treated.

All LSM systems are required to operate satisfactorily for larger events up to the discharge capacity of the stormwater drainage system in which the LSM systems are placed. The same operating criterion applies in the event of non-ideal conditions associated with situations when excessive inflow of gross pollutants has resulted in the trapping mechanisms being compromised or blocked. Above-design and non-ideal operation criteria include provision of the following:

- a controlled and predictable flow path for stormwater in excess of the design discharge (with predictable energy loss associated with these flow conditions);
- minimum reduction in the discharge capacity of the stormwater drainage system under above-design flow or non-ideal flow conditions; and
- protection of trapped material from being entrained with the flow and consequently transported out of the structure to the receiving waters.

Trapping efficiency

The trapping efficiency is defined as the proportion of the total mass of gross pollutants transported by stormwater that is retained by the LSM system. Common presentations of trapping efficiency data include reports of the weight or volume of gross pollutant removed. These reports are often provided without accompanying information that will enable computation of a mass balance between gross pollutant captured and that which has passed through the LSM system, making comparisons of different systems difficult.

While it is difficult to monitor field installations to satisfy a mass balance criterion, other data related to catchment area, rainfall, stormwater flow, and frequency and duration of above-design conditions, in association with the clean-out data, are helpful in developing a common basis for comparing performance.

Performance data for most LSM systems are confined to hydraulic behaviour under ideal conditions and without the interaction of the flow with urban derived litter and gross solids. This is considered inadequate for assessing the suitability and reliability of LSM systems in field conditions.

Continuous recording of water levels can often be used to assess the performance of a LSM system by identifying periods of non-ideal operating conditions. For example, observing the rate at which the water level in a GPT recedes at the conclusion of a storm event allows an assessment of the degree of blockage in the separation screen of the trap. Similarly, comparison of water levels on the upstream and downstream sides of a screen can often be used to estimate deterioration of the performance of the unit.

Gross pollutant characteristics and loading

Estimates of the gross pollutant loads are required when designing litter and sediment traps.

The composition by mass of gross pollutants and litter for Coburg, a suburb of Melbourne, is presented in Figures 4 and 5. Coburg is considered a typical example of inner city suburbs in Australian capital cities. The study found that in all land use types, a major proportion of the total gross pollutant load is made up of organic material such as leaves, twigs and grass clippings. When the gross pollutant data were sorted to examine the composition of litter, paper and plastics were found to be the dominant types.

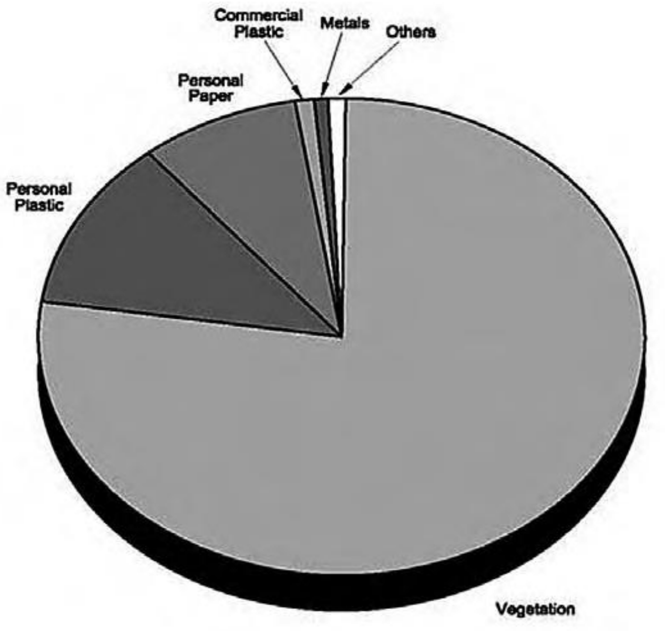

Figure 4. Composition of urban gross pollutants by mass. (Source: Allison et al. 1997.)

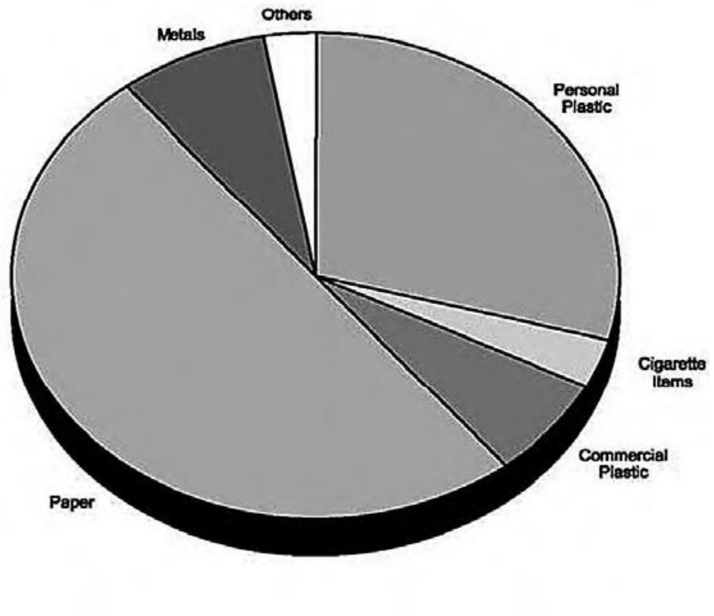

Figure 5. Composition of urban litter by mass. (Source: Allison et al. 1997.)

Studies by Allison et al. (1997, 1998a, 1998b) for the Coburg catchment provided nominal annual gross pollutant (i.e. material greater than 5 mm in size) load estimates of approximately 90 kg/ha/yr (wet weight). In their analysis, it was found that the typical pollutant density (wet) is approximately 250 kg/m^3 and the wet to dry mass ratio is approximately 3.3 to 1. This gives the expected volume of total gross pollutant load as approximately 0.4 m^3/ha/yr. The results of these studies can be applied to estimate gross pollutant loads in cities with similar rainfall and runoff patterns.

Stormwater runoff in suburbs on the WA coastal plain are likely to have lower loads of gross pollutants due to the higher infiltration rate and lower direct connectivity of the runoff pathways compared to

Melbourne. The higher infiltration rate reduces surface water discharge and hence the potential for gross pollutant transport.

The studies also found that a high proportion of the total gross pollutant load consists of vegetation (i.e. leaves) and that urban derived litter, food and drink refuse (from fast food consumers) and cigarette refuse, constitutes approximately 30% of the total gross pollutant load. These items entered the drainage network mainly from commercial areas. Data have indicated that approximately 10% of gross pollution remains buoyant for a significant length of time.

The study by Allison et al. (1997) found that gross pollutant concentrations are highest during the early stages of runoff; however, most of the load is transported during periods of high discharge. Similar loads and concentrations of gross pollutants were found in runoff from different storms that occurred on the same day. Therefore, LSM systems should aim to treat the maximum possible discharge and be able to accommodate multiple storms in one day.

Minimum dimensions

The minimum dimensions of the LSM system are dependent on the expected rate of sediment and gross pollutant exported from the catchment and the capture efficiency.

Efficient traps with small capacity for containment of trapped material require a high frequency of clean-out if the integrity of their trapping mechanism is not to be compromised.

Maintenance

Regular inspection and cleaning of LSM systems is essential to maintain their performance and prevent the devices from blockages or releasing pollutants. Poorly maintained devices can increase the risk of upstream flooding (Engineers Australia 2006).

The device should have a site-specific maintenance plan, providing guidance on a suitable inspection regime, maintenance practices (including guidelines on the equipment to be used, health and safety procedures, waste disposal arrangements, etc.) and responsibilities. These plans should be prepared in consultation with relevant maintenance personnel. Health and safety procedures need to address handling trapped litter that may contain needles and other sharp objects.

Frequent inspection is initially necessary following installation of the device to develop an appropriate inspection and cleaning regime. Maintenance schedules should not be fixed, but reviewed regularly to reflect the performance outcome from ongoing monitoring and optimise the maintenance regime. Flexibility of the maintenance regime is required given the seasonality and uncertainty of rainfall patterns and pollutant accumulation rates. Inspection and cleaning (if required) immediately prior to the wet season is essential.

Opinions on the frequency and timing of cleaning vary. However, experience suggests that fixed interval cleaning by contract cleaners, combined with regular council audits may be the best combination in most instances. It means that the LSM systems are being cleaned and the costs are budgeted for. A notable exception is where the systems are situated above ground and pollutant build-up can be easily sighted, in which case cleaning on ‘demand’ may be more effective. Where wet sumps are installed, trapped pollutants may break down and release contaminants and nutrients back into the stormwater system. Under these circumstances, cleaning may need to be undertaken much more frequently. Stormwater managers are required to critically assess the adequacy of manufacturers’ recommended maintenance schedules.

The type of land use and industries upstream of the LSM system should be considered in predicting what types of pollutants are likely to be trapped in the device or sediments. Sediments in open basins may contain iron monosulphide black oozes and will require special removal techniques to prevent oxygenation and subsequent acid release and deoxygenation of the water body. In regions like Perth, there is evidence to suggest that accumulated sediments in urban areas are enriched with nutrients, heavy metals and hydrocarbons (Swan River Trust 2003). Management of handling, drying and final disposal of materials removed during desilting operations needs to be considered. Spoil excavated from sediment basins should be placed where it cannot wash back into the basin or release contaminants back into the stormwater system. Areas disturbed by maintenance activities should be stabilised upon completion of the sediment removal works. Refer to BMP 2.2.2 'Maintenance of the stormwater network' in Chapter 7 for further guidance on managing sediments removed from the stormwater system.

Suitable equipment to extract the waste from the stormwater system needs to be used (e.g. for enclosed drains and pits, machinery that operates via suction rather than flushing). If the trap requires dewatering in order to remove solids settled at the base, then discharge of the liquid contents to the sewerage system or a wastewater tanker will need to be arranged. Depending on whether pollutants are collected on a solid surface or in a basket or sump, traps can be cleaned by hand or loader, by removing baskets by a crane truck, or by removing the contents of a sump with a vacuum truck.

An important factor is that there must be ready access to the device for the required type and size of vehicle. This service must be available in the area where the device is installed, otherwise transport costs become significant and there is the temptation to clean traps less frequently than required. The filter medium of some types of traps may need to be occasionally replaced if degraded or clogged.

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6.2 Hydrocarbon Management

Figure 1. Service stations are a common source of hydrocarbon pollutants.
Photograph: (Department of Water 2007.)

Background

The Unauthorised Discharge Regulations 2004 of the *Environmental Protection Act 1986* make it illegal to discharge substances, such as hydrocarbons, to groundwater or the stormwater system.

The primary aim of hydrocarbon pollution management is to provide at-source containment through the implementation of appropriate structural measures. Non-structural techniques, such as raising the awareness of operators or imposing heavy fines for illegal discharges, are also useful preventive measures. Compliance with control requirements incorporated into building approvals and industry operating licences, as well as pollution discharge inspection and monitoring, help to better regulate the principal sources of contamination.

On any site there may be one or more levels of containment. Primary containment deals with the tank or vessel in which the material is stored. It is therefore the first line of defence and must be fit for purpose. Secondary containment uses devices or structures that capture spills for treatment. These can either be 'local' containment, such as oil-water separators, or 'remote' containment such as floating booms installed on the inlets to ponds or wetlands. Remote containment can be an effective temporary measure for emergency spill response, but should not be considered in preference to local containment measures.

A risk assessment is useful in deciding the appropriate level of containment. The operator should consider the hazardous materials onsite, the risks posed by accidents, the likely failure mode of the primary containment, the sensitivity of receiving environments and the potential pathways for any resultant discharge to enter the stormwater system or be transported to receiving environments.

Commonly applied non-structural practices for hydrocarbon management include:

- preventing the mixing of stormwater and wastewater (for example from industrial processes or wash-down of vehicles or floors) and treating these water streams separately
- servicing, repairs and other activities that may result in contaminants such as oils, grease, solvents, acids, fuels, coolants and surfactants accumulating on hardstand areas should be undertaken in weatherproof and contained areas to prevent these contaminants entering the stormwater system
- activities should be undertaken on sealed concrete floors that prevent contaminants entering groundwater and enable comparatively easy clean-up of any spilt servicing fluids

- floors should be designed to drain to an internal collection sump and/or surrounded with an impervious perimeter bund. Any stormwater be diverted away from the workshop floor and chemical or parts storage areas
- wastes and wastewater should be disposed of in an approved manner, such as by removal off-site by a waste recycling and disposal contractor, or treatment and disposal to sewer where permitted
- chemicals and waste products should be stored in weatherproof and contained areas to prevent weathering of storage containers and to minimise the risk of contaminants from accidental spillage or ruptured containers entering the stormwater system or the environment. Storage tanks, such as underground fuel tanks, should be inspected and tested for leakages. All loading and unloading should also be undertaken in contained areas.

Oil-water separators are used to remove remnant pollutants that cannot be controlled using the practices outlined above.

Oil-water separators are often used in retrofit situations to provide some water quality treatment at a lot scale, particularly for small industrial or commercial lots where larger BMPs are not feasible due to site constraints. There is a variety of both proprietary and non-proprietary oil-water separators available, ranging from chambered designs to manhole types. Many of these systems are ‘drop in’ systems and incorporate some combination of filtration media, hydrodynamic sediment removal, oil and grease removal, or screening to remove pollutants from stormwater. The standardised designs allow for relatively easy installation.

These separators are best used in commercial, industrial and transportation type land uses (i.e. impervious areas that are expected to receive high sediment and hydrocarbon loadings, such as carparks and service stations). However, oil-water separators cannot be used for the removal of dissolved or emulsified oils such as coolants, soluble lubricants, glycols and alcohols.

For non-structural control information, refer to Sections 2.2.6. ‘Maintenance of premises typically operated by local government’, 2.2.8. ‘Maintenance of vehicles, plant and equipment (including washing)’, 2.2.10. ‘Stormwater management on industrial and commercial sites’, 2.3.4. ‘Education and participation campaigns for commercial and industrial premises’, 2.4.2 ‘Point source regulation of stormwater discharge and enforcement activities’ and 2.5.1 ‘Risk assessments and environmental management systems’ of Chapter 7: Non-structural controls.

For further information on managing stormwater and preventing pollution from industrial sites, see the following Water Quality Protection Notes: Light industry near sensitive waters (Department of Water 2009), Rural restaurants, cafés and taverns near sensitive water resources (Department of Water 2006) *Mechanical Servicing and Workshops* (Department of Water 2013), *Stormwater Management at Industrial Sites* (Department of Water 2010), *Mechanical Equipment Washdown* (Department of Water 2013), *Radiator Repair and Reconditioning* (Department of Water 2009), *Service Stations* (Department of Water 2013), and *Toxic and Hazardous Substances - storage and use* (Department of Water 2015), available at water.wa.gov.au/

Further information about spills and emergency response is available at www.der.wa.gov.au/our-work/pollution-response

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Performance efficiency

Selection of an appropriate oil-water separator is largely governed by the level of hydrocarbon interception that is required and the likely oil droplet size. Performance efficiencies for various types of oil-water separators are described below, based on information detailed in Engineers Australia (2006).

- **Flow density-based separators**: use a series of simple flow baffles to trap sediment and floating oil (Figure 2). The collected oil is removed by an oil skimmer to a separate storage tank or periodically removed by a suction tanker. The application of these separators is limited to medium (100-140 µm) size oil droplets (i.e. runoff conditions close to the source, with limited emulsification of the oil). The maximum treatable catchment area is typically less than 0.2 ha.
- **Coalescence plate-based separators**: use closely packed plates coated with a material that repels water and attracts oil, causing oil droplets to coalesce (i.e. join together). The accumulated oil on the plate then floats to the surface of the separation chamber. The close spacing of the plates reduces the distance that an oil droplet must travel before it reaches a collection surface. Therefore, to achieve the same degree of treatment as a flow density-based separator, a smaller device can be used. These separators are capable of high interception rates (> 90%) for small (50–60 µm) oil droplets that are typical of oil that has been highly emulsified by stormwater. The maximum treatable catchment area is typically less than 0.5 ha.
- **Vortex-based separators**: use the energy of the vortex to promote the density separation of oil and water. Vortex-based separators are capable of intercepting very fine (20–30 µm) oil droplets.

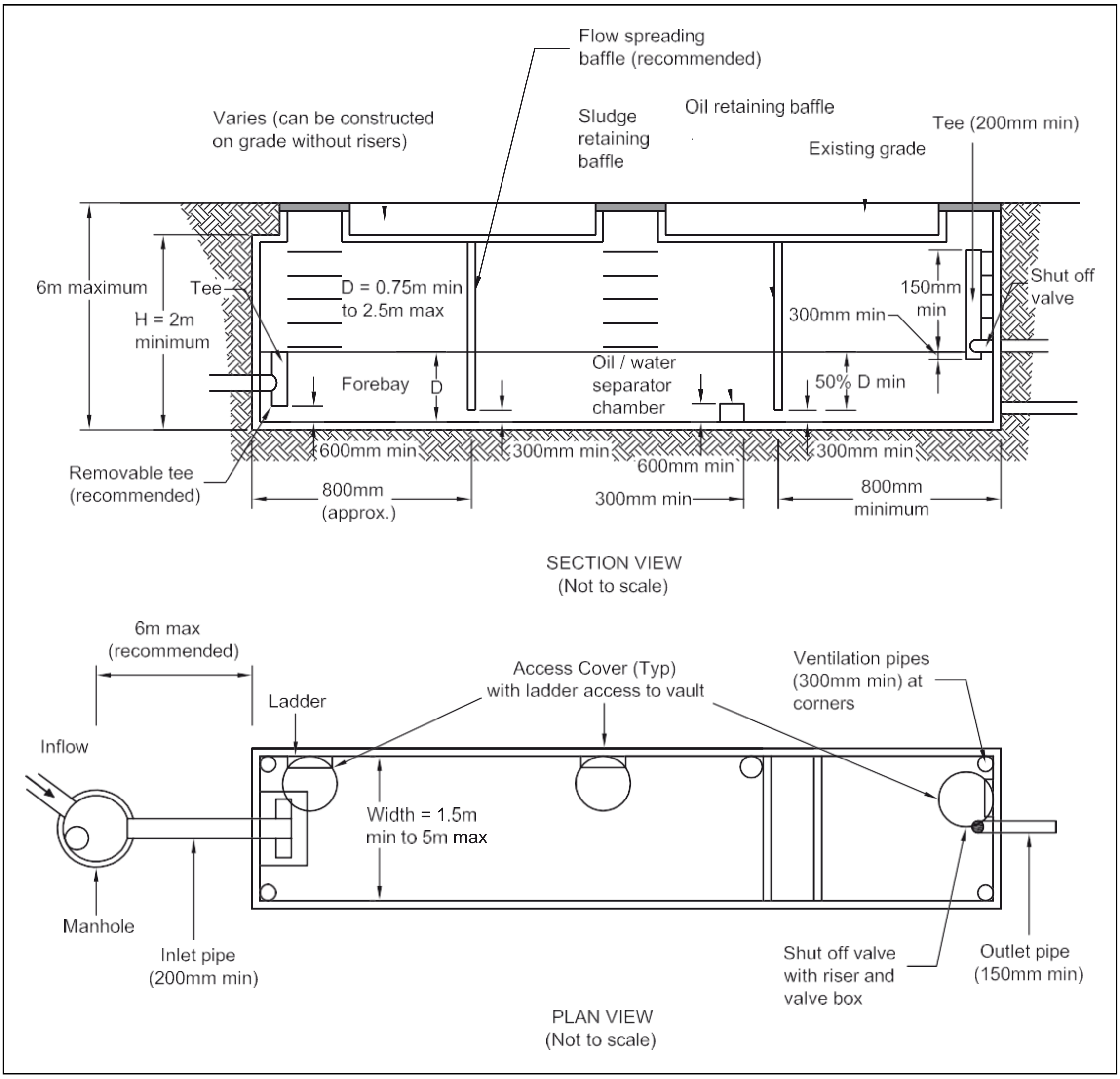

Figure 2. Typical flow density-based separator layout. (Source: Auckland Regional Council 2003.)

Cost

The construction costs for oil-water separators will vary greatly, depending on their size and depth.

A life cycle cost method is recommended in assessing the true costs of oil-water separator systems. This approach takes into consideration the capital costs as well as maintenance, servicing and disposal costs over the life of the system.

Due to the high variability and lack of standardisation of available cost data, it is recommended that capital costs for individual systems be assessed on a case by case basis.

Maintenance costs will also vary significantly depending on the size of the drainage area, the amount of residual collected and the clean-out and disposal methods available. The cost of residual removal, analysis and disposal can be a major maintenance expense, particularly if the residuals are toxic and are not suitable for disposal in a conventional landfill.

Design considerations

Only rainfall runoff that may contain hydrocarbons (e.g. runoff from carparks or areas adjacent to fuel pumps) should enter the oil-water separator that is part of the stormwater treatment system. Runoff that is relatively clean (e.g. roof runoff) should be managed separately to minimise the volume of stormwater that requires a high level of treatment. Oil-water separators installed to treat stormwater runoff at industrial or commercial sites should not be used to collect and treat wastewater or fluids from chemical or petroleum spills.

Careful evaluation of the maintenance and disposal issues is highly recommended. Higher residual hydrocarbon concentrations in trapped sediments cause maintenance and residual disposal costs associated with oil-water separators to be higher than other BMPs. Proper disposal of trapped sediment, oil and grease is required as trapped material is likely to have high concentrations of pollutants and might be toxic.

Ease of access for maintenance and inspection is required. In particular, lids should be kept as lightweight as practical.

Oil-water separators should be designed and constructed as offline systems only. In addition, it is recommended that the contributing area to any individual inlet be limited to approximately half a hectare or less of impervious surface.

Design guidelines

The following design guidelines for an oil-water separator are based on Auckland Regional Council (2003) and is provided as a design reference for oil-water separators in Engineers Australia (2006).

Rise velocity

The rise velocity for an oil droplet within a separator can be calculated, given the water temperature (which affects the viscosity of the water) and the density of the oil. This rise velocity is then used in the sizing calculation for the device.

$$V_r = \frac{g.D^2(1-s)}{18v}$$

Where:

$V_r = \textit{rise velocity of an oil droplet}\ (ms^{-1})$

$s = \textit{specific gravity}\ (e.g. oil\ 0.9, diesel\ 0.85, kerosene\ 0.79\ and\ gasoline\ 0.75)$

$D = \textit{droplet diameter}\ (m)$

$v = \textit{kinematic viscosity of water}\ (m^2s^{-1})$

$g = \textit{gravitational acceleration}\ (ms^{-2})$

Design flow

The required design (treatment) flow rate can be calculated using the Rational Method equation:

$$Q_d = \frac{CiA}{1000}$$

Where:

$Q_d = \textit{required design (treatment) flow rate}\ (m^3hr^{-1})$

$C = \textit{coefficient of runoff}$

$i = \textit{rainfall intensity for selected design rainfall event}\ (mmhr^{-1})$

$A = \textit{catchment area}\ (m^2)$

In WA, a design rainfall recurrence period of one in six months can be expected to achieve water quality treatment of at least 95% of the expected annual runoff volume. For small catchments, a critical storm of minimum 10 minute duration should be used.

Tank sizing for flow density-based separator

The base area of the tank (A_b) is a function of the rise velocity (V_r), expressed in m/hr, and design flow rate (Q_d), expressed in m^3/hr:

$$A_b = \frac{FQ_d}{V_r}$$

The factor F is dimensionless and accounts for short-circuiting and turbulence effects, which can degrade the performance of the tank. The factor depends on the ratio of horizontal velocity (U) to the rise velocity (V_r), as shown in Table 1 based on Auckland Regional Council (2003).

Table 1. Factor F for calculation of tank sizing

U/V_r	Factor (F)
3	1.28
6	1.37
10	1.52
15	1.64

The volume and area calculated by this method refer to the main compartment of the tank. Additional volume should be allowed for inlet and outlet sections of the tank.

Other key sizing requirements detailed in Auckland Regional Council (2003) for sizing the main compartment of the tank are:

- Length to be at least twice the width
- Depth to be at least 0.75 m
- $U \leq 15V_r$

Additionally, it is recommended that the width is typically between 1.5 m to 5 m, and depth is less than 2.5 m (and between 0.3 to 0.5 times the width). Some of these additional recommendations will not be appropriate for smaller catchments.

To avoid re-entrainment of oil and degradation of performance, it is recommended that the maximum horizontal flow velocity in the main part of the tank be less than 25 m/hr.

Tank sizing for coalescence plate-based separator

Plate separator suppliers can provide an approximate size to achieve the required oil droplet diameter separation at the chosen design flow rate. The plan area (A_{plan} in m^2) of each plate can be approximated from the following equation:

$$A_{plan} = \frac{Q_d}{V_r N}$$

Where N = the number of coalescing plates and the rise velocity (V_r) and design flow rate (Q_d) are expressed in m/hr and m^3/hr respectively.

Other considerations

A high-flow bypass may be required in certain situations so that flows above the design flow do not enter the oil-water separator and cause re-suspension of debris or entrainment of oils.

A bypass system may not be required where the catchment draining to the oil-water separator is small and therefore the volumetric increase in runoff can be accommodated by the tank size. An adequately sized tank is generally preferable to a bypass system, which will result in contaminants potentially reaching the main drainage system and receiving water bodies.

Where a bypass system is installed, an inlet baffle should be included. The inlet baffle prevents the collected oil from recirculating back into the bypass system and subsequently into the drainage system.

To achieve an even flow distribution across the tank at the inlet, a baffled inlet port or other device is used. The sizing of the inlet port or baffle should be such that some head loss is provided to spread the flow. It is recommended that velocities of the maximum separator flow should be less than 0.5 m/s to avoid oil emulsification.

Maintenance

The effectiveness of oil-water separators is highly dependent on regular maintenance. Regular inspection and maintenance is required to reduce the risk of re-suspension of debris or entrainment of oils. Failure of hydrocarbon management systems is usually caused by a lack of maintenance.

Recommended maintenance practices are outlined below:

- The device should have a site-specific maintenance plan, providing guidance on a suitable inspection regime, maintenance practices (including guidelines on the equipment to be used, health and safety procedures, waste disposal arrangements, etc.) and responsibilities. These plans should be prepared in consultation with relevant maintenance personnel.
- In the case of proprietary systems, use the manufacturers' recommended maintenance specification as a basis. However, stormwater managers are required to critically assess the adequacy of manufacturers' recommended maintenance schedules on a case by case basis. Where necessary, the maintenance requirements or cleaning frequency may need to be increased, particularly for high risk catchments (see Engineers Australia (2006) for further information on catchment risk assessment for hydrocarbon management). Frequent inspection is initially necessary following installation of the device to develop an appropriate inspection and cleaning regime. Maintenance schedules should not be fixed, but reviewed regularly to reflect the performance outcome from ongoing monitoring and optimise the maintenance regime.
- Periodic removal of sediment is required to maintain the capacity of oil compartments, prevent blockages of inlets and maintain the functioning of coalescence plates. As a general guide, in areas of high sediment loading, inlets should be inspected and cleaned after every major storm event, and inspected at least monthly. Typically, oil separators need to be maintained every one to six months.
- Suitable equipment to extract the waste from the drainage system needs to be used (e.g. machinery that operates via suction rather than flushing).
- Nuisance problems such as odours and mosquito breeding can occur with the use of wet chambers. Therefore, regular visual inspection of chambers is required during the mosquito risk breeding months and pollutants removed and mosquito control undertaken when necessary.
- The amount of material removed from each chamber should be documented so that the frequency of maintenance can be adjusted if required.
- A representative sample of the sediment should be analysed before disposal. If the sediment requires disposal in a landfill, refer to the *Landfill Waste Classification and Waste Definitions 1996* (As amended 2019) (Department of Water and Environmental Regulation 2019) to determine the appropriate landfill type and the waste acceptance criteria. The Department of Water and Environmental Regulation regulates the transportation of wastes that may cause

environmental or health risks. It does this through the application of the Environmental Protection (Controlled Waste) Regulations 2004. Controlled waste is generally defined as any waste that does not meet the acceptance criteria for a Class I, II or III landfill site. The *Guideline: Waste Categorisation of Controlled Waste* (Department of Water and Environmental Regulation 2021) specifies that a generator is a person whose activities produce or apparatus results in the production of controlled waste. Generators are required to use a Department of Water and Environmental Regulation controlled waste licensed carrier to transport the material off-site and be in possession of a controlled waste tracking form.

Worked example

Caution: The following worked examples use Rational Method as per the Australian Rainfall and Runoff (ARR) Book VIII (Institution of Engineers, Australia 2001). However, as per the updated ARR Book 9 'Runoff in Urban Areas', the Rational Method is only suitable for lot-scale scale catchments or simplistic small catchments where flood routing is not critical. This method is not suitable for a 'precinct' scale estimation of peak flows as it has 'limited' runoff generation and surface routing capabilities. If runoff volume management infrastructure forms part of a solution, or if an understanding of potential impacts on downstream flooding are required, then a 'strong' hydrologic estimation method such as a runoff-routing model should be used (ARR 2019). For further information on the limitations of Rational Method, please refer to Book 9 'Runoff in Urban Areas' of ARR 2019.

The following worked example has been adopted from Auckland Regional Council (2003) and amended to represent local hydrologic conditions.

A service station in Perth is to be fitted with an oil-water flow density-based separator to treat runoff that is potentially contaminated with hydrocarbons. Runoff from the roof should be separated from runoff that is likely to be contaminated with hydrocarbons (e.g. pavement runoff). The wastewater from the car wash area is to be directed to a water reuse system, which is connected to the sewer. The flow density-based oil-water separator installed to treat pavement runoff will have a catchment area of 300 m2 draining to the device.

The rainfall intensity for a 10 minute critical storm duration with a return period of 0.5 years (i.e. $0.5i_{10m}$) in Perth is 34 mm/hr. The separator design flow, using the Rational Method equation is:

$$Q_d = \frac{CiA}{1000}$$

$$Q_d = \frac{1.0 * 34 * 300}{1000}$$

$$Q_d = 10.2\ m^3 hr^{-1}$$

The separator is to be designed for this example to capture a 60 µm droplet of oil (s = 0.9) rising through water at 15°C (which has a kinematic viscosity $\nu = 1.139 \times 10^{-6}$ m^2/s)

To calculate the rise velocity:

$$V_r = \frac{gD^2(1-s)}{18v}$$

$$V_r = \frac{9.81 * (60 * 10^{-6})^2(1-0.9)}{18 * (1.139 * 10^{-6})}$$

$$V_r = 1.72 * 10^{-4}\ ms^{-1}$$

The rise velocity is 1.72×10^{-4} m/s (or 0.62 m/hr).

The maximum design flow horizontal velocity (U) at the separator is 15 V_r = 15 (0.62 m/hr) = 9.3 m/hr. Therefore the flow cross section (depth times the width) is Q_d /U = (10.2 m^3/hr) / (9.3 m/hr) = 1.1 m^2. The minimum required depth is 0.75 m, which gives a width of 1.5 m. These dimensions are within the recommended guidelines of the depth being typically half the width.

For U = 15 V_r, an F of 1.64 is then used (from Table 1) to calculate the base area A_b:

$$A_b = \frac{FQ_d}{V_r}$$

$$A_b = \frac{1.64 * 10.2}{0.62}$$

$$A_b = 27\ m^2$$

With this plan area and a width of 1.5 m, the length is 18.0 m. The volume of the main chamber of the tank will be 20.2 m^3 (excluding inlets and outlets). The tank will actually be longer to allow for an inlet chamber and an outlet section, which, as an approximate guide, could add an additional 20% to the total tank volume.

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Reference details

The recommended reference for overall manual is:

Department of Water and Environmental Regulation, 2007, *Stormwater management manual for Western Australia*, updated 2022, Government of Western Australia, Perth, available www.dwer.wa.gov.au

The recommended reference for this chapter is:

Department of Water and Environment Regulation, and Department of Biodiversity, Conservation and Attraction 2007, *Structural Controls, Stormwater management manual for Western Australia*, updated 2022, Government of Western Australia, Perth available www.dwer.wa.gov.au