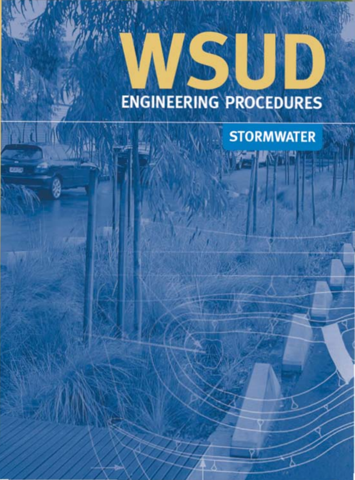
Water Sensitive Urban Design

WSUD

ENGINEERING PROCEDURES

STORMWATER



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The Municipal Association of Victoria

Association of Land Development Engineers

City of Kingston

City of Wyndham

The Plumbing Industry Commission

The City of Greater Shepparton

South Gippsland Shire Council

Institute of Public Works Engineers Australia

GHD

KLM

Mornington Peninsula Shire

Australian Institute of Landscape Architects

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Chris Chesterfield, Caroline Carvalho and Erin Pears of Melbourne Water all had significant input into the project.

Contents

Chapter 1 **Introduction**

1.1 Purpose of the Manual WSUD Engineering Procedures: Stormwater

Since the late 1990s there has been an increasing number of initiatives to manage the urban water cycle in a more sustainable way. These initiatives are underpinned by key sustainability principles of water consumption, water recycling, waste minimisation and environmental protection. The integration of management of the urban water cycle with urban planning and design is known as **Water Sensitive Urban Design (WSUD)**. WSUD has multiple environmental benefits including improving urban landscape, reducing pollutant export, retarding storm flows and reducing irrigation requirements.

Urban **stormwater** managed both as a resource and for the protection of receiving water ecosystems is a key element of WSUD. In Victoria, there have been many initiatives to improve the environmental management of urban stormwater. The publication of *Urban Stormwater: Best Practice Environmental Management Guidelines* (Victorian Stormwater Committee 1999) provided a framework for the development of Stormwater Management Plans by local councils. More recently, the release of *Melbourne 2030* (Department of Infrastructure 2002) in 2002, the Victorian government's planning strategy for sustainable growth in the Melbourne metropolitan area, clearly articulates the role of sustainable stormwater management.

The practice of WSUD espouses the innovative integration of urban water management technologies into the urban environment and that strategic planning and concept designs are underpinned by sound engineering practices in design and construction. Although there are several documents that provide guidance for planning and strategy development of WSUD, similar support for the next level of design detail, from concept plans to detailed plans suitable for construction, however, is not well covered; this level of support forms the focus of the current Manual.

WSUD Engineering Procedures: Stormwater complements existing resources that promote WSUD and provides advice on the design detail of WSUD elements. It is intended to provide a consistent approach to design that incorporates WSUD technologies into urban developments. It provides a set of design procedures that can be used equally by designers and by referral authorities when checking designs. These design procedures are intended to provide consistency for engineering details of WSUD elements in Victoria.

The Manual is not intended to be a decision-making guide to selecting, integrating and locating WSUD elements (i.e. site feasibility). These topics are covered by other documents, notably *Australian Runoff Quality Guidelines* (Engineers Australia 2003) and *Urban Stormwater:*

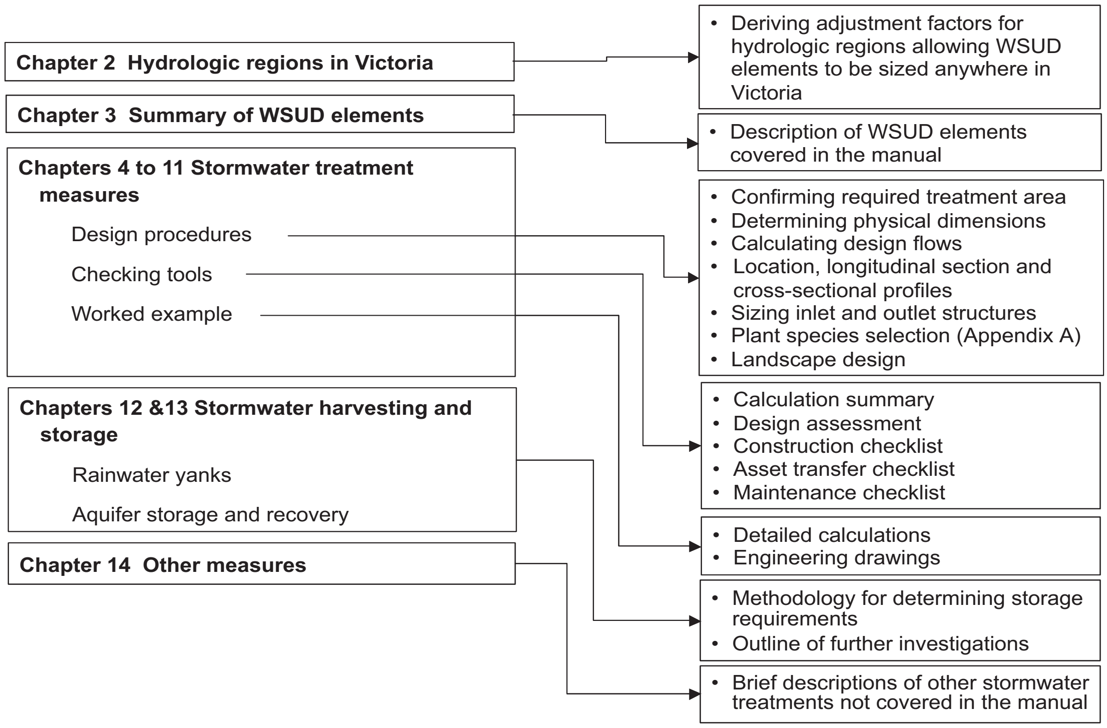

Figure 1.1 Map of manual contents.

Best Practice Environmental Management Guidelines (Victorian Stormwater Committee 1999). The purpose of the current Manual is to ensure that when an element is planned, its configuration will satisfy the engineering requirements of a stormwater system.

1.2 Target audience

As the aim of this Manual is to help ensure that engineering details of WSUD measures represent best practice and a consistent approach to their design is implemented, it will be used by referral authorities and assessment officers. It provides simplified tools and checklists to assess the adequacy of proposed works submitted for approval. The target audience, therefore, consists of both design engineers and local and state government approval officers. It is intended for professionals with some experience in urban hydrology and hydraulics and assumes a basic knowledge in these fields.

While primarily directed at engineers, it is, however, recognised that all WSUD developments require the involvement of a range of professionals to find a sustainable solution. Typically this would include planners, urban designers, landscape architects, environmental scientists and engineers to select and place WSUD elements. Sections of this Manual contain input from landscape architects, scientists and planners to reflect their expertise for components of a detailed design.

The Manual is applicable across Victoria.

1.3 Manual contents

The contents of this manual are grouped into four sections as shown in Figure 1.1.

All areas of Victoria are covered in the Manual through the use of **hydrologic design regions** (see Appendix B). The regions (see Chapter 2) are intended to allow the size of a particular WSUD element to be converted from that required at the reference site (i.e. Melbourne) to any location in Victoria in order to achieve the same level of pollutant reduction. Melbourne was selected as the reference site as it has the largest amount of collected performance data on WSUD elements.

Continuous improvements in sizing WSUD elements developed for the reference site can be applied to other areas using the **adjustment factors** provided for the various regions. Plots of adjustment factors for the nine regions of Victoria are provided.

In Chapter 3 an overview of the WSUD elements covered in this manual is presented, largely grouped into stormwater treatment measures, stormwater harvesting measures and other measures not covered in detail in this Manual.

In Chapters 4 to 11 the engineering design procedures for eight types of WSUD elements are presented: sediment basins, bioretention swales, bioretention basins, sand filters, swale/buffers, constructed wetlands, ponds, and infiltration systems. The format in each of the chapters follows a standard layout that includes a worked example to illustrate the design procedure, with associated design drawings. Checklists for design, construction and maintenance are provided to summarise key information related to the WSUD element to facilitate the design approval process.

The eight types of WSUD elements are described in the following paragraphs.

Confirming required treatment area: For each element covered typical configurations of the elements have been modelled to estimate their performance in removing pollutants. Expected pollutant reductions for Total Suspended Solids (TSS), Total Phosphorous (TP) and Total Nitrogen (TN) are presented for different sizes of WSUD elements (presented as a percentage of impervious **catchment**). The plots can be used during either designing or checking of a design. The curves, based on Melbourne data, can be adjusted for other areas in Victoria by using the hydrologic design regions. Should more local rainfall data be available, the performance for that specific treatment can be estimated through modelling more accurately.

Design procedure: For each element, a step-by-step process to perform detailed design calculations is presented. The procedures outline what equations should be followed as well as providing general commentary on the approach to design. The commentary emphasises important components that are critical to treatment performance and should not be compromised. Calculation summary sheets are provided to help ensure that designers follow each step in the design process. In addition, a section on construction advice is provided that summaries experiences from designing, building and maintaining WSUD elements around Australia.

Worked examples: A worked example is provided for each WSUD element to illustrate the use of the design procedure. The example goes through the steps from a concept design to a detailed design and discusses design decisions that are required as well as performing the calculations outlined in the design procedures. Working drawings that detail key elements of the system are also presented as examples.

Checking tools: A series of checking tools are provided for each WSUD element to guide referral agencies as well as designers. Checking tools for use during the construction process are provided for different stages of a WSUD element design.

Checklists: Checklists for design, construction, asset transfer and maintenance checklists are provided for checking the integrity of designs of each WSUD element prior to construction. Construction inspection forms are provided to allow checks of key elements on-site both during and after construction. In addition, an *Asset Transfer Checklist* is provided to ensure the WSUD element is functioning as designed following a defects period.

To aid maintenance of WSUD elements, *Maintenance Inspection Forms* are also provided for each element to highlight the important components of a system that should be routinely checked. These can be used as templates to develop more site-specific maintenance inspection forms.

Landscape designs: Illustrations of the WSUD elements showing possible landscapes are also provided. These help to show how the elements can fit into an urban landscape and are used to visualise the operation of a particular element.

Recommended plant species: Lists of recommended species are provided for different WSUD elements as well as for different zones within some WSUD elements (see Appendix A). Appendix A provides basic lists of plants that will enhance water quality. Recommended regions for each species across Victoria are also presented. Although these species will all improve water quality, the lists are not exhaustive and local indigenous species may be more appropriate.

Chapters 12 and 13 relate to stormwater harvesting and storage. They describe what needs to be investigated to design stormwater harvesting schemes at the allotment and regional levels.

Some additional treatment measures not specifically covered in this Manual are discussed in Chapter 14.

1.4 How to use this Manual

This Manual focuses on how to develop a WSUD strategy to ensure that the objectives of a stormwater system are maintained. In a typical project involving WSUD, the design process often starts with the development of a WSUD strategy. This would normally involve a series of workshops among the various stakeholders and include inputs from professionals from relevant disciplines. Some level of modelling is often involved to assist the workshop participants in arriving at a preferred WSUD strategy. In the case of stormwater management, further modelling is often undertaken to determine notional sizes of selected WSUD elements. Guidance in this process is provided by other documents such as *Australian Runoff Quality Guidelines* (Engineers Australia 2003) and *Urban Stormwater: Best Practice Environmental Management Guidelines* (USBPEM Guidelines) (Victorian Stormwater Committee 1999).

1.4.1 Designers

Once a WSUD element has been selected, detailed engineering calculations are required to size the various hydraulic components of the element, connection details to accommodate site constraints and to confirm the notional size (required to meet stated water quality objectives) determined during concept design. Calculations are also needed to demonstrate that a system is able to convey flood flows while maintaining treatment performance. Steps normally taken by a design engineer using this Manual would be as follows:

1 Refer to Chapter 2 to determine the appropriate design adjustment factors for the selected WSUD elements corresponding to the location of the specific site. This factor is used in subsequent chapters to confirm the selected size of treatment measures.
2 Refer to the relevant chapters for guidance in the detailed design of the components of a WSUD element. The steps are:
   a determine physical dimensions after confirming the required size of the treatment measure
   b calculate **design flows**
   c determine location, longitudinal section and cross-sectional profiles to suit the site characteristics
   d size inlet and outlet hydraulic structures
   e design the landscape
   f compile calculation summary sheets where basic information from the design process can be recorded and submitted as part of a development application.

1.4.2 Referral authorities

The current Manual is also intended to help when checking development submissions by ensuring that sufficient level of detail is presented for their assessment by referral authorities. The manual provides *Design Assessment Checklists* in each of the chapters on stormwater treatment measures (see Chapters 4–11) that can be followed to assess proposed design of WSUD elements.

There are also performance graphs that present relationships between the size of various WSUD elements and expected pollutant reductions. These graphs, based on Melbourne rainfall data, can be converted, however, into equivalent areas in other parts of Victoria with the use of the hydrologic design regions (see Chapter 2).

Following acceptance of a design, a project moves into construction, defects periods and ultimately a transfer of the asset to an owner. The inspection forms, asset transfer checklists and maintenance schedules can be used to help ensure WSUD elements are built as designed, are maintained and are in good operating condition prior to asset transfer to an authority.

1.5 Relevant WSUD guideline documents

Some existing documents are directly relevant to design detail of WSUD measures. The most relevant is the *Stormwater Management Devices: Design Guidelines Manual (TP10)* (Auckland Regional Council 2003). The Auckland Manual has many aspects relevant to Victorian practice and these are drawn upon here where appropriate.

Some recent relevant documents are described below.

1.5.1 Guidelines for Treatment of Road Runoff from Road Infrastructure

The *Guidelines for Treatment of Road Runoff from Road Infrastructure* (Austroads 2003) describes likely pollutants from road runoff, reviews design standards around Australia and describes a range of appropriate treatments. In addition, several case studies are presented that illustrate the design procedures as well as the required calculations.

Many of these case studies and the procedures directly relevant to this Manual and there is strong consistency between the approaches taken in both documents. Both demonstrate the application of a design procedure with real examples for design of **swales**, bioretention systems, infiltration systems and **wetlands**. Although specifically intended for road runoff, the procedures are equally applicable to other urban situations.

1.5.2 WSUD: Basic Procedures for 'SourceControl' of Stormwater Handbook

The Stormwater Industry Association and Urban Water Resources Centre (2002) draft of their document outlines detailed design considerations and procedures for stormwater detention and retention systems. These WSUD elements and much from the draft Handbook are relevant to the current Manual.

The Handbook describes the broad principles of WSUD and also various treatment measures (similar to the Victorian Stormwater Committee 1999). It distinguishes between measures intended for water quantity from those intended for water quality management. It divides descriptions into four categories of those intended to: reduce runoff quantity, remove particulate matter, harvest runoff and be used for multiple purposes.

The most relevant sections for the current Manual are the design procedures for infiltration and **Aquifer Storage and Recovery (ASR)** systems, which show a lot of rigour. The Handbook outlines detailed equations, sometimes with derivations from first principles. The information contained in the Handbook was reviewed and user friendly design tools were developed from this for the current Manual.

1.5.3 Stormwater Management Manual

The *Stormwater Management Manual* (City of Portland 2002) comprehensively describes the environmental management of stormwater as well as providing detailed technical guidance for many WSUD elements. Much of the material and procedures are directly relevant to the current Manual although simplified prescriptive design procedures (for North American conditions) are provided.

The *Stormwater Management Manual* is designed for developers requiring approval for stormwater treatments from new developments. The Manual uses a scoring system for a range of relatively simple WSUD elements to determine if compliance has been met (called the 'Presumptive Approach' as the systems are 'presumed' to comply if designed in accordance with the guidelines). Scores are based on the area of impervious surfaces of a development and the developer gets a credit for different treatment measures. In a similar way, this Manual compares the area of impervious surface draining to a WSUD element to determine the size required (depending on the location in Victoria) for a particular level of treatment.

To enable other WSUD elements to be used, and to encourage innovation, the *Stormwater Management Manual* allows a 'Performance Approach' that sets specific treatment levels for water quality and flow management that developers must demonstrate.

For a variety of WSUD elements, general design requirements are specified. In addition, specific design criteria are given as well as design curves for some elements. These are of a similar level of design detail as described in the current Manual and may be useful when designing elements that are not covered here.

The *Stormwater Management Manual* provides advice on maintenance of each WSUD element and presents templates for maintenance plans of the facilities. There are also some typical drawings of WSUD elements that show the general arrangements and landscaping requirements of specific WSUD elements. These features have also been included in the current Manual.

1.5.4 Stormwater Management Devices: Design Guidelines Manual (TP10)

The *Stormwater Management Devices: Design Guidelines Manual (TP10)* (Auckland Regional Council 2003) is intended to provide a common approach for selecting and designing stormwater management measures. It provides an overview of the effects of uncontrolled stormwater in urban areas and then sets out a framework to address the impacts. Some sections outline procedures to select a WSUD element showing suitable and unsuitable site conditions and pollutant removal rates. It then provides detailed design procedures for a range of WSUD elements that are most relevant to the proposed Technical Manual.

Topics include **ponds**, wetlands, filtration and infiltration systems, swales, oil and water separators, **rainwater tanks** and green rooftops. All these systems and the design approach adopted are relevant to the current Manual.

For each treatment system a broad description is provided, water quality performances are predicted and detailed design guidance with typical drawings presented. In addition, recommendations for construction are made, with the staging of elements outlined, recommended plant species suggested and comments of maintenance and operation outlined.

Case studies are used to illustrate the design concepts and inspection forms, for both the construction period and for routine maintenance, are given. This information was reviewed as part of the development of the current Manual and drawn upon (with author consent) for many checklists contained in the Manual.

1.5.5 Summary of existing WSUD manuals

WSUD Engineering Procedures: Stormwater provides more detailed design guidelines and engineering checks than the USBPEM Guidelines. There are no available Australian guidelines that cover the breadth of issues in sufficient detail that this Manual is intended to. Thus, the purpose of the current Manual is to address this gap in the industry knowledge.

1.6 References

Auckland Regional Council (ARC) (2003). *Stormwater Management Devices: Design Guidelines Manual* (TP10), 2nd edn, Auckland Regional Council, Auckland.

Austroads (2003). *Guidelines for Treatment of Road Runoff from the Road Infrastructure*, Publication Number AP-R232/03, Austroads Inc, Sydney.

City of Portland (2002). *Stormwater Management Manual*, City of Portland, USA.

Department of Infrastructure (2002). *Melbourne 2030: Planning for Sustainable Growth*, State Government of Victoria, Victoria.

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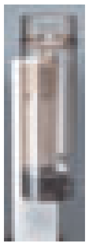
Stormwater Industry Association and Urban Water Resources Centre (2002). *Water Sensitive Urban Design: Basic Procedures for 'Source Control' of Stormwater, A Handbook for Australian Practice*, 1st edn, Draft, November. Final version published November 2004.

Victorian Stormwater Committee (1999). *Urban Stormwater: Best Practice Environmental Management Guidelines*, CSIRO Publishing, Melbourne.

Chapter 2 **Hydrologic design regions of Victoria**

2.1 Introduction

This chapter describes the development and use and of a simple design procedure for sizing **stormwater** treatment facilities across Victoria for small development projects (e.g. single or small clustered allotments). The procedure does not require any modelling. In addition, the procedure can be used as a simple tool to assess whether a proposed design is of sufficient size. The procedure is based on adjusting the size of the treatment measure from a reference site (Melbourne) to other parts of Victoria to achieve similar levels of pollutant removal.

To determine the **adjustment factors** a set of equations that only requires local Mean Annual Rainfall (MAR) data has been developed. This approach is based on defining nine **hydrologic design regions** within Victoria (four or which are in the Melbourne/Geelong metropolitan area) with adjustment factors for **wetlands**, **swales**, **ponds** and bioretention systems.

Melbourne was selected as the reference site. Estimated pollutant reductions from simulations for a range of treatment measures with different configurations for this reference site are presented in later chapters (see Chapters 4–10). These curves can then be adapted using the adjustment factors for use in different sites across Victoria. A required treatment area (i.e. the size of the facility) derived for the reference site (Melbourne) can be converted into an equivalent treatment area that will achieve the same level of treatment elsewhere in Victoria.

The results of this analysis are presented in this Chapter, while more details of the modelling approach and the model output are provided in Appendix B.

2.2 Approach to regionalisation

We have used a continuous simulation approach to help properly consider the influence of **antecedent conditions** on the design of stormwater treatment measures and the wide range of storm characteristics and hydraulic conditions that are relevant to individual treatments. Computer models such as the **Model for Urban Stormwater Improvement Conceptualisation (MUSIC)** (Cooperative Research Centre for Catchment Hydrology 2003), developed to enable continuous simulations of complex stormwater management **treatment trains**, aid in the development of stormwater management strategies and the design (sizing) of stormwater treatment measures.

The following approach was used to develop the hydrologic regions and adjustment factors presented in this chapter and Appendix B.

1 A measure of effectiveness was selected for different configurations of various stormwater treatment measures. In this case, the reduction in annual total nitrogen loads was adopted

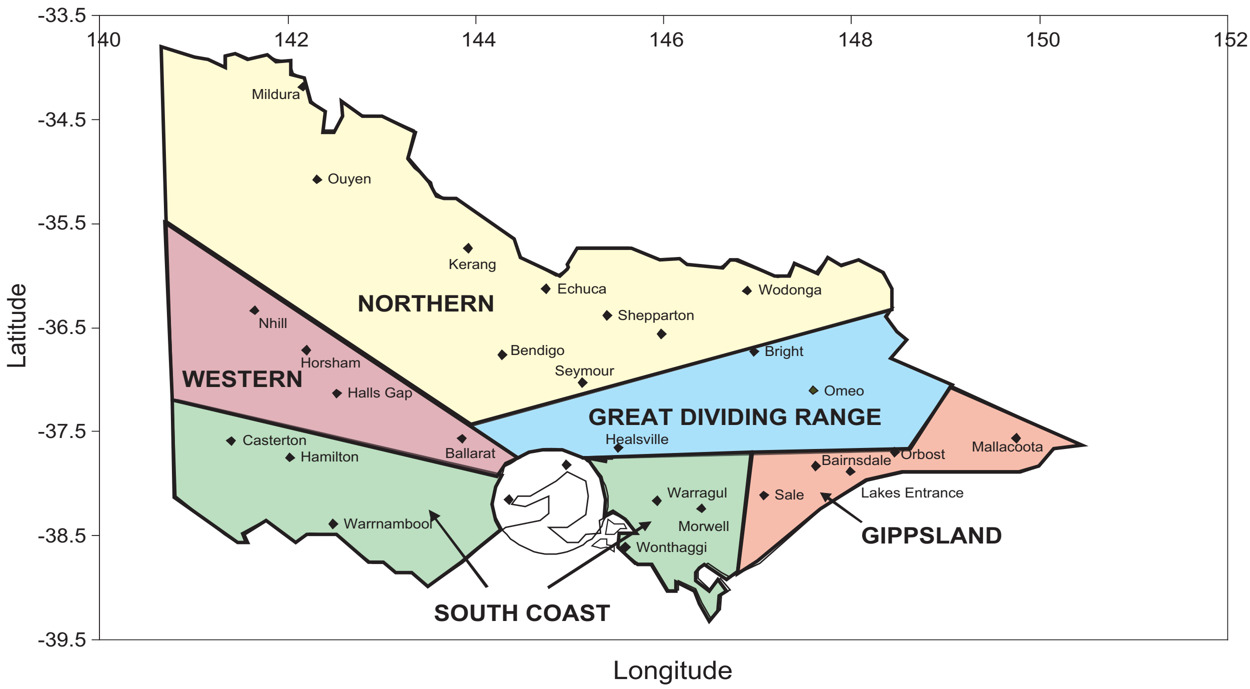


Figure 2.1 Hydrologic design regions for Greater Victoria (Melbourne and Geelong have been considered separately).

because it is commonly the limiting parameter in meeting the objectives for best practice stormwater quality.

2 A reference site was selected for which detailed performance curves were derived for different configurations (e.g. area, **extended detention** depth and **permanent pool** volume) of a range of stormwater treatment measures. Melbourne was selected as the reference site.

3 Hydrologic regions within Victoria were defined such that within each region the adjustment factor relationship was consistent. A simple equation for each region using MAR was developed that can be applied anywhere in the region.

Several geographical and meteorological factors were investigated for their influence on adjustment factors. They were limited to data that are readily available from Bureau of Meteorology website (http://www.bom.gov.au) and included mean annual rainfall, a measure of seasonal distribution of rainfall and raindays, site elevation and geographical location.

2.3 Determining hydrologic design regions

The hydrologic regions for **Water Sensitive Urban Design** (WSUD) were determined by using sufficiently long (i.e. about 20 years) sets of six-minute rainfall data for continuous simulations of the performance of stormwater treatment measures. We used 45 stations across Victoria in the analysis, of which 15 are in the Melbourne/Geelong metropolitan region (see Appendix B).

The MUSIC model was used to simulate the performance of wetlands, bioretention systems, vegetated swales and ponds to size these systems to meet best practice objectives. These sizes were then normalised against the sizes derived for Melbourne (i.e. the reference site) and expressed as the ratio of the size of the treatment area for Melbourne. This is thus the adjustment factor described in Step 3 in the methodology (see Section 2.2).

Following extensive testing and analysis of the significance of possible influencing factors, it was determined that MAR was the most significant. By using MAR it was possible to represent Victoria with five hydrologic regions (excluding the Melbourne/Geelong metropolitan region) (Figure 2.1).

Within the Melbourne/Geelong metropolitan region, a further four regions were used to provide a finer delineation of the influence of climatic conditions on the adjustment factor (Figure 2.2).

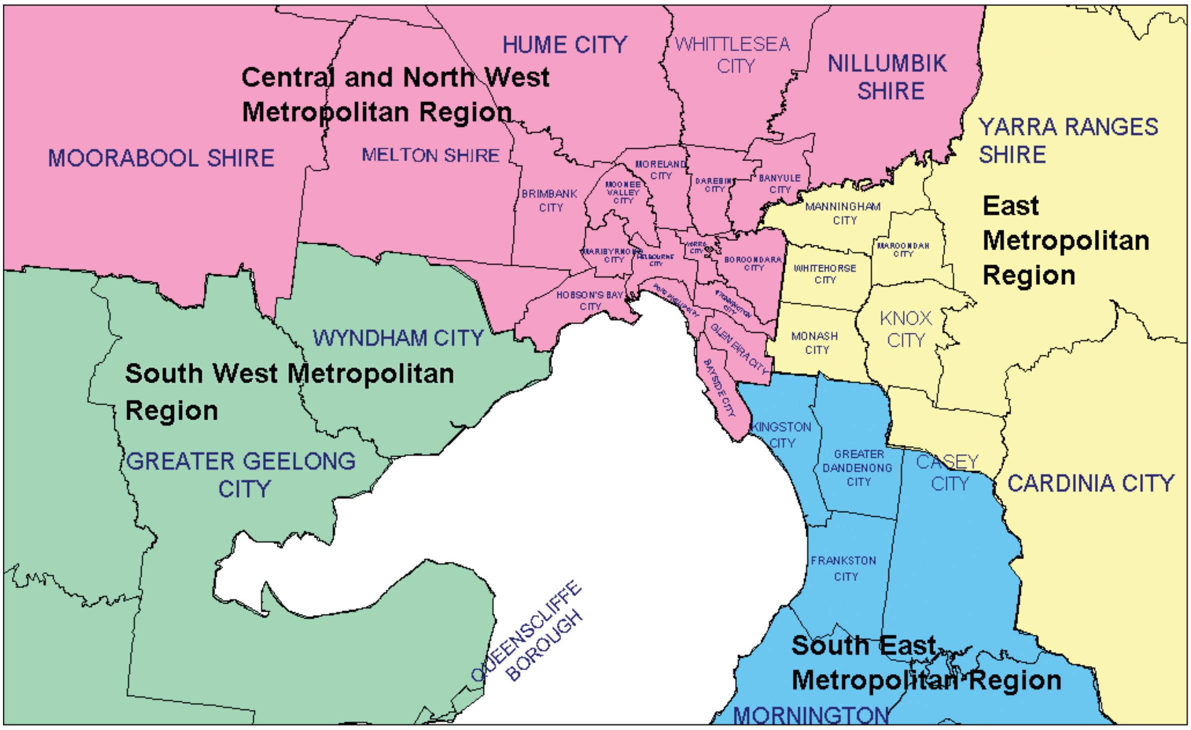


Figure 2.2 Hydrologic regions for the Melbourne/Geelong metropolitan area.

Boundaries of the hydrologic regions were determined to represent the results of the analysis. They were aligned so that they did not dissect major urban areas in Victoria or coincide with municipal boundaries, as much as possible, in the Melbourne/Geelong metropolitan area. The exceptions to this are in the Cities of Wyndham and Casey where the hydrologic regions are bounded by Skeleton Creek and the Monash Freeway, respectively.

2.4 Hydrologic region adjustment factors

Figure 2.3 presents an example plot of adjustment factor equations (for wetlands) derived from 30 stations in Greater Victoria grouped into the five hydrologic regions. This was performed for each of the treatment measures (wetlands, swales, ponds and bioretention systems) using all 45 stations.

A line of best fit was determined for the adjustment factors plotted against MAR for each region (e.g. Figure 2.3). The adjustment factor equations were determined from these relationships.

Modelling results indicated that the regional equations derived for the five state-wide hydrologic regions and four regions for the Melbourne/Geelong metropolitan region fall within a ±10% band (see Appendix B). Thus, by adopting adjustment factors that are 1.1 times (i.e. +10%) that predicted by these equations, it is expected that the predicted size of stormwater treatment measures using this method will provide adequate sizes for the treatment performance required with a high degree of certainty (i.e. they may be slightly conservatively designed). This preserves the opportunity (and incentive) for designers to adopt a more rigorous approach if desired (e.g. MUSIC modelling using local rainfall data) to further refine and reduce the size of treatments.

In three of the four hydrologic regions shown in Figure 2.2, the adjustment factor can be well represented for each treatment device by a single value (i.e. independent of rainfall) with the fourth region (Central Metropolitan and North West Metropolitan) represented as a function of MAR.

Inclusion of other factors such as raindays seasonality, rainfall seasonality and elevation did not appear to improve the estimation of the adjustment factors for the 45 **pluviographic** stations used in the analysis.

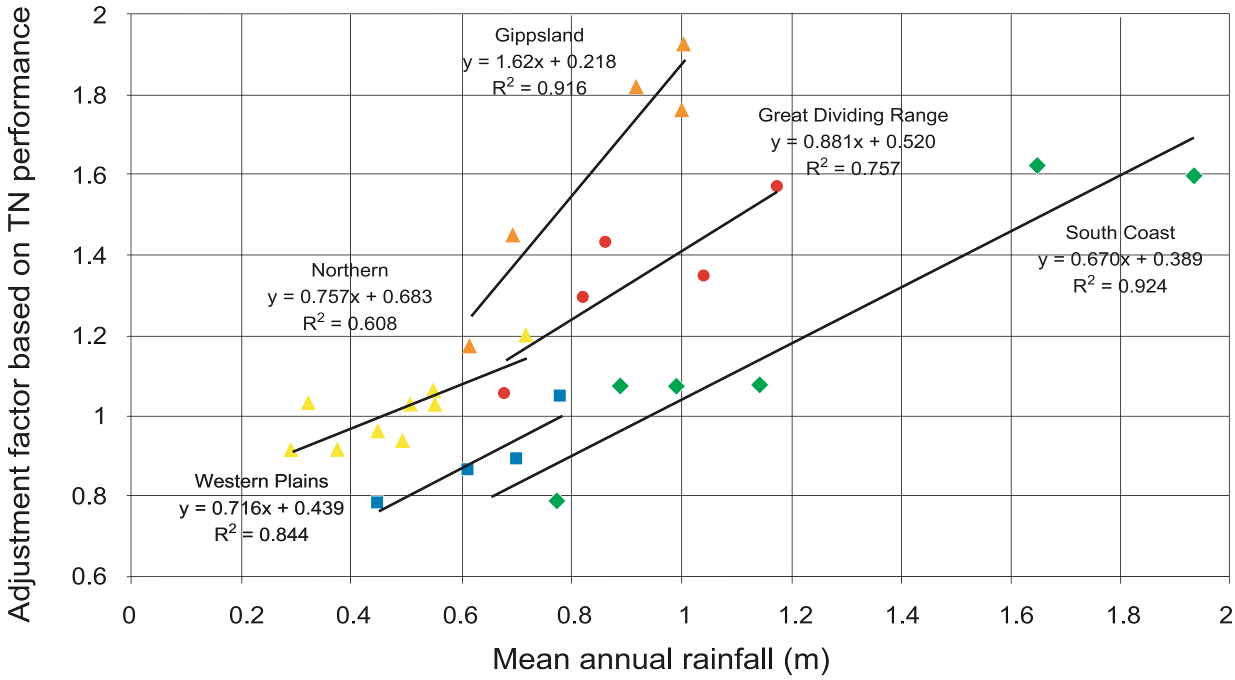


Figure 2.3 Plot of adjustment factor versus Mean Annual Rainfall (MAR) for wetlands in Greater Victoria.

The recommended equations and constants (including + 10% adjustment) for computing the appropriate adjustment factors for Victoria, including the Melbourne/Geelong metropolitan region, are summarised in Table 2.1 and Table 2.2.

Table 2.1 Greater Victoria adjustment factors
These figures are based on Figure 2.1

Region	Wetland	Bioretention	Swale	Pond
Northern	0.833 (MAR) + 0.751	0.383 (MAR) + 0.927	0.352 (MAR) + 0.956	1.85 (MAR) + 0.151
Western Plains	0.788 (MAR) + 0.483	0.059 (MAR) + 0.919	0.539 (MAR) + 0.622	1.91 (MAR) – 0.105
South Coast	0.737 (MAR) + 0.428	0.143 (MAR) + 0.719	0.15 (MAR) + 0.768	1.84 (MAR) – 0.160
Great Dividing Range	0.969 (MAR) + 0.572	0.316 (MAR) + 0.766	0.334 (MAR) + 0.813	2.20 (MAR) – 0.340
Gippsland	1.78 (MAR) + 0.273	0.325 (MAR) + 0.944	0.748 (MAR) + 0.670	2.28 (MAR) – 0.227

Table 2.2 Melbourne/Geelong metropolitan region adjustment factors
These figures are based on Figure 2.2

Region	Wetland	Bioretention	Swale	Pond
Central and North West Metropolitan	–0.463 (MAR) + 1.421	–0.259 (MAR) + 1.243	–0.144 (MAR) + 1.18	1.52 (MAR) + 0.117
South West Metropolitan	1.0	0.92	0.99	0.95
East Metropolitan	1.2	1.1	1.1	1.6
South East Metropolitan	0.99	0.89	0.94	1.3

2.5 Example application of mean annual rainfall method

Figure 2.4 shows a plot of treatment performance of a constructed stormwater wetland based on a series of MUSIC simulations using Melbourne rainfall. This is the reference plot for the sizing of **constructed wetlands** (with 0.75 m extended detention and 72 hour notional **detention time**) in Victoria.

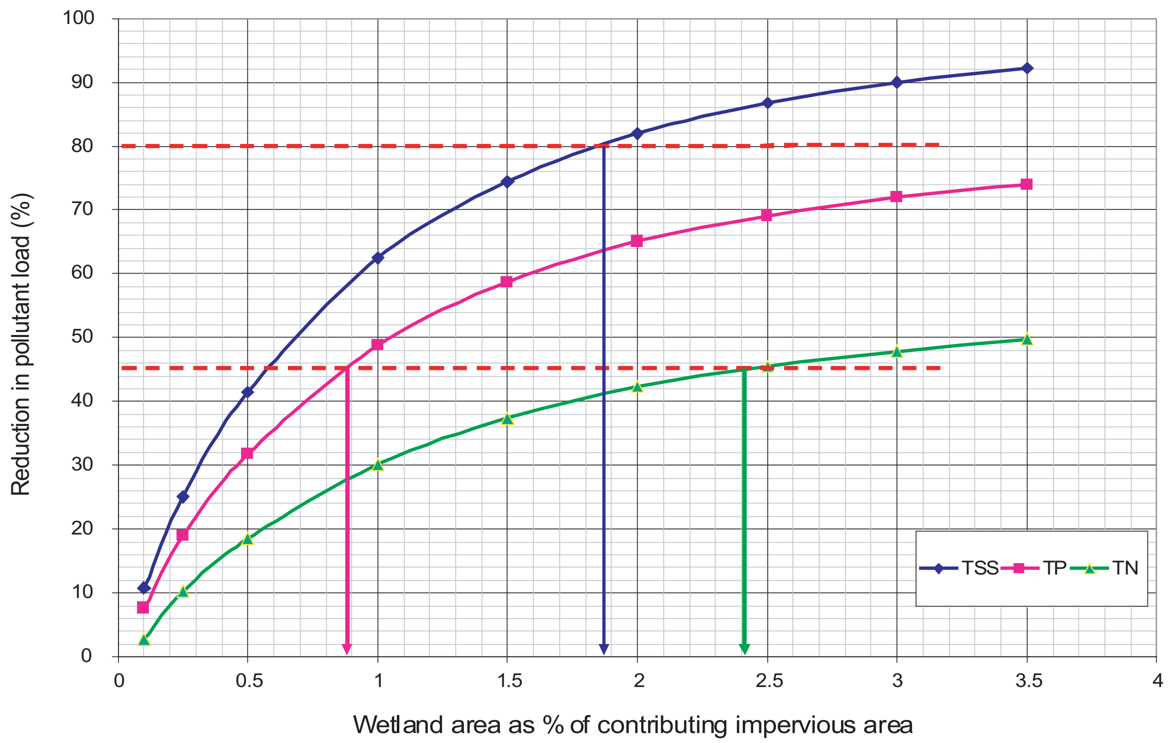


Figure 2.4 Performance curve for constructed wetlands in Melbourne.

To satisfy the objectives of stormwater treatment performance of 80% reduction in total soluble solids (TSS) and 45% reduction in total phosphorus (TP) and total nitrogen (TN) in Melbourne, the required wetland size is required to be approximately 2.4% of the contributing impervious area in the **catchment**. The required wetland size for reducing TN was the critical design condition in this case (i.e. a larger wetland is needed to meet the TN objectives than the TSS (1.86% impervious area) and TP (0.88% impervious area) objectives). For a site in another location in Victoria, this area will need to be adjusted with the appropriate wetland size adjustment factor derived from either Table 2.1 or Table 2.2.

Example

The required wetland area for a development in Gippsland with MAR of 850 mm, a catchment area of 50 ha and a fraction of impervious area of 0.5 is computed as follows:

1 From Figure 2.4, the reference wetland area needs to be 2.4% of the contributing impervious area to meet best practice objectives,
i.e. contributing impervious area = $0.5 \times 500\,000 = 250\,000$ m^2
reference wetland area = $0.024 \times 250\,000 = 6000$ m^2.

2 The adjustment factor for the Gippsland region is computed using the wetland adjustment equation for the Gippsland region in Table 2.1:
Adjustment factor = 1.78(MAR) + 0.273
= 1.78(0.85) + 0.273 = 1.8.

3 The required wetland area is $1.8 \times 6000 =$ **10 800** m^2.

Thus, a wetland in Gippsland (with 850 mm MAR) is required to have an area of 10 800 m^2 to give the same level of treatment as a 6000 m^2 wetland in Melbourne.

2.6 References

Cooperative Research Centre for Catchment Hydrology (CRCCH) (2003). *Model for Urban Stormwater Improvement Conceptualisation (MUSIC) User Guide*, Version 2.0, CRCCH, Monash University, Victoria.

Chapter 3 Summary of water sensitive urban design elements

3.1 Introduction

This chapter describes the **Water Sensitive Urban Design** (WSUD) elements for which detailed design procedures are presented in subsequent chapters. This Manual covers the most commonly used WSUD elements in Australia. Usually, a combination of these elements are used as a **treatment train** to effectively manage stormwater from a range of different land uses. The design procedures in Chapters 4–13 allow measures to be sized to target particular pollutant reductions (depending on their position in a treatment train).

Detailed design procedures are provided for the following WSUD elements:

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

In addition, Chapter 14 describes a range of 'other measures' and covers topics such as proprietary products (including **gross pollutant traps, GPTs**), **porous pavements** and other treatment devices. GPTs are not included as a separate chapter because there are many proprietary products available and detailed designs are typically not required other than the selection of treatment flows. The selection of treatment flows and other design considerations when selecting a proprietary product are contained in other texts (e.g. Engineers Australia 2003).

The following sections provide brief descriptions of the WSUD elements covered. The selection and placement of the elements within a **catchment** should be determined during a concept design of a stormwater treatment strategy and its consideration is outside the scope of this document.

3.2 Sediment basins

Sediment basins are used to retain coarse sediments from runoff and are typically the first element in a treatment train. They are important in protecting downstream elements from becoming overloaded or smothered with sediments. They operate by reducing flow velocities and encouraging sediments to settle out of the water column.

They are frequently used for trapping sediment in runoff from construction sites and as pretreatments for elements such as **wetlands** (e.g. an inlet pond). They can be designed to drain during periods without rainfall and then fill during runoff events or to have a **permanent pool**.

Sediment basins can have various configurations including hard edges and base (e.g. concrete) or a more natural form with edge vegetation creating an attractive urban landscape element. They are, however, typically turbid and maintenance usually requires significant disturbance of the system.

Maintenance of sediment basins involves dewatering and dredging collected sediments. This is required approximately every five years, but depends on the nature of the catchment. For construction sites that produce very large loads of sediment, desilting is required more frequently.

Figure 3.1 Sedimentation basins can be installed into hard or soft landscapes.

Sediment basins should be designed to retain coarse sediments only (recommended particle size is 0.125 mm. As the highest concentrations of contaminants such as hydrocarbons and metals are associated with fine sediments, waste disposal costs for this material can be much higher. Therefore, other treatment measures that assimilate these pollutants into a substrate are usually used to target this material.

3.3 Bioretention swales

Bioretention swales (or biofiltration trenches) are bioretention systems that are located within the base of a swale. They can provide efficient treatment of stormwater through fine filtration, **extended detention** and some **biological uptake** as well as providing a conveyance function (along the swale). They also provide some flow retardation for frequent rainfall events and are particularly efficient at removing nitrogen and other soluble or fine particulate contaminants.

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

Figure 3.2 Bioretention swales are commonly located in median strips of roads and carparks

Runoff is filtered through a fine media layer as it percolates downwards. It is then collected via perforated pipes and flows to downstream waterways or to storages for reuse. Unlike infiltration systems, bioretention systems are well suited to a wide range of soil conditions including areas affected by soil salinity and saline groundwater as their operation is generally designed to minimise or eliminate the likelihood of stormwater exfiltration from the filtration trench to surrounding soils.

Any loss in runoff can be mainly attributed to maintaining soil moisture of the filter media itself (which is also the growing media for the vegetation). Should soil conditions be favourable, infiltration can be encouraged from the base of a bioretention system to reduce runoff volumes (see Infiltration measures).

Vegetation that grows in the filter media enhances its function by preventing erosion of the filter medium, continuously breaking up the soil through plant growth to prevent clogging of the system and providing **biofilms** on plant roots that pollutants can adsorb to. The type of vegetation varies depending on landscape requirements and climate. The filtration process generally improves with denser and higher vegetation.

3.4 Bioretention basins

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

Like bioretention swales, bioretention basins can provide efficient treatment of stormwater through fine filtration, extended detention and some biological uptake, particularly for nitrogen and other soluble or fine particulate contaminants.

Bioretention basins have an advantage of being applicable at a range of scales and shapes and can therefore have flexibility for locations within a development. They can be located along streets at regular intervals and treat runoff prior to entry into an underground drainage system, or be located at outfalls of a drainage system to provide treatment for much larger areas (e.g. in the base of retarding basins).

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

Bioretention basins are, however, sensitive to any materials that may clog the filter medium. Traffic, deliveries and washdown wastes need to be kept from bioretenion basins to reduce any potential for damage to the vegetation or the filter media surface.

3.5 Sand filters

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

Prior to entering a sand filter, flows are generally subjected to a pretreatment to remove litter, debris and coarse sediments (typically a **sedimentation** chamber). Following pretreatment, flows are spread over the sand **filtration media** and water percolates downwards to perforated pipes located at the base of the sand. The perforated pipes collect the treated water for conveyance downstream. During higher flows, water can pond on the surface of the sand filter and increase the volume of water that can be treated. Very high flows are diverted around sand filters to protect the sand media from scour.

Figure 3.3 Bioretention basins are applicable at a range of scales and can be integrated with an urban landscape.

Figure 3.4 Sand filters can be installed above or below ground.

3.6 Swale or buffer systems

Vegetated swales are used to convey stormwater in lieu of pipes and provide a desirable **buffer** between receiving waters (e.g. creek or wetland) and impervious areas of a catchment. They use overland flows and mild slopes to slowly convey water downstream. The interaction with vegetation promotes an even distribution and slowing of flows thus encouraging coarse sediments to be retained. Swales can be incorporated in urban designs along streets or parklands and add to the aesthetic character of an area.

The longitudinal slope of a swale is the most important consideration. They generally operate best with slopes of 2% to 4%. Milder sloped swales can tend to become waterlogged and have stagnant ponding, although the use of underdrains can alleviate this problem. For slopes steeper than 4%, **check banks** along swales can help to distribute flows evenly across swales as well as slow velocities. Dense vegetation and drop structures can be used to serve the same function as **check dams** but care needs to be exercised to ensure that velocities are not excessively high.

Swales can use a variety of vegetation types. Vegetation is required to cover the whole width of a swale, be capable of withstanding design flows and be of sufficient density to provide good filtration. For best treatment performance, vegetation height should be above treatment flow water levels. If runoff enters directly into a swale, perpendicular to the main flow direction, the edge of the swale acts as a buffer and provides pre-treatment for the water entering the swale.

3.7 Constructed wetlands

Constructed wetland systems are shallow, extensively vegetated water bodies that use **enhanced sedimentation**, fine filtration and pollutant uptake processes to remove pollutants from stormwater. Water levels rise during rainfall events and outlets are configured to slowly release flows, typically over three days, back to the water levels of dry weather.

Figure 3.5 Swale vegetation is selected based on required appearance and treatment performance.

Wetlands generally consist of an **inlet zone** (sediment basin to remove coarse sediments), a **macrophyte zone** (a shallow heavily vegetated area to remove fine particulates and uptake of soluble pollutants) and a high flow bypass channel (to protect the macrophyte zone).

Wetland processes are engaged by slowly passing runoff through heavily vegetated areas. Plants filter sediments and pollutants from the water and biofilms that grow on the plants can absorb nutrients and other associated contaminants. In addition to being important in stormwater treatment, wetlands can also have significant community benefits. They provide habitat for wildlife and a focus for recreation, such as walking paths and resting areas. They can also improve the aesthetics of a development and be a central feature in a landscape.

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

Figure 3.6 Wetlands can be constructed on many scales.

3.8 Ponds

Ponds (or lakes) promote particle sedimentation, adsorption of nutrients by phytoplankton and ultraviolet (UV) disinfection. They can be used as storages for reuse schemes and urban landform features for recreation as well as wildlife habitat. Often wetlands will flow into ponds and the water bodies enhance local landscapes.

In areas where wetlands are not feasible (e.g. very steep terrain), ponds can be used for a similar purpose of water quality treatment. In these cases, ponds should be designed to settle fine particles and promote submerged macrophyte growth. Fringing vegetation, while aesthetically pleasing, contributes little to improving water quality. Nevertheless, it is necessary to reduce bank erosion. Ponds still require pretreatment such as sediment basins that need maintaining more regularly than the main, open water body. Poorly designed ponds can experience regular algal blooms. Reducing the risk of algal blooms is an integral component of design.

Ponds are well suited to steep, confined valleys where storage volumes can be maximised. Some limitations for ponds can be site specific, for example proximity to airports, as large

Figure 3.7 Ponds are popular landscape features in urban areas.

numbers of flocking birds can cause a disturbance to nearby air traffic. They also require regular inspection and maintenance to ensure that their aesthetic value is not diminished.

3.9 Infiltration measures

Infiltration measures encourage stormwater to infiltrate into surrounding soils. They are highly dependant on local soil characteristics and are best suited to sandy soils with deep groundwater. All infiltration measures require significant pretreatment of stormwater before infiltration to avoid clogging of the surrounding soils and to protect groundwater quality.

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

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

3.10 Rainwater tanks

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

Figure 3.8 Infiltration systems are best suited to sandy soils with deep groundwater.

Figure 3.9 Rainwater tanks are available in a range of sizes and shapes.

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

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

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

Tanks are generally sized for the demand they are intended for. For example, if tank water is intended to be used for toilet flushing and hot water systems, a desired level of **reliability** can be achieved with the selection of an appropriately sized tank given a site's rainfall pattern and the area of roof that drains to the tank. In most cases, where potable water is available, a connection to potable water supplies is recommended to ensure a high degree of reliability and provide a secondary source of supply.

Roof runoff that is reused also prevents stormwater pollutants (generated on roofs) from washing downstream. Depending on the roof area directed to the tank and the proportion of runoff reused, significant pollutant reductions can be made.

3.11 Aquifer storage and recovery

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

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

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

3.12 References

Engineers Australia (2003). *Australian Runoff Quality Guidelines*, Draft, June.

Chapter 4 **Sedimentation basins**

Sedimentation basin as an inlet zone for a constructed wetland.

4.1 Introduction

Reducing sediment loads is an important way to improve **stormwater** quality. **Sedimentation** basins are an integral component in a stormwater **treatment train** and are specifically employed to remove (by settling) coarse to medium-sized sediments. Sedimentation basins can take various forms and can be used as permanent systems integrated into an urban design or temporary measures to control sediment **discharge** during construction. They include all forms of stormwater detention systems that function primarily through sedimentation to promote settling of sediments through processes of temporary detention and reduction of flow velocities. Key design parameters are selecting a target sediment size, design discharge and sediment storage volume. Figure 4.1 shows the layout of a typical permanent sedimentation basin.

The required size of a sedimentation basin is calculated to match the settling velocity of a target sediment size with a design flow. Selecting a target sediment size is an important design consideration. As a pretreatment facility, selecting a sediment particle size of 125 μm is recommended as the target size.

Analysis of typical **catchment** sediment loads suggests that between 50% and 80% of suspended solids conveyed in urban stormwater are 125 μm or larger. Almost all sediment bed loads are larger than this target sediment size. However, coarse to medium-sized sediments have low concentrations of contaminant association compared to finer sediment and **colloidal particles**.

Analysis of the characteristics of particulate nutrients and metals indicate that these contaminants are mostly smaller than 50 μm and effective removal is best undertaken by treatment measures (e.g. **constructed wetlands**) other than sedimentation basins.

A sedimentation basin that is too small could have limited effectiveness and cause smothering of downstream treatment measures, thereby reducing their effectiveness in removing finer particulates and increasing maintenance.

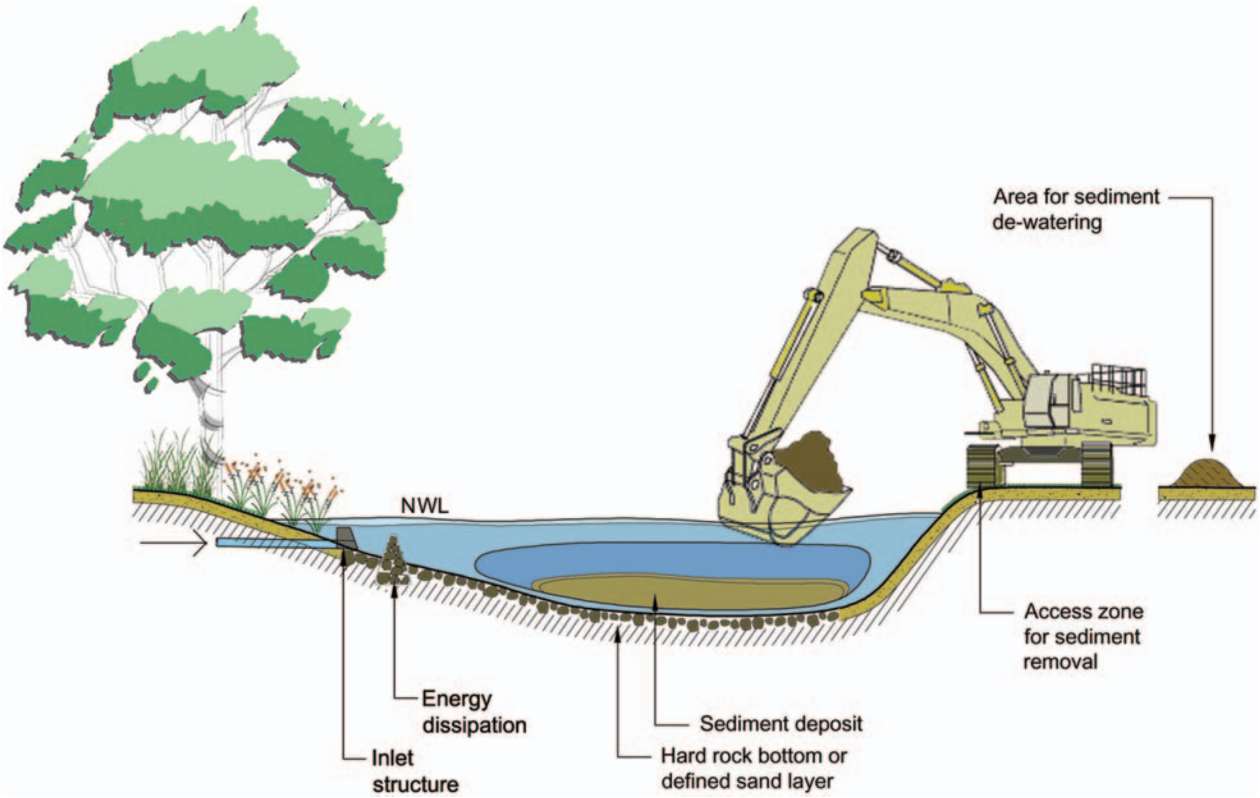


Figure 4.1 Sedimentation basin layout.

Basins that are sized to target coarse to medium-size sediment (e.g. 125 μm) are expected to capture sediment that has low levels of contamination (because of the larger sediment sizes) and is unlikely to require special handling and disposal. However, if a basin is oversized, there is an increased risk of much finer sediment accumulating and potentially having higher contaminant concentrations that could require specialist handling facilities for maintenance. Therefore, while a basin must have adequate size for capturing the target particles, they should not be grossly oversized.

A further consideration in sizing a sedimentation basin is providing adequate storage for settled sediment to prevent the need for frequent desilting. A desirable frequency of basin desilting for permanent facilities (not temporary basins for construction sites) is once every five years (triggered when sediment accumulates to half the basin depth).

Apart from needing to size a sedimentation basin appropriately for effective capture and retention of sediment, design considerations are similar to those for **ponds** and constructed wetlands.

4.2 Verifying size for treatment

Figure 4.2 shows relationships between a required basin area and design discharge for 125 μm sediment capture efficiencies of 70%, 80% and 90% using a typical shape and configuration ($\lambda = 0.5$, see Section 4.3.2). The influence of a **permanent pool** reduces flow velocities in the sedimentation basin and thus increases **detention times** in the basin (and hence removal efficiency). A typical permanent pool of 2 m depth was used to define the lower limit of the required sedimentation basin, thus forming three shaded areas in Figure 4.2 with the upper limit being defined using no permanent pool.

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

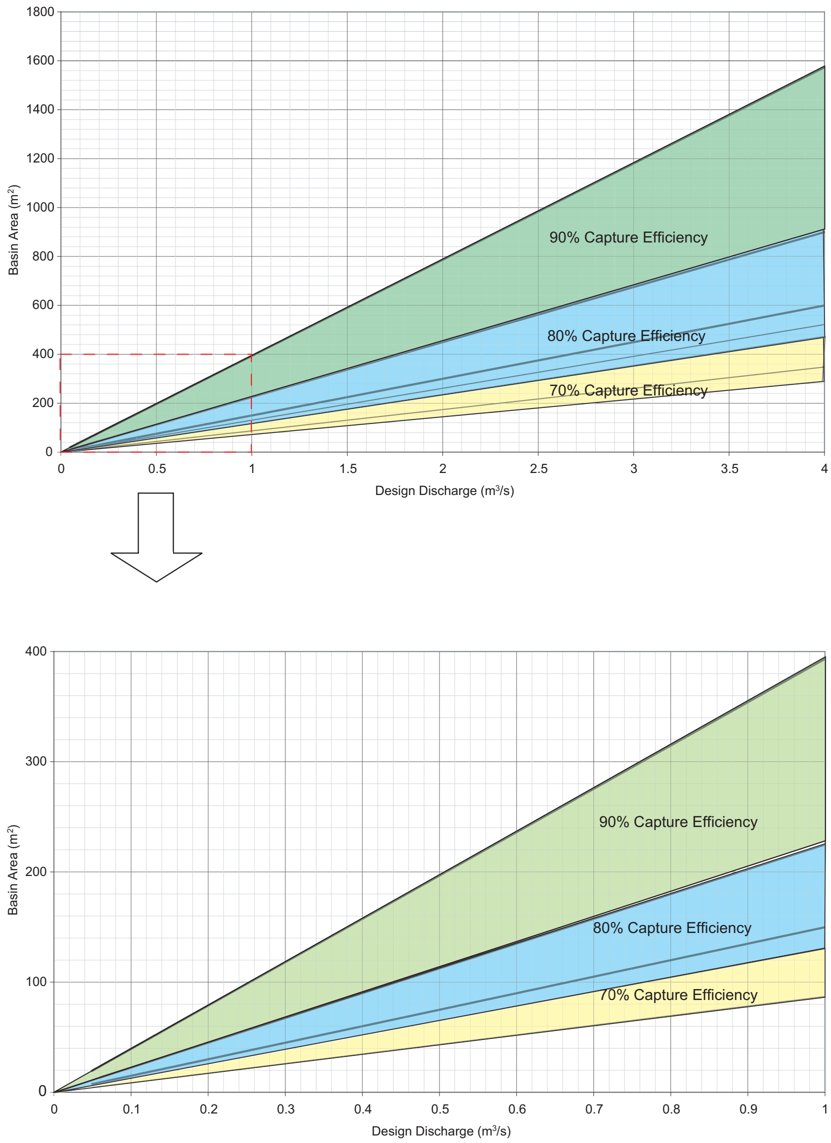


Figure 4.2 Sedimentation basin area versus design discharges for varying capture efficiencies of 125 μm sediment.

The volume of a permanent pool in a sedimentation basin should have sufficient capacity to ensure that desilting of the basin is not more frequent than once every five years (unless it is to be used for temporary sediment control when cleaning every six-months may be appropriate). A developing catchment can be expected to discharge between 50 m^3/ha and 200 m^3/ha of sediment each year. In a developed catchment, the annual sediment export is generally one to two orders of magnitude lower with an expected mean annual rate of 1.60 m^3/ha. There are different methods used to estimate sediment loads and some authorities have produced charts of sediment loading rates (ACT Department of Urban Services 1994; NSW Department of Housing 1998). Desilting should be required when the permanent pool is half full with deposited sediment.

4.3 Design procedure: sedimentation basins

4.3.1 Estimating design flows

4.3.1.1 Design discharges

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

- design flow for sizing the required basin size
- minor system design flow for the design of the inlet structure
- major flood flows for the design of the basin overflow structure.

Local councils and regional catchment management authorities may stipulate the design operation discharge for sedimentation basins, especially for temporary basins used to manage sediment discharge from construction sites. Normally the design discharge stipulated would be either the one-year Average Recurrance Interval (ARI) or two-year ARI peak discharge. The design operation flow for permanent sedimentation basins used as pretreatments for downstream stormwater treatment measures is normally the one-year ARI peak design.

An inflow structure of a sedimentation basin needs to have capacity to convey the design discharge of the minor stormwater drainage system. The design discharge varies according to location and requirements of local councils or regional drainage authorities (e.g. a 5-year or 10-year ARI peak discharge).

Sedimentation basins should not be designed to a have high flow bypass. All flows should be directed through a sedimentation basin such that some level of sedimentation is achieved even during high flow conditions. The design discharge for an overflow structure could be the same as that for the inflow structure if overland flow is not directed to the basin. In most drainage systems, a sedimentation basin forms part of the major drainage system, in which case, the design discharge for the overflow structure should correspond to the 100-year ARI event.

4.3.1.2 Minor and major flood estimation

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

4.3.2 Size and shape of sedimentation basins

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

$$R = 1 - \left(1 + \frac{1}{n}\frac{v_s}{Q/A}\right)^{-n} \qquad \text{(Equation 4.1)}$$

where R represents the fraction of target sediment removed;
v_S = settling velocity of target sediment (m/s);
Q/A = rate of applied flow (m^3/s) divided by basin surface area (m^2);
n = turbulence or short-circuiting parameter.

The above expression for sedimentation is applied with ***n*** being a turbulence parameter. Figure 4.3 provides guidance on estimating a λ value that is then used to calculate an appropriate ***n*** value (according to the configuration of the basin). The shape of a basin has a large impact on the effectiveness of the basin to retain sediments. Generally a length to width ratio of at least 3:1 should be achieved. In addition, the location of the inlet and outlet, flow spreaders and internal baffles affectct the hydraulic efficiency of the basin for stormwater treatment as the range of values in Figure 4.3 demonstrates. Figure 4.3 provides some guidance on what is considered to be good basin design with the higher values (of λ) representing basins with good sediment retention properties. Values of λ greater than 0.5 should be a design objective. If the basin configuration yields a lower value, modification to the basin configuration should be explored to increase the λ value (e.g. inclusion of baffles, islands or flow spreaders).

The maintenance access to a basin also needs to be considered when developing the shape of a basin as this can affect the allowable width (if access is from the banks) or the shape if access ramps into a basin are required. An area for sediment dewatering should also be accommodated which should drain back into the basin. This too may affect the footprint area required for a sedimentation basin system.

The value for λ is estimated from the configuration of the basin according to Figure 4.3. A value of ***n*** is estimated using the following relationship:

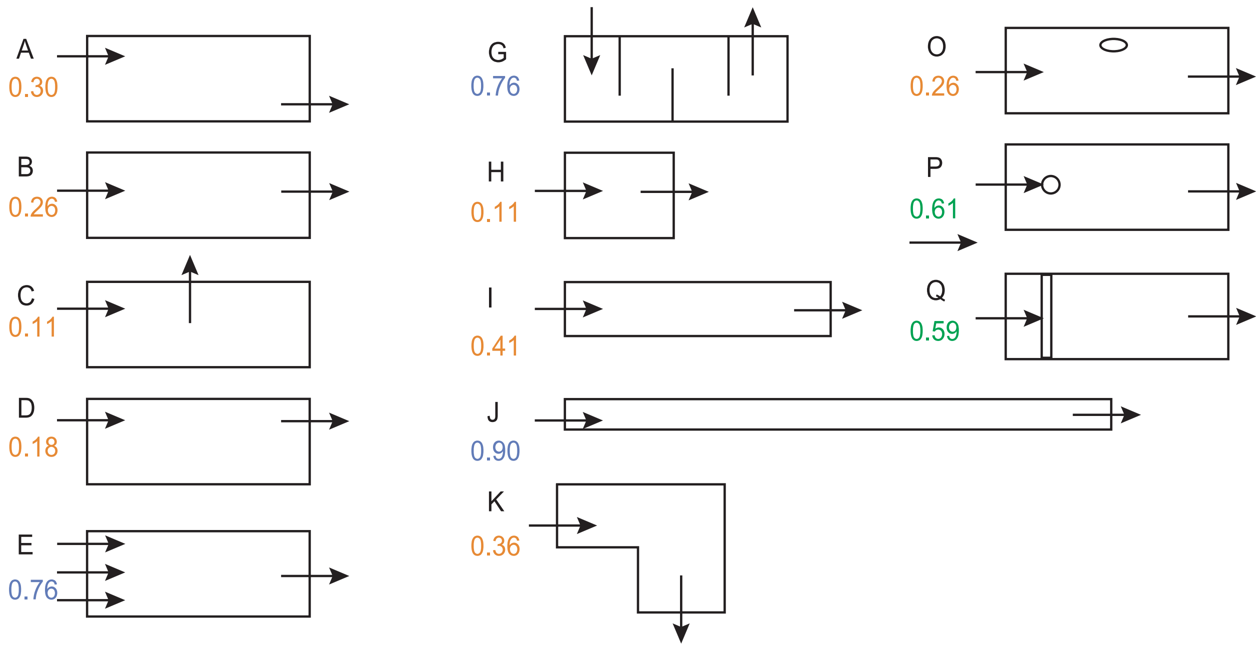


Figure 4.3 Hydraulic efficiency: λ is a measure of flow hydrodynamic conditions in constructed wetlands and ponds; range of λ is from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment (Persson et al. 1999).

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

The numbers in Figure 4.3 represent the values of λ that are used to estimate the turbulence parameter ***n*** for Equation 4.2. In Figure 4.3, 'o' in diagrams O and P represent islands in a waterbody and the double line in diagram Q represents a structure to distribute flows evenly.

Equation 4.1 is strictly applicable for systems with no permanent pool, and will generally overestimate the required area of a sedimentation basin. This equation is thus often considered to provide an upper limit estimate of the required size for sedimentation basins.

Good practice in the design of sedimentation basins will include a permanent pool to reduce flow velocities and provide storage of settled sediment. The presence of a permanent pool reduces flow velocities in the sedimentation basin and thus increases detention times. With the outlet structure being located some distance above the bed of a sedimentation basin, it is also not necessary for sediment particles to settle all the way to the bed of the basin to be effectively retained. It is envisaged that sediments need only settle to an effective depth which is less than the depth to the bed of the sediment. This depth is considered to be about 1 m below the permanent pool level. Equation 4.1 can thus be re-derived to account for the effect of the permanent pool storage as follows:

$$R = 1 - \left[1 + \frac{1}{n} \times \frac{v_s}{Q/A} \times \frac{(d_e + d_p)}{(d_e + d^\star)}\right]^{-n} \qquad \text{(Equation 4.3)}$$

where d_e represents the **extended detention** depth (m) above the permanent pool level;
d_p is the depth (m) of the permanent pool;
$d^\star$ is the depth below the permanent pool level that is sufficient to retain the target sediment (m) – adopt 1.0 or d_p whichever is lower.

Table 4.1 lists the typical settling velocities of sediments.

A further check to confirm the size of a sedimentation basin is the required volume for storage of accumulated sediments and the impact of this volume on required cleaning frequencies. Estimates of the loading rates are required (depending on whether the basin is for sediment control during construction work or post development) (see Section 4.2).

Table 4.1 Settling velocities under ideal conditions

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

Loading rates (e.g. 1.6 m^3/ha per year for developed catchments) can then be used to estimate the required storage volume for each clean-out and this volume checked against the volume of allowable sediment accumulation given the basin configuration (estimated using Equation 4.1 or 4.3). The allowable sediment storage volume should be estimated using half the permanent pool volume (as this level of accumulation should trigger a clean-out).

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

The required volume of sediment storage (*S*) can be estimated using Equation 4.4:

$$S_t = C_a \times R \times L_0 \times F_r \qquad \text{(Equation 4.4)}$$

where S_t represents the volume of storage required (m^3);
C_a = contributing catchment area (ha);
R = capture efficiency (%), estimated from Equation 4.1 or 4.3;
L_o = sediment loading rate (m^3/ha per year);
F_r = desired clean-out frequency (years).

A calculation spreadsheet which calculates sedimentation basin areas is included on the attached CD.

4.3.3 Cross sections

With the exception of temporary sedimentation basins used for construction sites, **batter slopes** on approaches and immediately under the water line of a basin should be configured with consideration of public safety. Both hard and soft edge treatments can be applied to complement the landscape of a surrounding area. Soft edge treatments involve using gentle slopes to the water's edge, extending below the water line for a distance before batter slopes steepen into deeper areas (Figure 4.4).

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

Figure 4.5 shows two options for hard edge details. One has a larger vertical wall and associated handrail for public safety and the other is a low vertical wall. In both hard edge details, it is proposed to line the bottom of the waterbody with rock to prevent vegetation (particularly weed) growth.

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

4.3.4 Hydraulic structures

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

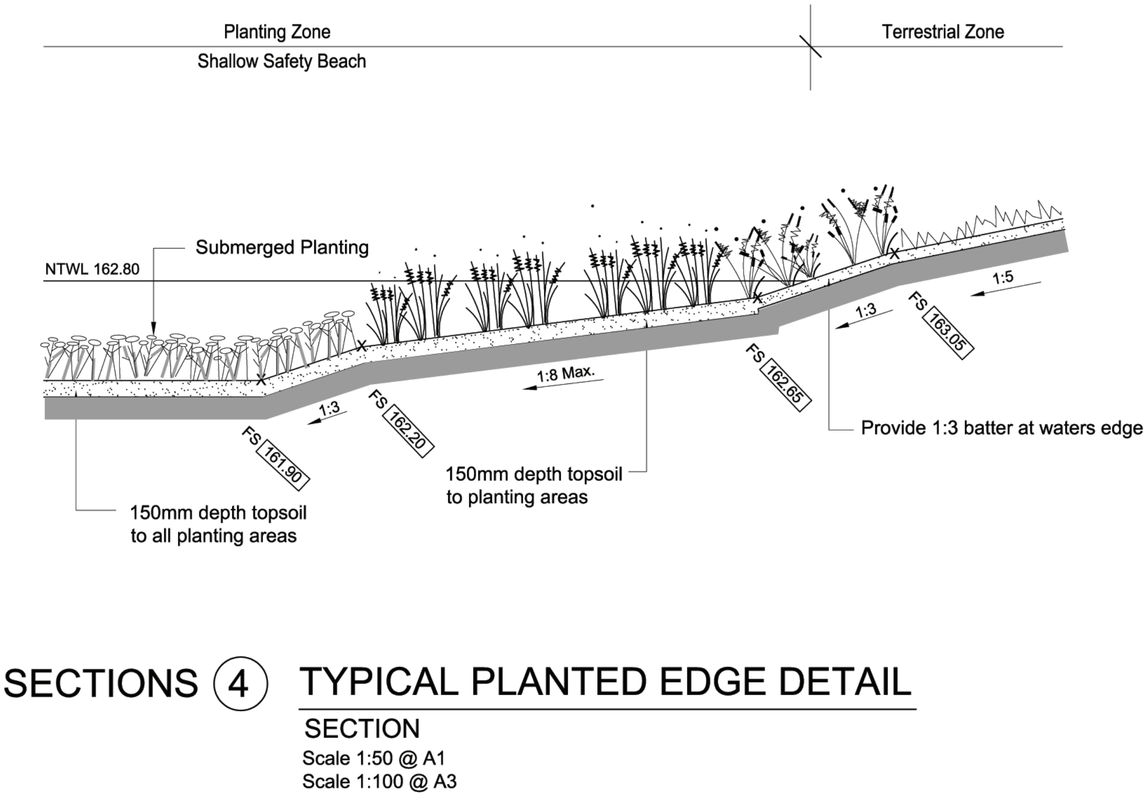


Figure 4.4 A soft edge treatment for open waterbodies (Graeme Bentley Landscape Architects 2004).

4.3.4.1 Inlet structure

Stormwater conveyed by a pipe or open channel would normally discharge directly into a sedimentation basin as this is often the first element of a stormwater treatment train. Inflow energy needs to be adequately dissipated so as not to cause localised scour near a pipe or channel outfall. Design of the inlet structures for adequate protection against scour is common hydraulic engineering practice (see e.g. Chow 1959; Henderson 1966).

Litter control is also normally required at an inlet structure and it is generally recommended that some form of **gross pollutant trap (GPT)** be installed as part of an inlet structure. The provision of a GPT will depend on catchment activities as well as any upstream measures in place. Several proprietary products are available for removing gross pollutants (see Engineers Australia 2003, Chapter 7). The storage capacity of GPTs should be sized to ensure that maintenance (clean-out) frequency is not greater than once every three months.

4.3.4.2 Outlet structure

An outlet structure of a sedimentation basin can be configured in many ways and depends on the specified operation of the system (e.g. whether as a 'stand-alone' sedimentation basin for managing construction site runoff or as part of a wetland). The outlet structure generally consists of an outlet pit and a discharge control structure to control the rate of discharge from the basin under normal operation. The discharge control structure should have adequate capacity to convey the design operation flow.

Landscape amenity is not an important design outcome for a sedimentation basin used for managing runoff from a construction site.. Therefore, floating discharge control structures are considered to be the most effective outlets for sedimentation basins for construction sites (Figure 4.6). They draw flows from the surface, which generally have the lowest suspended sediment concentrations. The discharge control structure consists of one or more slotted pipes mounted with floats to enable them to rise with the progressive filling of the basin (Figure 4.6). Discharge from the basin is maintained at a relatively constant rate independent of the depth of water in the basin.

With sedimentation basins that also serve as a landscape element, a more appropriate discharge control structure is a **weir**. Where possible, a narrow weir (Figure 4.6) should be

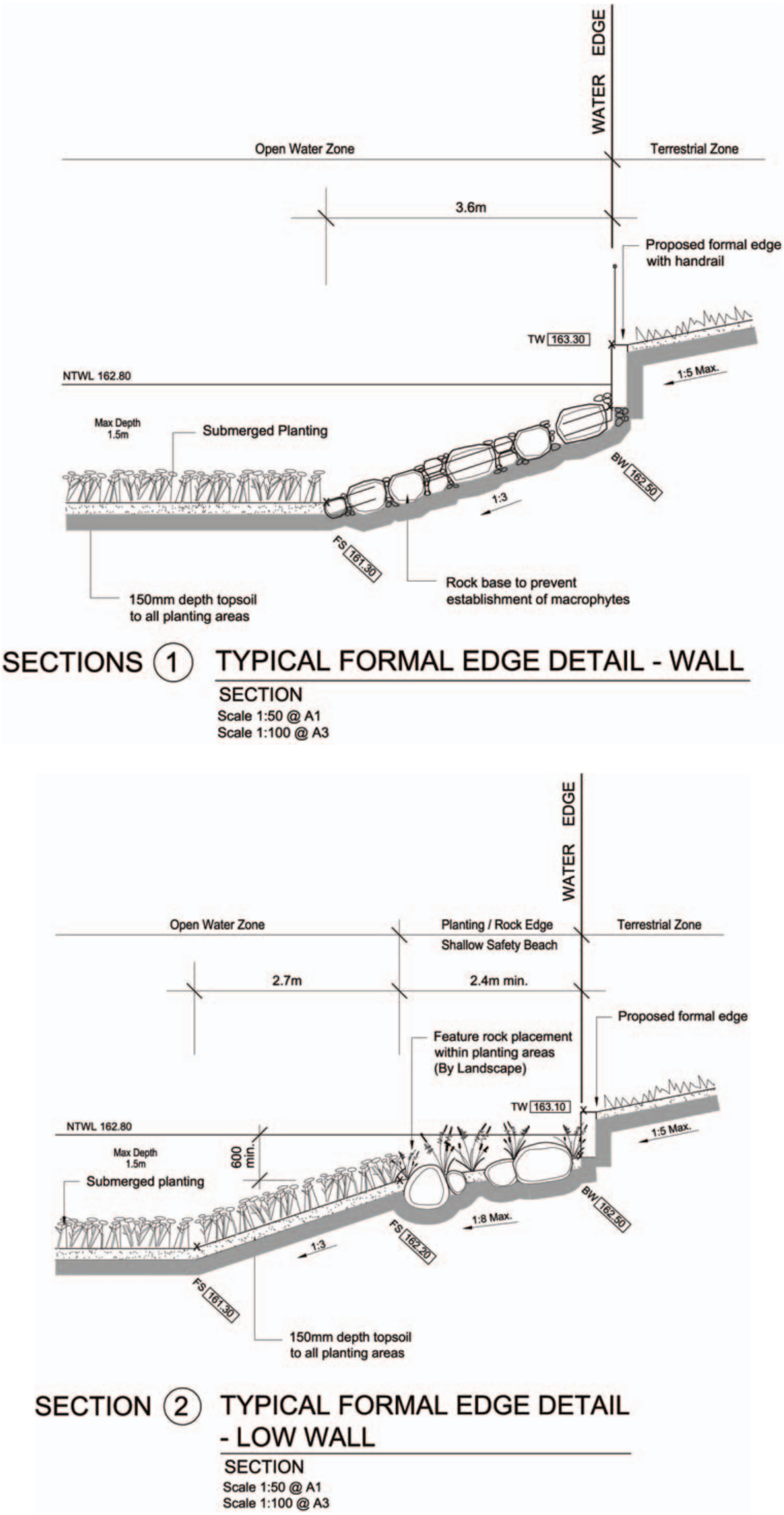


Figure 4.5 Hard edge treatment for open waterbodies (Graeme Bentley Landscape Architects 2004).

Figure 4.5 (Continued)

Figure 4.6 Sedimentation basin outlet structures: (a) a floating skimmer and (b) a narrow weir.

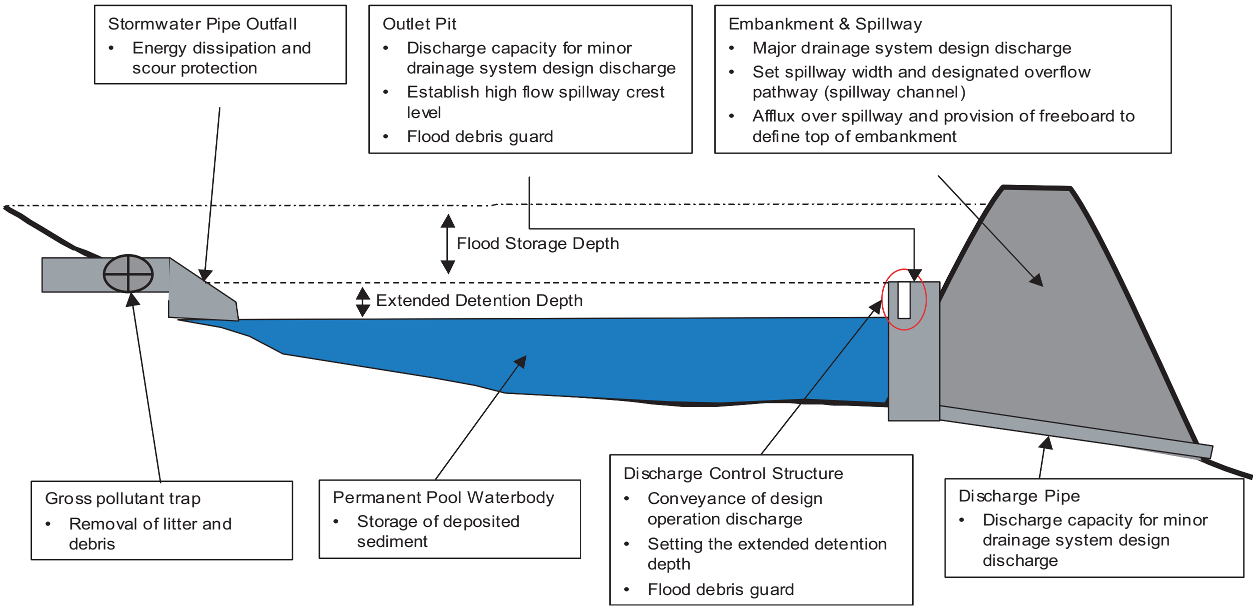


Figure 4.7 Overview of design elements of a sedimentation basin and main design considerations.

adopted to promote a larger range of extended detention depths while ensuring adequate capacity to convey the design discharge.

Dimensions of an outlet should ensure that the perimeter is sufficiently long to pass the design discharge into the connecting pipe, to either (i) a downstream treatment measure or (ii) receiving waters or downstream stormwater drainage infrastructure. In (i), the one-year ARI peak discharge is normally adopted as the design discharge whereas in (ii), the peak discharge corresponding to the design discharge for the minor stormwater drainage system should be adopted.

Design of an outlet pit and associated discharge control structure include the following:

- placement of the crest of the pit at or above the permanent pool level of the sedimentation basin
- sizing the pit to provide discharge capacity that is greater than the discharge capacity of the outlet culvert
- protection against blockage by flood debris.

Figure 4.7 summarises the design elements of the various components of a sedimentation basin.

Outlet pit

An outlet pit is sized with a discharge capacity of the minor drainage system (e.g. five-year ARI). The dimension of an outlet pit is determined by considering two flow conditions, (i) weir and (ii) orifice flow (Equations 4.5 and 4.6).

A blockage factor (B) is also used to account for any debris blockage. An assumption that the outlet is 50% blocked is recommended (i.e. $B = 0.5$). Generally it will be the discharge pipe from the sediment basin (and downstream water levels) that controls the maximum flow rate from the basin; it is therefore less critical if the outlet pit is oversized to allow for blockage.

1. Weir flow condition – usually when the extended detention storage of the retarding basin is not fully engaged, that is:

$$P = \frac{Q_{des}}{B \times C_w \times H^{1.5}} \qquad \text{(Equation 4.5)}$$

where P = perimeter of the outlet pit;
B = blockage factor (0.5);
H = depth of water above the crest of the outlet pit (m);

Q_{des} = Design discharge (m^3/s);
C_w = weir coefficient (1.7).

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

$$A_o = \frac{Q_{des}}{B \times C_d\sqrt{2gH}} \qquad \text{(Equation 4.6)}$$

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);
g = Acceleration due to gravity (9.81 m/s^2)

It is important that an outlet pit is prevented from blockage by debris. Design consideration needs to include means of preventing blockage of the outlet structure (Figure 4.8).

Figure 4.8 Debris protection for outlet pits.

Discharge control structure

Three types of discharge control structures can be used.

1. Overflow weir – the length of the weir is computed with the weir flow equation (Equation 4.5) but checked to ensure that there is adequate discharge capacity when the structure operates under submerged conditions using the orifice flow equation (Equation 4.6).
2. **Riser outlet** – a vertical pipe with orifices located along the length of the pipe. The placement of outlet orifices and determining their appropriate diameters is designed iteratively by varying outlet diameters and levels, using the orifice equation (Equation 4.6) applied over discrete depths along the length of a riser up to the maximum detention depth. This can be performed with a spreadsheet as illustrated in the worked example (See Section 4.6 and Chapter 9).
3. Floating slotted pipe – the size and number of slots required to pass the operation design flow can be computed using the orifice flow equation (Equation 4.6).

With riser-type structures to control discharge , an outlet orifice is likely to be small and it is important that these are prevented from clogging by debris. Some form of debris guard is recommended (Figure 4.9).

Figure 4.9 Debris guards for outlet structures.

4.3.5 Overflow structure

The provision of a high-flow overflow structure is an essential design element. An overflow structure is normally a weir spillway structure. The required length of the spillway can be computed using the weir flow equation (Equation 4.5) with the design discharge being selected according to discussion in Section 4.3.1.1.

4.3.6 Vegetation specification

Vegetation planted along the **littoral zone** of a sedimentation basin serves the primary function of inhibiting public access to the open waterbody and preventing edge erosion. Terrestrial

Figure 4.10 Overflow structure of a sedimentation basin.

planting beyond the littoral zone may also be recommended to screen areas and provide an access barrier to uncontrolled areas of the stormwater treatment system. A list of suggested plant species suitable for a sedimentation basin littoral zone in Victoria is provided in Appendix A.

4.3.7 Design calculation summary

The *Sedimentation Basin Calculation Checklist* is a design calculation summary sheet for the key design elements of a sedimentation basin. It has been included to aid the design process.

Sedimentation Basin — **CALCULATION CHECKLIST**

	CALCULATION TASK	OUTCOME		CHECK
1	**Identify design criteria**			☐
	Design ARI flow for inlet hydraulic structures		year	
	Design ARI flow for outlet hydraulic structures		year	
	Design ARI for overflow hydraulic structures		year	
2	**Catchment characteristics**			☐
	Residential		ha	
	Commercial		ha	
	Roads		ha	
	Fraction impervious			☐
	Residential			
	Commercial			
	Roads			
3	**Estimate design flow rates**			
	Time of concentration			
	Estimate from flow path length and velocities		minutes	☐
	Identify rainfall intensities			
	station used for IFD data:			
	Design rainfall intensity for inlet structure(s)		mm/hr	☐
	Design runoff coefficient			
	Inlet structure(s)			☐
	Peak design flows			☐
	Inlet structure(s)		m^3/s	
	Outlet structure(s)		m^3/s	
	Overflow structure(s)		m^3/s	
4	**Basin dimension and layout**			☐
	Area of sedimentation basin		m^2	
	Aspect ratio		L:W	
	Hydraulic efficiency			
	Depth of permanent pool		m	
	Permanent pool volume		m^3	
	Cross section batter slope		V:H	
5	**Basin performance**			☐
	Capture efficiency (of 125 μm sediment)		%	
	Sediment cleanout frequency		years	
6	**Hydraulic structures**			
	Inlet structure			☐
	Provision of energy dissipation			
	Outlet structure			☐
	Pit dimension		L x B	
	or		mm diam	
	Discharge capacity of outlet		m^3/s	
	Provision of debris trap			
	Discharge pipe			☐
	Discharge capacity of discharge pipe		m^3/s	
7	**Spillway**			☐
	Discharge capacity of spillway		m^3/s	

4.4 Checking tools

Checking aids are included for designers and referral authorities. In addition, advice on construction techniques and lessons learnt from building sediment basins are provided.

Checklists are provided for:

- design assessments
- construction (during and post)
- operation and maintenance inspections
- asset transfer (following defects period).

4.4.1 Design assessment checklist

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

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

Where an item results in an 'N' when reviewing the design, the design procedure should be assessed to determine the effect of the omission or error.

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

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

4.4.2 Construction advice

General advice is provided for the construction of sedimentation basins. It is based on observations from construction projects around Australia.

Building phase damage

It is important to protect a sediment basin from upstream flows during its construction. A mechanism to divert flows around a construction site, protection from litter and debris is required.

High flow contingencies

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

Maintenance access

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

Solid base

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

Dewatering removed sediments

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

Inlet checks

It is good practice to check the operation of inlet erosion protection measures following the first few rainfall events. These need to be checked early in the system's life, to avoid continuing problems. If problems occur in these events, then erosion protection should be enhanced.

4.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Sediment basin

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

SITE: ____________________

CONSTRUCTED BY: ____________________

DURING CONSTRUCTION									
'tems inspected	Checked		Satisfactory	Unsatisfactory		Checked		Satisfactory	Unsatisfactory
Preliminary works	Y	N			**Structural components**	Y	N		
1. Erosion and sediment control plan adopted					11. Location and levels of outlet as designed				
					12. Safety protection provided				
2. Limit public access					13. Pipe joints and connections as designed				
3. Location same as plans					14. Concrete and reinforcement as designed				
4. Site protection from existing flows					15. Inlets appropriately installed				
Earthworks					16. Inlet energy dissipation installed				
5. Integrity of banks					17. No seepage through banks				
6. Batter slopes as plans					18. Ensure spillway is level				
7. Impermeable (solid) base installed					19. Provision of maintenance drain				
8. Maintenance access (e.g. ramp) installed					20. Collar installed on pipes				
9. Compaction process as designed					**Vegetation**				
10. Levels of base, banks and spillway as designed					21. Stablisation immediately following earthworks				
					22. Planting as designed (species and densities)				
					23. Weed removal before stabilisation				
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of banks				
2. Confirm structural element sizes					7. Inlet erosion protection working				
3. Check batter slopes					8. Maintenance access provided				
4. Vegetation as designed					9. Construction generated sediment removed				
5. Draining area for maintenance provided									

COMMENTS ON INSPECTION

ACTIONS REQUIRED

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

4.4.4 Asset handover checklist

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

4.5 Maintenance requirements

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

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

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

4.5.1 Operation and maintenance inspection form

The *Sediment Basin Maintenance Checklist* is designed to be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

4.6 Sedimentation basin worked example

4.6.1 Worked example introduction

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

Sediment Basin Maintenance Checklist			
Inspection frequency:	3 monthly	Date of visit:	
Location:			
Description:			
Site visit by:			
Inspection items	Y	N	Action required (details)
Litter within inlet or open water zones?			
Sediment within inlet zone requires removal (record depth, remove if >50%)?			
Overflow structure integrity satisfactory?			
Evidence of dumping (building waste, oils etc.)?			
Terrestrial vegetation condition satisfactory (density, weeds etc.)?			
Weeds require removal from within basin?			
Settling or erosion of bunds/batters present?			
Damage/vandalism to structures present?			
Outlet structure free of debris?			
Maintenance drain operational (check)?			
Comments:			

system. The site is triangular with a surface area of 500 m^2 (Figure 4.11). Road runoff is conveyed by conventional stormwater pipes (up to the 100-year ARI event) and there are two freeway outfall pipes that discharge to the two top apexes of the site. Each outfall services about 1 km of the freeway with the total contributing area of 4 ha (90% impervious) to each outfall. The site of the sedimentation basin has a fall of about 2 m (from 5 m-AHD (Australian Height Datum) to 3 m-AHD) towards a degraded watercourse.

Site constraints limit the size available for the stormwater treatment system. In principle, when available space is constrained, the size of the inlet zones (i.e. sedimentation basins) should not be compromised, to ensure that larger sediments are effectively trapped and prevented from smothering the **macrophyte zone** (thereby creating future maintenance problems).

Therefore, if the site constrains the total size of the treatment system, the macrophyte zone should be reduced accordingly. This will reduce the overall **hydrologic effectiveness** of the system (i.e. the proportion of Mean Annual Runoff, MAR, subjected to the full wetland treatment), but not its functional integrity.

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

4.6.1.1 Design objectives

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

- promote sedimentation of particles larger than 125 μm with a 90% capture efficiency for flows up to the one-year ARI (unattenuated) peak discharge
- provide for connection to the downstream macrophyte zone with discharge capacity corresponding to the one-year ARI (unattenuated) peak discharge
- provide for a bypass operation when the inundation of the downstream macrophyte zone reaches the design maximum extended detention depth with a discharge capacity corresponding to the 100-year ARI peak discharge.

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

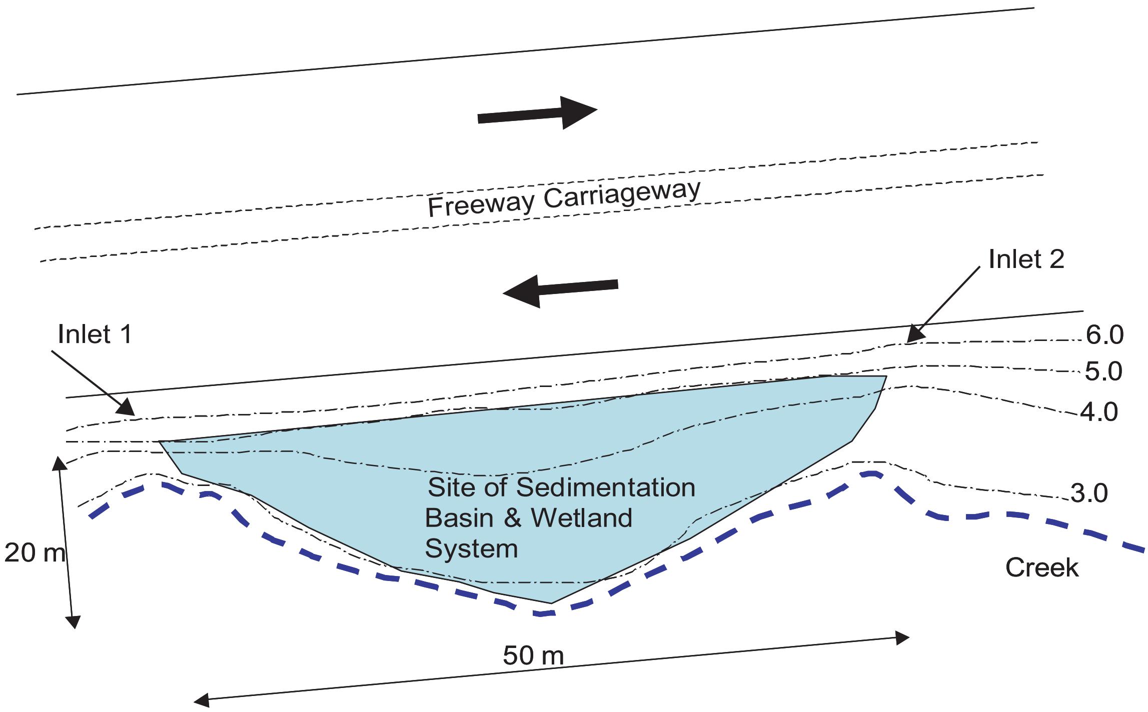


Figure 4.11 Layout of proposed site for sedimentation basin.

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

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

- littoral zone vegetation
- terrestrial vegetation.

4.6.2 Estimating design flows

The procedures in Australian Rainfall and Runoff (ARR) (Institution of Engineers 1987) are used to estimate the design flows. The site has two contributing catchments, each catchment is 4 ha, 1 km long (along the freeway) and is drained by culverts. Velocity within the pipes is assumed to be 1 m/s for the purposes of estimating the time of concentration (t_c):

$$
\begin{aligned}
t_c &= 1000 \text{ m}/1 \text{ m/s} \\
&= 1000 \text{ s} = 17 \text{ minutes)}
\end{aligned}
$$

Rainfall intensities for Geelong (for the 1, 10 and 100-year average recurrence intervals) are estimated using ARR (Institution of Engineers 2001) with a time of concentration of 17 minutes and are:

$$
\begin{aligned}
I_1 &= 27 \text{ mm/hr} \\
I_{10} &= 56 \text{ mm/hr} \\
I_{100} &= 95 \text{ mm/hr}
\end{aligned}
$$

Runoff coefficients as per ARR (Institution of Engineers 2001):

$$ {}^{10}I_1 = 26.4 \text{ mm/hr} $$

Fraction impervious, $F_{imp} = 0.9$:

$$ C_{10}^{1} = 0.12 $$
$$ C_{10} = 0.82 $$

Runoff coefficients are as per Table 1.6 Institution of Engineers, Book VIII of ARR (2001)

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

From the Rational Method Design Procedure:

$Q = CIA/360$; $Q_1 = 0.20 \text{ m}^3/\text{s}$; $Q_{10} = 0.51 \text{ m}^3/\text{s}$; and $Q_{100} = 1.0 \text{ m}^3/\text{s}$.

Operation design discharge = $0.20 \text{ m}^3/\text{s}$
Design discharge for connection to macrophyte zone = $0.20 \text{ m}^3/\text{s}$
Spillway design discharge = $1.0 \text{ m}^3/\text{s}$

4.6.3 Size and shape of sedimentation basin

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

Pollutant removal is estimated using Equation 4.3:

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

An aspect ratio of 1 (w) to 4 (L) is adopted based on the available space (Figure 4.11). Using Figure 4.3, the hydraulic efficiency λ is estimated to be 0.4. This value is less than desirable; however, site constraints prevent any other configuration. The turbulence factor $\boldsymbol{n}$ is computed from Equation 4.2 to be 1.67. Thus,

Hydraulic efficiency $\lambda = 0.4$

Turbulence factor ($\boldsymbol{n}$) = 1.67.

The proposed extended detention depth of the basin is 0.25 m (as outlined in Section 4.6.1.1) and a notional permanent pool depth of 2 m has been adopted, that is:

$$ d_p = 2.0 \text{ m} $$
$$ d^* = 1.0 \text{ m} $$
$$ d_e = 0.25 \text{ m} $$

$v_s = 0.011$ m/s for 125 μm particles
Q = design operation flow rate = $0.20 \text{ m}^3/\text{s}$.

From Equation 4.3, the required sedimentation basin area to achieve target sediment (125 μm) capture efficiency of 90% is 50 m^2. With a W to L ratio of 1:4, the notional dimensions of the basin are 3.5 m × 14 m. This size is validated against the curves presented in Figure 4.2.

The available sediment storage is 50 × 2 = 100 m^3. Clean-out is to be scheduled when the storage is half full, therefore the available sediment storage prior to clean-out is 50 m^3.

The required volume of sediment storage to ensure cleaning is not required more frequently than every five years is estimated using Equation 4.4 (using a sediment discharge rate of 1.6 m^3/ha per year).

$$\text{Required storage } (S_t) = C_a \times R \times L_0 \times F_r$$
$$= 4 \times 0.9 \times 1.6 \times 5 = 29 \text{ m}^3$$

Available storage volume is 50 m^3, and therefore it is OK.

The required clean-out frequency is estimated to be (by rearranging Equation 4.4):

$$\text{Frequency fo basin desilting} = \frac{0.5 \times 100}{1.6 \times 4 \times 0.9} = 8.6 \text{ years} > 5 \text{ years} \rightarrow \text{OK}$$

Open water area = 50 m^2
Width = 3.5 m; Length = 12 m
Depth of permanent pool (d_p) = 2.0 m
Depth of extended detention (d_e) = 0.25 m.

4.6.4 Hydraulic structure design

4.6.4.1 Inlet structure

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

$$(Q_{10} = 0.51 \text{ m}^3)/\text{s}$$

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

Energy dissipation and erosion protection will need to be provided in the form of rock beaching at the inlet structure; Q_{des} = 0.51 m^3/s (see Section 4.6.2).

4.6.4.2 Outlet structure

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

According to Section 4.4.2, two possible flow conditions need to be checked [i.e. weir flow conditions (with extended detention of 0.25 m) and orifice flow conditions].

Weir flow conditions

From Equation 4.5, the required perimeter of the outlet pit to pass 0.2 m^3/s with an **afflux** of 0.25 m can be calculated:

$$P = \frac{Q}{B \times C_w \times H^{1.5}} = \frac{0.2}{0.5 \times 1.7 \times 0.25^{1.5}} = 1.88 \text{ m}$$

Orifice flow conditions

From Equation 4.6, the required area of the outlet pit can be calculated as follows:

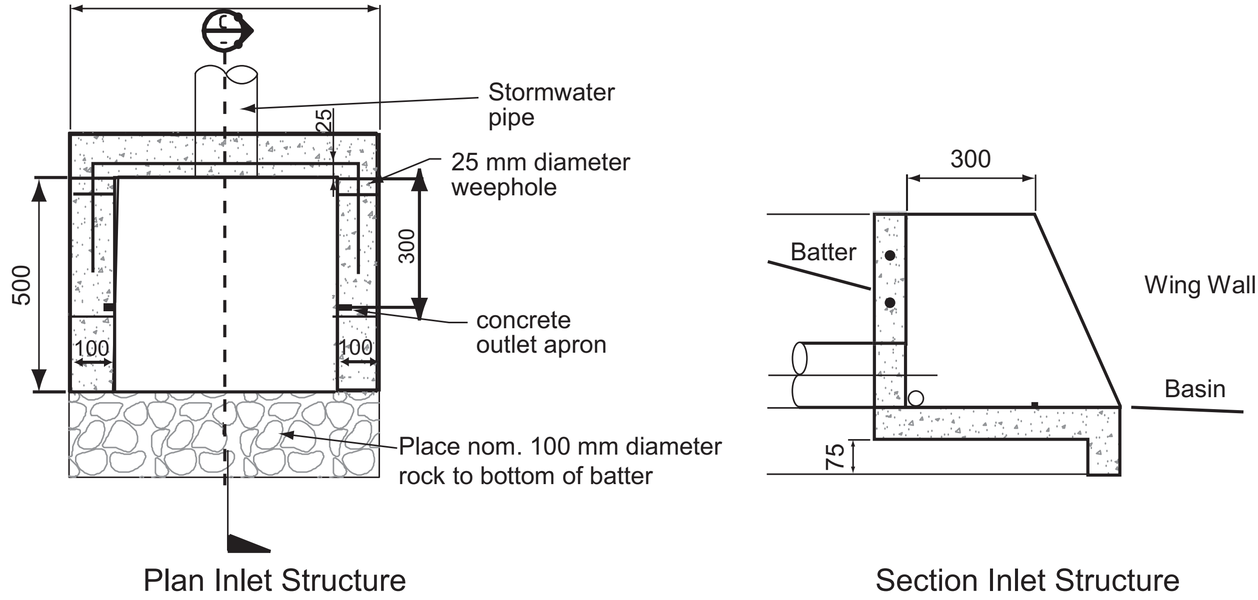


Figure 4.12 Inlet structure.

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

Adopt 600 × 600 mm pit: area = 0.36 m^2; perimeter = 2.4 m; Q_{cap} = 0.24 m^3/s → OK.

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

Outlet pit = 600 x 600 mm diameter with standard grating.

4.6.4.3 Overflow structure

The overflow structure is to discharge Q_{100} peak flow. The overflow structure is an overflow weir with a crest elevation set at 0.25 m (i.e. d_e) above the permanent pool level. The length of this weir determines the afflux for the 100-year ARI peak discharge and sets the top of embankment of the sedimentation basin. It is common practice to allow for 300 mm of freeboard above the afflux level when setting the top of embankment elevation. An afflux of 0.3 m has been adopted in defining the length of the spillway weir. This value was adopted as a trade off between the bank height and the width of the weir. A bank height of 600 mm (300 mm afflux and 300 mm freeboard) above the normal water level was deemed acceptable. The length is calculated using the weir flow equation with a weir coefficient of 1.7, that is:

$$L = \frac{Q_{des}}{C_w \times H^{1.5}} = \frac{1.0}{1.7 \times 0.3^{1.5}} = 3.6 \text{ m} \qquad \text{(Equation 4.7)}$$

where L represents the length of the weir.

A bypass weir is located adjacent to inflow culvert to minimise risk of sediment scour.
Spillway length = 3.6 m set at 0.25 m above permanent pool level.
Top of embankment set at 0.6 m above the permanent pool level.

4.6.4.4 Discharge to macrophyte zone

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

The design calculation and configuration of this connection is described in Chapter 9 on constructed wetland design.

4.6.5 Design calculation summary

Sedimentation Basin	CALCULATION SUMMARY		
CALCULATION TASK	**OUTCOME**		**CHECK**
1 Identify design criteria			✓
Design ARI flow for inlet hydraulic structures	10	year	
Design ARI flow for outlet hydraulic structures	1	year	
Design ARI for overflow hydraulic structures	100	year	
2 Catchment characteristics			✓
Residential	0	ha	
Commercial	0	ha	
Roads	4	ha	
Fraction impervious			✓
Residential	N/A		
Commercial	N/A		
Roads	0.9		
3 Estimate design flow rates			
Time of concentration			
Estimate from flow path length and velocities	17	minutes	✓
Identify rainfall intensities			
station used for IFD data:	Geelong		
Design rainfall intensity for inlet structure(s)	27 to 56	mm/hr	✓
Design runoff coefficient			
Inlet structure(s)	0.66 to 0.98		✓
Peak design flows			✓
Inlet structure(s)	0.51	m^3/s	
Outlet structure(s)	0.20	m^3/s	
Overflow structure(s)	1.00	m^3/s	
4 Basin dimension and layout			✓
Area of sedimentation basin	50	m^2	
Aspect ratio	4(L):1(W)	L:W	
Hydraulic efficiency	0.4		
Depth of permanent pool	2	m	
Permanent pool volume	100	m^3	
Cross section batter slope	1(V):8(H)	V:H	
5 Basin performance			✓
Capture efficiency (of 125 μm sediment)	90	%	
Sediment cleanout frequency	8.6	years	
6 Hydraulic structures			
Inlet structure			✓
Provision of energy dissipation	Y		
Outlet structure			✓
Pit dimension	600 x 600	L x B	
or		mm diam	
Discharge capacity of outlet	0.21	m^3/s	
Provision of debris trap	Y		
Discharge pipe			✓
Discharge capacity of discharge pipe	0.2	m^3/s	
7 Spillway			✓
Discharge capacity of spillway	1	m^3/s	

4.6.6 Construction drawings

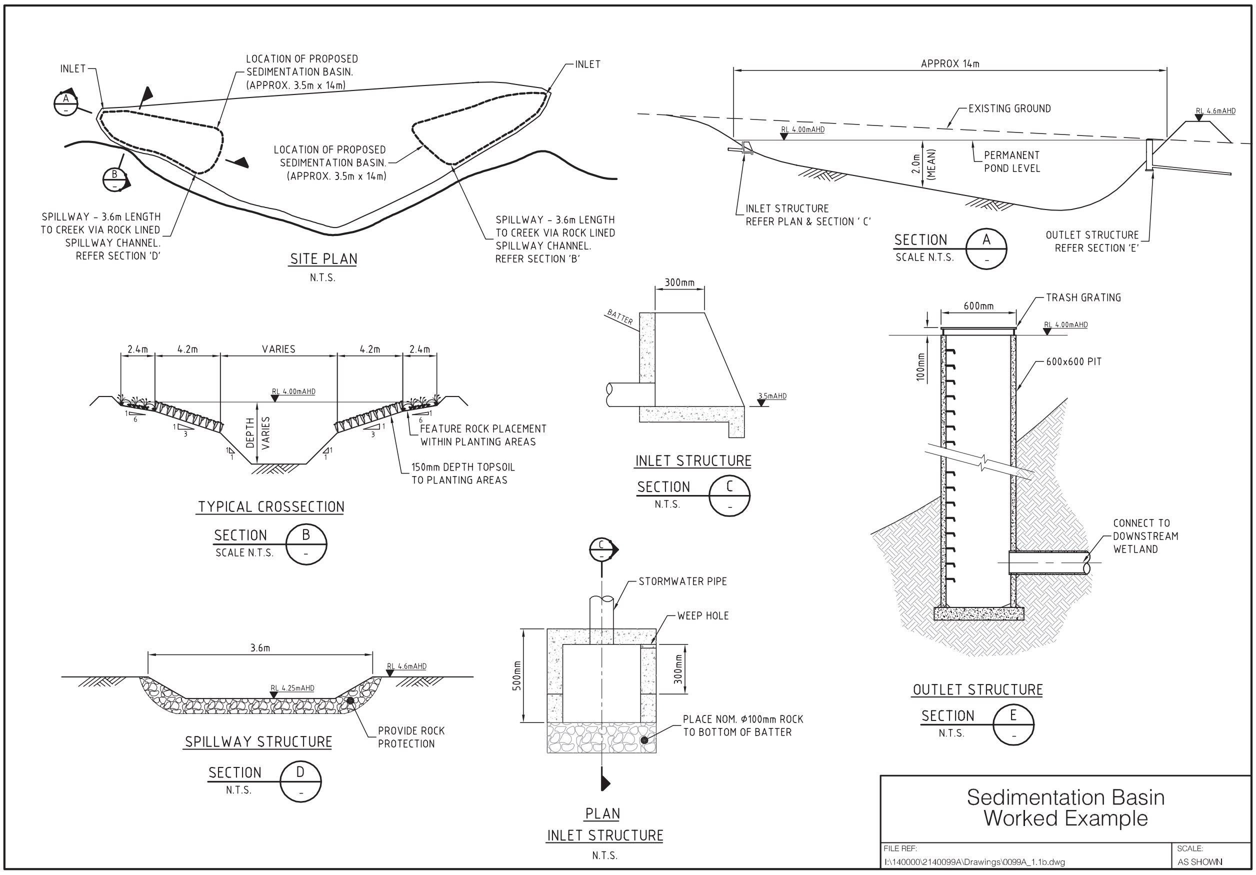


Figure 4.13 Construction drawing of the sedimentation basin worked example.

4.7 References

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Chapter 5 **Bioretention swales**

Bioretention swale in Zetland, NSW

5.1 Introduction

Bioretention swales provide both **stormwater** treatment and conveyance functions. A bioretention system is installed in the base of a **swale** that is designed to convey minor floods. The swale component provides pretreatment of stormwater to remove coarse to medium sediments while the bioretention system removes finer particulates and associated contaminants. Figure 5.1 shows the layout of a bioretention swale.

A bioretention system can be installed in part of a swale, or along the full length of a swale, depending on treatment requirements. Typically, these systems should be installed with slopes of between 1% and 4 %. In steeper areas, **check dams** are required to reduce flow velocities. For milder slopes, adequate drainage needs to be provided to avoid nuisance ponding (a bioretention system along a full length of the swale will provide this drainage).

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

To design the bioretention swale, separate calculations are performed to design the swale and the bioretention system, with iterations to ensure appropriate criteria are met in each section. Depending on the length of the swale and steepness of the terrain, check dams can be used to manage steep slopes and also to provide ponding over a bioretention surface. In this way increased volumes of runoff can be treated through a bioretention system prior to bypass.

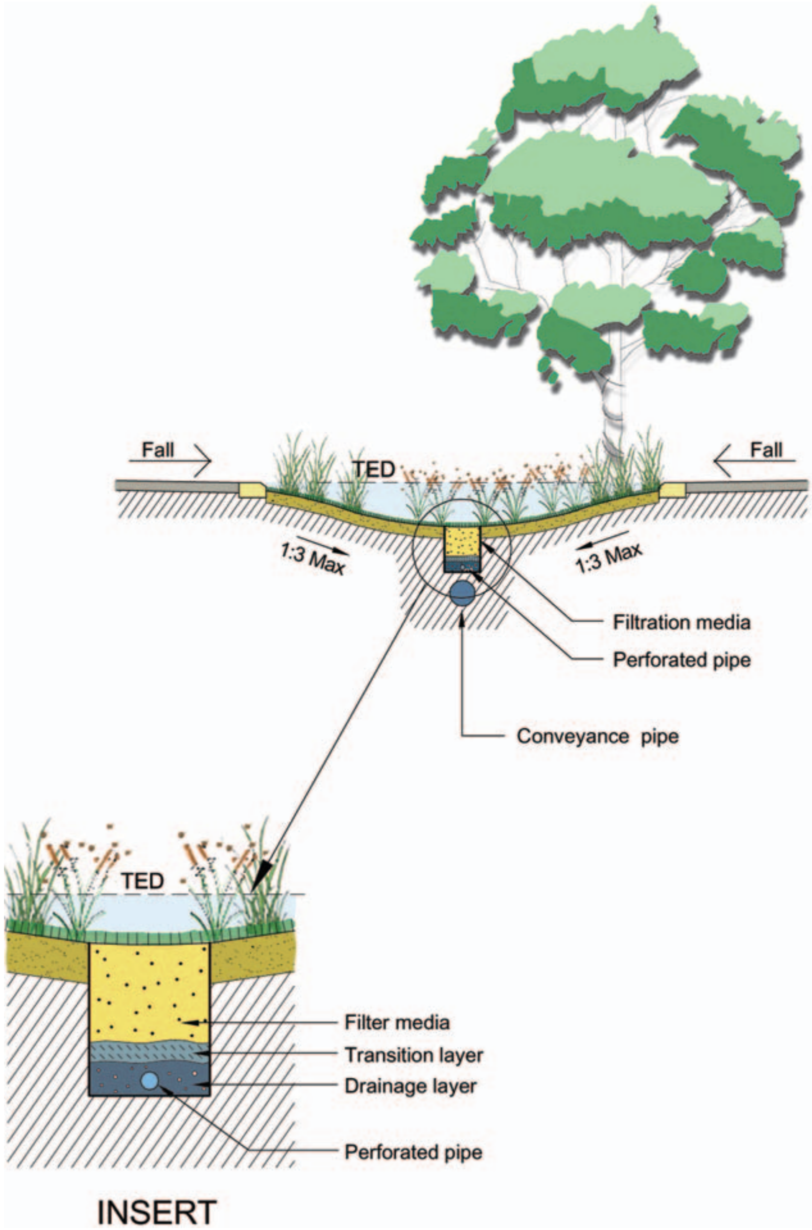


Figure 5.1 Bioretention swale as a centre road median

In many urban situations, the width available for a swale system will be fixed (as well as the longitudinal slope); therefore, the length of the swale to convey a minor storm safely will also be fixed. A common way to design these systems is as a series of discrete 'cells' (e.g. Figure 5.2). Each cell has an overflow pit that **discharges** flow to an underground pipe system. Bioretention systems can then be installed directly upstream of the overflow pits. This also allows an area for ponding over the **filtration media**.

As flood flows are conveyed along the bioretention surface, velocities need to be kept low to avoid scouring of collected pollutants and vegetation.

Bioretention swales can be installed at various scales, for example, in local streets or on large highways.

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

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

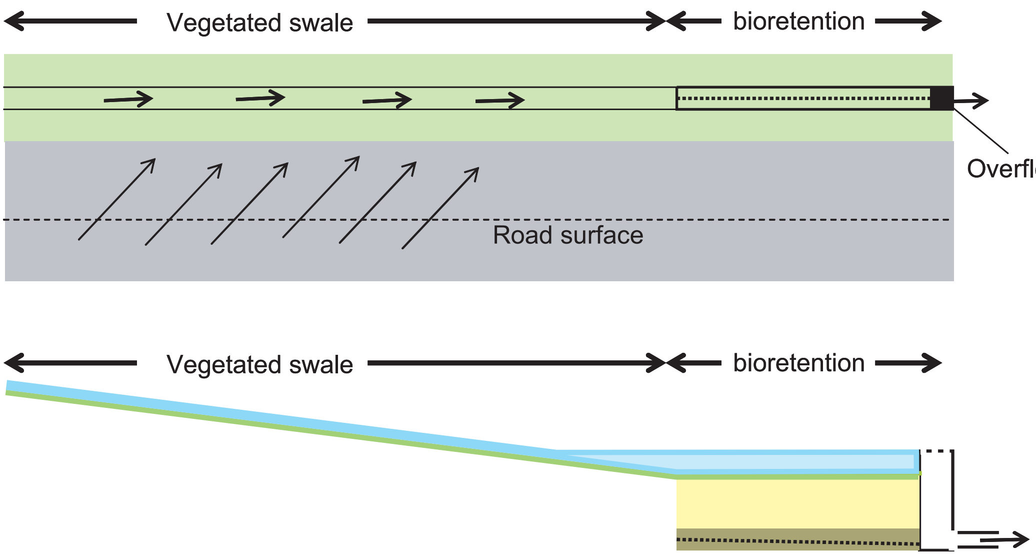


Figure 5.2 Bioretention swale example layout

Where bioretention systems are not intended to be infiltration systems, the dominant pathway for water is not via discharge into groundwater. Rather, these systems convey collected water to downstream waters (or collection systems for reuse) with any loss in runoff mainly attributed to maintaining soil moisture of the filter media itself (which is also the growing media for the vegetation).

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

Runoff is filtered through a fine media layer as it percolates downwards. It is then collected via perforated pipes and flows to downstream waterways or storages for reuse (e.g. Figure 5.3).

Vegetation that grows in the filter media enhances its function by preventing erosion of the filter medium, continuously breaking up the soil through plant growth to prevent clogging of the system and providing **biofilms** on plant roots that pollutants can adsorb to. The type of vegetation varies depending on landscaping requirements. Generally, the denser and higher the vegetation, the better the filtration. Vegetation is critical to maintaining porosity of the filtration layer. Selection of an appropriate filtration media is a key issue. Sufficient hydraulic conductivity (i.e. passing water through the filtration medium as quickly as possible) needs to be balanced with stormwater detention for treatment and provision of a suitable growing medium to support vegetation growth (i.e. retaining sufficient soil moisture and organic content). Typically a sandy loam is suitable, but soils can be tailored to a vegetation type.

A bioretention trench could consist of three layers (Figure 5.3). In addition to the filtration media, a drainage layer is required to convey treated water into the perforated underdrains. This

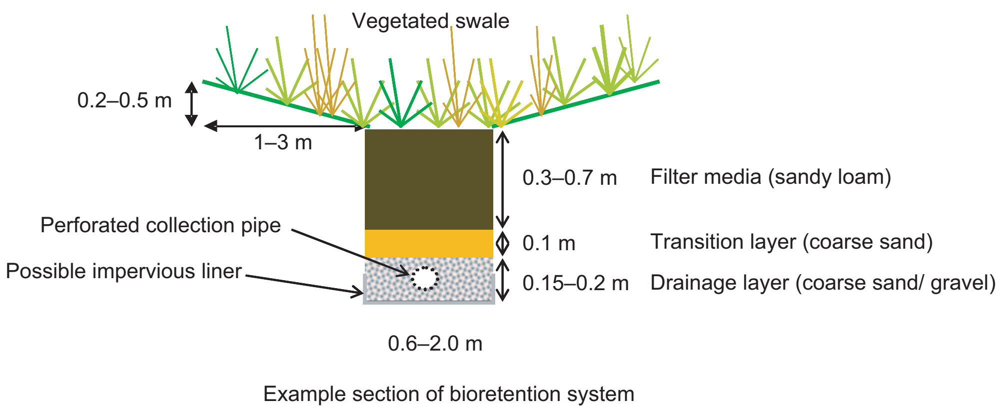


Figure 5.3 Typical section of a bioretention swale.

material surrounds the perforated underdrainage pipes and can be either coarse sand (1 mm) or fine gravel (2–5 mm). Should fine gravel be used, it is advisable to install a **transition layer** of sand or a geotextile fabric (with a mesh size equivalent to sand size) to prevent any filtration media being washed into the perforated pipes.

Another design component is keeping traffic and deliveries off bioretention swales. Traffic tends to ruin the vegetation, provide ruts that cause preferential flow paths that do not offer filtration and compact the filter media, thus reducing treatment flows. Traffic can be controlled by selecting vegetation that discourages the movement of traffic or by providing physical barriers. For example, barrier kerbs with breaks in them (to allow distributed water entry, albeit with reduced uniformity of flows compared with flush kerbs) or bollards along flush kerbs can be used to prevent vehicle movement onto swales.

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

Key design issues to be considered are:

1 verifying size and configuration for treatment
2 determining design capacity and treatment flows
3 calculating dimensions of the swale
4 specifying details of the filtration media
5 checking above-ground components:
   - velocities
   - design of inlet zone and overflow pits
   - above design flow operation
6 checking below-ground components:
   - soil media layer characteristics (filter, transition and drainage layers)
   - underdrain design and capacity
   - requirements for bioretention lining
7 recommending plant species and planting densities
8 providing maintenance.

5.2 Verifying size for treatment

The curves (Figures 5.4–5.6) show the pollutant removal performance expected for bioretention systems (either swales or basins) with varying depths of ponding. An important consideration with bioretention swales is to estimate an average ponding depth as the average depth is less than the maximum depth if the surface of the bioretention system is sloped with the swale.

The curves are based on the performance of the system in Melbourne and were derived using the Model for Urban Stormwater Improvement Conceptualisation (**MUSIC**) (Cooperative Research Centre for Catchment Hydrology 2003). To estimate an equivalent performance at other locations in Victoria, the hydrologic design region relationships should be used to convert the treatment area into an equivalent treatment area in Melbourne (reference site) (see Chapter 2). In preference to using the curves, local data should be used to model the specific treatment performance of the system.

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

- hydraulic conductivity of 180 mm/hr
- filtration media depth of 600 mm
- filter media particle size (d_{50}) of 0.45 mm.

These curves can be used to check the expected performance of the bioretention system for removal of Total Soluble Solids (TSS), Total Phosphorus (TP) and Total Nitrogen (TN). The X-axis is the area of bioretention expressed as a percentage of the bioretention area of the *impervious* contributing **catchment** area.

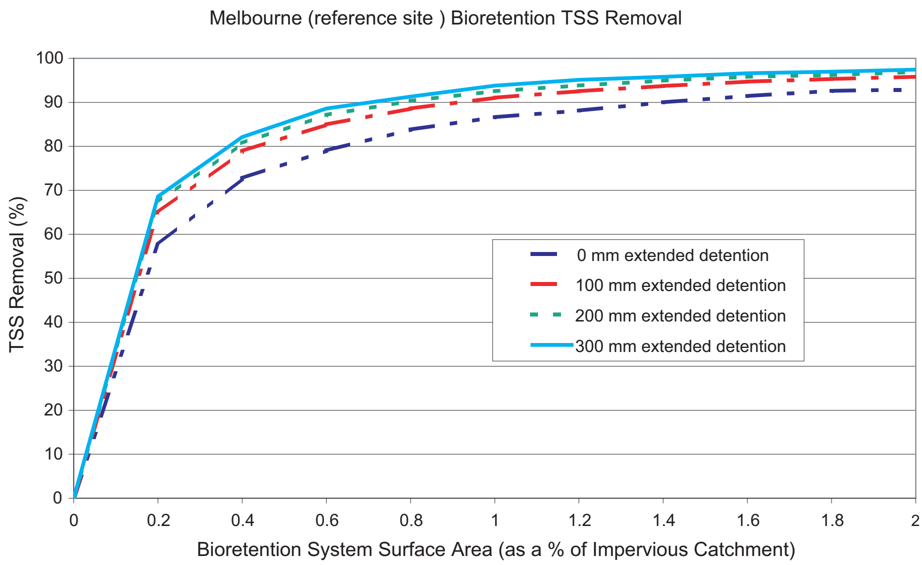


Figure 5.4 Performance of a bioretention system in removing Total Soluble Solids (TSS) in Melbourne.

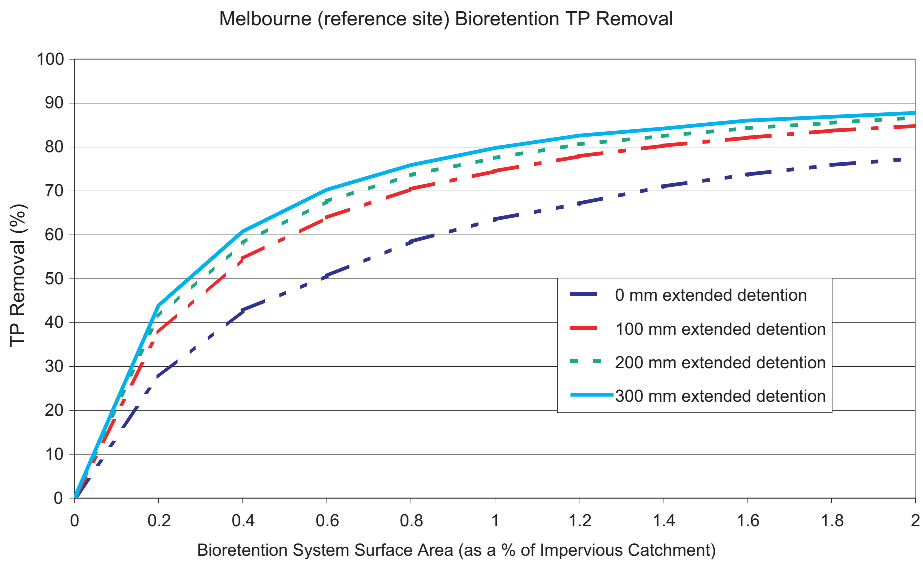


Figure 5.5 Performance of a bioretention system in removing Total Phosphorus (TP) in Melbourne.

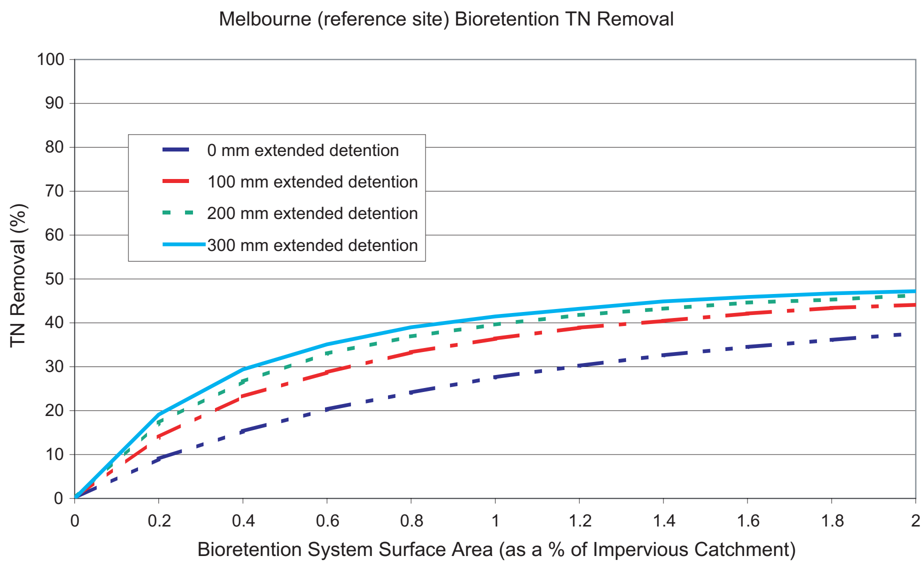


Figure 5.6 Performance of a bioretention system in removing Total Nitrogen (TN) in Melbourne.

5.3 Design procedure: bioretention swales

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

5.3.1 Estimating design flows

Three design flows are required for bioretention swales:

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

5.3.1.1 Minor and major flood estimation

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

5.3.1.2 Maximum infiltration rate

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

A maximum infiltration rate (Q_{max}) can be estimated by applying Darcy's equation (Equation 5.1):

$$Q_{max} = k \times L \times W_{base} \times \frac{h_{max} + d}{d} \qquad \text{(Equation 5.1)}$$

where k is the hydraulic conductivity of the soil filter (m/s);
W_{base} is the base width of the ponded cross section above the soil filter (m);
L is the length of the bioretention zone (m);
h_{max} is the depth of pondage above the soil filter (m);
d is the depth of filter media.

5.3.2 Swale design

The swale design of a bioretention swale needs to be determined first to set the broad dimensions of the system. Typically the swale will be trapezoidal in shape with side slopes ranging from 1:9 to 1:3 (gradient) depending on local council regulations and any requirements for driveway crossings. The base of the swale is where a bioretention system can be installed. A minimum base width of 300 mm is suggested; however, this would more typically be 600–1000 mm.

The swale design either involves determining the width of swale required to pass the design flow for the minor drainage system if the catchment areas are known or determining the maximum length of swale prior to discharge into an overflow pit (i.e. maximum length of each cell) for a given width of swale.

Manning's equation is used to size the swale given the site conditions. Selection of an appropriate **Manning's *n*** is a critical consideration (see Section 5.3.2.2) and this will vary depending on the vegetation type. Consideration of landscape and maintenance elements of vegetation will need to be made before selecting a vegetation type.

5.3.2.1 Slope considerations

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

Selection of an appropriate side slope depends on local council regulations and will relate to traffic access and the provision of driveway crossings (if required). The provision of driveway

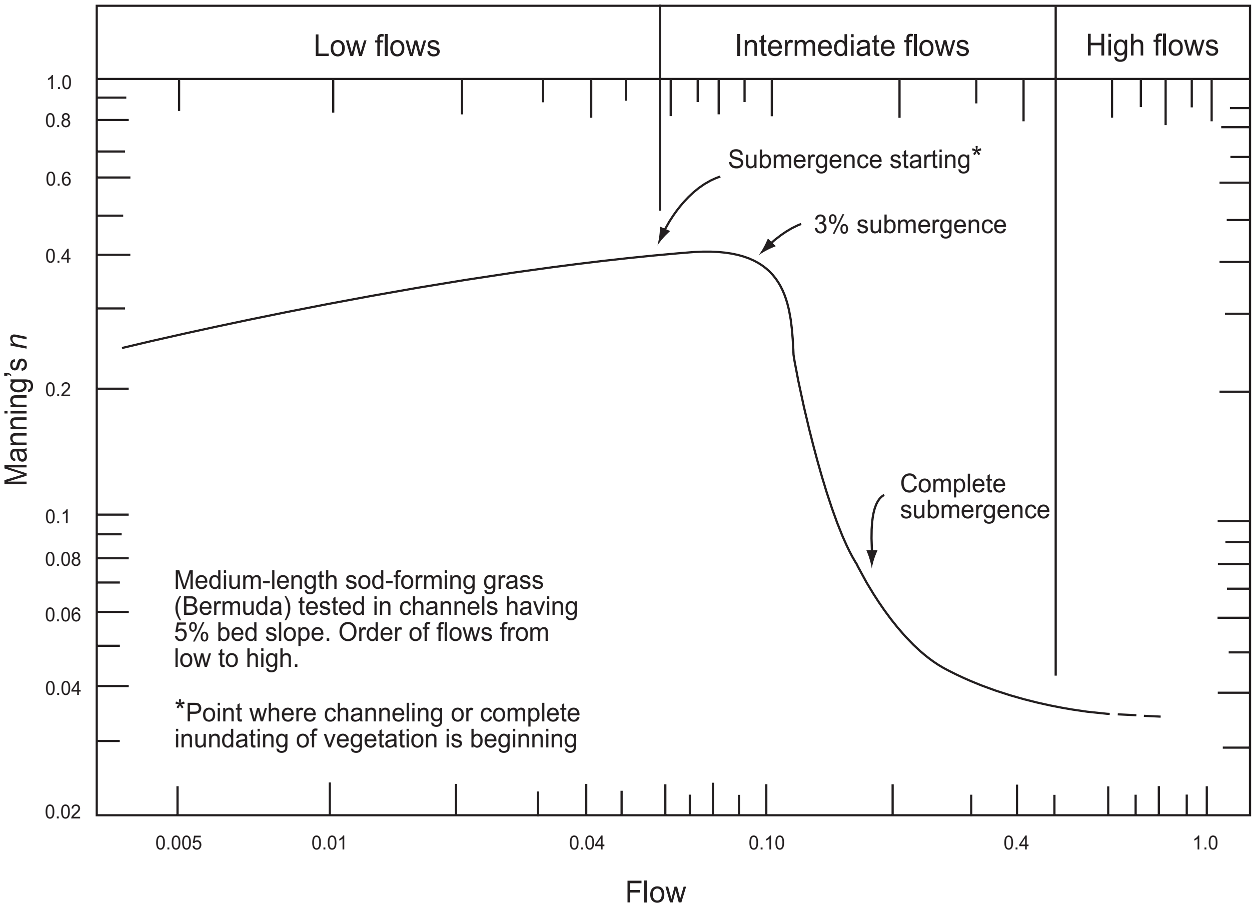


Figure 5.7 The effect of flow depth on hydraulic roughness (after Barling and Moore 1993).

crossings can significantly affect the required width of the swale/bioretention system. Driveway crossings can either be 'elevated' or 'at-grade'. Elevated crossings provide a culvert along the swale to allow flows to continue downstream, whereas at-grade crossings act as small fords and flows pass over the crossings. The slope of at-grade crossings (and therefore the swale) are governed by the trafficability of the change in slope across the base of the swale. Typically 1:9 side slopes, with a small flat base, will provide sufficient transitions to allow for suitable traffic movement.

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

Crossings can provide good locations for promoting extended detention within the bioretention swale and also for providing overflow points in the bioretention swale that can also be used to achieve ponding over a bioretention system (e.g. Figure 5.2). The distance between crossings will determine the feasibility of having overflow points at each one.

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

5.3.2.2 Selection of Manning's *n*

Manning's *n* is a critical variable in the Manning's equation relating to roughness of the channel. It varies with flow depth, channel dimensions and the vegetation type. For constructed swale systems, the values are recommended to be between 0.15 and 0.4 for flow depths shallower than the vegetation height (preferable for treatment) and significantly lower (e.g. 0.03) for flows with greater depth than the vegetation. It is considered reasonable for Manning's *n* to have a maximum at the vegetation height and then sharply reduce as depths increase. Figure 5.7 shows a plot of varying Manning's *n* with flow depth for a grass swale. It is reasonable to expect the shape of the Manning's *n* relation with flow depth to be consistent with other swale configurations, with the vegetation height at the boundary between 'Low flows' and

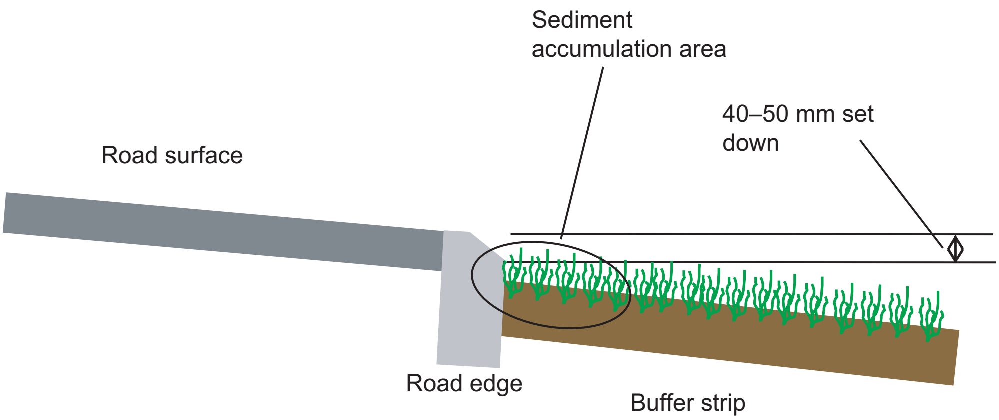


Figure 5.8 A flush kerb without setdown (photograph), edge detail showing setdown.

'Intermediate flows' (Figure 5.7) on the top axis of the diagram. The bottom axis of the plot has been modified from Barling and Moore (1993) to express flow depth as a percentage of vegetation height.

Further discussion on selecting an appropriate Manning's *n* for swales is provided in Appendix E of the MUSIC modelling manual (Cooperative Research Centre for Catchment Hydrology 2003).

5.3.3 Inlet details

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

5.3.3.1 Distributed inflows

An advantage of flows entering a swale system in a distributed manner (i.e. entering perpendicular to the direction of the swale) is that inflows are distributed and inflow depths are shallow which maximises contact with vegetation. This provides good pretreatment prior to flows entering the bioretention system. Creating distributed inflows can be achieved either by having flush kerbs or by using kerbs with regular breaks (Figure 5.9).

For distributed inflows, an area off the road surface is needed for coarse sediments to accumulate.. Sediment can accumulate on a street surface where the vegetation is at the same level as the road (Figure 5.8, photograph). To avoid this accumulation, a tapered flush kerb can be used that sets the top of the vegetation between 40 mm and 50 mm lower than the road surface (Figure 5.8), which requires the top of the ground surface (before turf is placed) to be between 80 mm and 100 mm below the road surface. This allows sediments to accumulate off any trafficable surface.

Figure 5.9 Kerbs with breaks to distribute inflows.

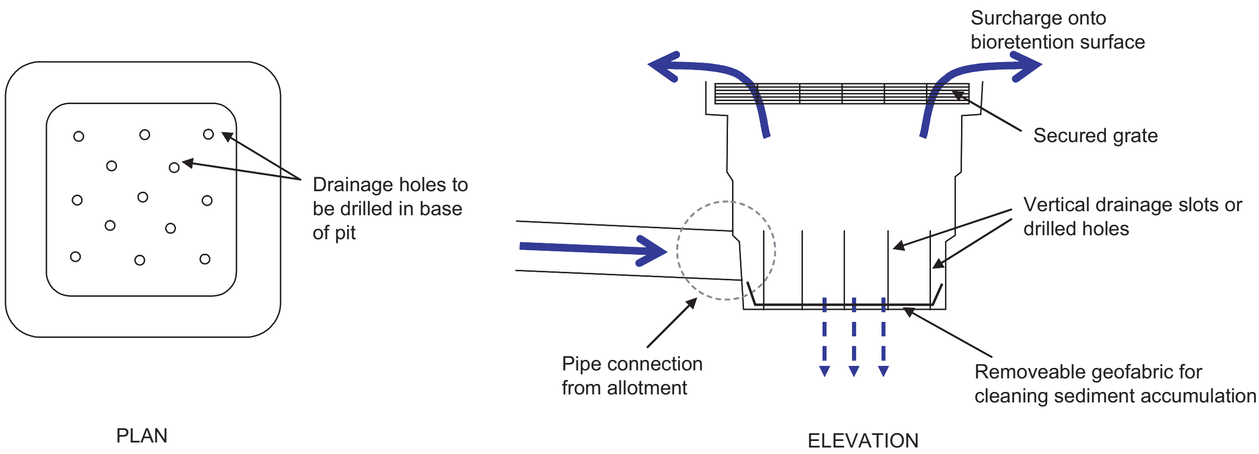


Figure 5.10 A surcharge pit for discharging allotment runoff into a bioretention swale.

5.3.3.2 Direct entry points

Direct entry of flows can be either through a break in a kerb or from a pipe system. Entrances through kerb breaks may cause some level of water ponding around the entry points. The width of the flow inundation on the road prior to entry will need to be checked and the width of the required opening determined to meet Council requirements (see Chapter 6, Section 6.3.2.1).

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

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

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

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

These systems are most frequently used when allotment runoff is required to cross a road into a swale on the opposite side. Several allotments can usually be combined prior to crossing the road to minimise the number of road crossings. Figure 5.10 shows an example of a surcharge pit discharging into a bioretention swale.

5.3.4 Vegetation scour velocity check

Scour velocities over the vegetation along the swale need to be checked. Manning's equation is used to estimate the mean velocity in the swale. An important consideration is the selection of an appropriate Manning's *n* that suits the vegetation height (see Section 5.3.2.2).

Manning's equation should be used to estimate flow velocities and ensure that they are below:

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

5.3.4.1 Velocity check – safety

As swales are generally accessible by the public, the combined depth and velocities product needs to be from a public risk perspective. To avoid people being swept away by flows along swales, a velocity–depth product check should be performed for design flow rates (see Institution of Engineers 2001, Book VIII Section 1.10.4). Thus, the following standard needs to be met:

$$\text{Velocity (m/s)} \times \text{depth (m)} < 0.4\ \text{m}^2/\text{s}$$

Note: 0.35 m^2/s is used in the Melbourne Water region.

5.3.5 Size perforated collection pipes

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

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

Considerations for the selection of a drainage layer include the slot widths in the perforated pipes as well as construction techniques. In addition, where the bioretention system can only have limited depth (e.g. maximum depth to perforated pipe < 0.5 m) it will be preferable to install just one drainage layer with a geotextile fabric providing the function of the transition layer.If gravel is used around the perforated pipes and the filtration media is finer than sand, it is recommended to install an additional 'transition' layer to prevent the fine filtration media being washed into the perforated pipes. Typically this is sand to coarse sand (0.7 mm–1.0 mm). Alternatively, a geotextile fabric could be used above the drainage layer to prevent finer material from reaching the perforated pipes; however, caution should be taken to ensure this material is not too fine as if it becomes blocked, the whole system will require resetting.

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

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

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

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

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

5.3.5.1 Perforations inflow check

To estimate the capacity of flows through the perforations ($Q_{perforations}$), orifice flow conditions are assumed and a sharp-edged orifice equation can be used (Equation 5.2). First, the number and size of perforations needs to be determined (typically from manufacturer's specifications) and used to estimate the flow rate into the pipes using a head of the filtration media depth plus the ponding depth. Second, it is conservative but reasonable to use a blockage factor (B) to account for partial blockage of the perforations by the drainage layer media. A factor of two is recommended.

$$Q_{perforations} = C \times A\sqrt{2gh}/B \qquad \text{(Equation 5.2)}$$

where g = Acceleration due to gravity (9.81 m/s^2)
A = total area of the orifice
h = maximum depth of water above the pipe
C = orifice coefficient
B = blockage factor

5.3.5.2 Perforated pipe capacity

The Colebrook-White equation (Equation 5.3) can be applied to estimate the flow rate in the perforated pipe. Manning's equation could be used as an alternative. The capacity of this pipe needs to exceed the maximum infiltration rate.

$$Q = [-2(2gDS_f)^{0.5}\log_{10}(k/(3.7D) + 2.51\nu/D(2gDS_f)^{0.5})] \times A \qquad \text{(Equation 5.3)}$$

Where Q = flow (m^3/s)
D = pipe diameter (m)
A = area of the pipe
S_f = pipe slope
k = wall roughness
ν = viscosity
g = gravity constant.

5.3.5.3 Drainage layer hydraulic conductivity

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

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

5.3.5.4 Impervious liner requirement

When bioretention systems are used as conveyance filtration devices (i.e. infiltration is not an objective) it is important to contain flows in the bioretention system. Stormwater is treated via filtration through a specified soil media with the filtrate collected by a subsurface drainage system to be either discharged as treated surface flow or collected for reuse. The amount of water lost to surrounding soils depends largly on local soils and the hydraulic conductivity of the filtration media in the bioretention system. Typically the hydraulic conductivity of filtration media should be selected such that it is 1–2 orders of magnitude greater than the native surrounding soil profile to ensure that the preferred flow path is into the perforated underdrainage system.

During detailed design, it is good practice to provide an impervious liner when infiltration is not desired and where the saturated hydraulic conductivity of the surrounding soils is more than one order of magnitude lower than the filtration media (see chapter 11 of Engineers Australia 2003). This is only expected to be required in sandy loam to sandy soils and where infiltration is expected to create problems.

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

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

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

5.3.6 High-flow route and overflow design

The design for high flows must safely convey flows up to the design storm for the minor drainage system (e.g. five-year ARI flows) to the same level of protection that a conventional stormwater system provides. Flows are to be contained within the bioretention swale. Where the capacity of the swale system is exceeded at a certain point along its length, an overflow pit is required. This discharges excess flows into an underground drainage system for conveyance downstream. The frequency of overflow pits is determined in the swale design (see Section 5.3.2 for a method to dimension the overflow pits).

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

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

To size a grated overflow pit, two checks should be made to estimateeither drowned or free flowing conditions. A broad-crested **weir** equation (Equation 5.4) can be used to determine the length of weir required (assuming free-flowing conditions) (L) and an orifice equation (Equation 5.5) used to estimate the area between opening required (assumed drowned outlet conditions). The larger of the two pit configurations should be adopted. In addition, a blockage factor (B) is to be used that assumes the orifice is 50% blocked.

For free overfall conditions (weir equation) (solving for L):

$$Q_{\text{minor}} = B \times C \times L \times H^{3/2} \qquad \text{(Equation 5.4)}$$

where Q_{minor} represents the flow through the minor drainage system (m^3/s), B = blockage factor (0.5), C = 1.7 and H = available head above the weir crest, and L = length of weir (m).

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

For drowned outlet conditions (orifice equation) (Equation 5.5):

$$Q_{\text{minor}} = B \times C \times A\sqrt{2gh} \qquad \text{(Equation 5.5)}$$

where B = blockage factor (0.5), C = 0.6 and H = available head above weir crest.

5.3.7 Soil media specification

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

A filter media layer provides most of the treatment function, through fine filtration and also by supporting vegetation that enhances filtration. The vegetation also helps to keep the filter media porous and provides some uptake of nutrients and other contaminants in the stormwater. The filter media is required to have sufficient depth to support vegetation, and is usually between 300 mm and 1000 mm.

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

Materials similar to those described in the following Sections should provide adequate substrate for vegetation to grow in and sufficient conveyance of stormwater through the bioretention system.

5.3.7.1 Filter media specifications

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

The material shall meet the geotechnical requirements set out below:

- **Material** – Sandy loam or equivalent material (i.e. similar hydraulic conductivity, 36–180 mm/hr) free of rubbish and deleterious material.
- **Particle size** – Soils with infiltration rates in the appropriate range typically vary from sandy loams to loamy sands. Soils with the following composition are likely to have an infiltration rate in the appropriate range: clay 5%–15%, silt < 30%, sand 50%–70%, assuming the following particle sizes (clay < 0.002 mm, silt 0.002 mm–0.05 mm, sand 0.05 mm–2.0 mm). Soils with most particles in this range would be suitable. Variation in large particle size is flexible (i.e. an approved material does not have to be screened). Substratum materials should avoid the lower particle size ranges unless hydraulic conductivity tests can demonstrate an adequate hydraulic conductivity (36–180 mm/hr).
- **Organic content** – between 5% and 10%, measured in accordance with AS1289 4.1.1.
- **pH** – is variable, but preferably neutral, with nominal pH 6.0 to pH 7.5 range. Optimum pH for denitrification, which is a target process in this system, is pH 7–8. Siliceous materials may have lower pH values.

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

5.3.7.2 Transition layer specifications

Transition layer material shall be sand/coarse sand material. A typical particle size distribution (per cent of particles passing through different sieve sizes) is provided below:

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

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

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

Table 5.1 Saturated hydraulic conductivity for a range of media particle sizes (d_{50})
Engineers Australia (2003)

Soil type	Particle size (mm)	Saturated hydraulic conductivity (mm/hr)(m/s)	
Gravel	2	36 000	1×10^{-2}
Coarse sand	1	3 600	1×10^{-3}
Sand	0.7	360	1×10^{-4}
Sandy loam	0.45	180	5×10^{-5}
Sandy clay	0.01	36	1×10^{-5}

5.3.7.3 Drainage layer specifications

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

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

5.3.8 Vegetation specification

Table A.1 (see Appendix A) provides lists of plants that are suitable for bioretention swales. Consultation with landscape architects is recommended when selecting vegetation to ensure the treatment system complements the landscape of the area.

5.3.9 Design calculation summary

Bioretention Swales — CALCULATION CHECKLIST

CALCULATION TASK	OUTCOME	CHECK
1 Identify design criteria		
Conveyance flow standard (ARI)		year
Area of bioretention		m²
Maximum ponding depth		mm
Filter media type		mm/hr
2 Catchment characteristics		
Cell A		m²
Cell B		m²
Slope		%
Fraction impervious		
Cell A		
Cell B		
3 Estimate design flow rates		
Time of concentration		
Estimate from flow path length and velocities		minutes
Identify rainfall intensities		
Station used for IFD data:		
Major flood – 100 year ARI		mm/hr
Minor flood – 5 year ARI		mm/hr
Peak design flows	Cell A, Cell B	
Q_{minor}		m³/s
Q_{100}		m³/s
Q_{infil}		m³/s
3 Swale design		
Appropriate Manning's *n* used?		
4 Inlet details		
Adequate erosion and scour protection?		
5 Velocities over vegetation		
Velocity for 5-year flow (<0.5 m/s)		m/s
Velocity for 100-year flow (<1.0 m/s)		m/s
Safety: Vel x Depth (<0.4)		m/s
6 Slotted collection pipe capacity		
Pipe diameter		mm
Number of pipes		
Pipe capacity		m³/s
Capacity of perforations		m³/s
Soil media infiltration capacity		m³/s
8 Overflow system		
System to convey minor floods		
9 Surrounding soil check		
Soil hydraulic conductivity		mm/hr
Filter media		mm/hr
MORE THAN 10 TIMES HIGHER THAN SOILS?		
10 Filter media specification		
Filtration media		
Transition layer		
Drainage layer		
11 Plant selection		

5.4 Checking tools

Checking aids are included for designers and referral authorities. In addition, advice on construction techniques and lessons learnt from building bioretention systems are provided.

Checklists are provided for:

- design assessments
- construction (during and post)
- operation and maintenance inspections
- asset transfer (following defects period).

5.4.1 Design assessment checklist

The *Bioretention Swale Design Assessment Checklist* presents the key design features that should be reviewed when assessing a design of a **bioretention swale**. These considerations include configuration, safety, maintenance and operational issues that should be addressed during the design phase.

Where an item results in an 'N' when reviewing the design, the design procedure should be assessed to determine the effect of the omission or error.

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

Land and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the *Asset Handover Checklist* (see Section 6.4.4).

5.4.2 Construction advice

General advice is provided for the construction of bioretention basins. It is based on observations from construction projects around Australia.

Building phase damage

It is important to protect filtration media and vegetation during the building phase as uncontrolled building site runoff is likely to cause excessive **sedimentation**, introduce weeds and litter and require replanting after building. A staged implementation can be used [i.e. during building, use geofabric, some soil (e.g. 50 mm) and instant turf (laid perpendicular to flow path)] to provide erosion control and sediment trapping. After building, remove the interim measures and revegetate, possibly reusing turf at subsequent stages. Also divert flows around swales during building (i.e. divert to sediment controls).

Traffic and deliveries

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

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

Inlet erosion checks

It is good practice to check the operation of inlet erosion protection measures following the first few rainfall events. These need to be checked early in the system's life, to avoid continuing problems. If problems occur in these events, then erosion protection should be enhanced.

Bioretention Swale Design Assessment Checklist					
Bioretention location:					
Hydraulics	Minor flood: (m³/s)		Major flood: (m³/s)		
Area	Catchment area (ha):		Bioretention area (ha)		
Treatment				Y	N
Treatment performance verified from curves?					
Inlet zone/hydraulics				Y	N
Station selected for IFD appropriate for location?					
Longitudinal slope of invert >1% and <4%?					
Mannings '*n*' selected appropriate for proposed vegetation type?					
Overall flow conveyance system sufficient for design flood event?					
Maximum flood conveyance width does not impact on traffic amenity?					
Overflow pits provided where flow capacity exceeded?					
Inlet flows appropriately distributed?					
Energy dissipation provided at inlet?					
Velocities within bioretention cells will not cause scour?					
Set down of at least 50 mm below kerb invert					
Collection system				Y	N
Slotted pipe capacity > infiltration capacity of filter					
Transition layer/geofabric barrier provided to prevent clogging of drainage layer?					
Cells				Y	N
Maximum ponding depth and velocity will not impact on public safety (V x D <0.4)?					
Selected filter media hydraulic conductivity > 10x hydraulic conductivity of surrounding soil?					
Maintenance access provided to invert of conveyance channel?					
Protection from gross pollutants provided (for larger systems)?					
Vegetation				Y	N
Plant species selected can tolerate periodic inundation and design velocites?					
Plant species selected integrate with surrounding landscape design?					
Detailed soil specification included in design?					

Sediment build-up on roads

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

Tolerances

Tolerances are very important in the construction of bioretention swales (e.g. base, longitudinal and batters) – having flat surfaces is particularly important for well-distributed flow paths and even ponding over the surfaces. Generally plus or minus 50 mm is acceptable.

Erosion control

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

Timing for planting

Timing of planting vegetation depends on a suitable time of year (and potential irrigation requirements) as well as timing in relation to the phases of development. For example, temporary planting set up during construction for sediment control (e.g. with turf) can then be removed and the area planted out with long-term vegetation. Alternatively temporary (e.g. turf or sterile grass) can be used until a suitable season for long-term vegetation.

Planting strategy

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

Perforated pipes

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

Clean filter media

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

5.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Bioretention swales

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

SITE: ____________________

CONSTRUCTED BY: ____________________

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

COMMENTS ON INSPECTION

ACTIONS REQUIRED

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

5.4.4 Asset handover checklist

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

5.5 Maintenance requirements

Bioretention swales treat runoff by filtering it through vegetation and then passing the runoff vertically through a filtration media which filters the runoff. In addition, they are used for flood conveyance and need to be maintained to ensure adequate flood protection for local properties.

Besides vegetative filtration, treatment relies upon detention, soil filtration and collection of runoff into an underdrain. Vegetation is key in maintaining the porosity of the soil media of the bioretention system and a strong healthy growth of vegetation is critical to its performance. The potential for rilling and erosion along the swale component of the system needs to be carefully monitored during establishment stages of the system.

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

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

Maintenance is primarily concerned with:

- flow to and through the system
- maintaining vegetation
- preventing undesired vegetation from taking over the desirable vegetation
- removal of accumulated sediments
- litter and debris removal.

Vegetation maintenance will include:

- removal of noxious plants or weeds
- re-establishment of plants that die.

Sediment accumulation at the inlet points needs to be monitored. Depending on the catchment activities (e.g. building phase) the deposition of sediment can tend to smother plants and reduce the available ponding volume. Should excessive sediment build-up, it will affect plant health and require removal before it reduces the infiltration rate of the filter media.

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

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

5.5.1 Operation and maintenance inspection form

The *Bioretention Swale Maintenance Checklist* is designed to be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

5.6 Bioretention swale worked example

5.6.1 Worked example introduction

Modelling using MUSIC was undertaken in developing a stormwater quality treatment system for a residential estate in Melbourne. The bioretention system makes up part of a larger system including downstream reuse. Because of the downstream treatment, TSS is the limiting pollutant for the bioretention system itself. This worked example describes the detailed design of a grass swale and bioretention system located in a median separating an arterial road and a local road within the residential estate. The layout of the catchment and bioretention swale is shown in Figure 5.11. A photograph of a similar bioretention swale in a median strip is shown in Figure 5.12 (although the case study is all turf).

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

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

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

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

Simulation using MUSIC found that the required area of the bioretention system to achieve a 80% reduction in TSS from values typically generated from urban catchments is approximately 61 m^2 and 22 m^2 for Cell A and B, respectively. The filtration medium used is sandy loam with a notional saturated hydraulic conductivity of 180 mm/hr. The required area of the filtration zone is distributed to the two cells according to their catchment area.

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

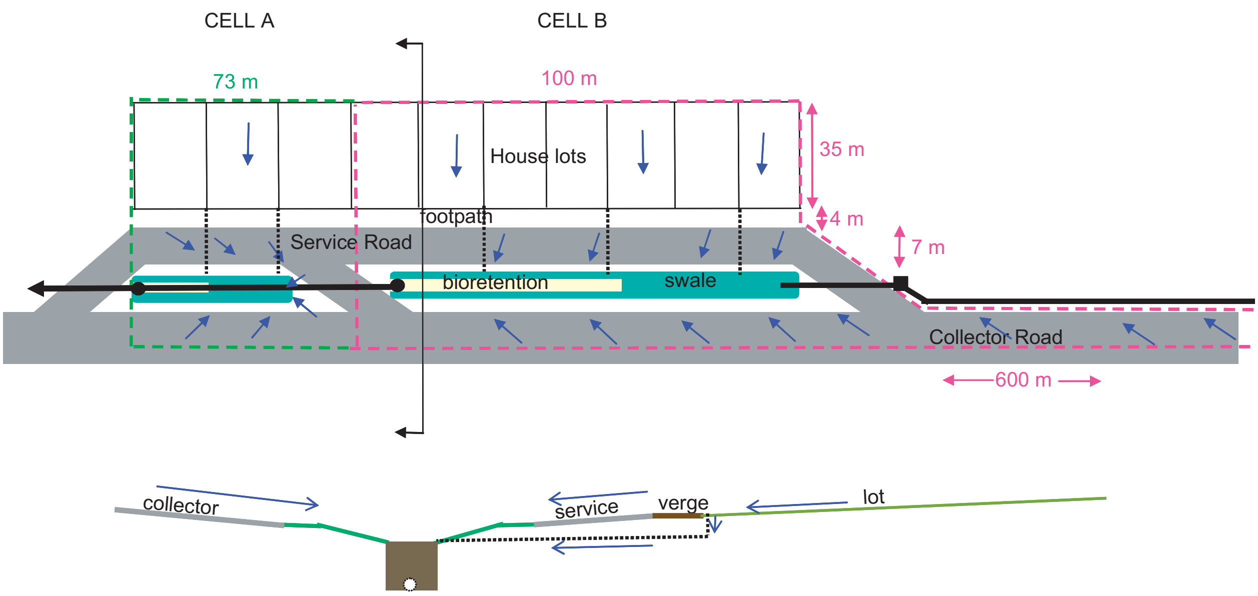


Figure 5.11 Catchment area layout and section for worked example of a stormwater quality treatment system for a residential estate in Melbourne.

5.6.1.1 Design objectives

The design objectives of the bioretention swale are to:

- use treatment to achieve a 80% reduction of TSS
- design the subsoil drainage pipe to ensure that the capacity of the pipe exceeds the saturated infiltration capacity of the filtration media (both inlet and flow capacity)

Figure 5.12 A bioretention swale.

- be able to convey safely design flows within up to 10-year ARI range into a piped drainage system without any inundation of the adjacent road
- check the hydraulics for the swale to confirm flow capacity for the 10-year ARI peak flow.
- create acceptable safety and scouring behaviour for an 100-year ARI peak flow.

5.6.1.2 Constraints and concept design criteria

The constraints and concept design criteria for the bioretention swale are that:

- depth of the bioretention filter layer shall be a maximum of 600 mm
- maximum ponding depth allowable is 200 mm
- width of median available for siting the system is 6 m
- the filtration medium available is a sandy loam with a saturated hydraulic conductivity of 180 mm/hr.

5.6.1.3 Site characteristics

The site characteristics for the bioretention swale are:

- urban, low density residential land use
- a 1.3% overland flow slopes for Cell A and B
- soil is clay
- fraction impervious is: 0.60 (lots); 0.90 (roads); 0.50 (footpaths); 0.0 (swale)
- catchment areas are as shown in Table 5.2.

Table 5.2 Catchment areas for the worked example of the bioretention swale
All measurements are in metres (length x width)

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

5.6.2 Confirm size for treatment

Interpretation of Figures 5.4 to 5.6 with the input parameters below is used to estimate the reduction performance of the bioretention system to ensure the design will achieve target pollutant reductions.

- Melbourne location
- 200 mm extended detention
- treatment area to impervious area ratio: Cell A – 61 m^2/ 6710 m^2 = 0.89%; Cell B – 22 m^2/ 2599 m^2 = 0.85%.

From the TSS graph, the expected pollutant reduction is 92% for TSS which exceeds the design requirement of 80%.

5.6.3 Estimating design flows

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

5.6.3.1 Major and minor design flows

Time of concentration (t_c)

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

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

Following the procedures in *Australian Rainfall and Runoff* (Institution of Engineers 2001), the following t_c values are estimated:

t_c – Cell A : 10 min
t_c – Cell B: 8 min.

Design rainfall intensities

Adopt the values from IFD (Intensity–Frequency Duration) table for Melbourne (Table 5.3).

Table 5.3 Rainfall intensities for selected catachments

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

Design runoff coefficient

To calculate the design runoff coefficient, apply the method outlined in ARR (Institution of Engineers 2001, Book VIII, Section 1.5.5 iii):

$C^1{}_{10} = 0.1 + 0.0133\ ({}^{10}I_1 - 25)$, where $C^1{}_{10}$ is the pervious runoff coefficient
$C_{10} = 0.9f + C^1{}_{10}\ (1\ [-]\ f)$, where f is the fraction impervious.
The fraction impervious is calculated using the following f values:

- roads, $f = 0.90$
- footpaths, $f = 0.5$
- swales, $f = 0.0$
- lots, $f = 0.6$.

Therefore, for Cell A (area weighted), $f = 0.70$
For Cell B (area weighted), $f = 0.61$
C_{10} for Cell A = 0.67

C_{10} for Cell B = 0.61
$C_y = F_y\ C_{10}$ (F_y from Table 1.6 Institution of Engineers 2001 Book VIII)

Table 5.4 Design runoff coefficients and flows

	C_{10}	C_{100}
Cell A	0.67	0.80
Cell B	0.61	0.73

Peak design flows

The peak design flows are calculated by using the Rational Method as follows:

$$Q = 0.00278.\ CIA\ (\mathrm{m^3/s})$$

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

5.6.3.2 Maximum infiltration rate

The maximum infiltration rate reaching the perforated pipe at the base of the soil media is estimated by using the hydraulic conductivity (k) of the media and the head above the pipes (h_{max}) and applying Darcy's equation (see Equation 5.1):
Hydraulic conductivity = 180 mm/h
Flow capacity of the infiltration media, Q_{max} (assuming no blockage)

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

$$Q_{max} = 5E10^{-5} \times L \times W_{base}\left(\frac{0.2 + 0.6}{0.6}\right) \quad \text{(Equation 5.6)}$$

Therefore, completing the above calculations gives a result of maximum infiltration rate Cell A = 0.004 m^3/s, and maximum infiltration rate Cell B = 0.001 m^3/s.

5.6.4 Swale design

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

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

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

The maximum capacity of the swale (Equation 5.7) is estimated by adopting a 150 mm freeboard (i.e. maximum depth is 0.61 m).

$$Q_{cap} = 2.1\ \mathrm{m^3/s} > 0.14\ \mathrm{m^3/s} \quad \text{(Equation 5.7)}$$

Q_{cap} = flow capacity of the swale

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

5.6.5 Inlet details

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

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

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

5.6.6 Vegetation scour velocity check

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

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

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

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

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

5.6.6.1 Velocity check – safety

The velocity–depth product in Cell A during peak 100-year ARI flow must be checked for pedestrian safety criteria (Equation 5.8). As v = 0.49 m/s (calculated in Section 5.6.6), and d = 0.32 m, then:

$$v \times d = 0.49 \times 0.32 = 0.16 < 0.4\,\text{m}^2/\text{s} \qquad \text{(Equation 5.8)}$$

(Institution of Engineers 2001 Book VIII Section 1.10.4)

Therefore, velocities and depths are OK.

5.6.7 Sizing of perforated collection pipes

5.6.7.1 Perforations inflow check

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

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

Assume subsurface drains with half of all pipes blocked:
Clear opening = 2100 mm^2/m, hence blocked openings are 1050 mm^2/m.
Slot width is 1.5 mm
Slot length, 7.5 mm,
No. of rows, 6
Diameter = 100 mm,
Number of slots per metre = (1050)/(1.5 [x] 7.5) =93.3
Assume orifice flow conditions – $Q = CA\sqrt{2gh}$ (see Equation 4.6)
C = 0.61 (assume slot width acts as a sharp-edged orifice, see Equation 5.2).

Inlet capacity per metre of pipe =

$$[0.61 \times (0.0015 \times 0.0075) \times \sqrt{2} \times 9.81 \times 0.85] \times 93.3$$
$$= 0.0025 \text{ m}^3/\text{s}$$

Inlet capacity per metre × total length for each of Cells A and B:
Cell A = 0.0025 × 61 = 0.15 m^3/s > 0.003 (max infiltration rate), hence one pipe has sufficient perforation capacity to pass flows into the perforated pipe.
Cell B = 0.0025 × 22 = 0.05 m^3/s > 0.001 (max infiltration rate), hence 1 pipe is sufficient.

5.6.7.2 Perforated pipe capacity

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

Estimate applying the Colebrook-White equation (see Equation 5.3):

$$Q = [-2(2gDS_f)^{0.5}\log_{10}(k/(3.7D) + 2.51\nu/D(2gDS_f)^{0.5})] \times A$$

Adopt: $D = 0.10$ m
$S_f = 0.005$ m/m
$g = 9.81$ m^2/s
$k = 0.007$ m
$\nu = 1.007 \times 10^{-6}$

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

Adopt 1 × ϕ (diameter) 100 mm perforated pipe for the underdrainage system in both Cell A and Cell B.

5.6.7.3 Drainage layer hydraulic conductivity

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

5.6.7.4 Impervious liner requirement

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

5.6.8 Overflow design

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

The size of the pits are calculated using a broad-crested weir equation with the height above the maximum ponding depth and below the road surface, less freeboard (i.e. 0.76 – (0.2 + 0.15)= 0. 41 m).

First, check using a broad-crested weir equation (see Equation 5.4):

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

where $B = 0.5$, $C = 1.7$ and $H = 0.41$, and solving for L

Gives $L = 0.62$ m of weir length required (equivalent to 155 × 155 mm pit).

Second, check for drowned conditions (see Equation 5.5):

$$Q = B \times C \times A\sqrt{2gh} \text{ with } C = 0.6$$

$$0.14 = 0.5 \times 0.6 \times \mathrm{A} \times \sqrt{2\mathrm{g}} \times 0.41 \text{ gives } A = 0.16 \text{ m}^2 \text{ (equivalent to } 400 \times 400 \text{ pit).}$$

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

5.6.9 Soil media specification

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

5.6.9.1 Filter media specifications

The filter mediium is to be a sandy loam with the following criteria and shall meet the geotechnical requirements set out below:

- hydraulic conductivity between 50 mm/hr and200 mm/hr
- particle sizes of between: clay 5%–15%, silt < 30%, sand 50%–70%
- between 5% and 10% organic content, measured in accordance with AS1289 4.1.1
- pH neutral.

5.6.9.2 Transition layer specifications

Transition layer material shall be coarse sand material (such as Unimin 16/30 FG sand grading or equivalent). A typical particle size distribution is as follows: percentage passing 1.4 mm, 100%; 1.0 mm, 80%; 0.7 mm, 44%; 0.5 mm, 8.4%.

5.6.9.3 Drainage layer specifications

The drainage layer is to be 5 mm screenings.

5.6.10 Vegetation specification

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

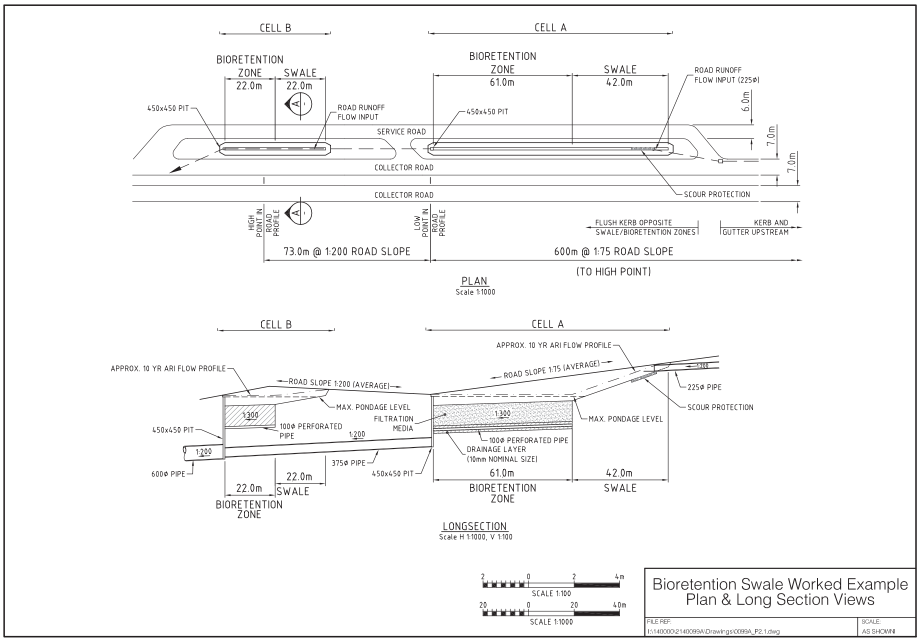
5.6.11 Calculation summary

The completed *Bioretention Swales Calculation Summary* shows the results of the design calculations.

	Bioretention Swales	CALCULATION SUMMARY		
	CALCULATION TASK	**OUTCOME**		**CHECK**
1	**Identify design criteria**			
	Conveyance flow standard (ARI)	10	year	
	Area of bioretention	61 and 22	m^2	
	Maximum ponding depth	200	mm	
	Filter media type	180	mm/hr	✓
2	**Catchment characteristics**			
	Cell A	9600	m^2	
	Cell B	4200	m^2	
	Slope	1.3	%	
	Fraction impervious			
	Cell A	70		
	Cell B	0.61		✓
3	**Estimate design flow rates**			
	Time of concentration			
	Estimate from flow path length and velocities	Cell A – 10 Cell B – 8	minutes	✓
	Identify rainfall intensities			
	Station used for IFD data:			
	Major flood – 100 year ARI		mm/hr	
	Minor flood – 5 year ARI		mm/hr	
	Peak design flows	Cell A, Cell B		
	Q_{minor}	0.14, 0.06	m^3/s	
	Q_{100}	0.29, 0.11	m^3/s	
	Q_{infil}	0.003, 0.001	m^3/s	✓
3	**Swale design**			
	Appropriate Manning's *n* used?	yes		✓
4	**Inlet details**			
	Adequate erosion and scour protection?	rock pitching		✓
5	**Velocities over vegetation**			
	Velocity for 5-year flow (<0.5 m/s)	0.09	m/s	
	Velocity for 100-year flow (<1.0 m/s)	0.49	m/s	
	Safety: Vel x Depth (<0.4)	0.16	m/s	✓
6	**Slotted collection pipe capacity**			
	Pipe diameter	100	mm	
	Number of pipes	1		
	Pipe capacity	0.004	m^3/s	
	Capacity of perforations	0.15	m^3/s	
	Soil media infiltration capacity	0.003	m^3/s	✓
8	**Overflow system**			
	System to convey minor floods	grated pits 450 x 450		✓
9	**Surrounding soil check**			
	Soil hydraulic conductivity	3.6	mm/hr	
	Filter media	180	mm/hr	
	MORE THAN 10 TIMES HIGHER THAN SOILS?	YES		✓
10	**Filter media specification**			
	Filtration media	sandy loam		
	Transition layer	sand		
	Drainage layer	gravel		✓
11	**Plant selection**	turf		

5.6.12 Construction drawings

Figure 5.13 shows the construction drawing for the worked example.



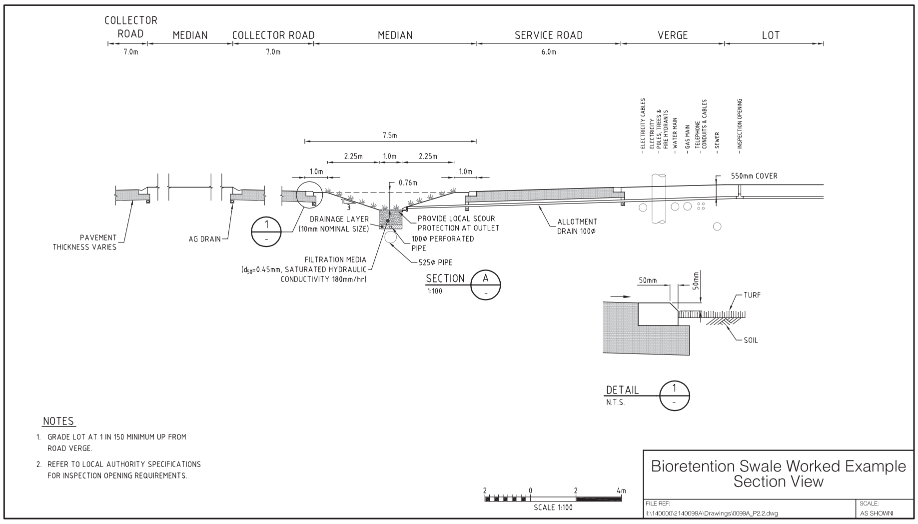


Figure 5.13 Construction drawing of the bioretention swale worked example, and a section view.

5.7 References

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Chapter 6 **Bioretention basins**

Bioretention basin in Richmond, Victoria.

6.1 Introduction

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

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

Bioretention basins operate by passing runoff through prescribed filtration media, commonly planted with vegetation that provides treatment through fine filtration, **extended detention** and some **biological uptake**. They also provide flow retardation and are particularly efficient at removing nutrients.

Figure 6.1 shows an example of a basin integrated into a local streetscape and a car park.

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

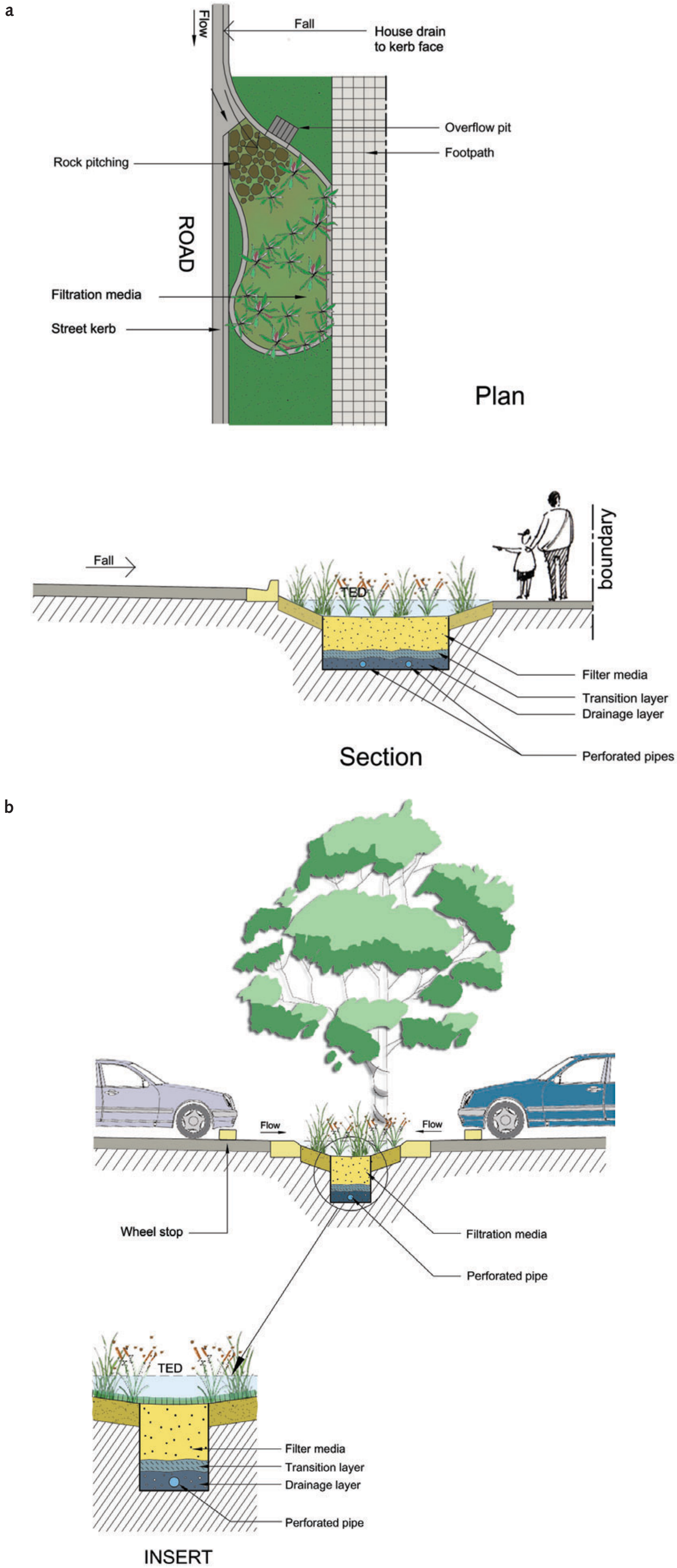

Figure 6.1 Bioretention basin integrated into: (a) a local streetscape and (b) a car park.