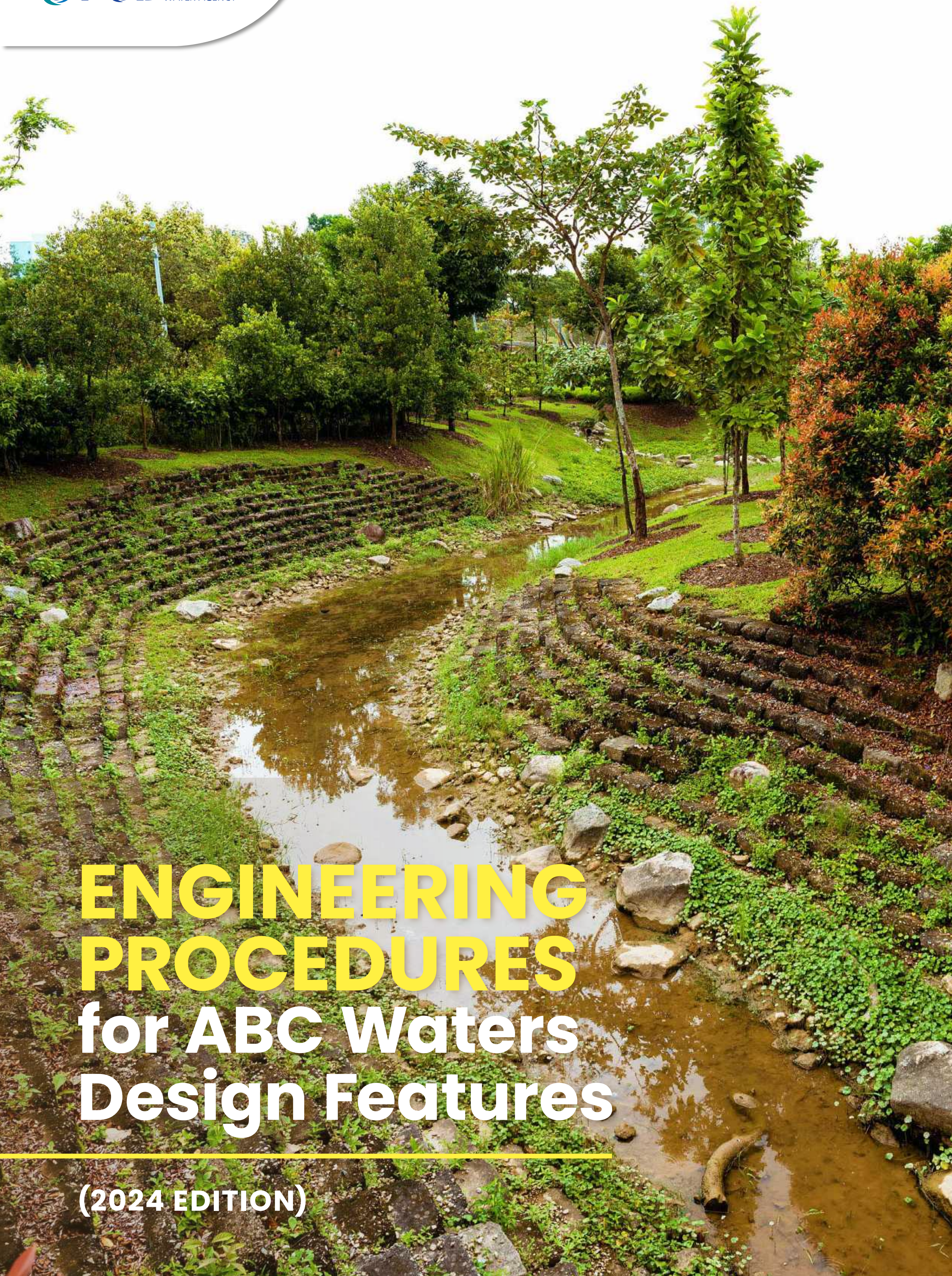
SINGAPORE'S NATIONAL WATER AGENCY



ENGINEERING PROCEDURES for ABC Waters Design Features

(2024 EDITION)

Introduction 1

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Chapter 1 Introduction

1.1 Introduction

Singapore is one of the world's most water-scarce countries – the 20th off the bottom of the rung of freshwater availability (170 out of 193 countries in the 2006 United Nations World Water Report 2 for total annual water resources volume per capita). Adding to this is the dramatic growth in population since the turn of the 20th century, which has consistently put a squeeze on land available on land available for water catchment. In its efforts to manage water resources, Singapore's Water management is linked to many other developmental policies, including housing, land use and infrastructure.

Today, Singapore has four main sources of water supply, commonly known as the "Four National Taps"

- Imported water from Malaysia
- Water from local reservoirs
- NEWater (reclaimed used water)
- Desalinated water

Singapore is a good example of an urban environment with limited natural resources facing the challenges of a growing population. The built urban environment is now a critical focal point for Ecologically Sustainable Development practices.

The pursuit of sustainable urban environments involves development that aimed to minimize depleting natural resources and degrading the health and amenity of the pre-existing land and water environments. As growing urban communities look to minimize their impact on already stressed water resources, designing for resilience to the impacts of climate change, including the protection of downstream water environments and mitigating increased flood risk, play a key role in securing the future of water resources in Singapore.

Successful urban communities are extremely complex socio-physical systems that are fully integrated and constantly evolving. Harmony of the built, social and natural environments within a city is the result of complex interactions between the quality of the natural and built environment, the social and institutional capital, and the natural resources that support a city. The ability of a city to meet current and emerging challenges in relation to achieving this harmony is closely linked to the strength of the urban economy.

Following a series of practitioner envisioning workshops, Binney *et al*. (2010) presented a vision for Cities of the Future comprising twelve principles arranged under four themes as shown in Figure 1. Many of these principles would apply to the way we manage urban stormwater, as a component of the total urban water cycle. The way we manage urban water, particularly urban stormwater, influences almost every aspect of our urban environment and the quality of life. Water is an essential element of place making, both in maintaining/enhancing the environmental values of surrounding waterways and in the amenity and cultural connection of the place.

Wong *et al*. (2011) noted that the link between sustainable urban water management and the vitality and prosperity of urban environments is only beginning to be recognized and these linkages include

- Access to secured and clean water supply
- Clean water environment
- Flood protection
- Urban design strategies
- Mitigating urban heat
- Creating productive landscapes
- Quality of public spaces.

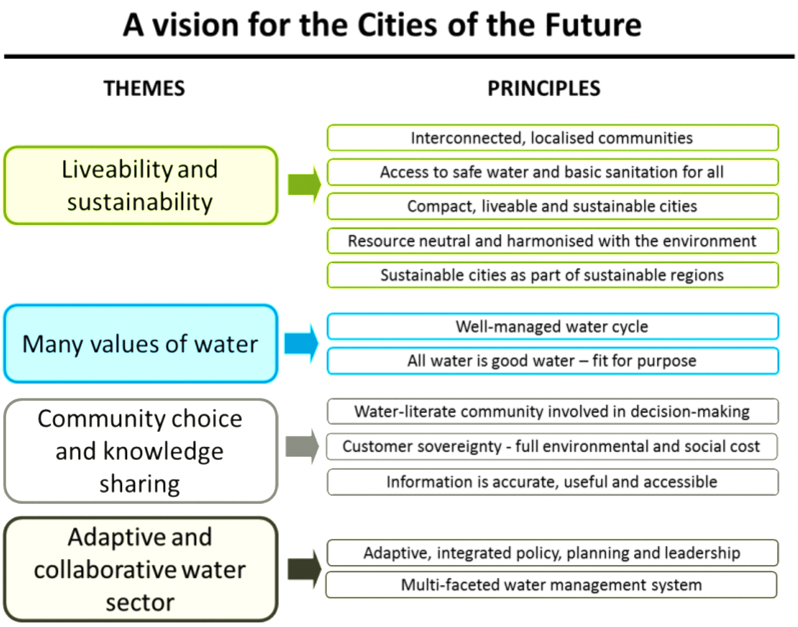

Figure 1 – Principles for a City of the Future.

Adapte from Binney, P., Donald, A., Elmer, V., Eert, J., Phillis, O., Skinner, R. and Young R. (2010)IWA Cities of the Future Program, Spatial Planning and Institutional Reform Conclusions from the World Water Congress, Montreal, September 2010.

1.2 Creating water sensitive cities

1.2.1 Vision for water sensitive city

Contemporary research in integrated urban water cycle management highlights that a Water Sensitive City will involve significant departures from conventional urban water management approaches and that the transformation of cities to water sensitive cities will require a major social-technical overhaul of conventional approaches.

A holistic philosophy incorporating flexibility in supply and demand to meet the needs of users and the environment is required. This philosophy will inform the collection, storage, treatment and movement of water. It also underpins the technologies that support these activities in a way that provides a sensory manifestation of process for all to acknowledge and appreciate.

1.2.2 Urban Stormwater management in a water sensitive city

Traditional approaches to stormwater management are based on a single management objective that considers stormwater as a source of potential hazard to public safety. Stormwater management was essentially that of stormwater drainage using two general methods, ie. (i) conveyance of stormwater to receiving waters in an hydraulically efficient manner; and (ii) detention and retardation of stormwater. Recent developments involving the concept of major/minor drainage systems (e.g. Institution of Engineers, Australia, 2001) take into account an economic risk-based approach to stormwater drainage but stormwater management essentially remained a single objective exercise.

A growing public awareness of environmental issues in recent times has highlighted the importance of environmental management of urban stormwater. It is well documented that urban stormwater runoff are generally of poorer overall quality than runoff from a rural catchment. The impact of poor stormwater quality is becoming an increasing issue of concern amongst catchment managers. The impacts can include the deposition of suspended material, which can smother aquatic habitats, increased concentrations of nutrients, oxygen-demanding materials, micro-organism and toxic materials and the deposition of litter. Increase catchment runoff can lead to significant changes to the morphology of creeks and rivers leading to degradation of aquatic habitats. Stormwater contaminants causes dissolve oxygen depletion and increased toxicity levels with the consequential degradation of ecological health of the receiving waters.

Singapore is progressively moving towards attaining a higher level of self sustainability in water resources. One of the strategies to achieve this is in the consideration of urban stormwater as a resource. PUB had therefore embarked on a project which would convert Marina Bay, located at the heart of Singapore City, into a freshwater reservoir that harvests stormwater from one-sixth of the area of metropolitan Singapore. Stormwater quality management and reducing potential impacts of stormwater pollution are therefore important water resource management considerations in Singapore. The management of urban stormwater to meet these objectives can fundamentally be categorised into stormwater quantity and stormwater quality management.

There have been a number of initiatives to change the traditional means by which urban stormwater is managed. One such initiative is the practice of Water Sensitive Urban Design (WSUD). WSUD reflects the new paradigm in the planning and design of urban environments that is ‘sensitive’ to the issues of water sustainability and environmental protection. Water Sensitive Urban Design is the process and Water Sensitive Cities are the outcome. This Australian innovation of Water Sensitive Urban Design has evolved from its early association with stormwater management to provide a broader framework for sustainable urban water management (Wong 2006a, 2006b), and building water sensitive cities. In line with WSUD in terms of stormwater management, PUB's ABC Waters Design Guidelines encourage the use of ABC Waters Design Features to slow down stormwater run-off and to keep Singapore's waterways and waterbodies clean.

Like WSUD, ABC Waters Design Guidelines bring 'sensitivity to water' into urban design, as it aims to ensure that water is given due prominence within the urban design process through the integration of urban design with the various disciplines of engineering and environmental sciences associated with the provision of water services including the protection of aquatic environments in urban areas. Community values and aspirations of urban places necessarily govern urban design decisions and therefore water management practices.

The practicalities of urban stormwater management often require that stormwater quantity management issues such as flood protection, public safety and drainage infrastructure economics are addressed. This should occur in the first instance before stormwater quality issues are considered. This does not suggest that these two fundamental issues are mutually exclusive. Many measures designed for stormwater quantity control have inherent water quality management functions while others can be retrofitted to serve the dual functions of stormwater quantity and quality management.

The guidelines for the planning and design of these stormwater quality management systems (termed ABC Waters Design Guidelines) are to aid the developer, design engineer, planner and architect to meet urban stormwater management objectives. Stormwater quality management involves the use of structural and non-structural changes to catchment management. This document concentrates primarily on the implementation of structural treatment measures although issues of catchment planning are discussed in some detail in Chapter 3.

1.3 Overview on Water Sensitive Urban Design (WSUD) and ABC Waters Design

WSUD can be best explained as the interactions between the urban built form (including urban landscapes) and the urban water cycle as defined by the three urban water streams of potable water, wastewater, and stormwater. This is shown in Figure 1.1, which shows how WSUD fits into an Ecologically Sustainable Development framework and lists key WSUD initiatives to address potable water, wastewater and stormwater issues, whereby benefits from one element can often have flow on benefits.

A key principle espoused by the framework presented in Figure 1.1 is a holistic approach to urban water cycle management that include all water flows, such as water supply, stormwater and wastewater. Singapore has covered good mileage in area (i) and (ii) through the Water Efficiency Strategies and NEWater initiatives. PUB's ABC Waters Design Guidelines aim to achieve (iii) and (iv). All streams of water should be managed as a resource as they have quantitative and qualitative impacts on land, water and biodiversity, and the community's aesthetic and recreational enjoyment of waterways. This applies at all level of urban water governance, ie. community, institutional and government. In Singapore, the integrated management for potable water, used water and stormwater is under the purview of PUB, Singapore's National Water Agency.

In ABC Waters design, stormwater is to be managed both as a resource and for protection of the environmental and use values of receiving waters. When applied to the design and operation of urban developments, ABC Waters Design strategy adopts an integrated approach of combining stormwater quantity and quality management measures across the range of scale in an urban environment. The outcome is a more site-responsive range of design solutions including detention and/or retention of stormwater at, or near, its origin, with subsequent slow release to groundwater or downstream receiving bodies.

This integrated approach has begun to gain favour over the traditional conveyance-oriented approach because it has the potential to reduce development costs and minimise pollution and water balance problems by ensuring hydrological regimes are changed minimally from pre-development conditions. The approach also has ecological benefits that contribute to making Singapore a biophilic City in Nature, thriving with biodiversity and greenery enveloping our urban landscape. However, the adoption of the integrated approach has been constrained because it is perceived to have post-development operation and maintenance costs, and in some cases can cause a reduction in developable land.

This reduction in developable land may be the case if detention/retention facilities are used solely to control the amount of stormwater runoff. Detention/retention facilities however, have increasingly been used in a multi-purpose role, providing recreational and aesthetic value, thereby offsetting any loss in developable land by increasing land value for nearby residential areas.

Furthermore, the integrated approach aims to control pollutants such as nutrients, pesticides, heavy metals and bacteria. Diffuse source pollution control can be achieved by detention/retention techniques that settle and capture particulates and prevent erosion by maintaining the hydrological regime.

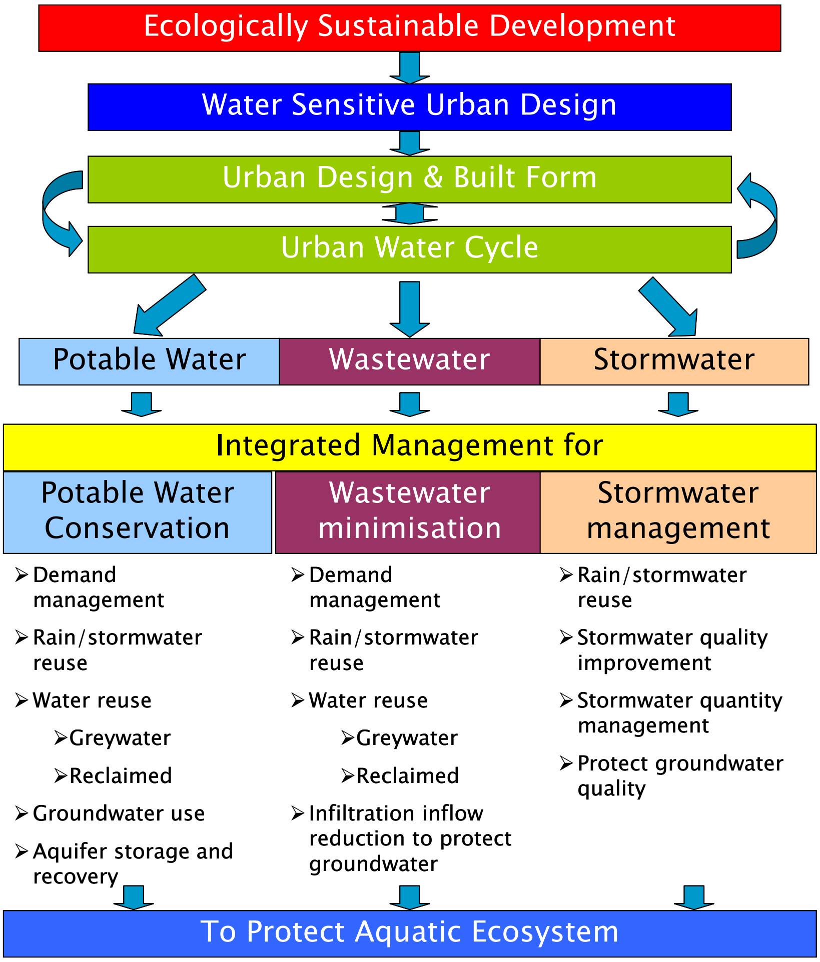

Figure 1.1 Water Sensitive Urban Design Framework

1.4 Stormwater Quality

Stormwater pollutants from urban developments originate from a variety of sources in the catchment. Table 1.1 summarises the sources of some of the more common urban runoff pollutants. Suspended solids, nutrients, BOD_5 and COD and micro-organisms are usually considered the most significant parameters in terms of ecological impacts. Oils and surfactants, and litter have aesthetic impacts which are more renowned for generating community concern and action. Organic load in stormwater originates mainly from leaves and garden litter. As a significant amount of inorganic pollutants is sediment bound, effective treatment of suspended solids is often a minimum criterion in stormwater quality management with the expectation that a significant amount of organic and inorganic pollutant will also be treated.

Table 1.1

Typical Urban Runoff Pollutant Sources

Pollutant Source	Solids	Nutrients	Pathogens	DO Demands	Metals	Oils	Synthetic Organics
Soil Erosion	✓	✓		✓	✓		
Cleared Land	✓	✓	✓				
Fertilisers		✓			✓		
Human Waste	✓	✓	✓	✓			
Animal Waste	✓	✓	✓	✓	✓		
Vehicle Fuels and Fluids	✓		✓	✓	✓		
Fuel Combustion		✓			✓	✓	
Vehicle Wear	✓				✓		
Industrial and Household Chemicals	✓	✓			✓	✓	✓
Industrial Processes	✓	✓			✓	✓	✓
Paint and Preservatives					✓	✓	
Pesticides					✓	✓	✓
Wastewater Facilities	✓	✓	✓	✓	✓		

As described by Schueler (1995), as much as 70% of the impervious area is related to transport-related functions such as roads, driveway, car-parks etc. This component of the impervious areas in an urbanised catchment is identified as a prominent source of stormwater pollutants such as suspended solids and associated trace metals, polycyclic aromatic hydrocarbons and nutrients. Urban commercial activities have also been identified as the main source of litter generation. An overview of some key urban stormwater pollutants as presented in the following sections below.

1.4.1 Suspended Solids

Suspended solids comprise of inorganic and organic materials. Sources of inorganic suspended solids include soil particles from erosion and land degradation, streets, households and buildings, and airborne particulate matter. Contributors to organic suspended solids are bacteria and microorganisms such as those found in sewage. The level of suspended solids in urban runoff is comparable to raw sewage and, inorganic soil particles are particularly of concern. Large amounts of inorganic soil particles are often associated with urban construction and the development of supporting services including roads, sewers and drainage systems. Levels of inorganic soil particles generated from these activities are at least two to six times, and can be up to several hundred times, pre-development levels.

Turbid waters often result from the presence of suspended solids. In general the community associates turbid waters with environmental pollution and degradation of the water's aesthetic value.

Nutrients and toxins such as phosphorus, heavy metals and organic chemicals utilise sediment as the medium for transportation in urban runoff. The deposition of sediments can result in the release of these toxins and nutrients at a later time when the ambient conditions related to the redox potential of the sediment and water column becomes favourable for their release. This mechanism provides the opportunity for pollutant re-mobilisation in later flow events enhancing the risk of further downstream degradation.

Suspended solids also reduce the penetration of light through water, and this adversely affects the feeding and respiration of aquatic plants.

1.4.2 Nutrients

Nutrients are fed into the water system through many different sources. These include sewerage, plant matter, organic wastes, fertilisers, kitchen wastes (including detergents), nitrous oxides produced from vehicles exhausts and ash from bushfires. Nutrients contain natural compounds consisting of nitrogen and phosphorus.

There are problems associated with high levels of nutrients in waterbodies. Nutrients promote growth of aquatic plant life including floating macrophytes, which if in large concentrations, produce algal blooms on the water surface. Algae are microscopic plants which occur naturally in waterways. With an increase in nutrients algal growth becomes excessive often resulting in a build up of toxins. Toxic algal blooms cause the closure of fisheries, water farming industries and public beaches.

1.4.3 Litter

Litter is generally the most noticeable indicator of water pollution to the community. Litter is also commonly thought of as the pollutant most detrimental to waterways because of its visibility. Pollution of the environment including the export of litter and gross pollutants has intensified over the last 30 years due to the production of easily disposable, non-biodegradable packaging and household and industrial items. The sources of litter are varied and they include dropping of rubbish, overflows of rubbish containers and material blown away from tips and other rubbish sources.

1.4.4 Metals

Analysis of contaminants associated with urban dust and dirt by Dempsey *et al* (1993) found highest concentrations of Cu, Zn and TP to be associated with particles in the 74 μm to 250 μm. The particle size range with high Pb association extends to 840 μm. One possible explanation for a higher contaminant concentration is that the particular size range has a higher specific surface area (and thus contaminant binding sites). For example, Sansalone and Buchberger (1997) found that specific surface area of solids transported from an urban roadway surface decrease with increasing particle size as is normally the case for spherical particles. With irregularly shaped particles, there is the general tendency for larger sized particles to have higher specific surface area than are normally expected.

Table 1.2 reproduces the table of particle sizes and associated pollutants presented by Dempsey et al (1993) for dust and dirt generated from road surfaces. The data presented in the Table 1.2 indicates that treatment measures with capability of settling particles of sizes down to 74 µm will be necessary to facilitate treatment of metals and nutrients in stormwater runoff generated from these areas. The particle size distribution of sediment transported in stormwater is dependent on the geology of the catchment and other studies (eg. Oliver et al (1993)) have found high concentrations of nutrients in colloidal particles which are much finer than 74 µm. Under such circumstances, treatment measures involving significant periods of detention and enhanced sedimentation, using wetland macrophytes, will be necessary (Lloyd, 1997).

Table 1.2
Pollutants Associated with Urban Dust and Dirt (mg/g per mg/L)
(ref. Dempsey et al, 1993)

Contaminant	Particle Size Range					
	<74 µm	74-105 µm	105-250 µm	250-840 µm	840-2000 µm	>2000 µm
Cu	7,100	12,000	66,000	5,900	1,600	344
Zn	28,000	41,000	31,000	11,000	4,100	371
Pd	37,000	55,000	62,000	86,000	19,000	15,000
Total P	3,000	4,800	5,400	2,500	3,000	3,900

1.5 Stormwater Quality Objectives

The specific objectives or performance targets of ABC Waters Design Strategies for Singapore catchments are still evoluting. However, best practice involves a risk-based approach to the protection of environmental values and beneficial uses of urban waterways and aquatic ecosystems. In Australia, reference is made to the objectives of ANZECC/ARMCANZ (2000) and the setting of acceptable risk ambient water quality values based on comparison with reference ecosystems, will be a necessary first step towards setting stormwater management objectives.

In cases of receiving water bodies of environmental significance, the relevant management authority will prescribe water quality guidelines, developed from in-depth investigations. It is envisaged that a similar approach directed at the protection of urban waterbodies in Singapore such as the Marina Reservoir would be appropriate. In most cases, it will be required for land development agencies and enterprises to demonstrate that the development and associated stormwater management strategy has adequately addressed the environmental threats of the project to the receiving waters, and also the opportunities for improved environmental outcomes from the project.

Guidelines for treatment objectives for stormwater quality have been defined in many states in Australia and overseas, to represent achievable targets using best practice. Treatment objectives for stormwater are often expressed in mean annual reductions of pollutant loads from typical urban areas with no stormwater treatments installed and are summarised in Table 1.3 for Australian states. These objectives are used in conjunction with any local site-specific conditions to determine the environmental objectives for stormwater at a site and are recommended for interim applications in Singapore.

It is expected that the treatment objectives will be revised progressively to reflect expected best practice improvements in design. Achieving these objectives does not necessarily suggest that the ultimate receiving water quality outcomes for protecting the health of aquatic ecosystems have been attained. However, it is often seen as a practical approach to institutionalising best practices in stormwater quality management, particularly in built up catchments.

In Singapore, a stormwater characterisation study carried out in August 2012 to Nov 2013. The results revealed that stormwater runoff in Singapore has quite low concentrations of suspended solids and nutrients, except for parkland, agricultural land, areas with high human traffic and some busy roads.

Table 1.3 Stormwater treatment objectives for Victoria and New South Wales, Australia

Pollutant	Stormwater treatment objective
Suspended solids	80% retention of average annual load
Total phosphorus	45% retention of average annual load
Total nitrogen	45% retention of average annual load
Litter	Retention of litter greater than 50mm for flows up to the 3-month ARI peak flow
Coarse sediment	Retention of sediment coarser than 0.125 mm* for flows up to the 3-month ARI peak flow
Oil and grease	No visible oils for flows up to the 3-month ARI peak flow

* Based on ideal settling characteristics

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Stormwater Treatment Elements 2





2

Chapter 2 Stormwater Treatment Elements

2.1 Introduction

This chapter gives an overview of the Water Sensitive Urban Design stormwater treatment elements.

The ABC Waters Design Guidelines cover the most commonly used stormwater quality treatment elements that are applicable in Singapore. Usually, combinations of these elements are used as a treatment train to effectively manage stormwater from a range of different land uses.

Detailed design procedures are provided for the following ABC Waters Design Features in the subsequent chapters:

- Sedimentation basins
- Swale/ buffer systems
- Bioretention swales
- Bioretention basins (Rain Garden)
- Cleansing Biotopes
- Bioengineering
- Constructed wetlands

The selection and placement of the elements within a catchment should be determined during a concept design of a stormwater treatment strategy and is outside the scope of this document.

2.2 Stormwater Quality Management

The impact of poor stormwater quality discharged to receiving environments has in the past decade become an issue of significant concern among catchment managers. The impacts can include increased turbidity and suspended solid concentrations, deposition of suspended material, increased concentrations of nutrients, oxygen-demanding materials, micro-organism and toxic materials, and the deposition of litter. Deposition of suspended material and gross pollutants can smother aquatic habitats. Stormwater contaminants can deplete dissolved oxygen and increase toxicity levels, causing degradation of ecological health of receiving waters.

Increased magnitude and frequency of storm flows can lead to significant changes to the morphology of creeks and rivers leading to degradation of aquatic habitats. The problem is exacerbated by a hydraulically efficient stormwater drainage system within the catchment, leading to frequent flash-flood flow conditions and physical disturbance of aquatic habitats.

The nature of the effects of catchment urbanisation on stormwater and the consequent impact on the environment are short term and long term. It is often not possible to distinguish which of these two factors (ie. poor water quality and hydrologic change) is the dominant cause of environmental degradation of urban aquatic ecosystems.

Singapore is progressively moving towards attaining a higher level of self sustainability in water resources. One of the strategies to achieve this is in the consideration of urban stormwater as a resource. Singapore Government has therefore embarked on a project which will convert Marina Bay, located at the heart of Singapore City, into a freshwater reservoir that will harvest stormwater from one-sixth of the area of metropolitan Singapore. Stormwater quality management and reducing potential impacts of stormwater pollution are therefore important water resource management considerations in Singapore

In formulating stormwater management strategies for multiple objectives, it is vital that the cause-and-effect relationships of stormwater-related environmental problems are first clearly understood. Remedial and preventative measures for improving urban stormwater quality encompass non-structural and structural interventions in urban catchment management practices. Effective and sustainable stormwater management requires the coordinated and integrated implementation of non-structural and structural measures, formulated to accommodate the constraints and opportunities posed by individual catchments.

Best practice urban stormwater management aims to meet multiple objectives including:

- providing flood protection and drainage
- protecting downstream aquatic ecosystems (including groundwater systems)
- removing contaminants
- promoting stormwater elements as part of the urban form.

A fundamental requirement of a stormwater system is to provide a conveyance system for safe passage of stormwater runoff, to avoid nuisance flooding and flood damage to public and private property. In contrast to this requirement, a stormwater system should also provide on-site stormwater retention to protect downstream aquatic ecosystems from increased flow volumes and rates associated with urbanisation. This also avoids increased flooding along downstream waterways and drainage systems, and helps to maintain the hydrological regime of the downstream system.

Typical urbanisation produces many contaminants that can be blown or washed into waterways and affect the health of streams and waterways. Best practice stormwater management provides for treatment of runoff to remove waterborne contaminants, to protect or enhance the environmental, social and economic values of receiving waterways.

As a general rule, site conditions and the characteristics of the target pollutant(s) influence the selection of an appropriate type of treatment measure. Climatic conditions influence the hydrological design and ultimately the overall pollutant removal effectiveness of the measures.

An overriding management objective can help determine what treatment process is likely to be feasible. Figure 2.1 shows a relationship between management issues, likely pollutant sizes and appropriate treatment processes to address those pollutants.

A series of treatment measures that collectively address all stormwater pollutants is termed a 'treatment train'. A treatment train consists of a combination of treatment measures that can address the range of particle size pollutant found in stormwater. A treatment train, therefore, employs a range of processes to achieve pollutant reduction targets (such as physical screening, filtration and enhanced sedimentation). The selection and order of treatments is a critical consideration in developing a treatment train. The coarse fraction of pollutants usually requires removal so that treatments for fine pollutants can operate effectively. Other considerations when determining a treatment train are the proximity of a treatment to its source, as well as the distribution of treatment throughout a catchment.

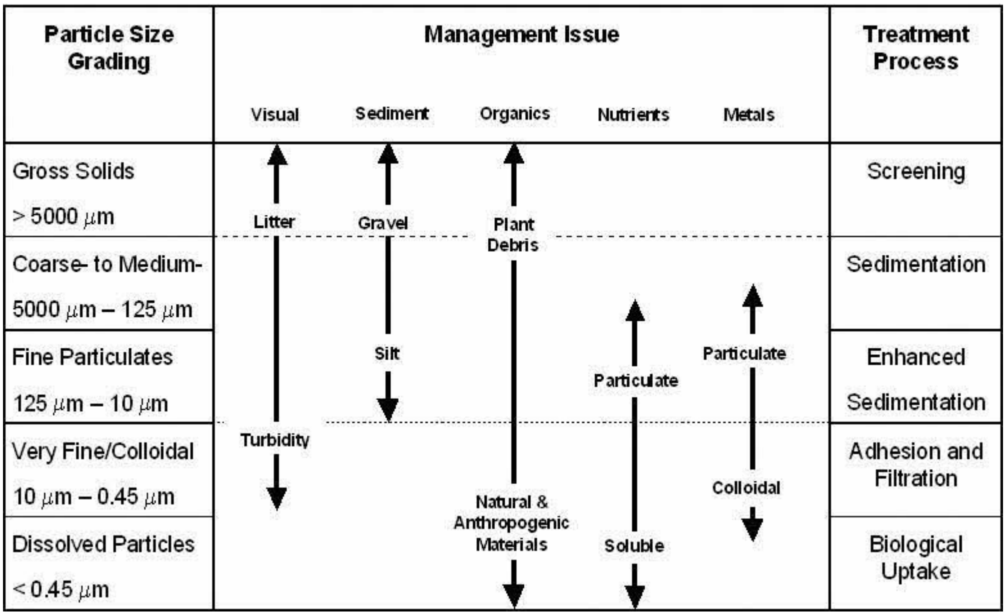


Figure 2.1 Stormwater management issues, pollutants and treatment processes (Ecological Engineering 2003)

Figure 2.2 shows the inter-relationship between stormwater pollutant types (as expressed somewhat simplistically by its physical size), suitable types of treatment measures (based on their treatment process) and appropriate hydraulic loading (expressed as the ratio of the design flow to the area of the treatment measure). The hydraulic loading value can be used to provide an indication of the 'footprint' of a given treatment measure necessary to accommodate the design treatment flow).

As the physical size of the target pollutant reduces (e.g. for treatment of nutrients and metals) the nature of the treatment changes, to include enhanced sedimentation, biofilm adsorption and biological transformation of the pollutants. These treatments use vegetation to provide the filtering surface area, spread, and reduced flow velocities, to allow sedimentation as well as providing a substrate for biofilm growth and hence biological uptake of soluble pollutants. These measures, such as grass swales, vegetated buffer strips, surface wetlands and infiltration systems, require long detention times to allow the various pollutant removal processes to occur. Consequently, the hydraulic loading on these treatment measures is small relative to the measures used for removal of gross solids (and therefore require a larger proportion of land for treatment flows).

Stormwater elements (such as waterways and wetlands) can become an asset for conservation and recreation in developments. Integration of stormwater conveyance and treatment systems into the urban and landscape design of residential areas is now an essential part of urban design, and can lead to better accepted, more environmentally friendly urban areas.

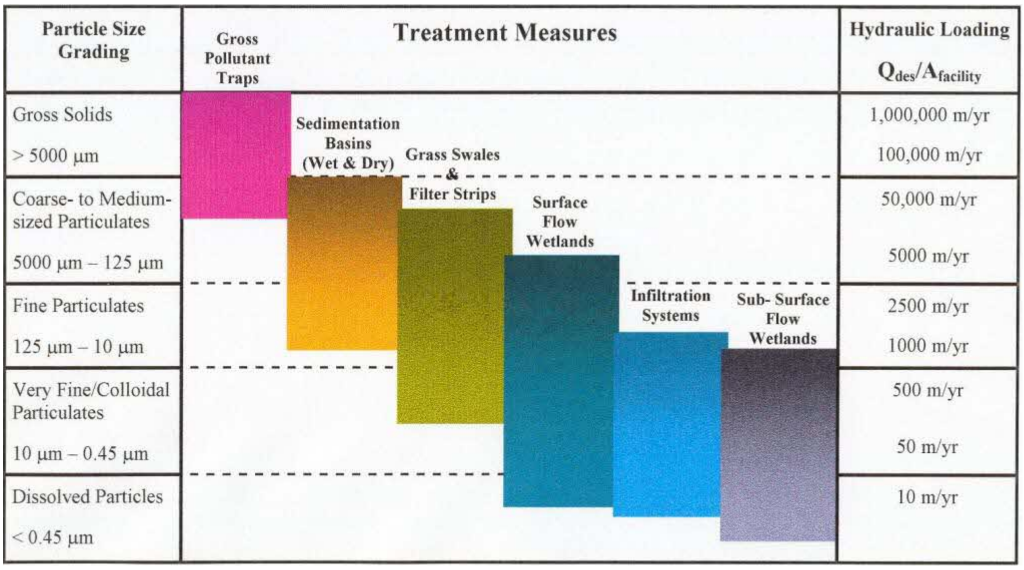


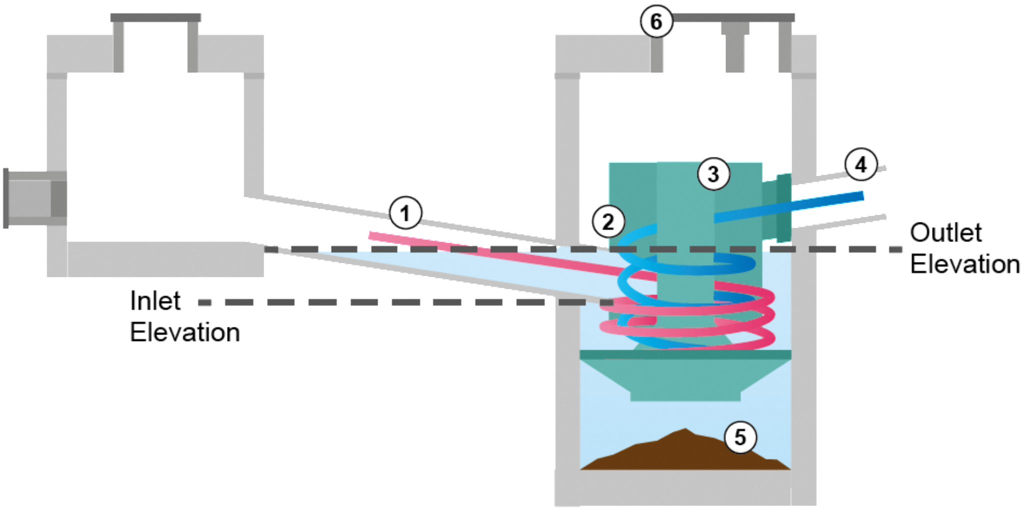
Figure 2.2 **Pollutant size ranges for various stormwater treatment measures (Ecological Engineering 2003)**

2.3 Sedimentation basins

Sedimentation basins are used to retain coarse sediments from runoff and are typically the first element in a treatment train. They play an important role by 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.

Gross pollutants traps are structures that use physical processes to trap solid waste such as litter and coarse sediment. Sedimentation basins are normally used with Gross Pollutant Trap or GPT at the inlet to remove debris, floatable trash and oil from the incoming runoff to prevent these pollutants from entering the Sedimentation basin. A typical schematic diagram for GPT is as shown below.



Components
1. Inlet to Precast Vortex Chamber
2. Cylindrical Baffle
3. Center Shaft
4. Outlet Pipe
5. Sediment Storage Sump
6. Access Lid

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

Maintenance of sedimentation basins involves dewatering and dredging collected sediments. This is required every 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.

Sedimentation basins should be designed to retain coarse sediments only (recommended particle size is 0.125mm). 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, hence other treatment measures that assimilate these pollutants into a substrate are usually used to target this material.

Figure 2.3 Sedimentation basins can be installed into hard or soft landscapes

2.4 Swale/buffer systems

Vegetated swales are used to convey stormwater in lieu of concrete drains and provide a desirable ‘buffer’ between receiving waters (e.g. downstream drain, wetland etc.) 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 dams 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.

Figure 2.4 Swale vegetation is selected based on required appearance and design requirements

2.5 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 uptakes 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 side table of roads.

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 volume.

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

Figure 2.5 Bioretention swales are commonly located in side-table of roads and parks

2.6 Bioretention basins

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

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.

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

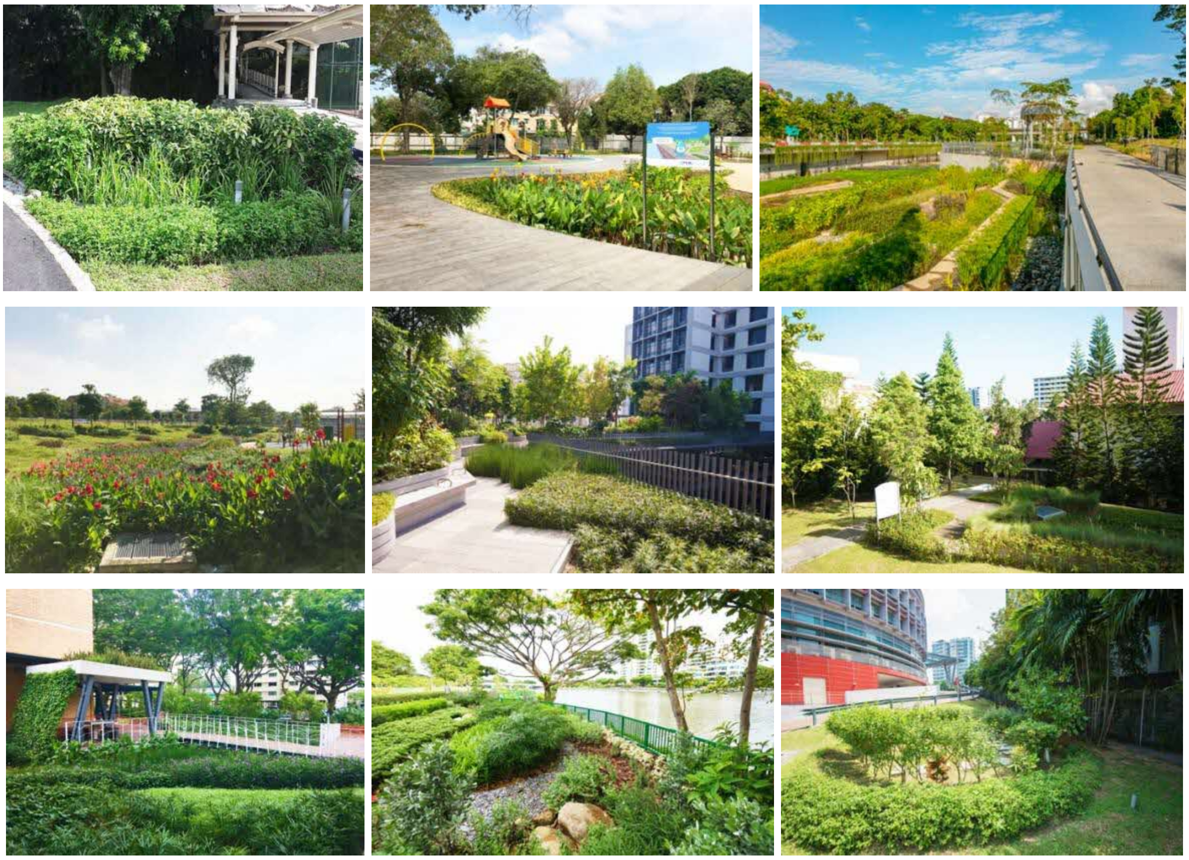


Figure 2.6 Bioretention basins are applicable at a range of scales and can be integrated with an urban landscape

2.7 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.

2.8 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 dry weather water levels.

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

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 playing an important role in stormwater treatment, wetlands can also have significant community benefits. They provide habitat for wildlife and a focus for recreation, such as walking paths and resting areas. They can also improve the aesthetics of a development and be a central feature in a landscape.

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

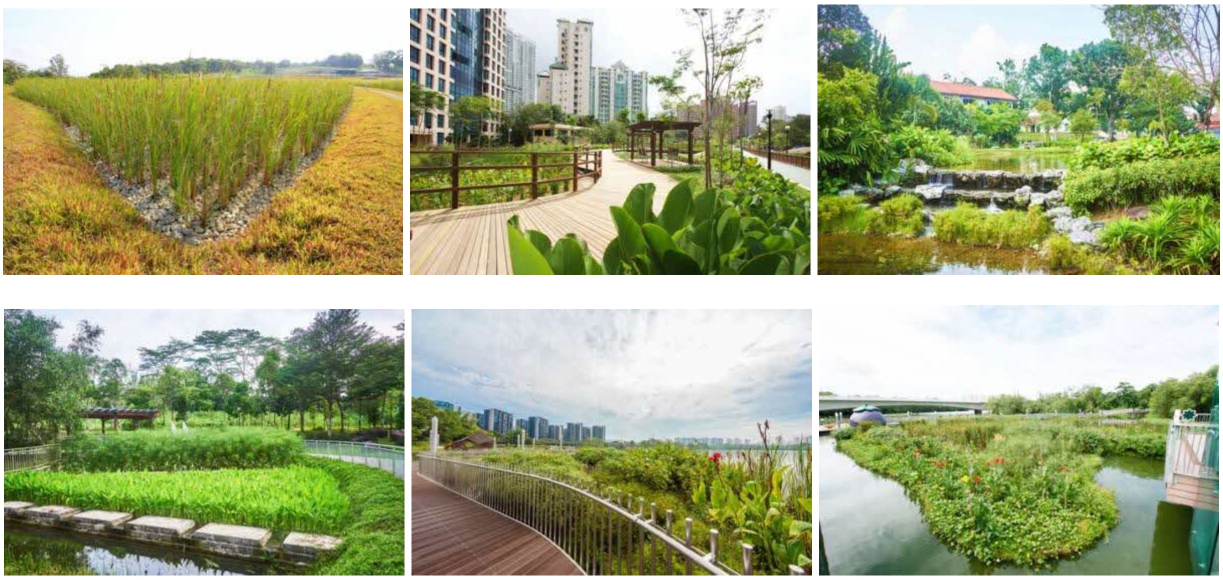


Figure 2.7 Wetlands can be constructed on many scales

2.9 Ponds

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

In areas where wetlands are not feasible (eg. very steep terrain), ponds can be used for a similar purpose of water quality treatment. In these cases, ponds should be designed to settle fine particles and promote submerged macrophyte growth. Fringing vegetation, while aesthetically pleasing, contributes little to improving water quality. Nevertheless, it is necessary to reduce bank erosion. Ponds still require pretreatment such as a sedimentation basin that need to be maintained more regularly than the main open waterbody.

Ponds are well suited to steep confined valleys where storage volumes can be maximised. Some limitations for ponds can be site specific for example; proximity to airports, as large numbers of flocking birds can cause a disturbance to nearby air traffic. They also require regular inspection and maintenance to ensure that there's no stagnant zone and their aesthetic value is not diminished.

Figure 2.8 Ponds are popular landscape features in urban areas

2.10 Rainwater tanks

Rainwater tanks collect runoff from roof areas or other areas for subsequent reuse that reduces the demand on potable mains supplies and reduces stormwater pollutant discharges. Sometimes rainwater tanks are designed to integrate with ABC Waters design features so that treated effluent can be stored for reuse.

There are many forms and sizes of rainwater tanks available. They can be incorporated into building designs so they do not impact on the aesthetics of a development or located underground.

The core sustainability objective of using rainwater tanks is to conserve potable water. In addition to conserving potable water, rainwater and stormwater harvesting on individual allotments are some of the initiatives that can be implemented to deliver such a potable water conservation objective.

The use of rainwater tanks to reduce demand on reticulated potable water supplies and stormwater runoff volume need to consider a number of issues. These are:

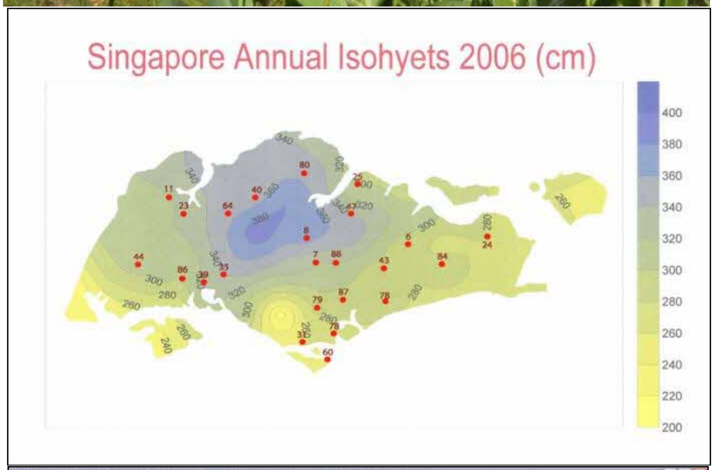
- Supply and demand – conditions such as a low roof area to occupancy ratio (e.g. high density development) can result in large tank volumes to provide a “reliable” supplementary water supply to the end-uses connected to a tank.
- Water quality – the quality of water from rainwater tanks needs to be compatible with the water quality required by the connected “end-use”. There are a number of ways in which the water quality in rainwater tanks can be affected and it is important to understand these so that appropriate management measures can be implemented.
- Stormwater quality benefits – the quantity of the stormwater that is reused from a tank system reduces the quantity of runoff and associate pollutants discharging into a stormwater system. The benefits, in terms of pollutant reduction, should be considered as part of a stormwater treatment strategy.
- Cost – the cost of rainwater tanks needs to be considered against alternative demand management initiatives and alternative water sources.
- Available space - small lots with large building envelopes may preclude the use of external, above ground, rainwater tanks.
- Competing uses for stormwater runoff – there may be situations where a preferred beneficial use for stormwater runoff (such as irrigation of a local public park, oval, or golf course) may provide a more cost-effective use of runoff from roofs than the use of rainwater tanks on individual allotments.
- Maintenance – Pumps, valves and filtration system may be integrated with rainwater harvesting tank. Most rainwater tanks will need to be maintained by M&E contractor engaged by the householder or the MCST (or similar).

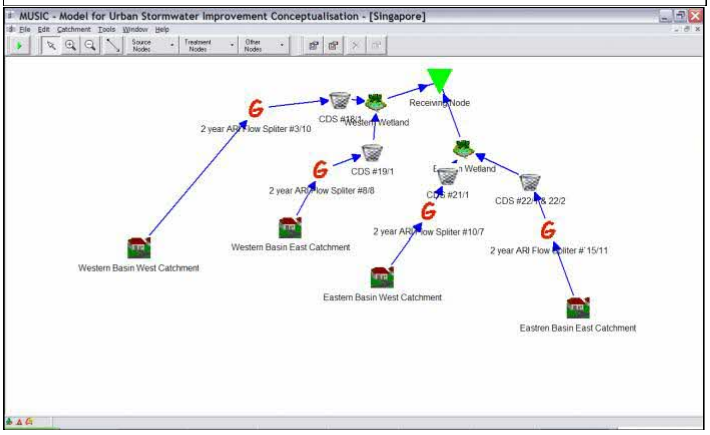
Figure 2.9 Rainwater tanks are available in a range of sizes and shapes

2.11 References

Ecological Engineering (2003), Landcom Water Sensitive Urban Design Strategy – Design Philosophy and Case Study Report, report prepared for Landcom, NSW

Planning and Sizing Treatment Systems 3





3

Chapter 3 Planning and Sizing Treatment Systems

3.1 Stormwater Quality

Stormwater pollutants from urban developments originate from a variety of sources in the catchment. Table 3.1 summarises the sources of some of the more common urban runoff pollutants as outlined by Lawrence and Breen (2006). Suspended solids, nutrients, BOD_5 and COD and micro-organisms are usually considered the most significant parameters in terms of ecological impacts. Oils and surfactants, and litter have aesthetic impacts which are more renowned for generating community concern and action. Organic load in stormwater originates mainly from leaves and garden litter. As a significant amount of inorganic pollutants is sediment bound, effective treatment of suspended solids is often a minimum criterion in stormwater quality management with the expectation that a significant amount of organic and inorganic pollutant will also be treated.

Table 3.1 Typical Urban Runoff Pollutant Sources

Pollutant Source	Solids	Nutrients	Pathogens	DO Demands	Metals	Oils	Synthetic Organics
Soil Erosion	✓	✓		✓	✓		
Cleared Land	✓	✓	✓				
Fertilisers		✓			✓		
Human Waste	✓	✓	✓	✓			
Animal Waste	✓	✓	✓	✓	✓		
Vehicle Fuels and Fluids	✓		✓	✓	✓		
Fuel Combustion		✓			✓	✓	
Vehicle Wear	✓				✓		
Industrial and Household Chemicals	✓	✓			✓	✓	✓
Industrial Processes	✓	✓			✓	✓	✓
Paint and Preservatives					✓	✓	
Pesticides					✓	✓	✓
Stormwater Facilities	✓	✓	✓	✓	✓		

3.1.1 Suspended Solids

Suspended solids comprise of inorganic and organic materials. Sources of inorganic suspended solids include soil particles from erosion and land degradation, streets, households and buildings, and airborne particulate matter. Contributors to organic suspended solids are bacteria and microorganisms such as those found in sewage. The level of suspended solids in urban runoff is comparable to raw sewage and, inorganic soil particles are particularly of concern. Large amounts of inorganic soil particles are often associated with urban construction and the development of supporting services including roads, sewers and drainage systems. Levels of inorganic soil particles generated from these activities are at least two to six times, and can be up to several hundred times, pre-development levels.

Turbid waters often result from the presence of suspended solids. In general, the community associates turbid waters with environmental pollution and degradation of the water's aesthetic value.

Nutrients and toxins such as phosphorus, heavy metals and organic chemicals utilise sediment as the medium for transportation in urban runoff. The deposition of sediments can result in the release of these toxins and nutrients at a later time when the ambient conditions related to the redox potential of the sediment and water column becomes favourable for their release. This mechanism provides the opportunity for pollutant re-mobilisation in later flow events enhancing the risk of further downstream degradation.

Suspended solids also reduce the penetration of light through water, and this adversely affects the feeding and respiration of aquatic plants. Duncan (2006) presents typical concentrations of suspended solids in urban stormwater runoff from different land use, expressed as a log-normal distribution (Figure 3.).

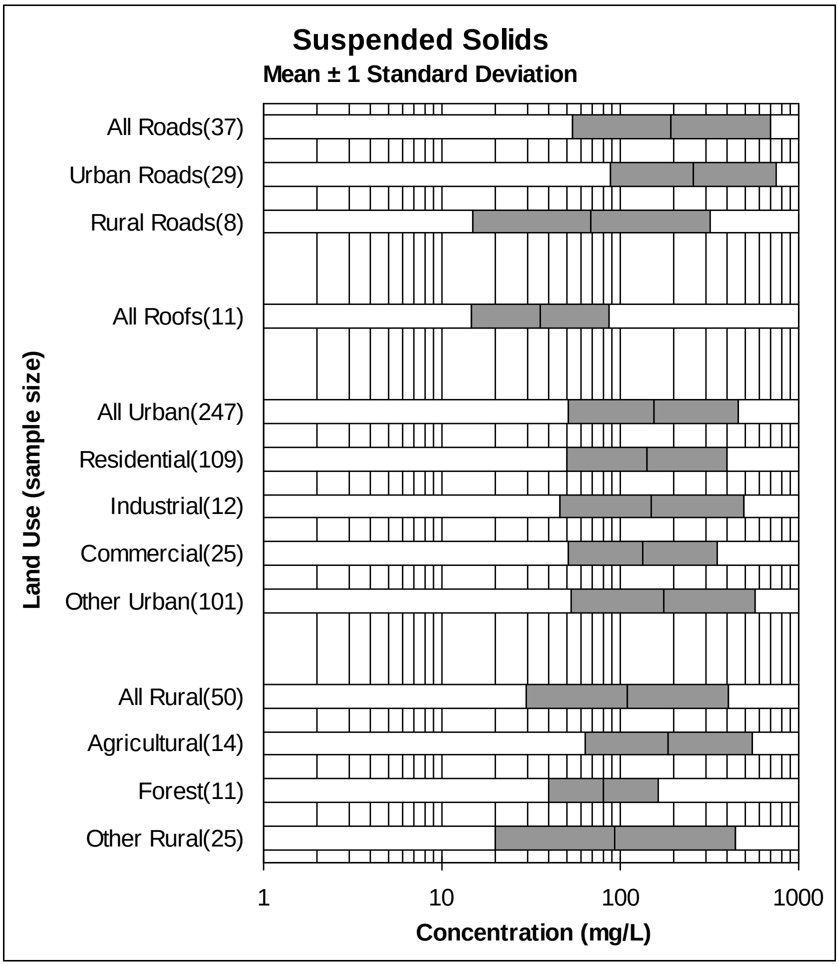


Figure 3.1 Suspended Solids Concentration vs Land Use (Duncan, 2006)

3.1.2 Nutrients

Nutrients are fed into the water system through many different sources. These include sewerage, plant matter, organic wastes, fertilisers, kitchen wastes (including detergents), nitrous oxides produced from vehicles exhausts and ash from bushfires. Nutrients contain natural compounds consisting of nitrogen and phosphorus.

There are problems associated with high levels of nutrients in waterbodies. Nutrients promote growth of aquatic plant life including floating macrophytes, which if in large concentrations, produce algal blooms on the water surface. Algae are microscopic plants which occur naturally in waterways. With an increase in nutrients algal growth becomes excessive often resulting in a build up of toxins. Toxic algal blooms cause the closure of fisheries, water farming industries and public beaches.

Key nutrients of interest in managing urban waterways are phosphorus and nitrogen. Phosphorus concentration in urban stormwater is often expressed as Total Phosphorus (TP) and is the sum of dissolved and particulate phosphorus. Each fraction can be subdivided into reactive, acid-hydrolysable, and organically bound phosphorus, according to its chemical availability. Reactive phosphorus is readily available for uptake by organism (e.g. algae). In urban stormwater, between 10% and 30% of TP is made up of soluble phosphorus. Duncan (2006) presents typical concentrations of Total Phosphorus in urban stormwater runoff from different land use, expressed as a log-normal distribution (Figure 3.).

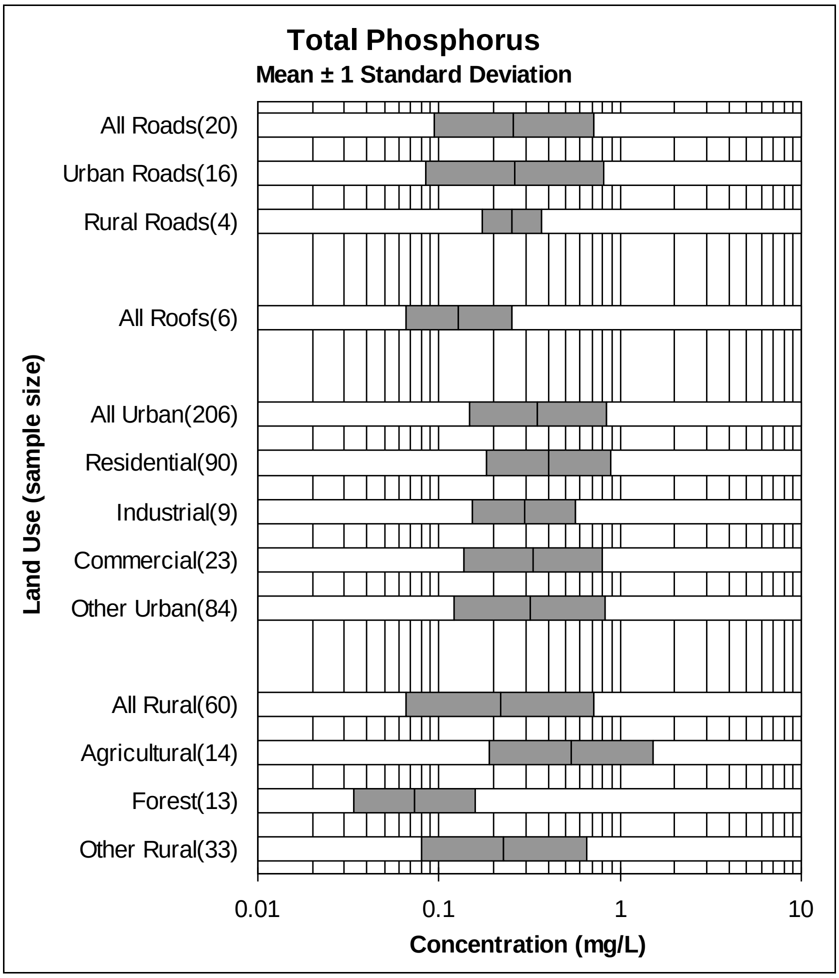


Figure 3.2 Total Phosphorus Concentration vs Land Use (Duncan, 2006)

Nitrogen concentration in urban stormwater is often expressed as Total Nitrogen which is the sum of several forms. Organic nitrogen plus ammonia nitrogen comprises total Kjeldahl nitrogen. Nitrite plus nitrate comprise oxidised nitrogen. Total Kjeldahl nitrogen and oxidised nitrogen together make up total nitrogen. Nitrogen can be converted between these forms, and also to nitrogen gas, by chemical and biological action. It is a common but not universal practice to quote concentrations in terms of the mass of nitrogen only, rather than the mass of the compound in which it occurs. Nitrite and nitrate, in particular, may be expressed in either form in the published literature.

Duncan (2006) presents typical concentrations of Total Nitrogen (TN) in urban stormwater runoff from different land use, expressed as a log-normal distribution.

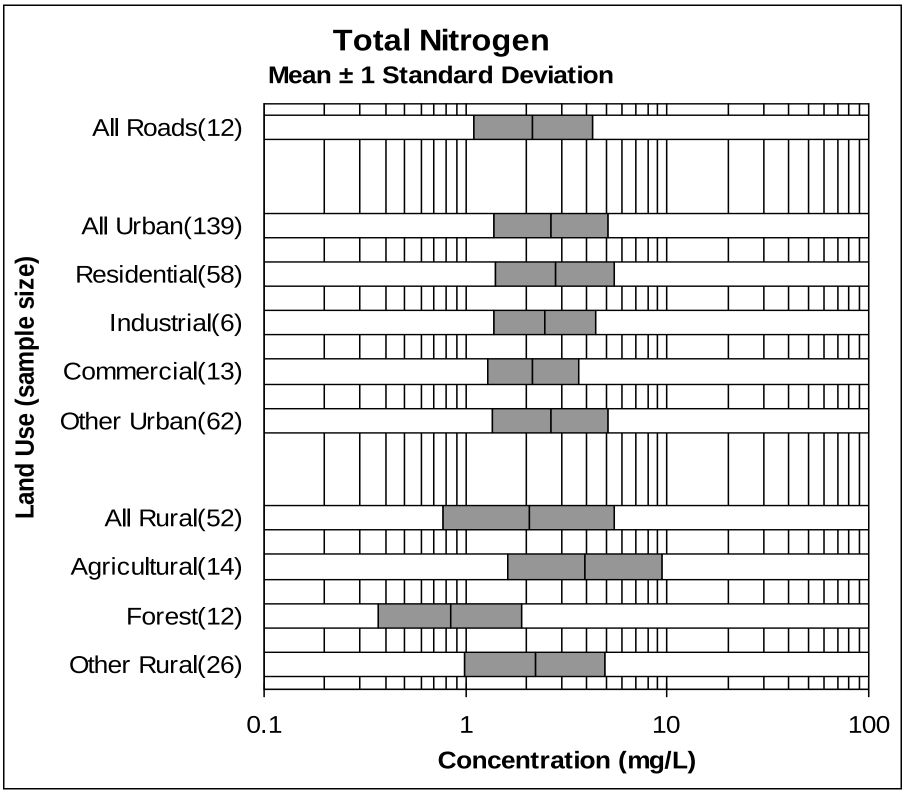


Figure 3.3 Total Nitrogen Concentration vs Land Use (Duncan, 2006)

3.1.3 Litter

Litter is generally the most noticeable indicator of water pollution to the community. Litter is also commonly thought of as the pollutant most detrimental to waterways because of its visibility. Pollution of the environment including the export of litter and gross pollutants has intensified over the last 30 years due to the production of easily disposable, non-biodegradable packaging and household and industrial items. The sources of litter are varied and they include dropping of rubbish, overflows of rubbish containers and material blown away from tips and other rubbish sources. Allison et al. (1998) defines gross pollutants as the material that would be retained by a five-millimetre mesh screen, thus eliminating practically all sediment except that attached to litter and other large debris. Figure 3. 4 shows the gross pollutant load generated in urban catchments in Australia.

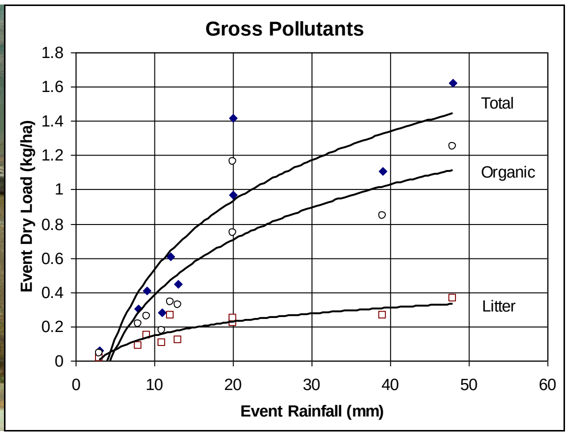


Figure 3. 4 Gross Pollutant Event Loads vs Rainfall (redrawn from Allison et al. (1998))

3.1.4 Metals

Analysis of contaminants associated with urban dust and dirt by Dempsey et al (1993) found highest concentrations of Cu, Zn and TP to be associated with particles in the 74 μm to 250 μm. The particle size range with high Pb association extends to 840 μm. One possible explanation for a higher contaminant concentration is that the size range is the higher specific surface area (and thus contaminant binding sites) of particles in this range. For example, Sansalone and Buchberger (1997) found that specific surface area of solids transported from an urban roadway surface decrease with increasing particle size as is normally the case for spherical particles. With irregularly shaped particles, there is the general tendency for larger sized particles to have higher specific surface area than are normally expected.

Table 3.2 reproduces the table of particle sizes and associated pollutants presented by Dempsey et al (1993) for dust and dirt generated from road surfaces. The data presented in the Table 3.2 indicates that treatment measures with capability of settling particles of sizes down to 74 μm will be necessary to facilitate treatment of metals and nutrients in stormwater runoff generated from these areas. The particle size distribution of sediment transported in stormwater is dependent on the geology of the catchment and other studies (eg. Oliver et al (1993)) have found high concentrations of nutrients in colloidal particles which are much finer than 74 μm. Under such circumstances, treatment measures involving significant periods of detention and enhanced sedimentation, using wetland macrophytes, will be necessary (Lloyd, 1997).

Table 3.2 Pollutants Associated with Urban Dust and Dirt (mg/g per mg/L) (ref. Dempsey et al, 1993)

Contaminant	Particle Size Range					
	<74 μm	74-105 μm	105-250 μm	250-840 μm	840-2000 μm	>2000 μm
Cu	7,100	12,000	66,000	5,900	1,600	344
Zn	28,000	41,000	31,000	11,000	4,100	371
Pb	37,000	55,000	62,000	86,000	19,000	15,000
Total P	3,000	4,800	5,400	2,500	3,000	3,900

Duncan (2006) presents typical concentrations of Cu, Zn and Pb in urban stormwater runoff from different land use, expressed as log-normal distributions.

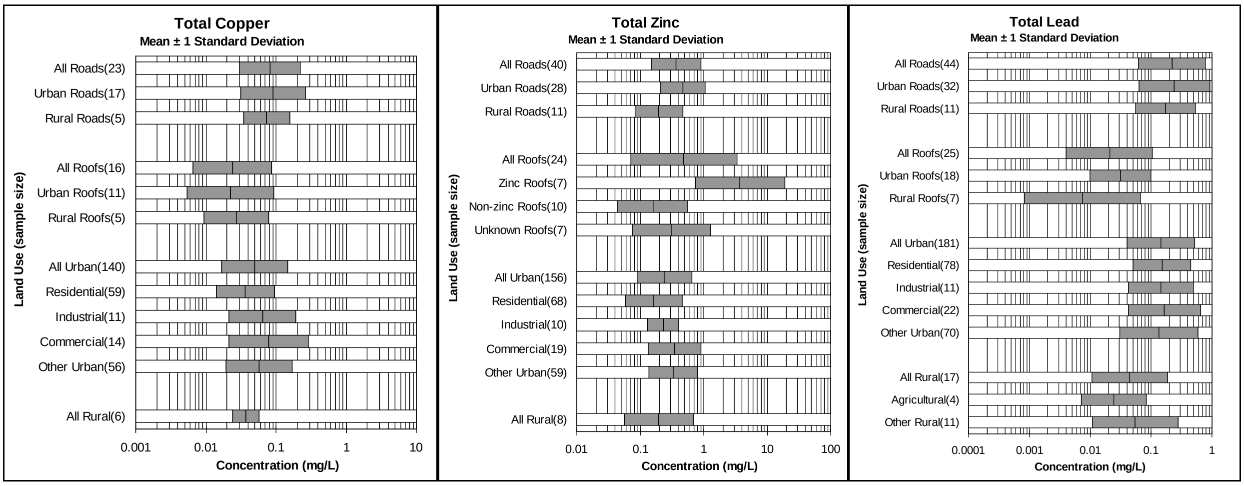


Figure 3.5 Copper, Zinc and Lead Concentrations vs Land Use (Duncan, 2006)

3.2 Formulating a Stormwater Management Strategy

3.2.1 General

Structural and non-structural stormwater management measures take many forms and can often be directed at addressing specific problems. In most instances, a number of management measures can be implemented in series or concurrently forming a treatment train approach to stormwater management. Figure 3. shows the various types of treatment works within an overall regional management flow chart, which could form an integrated catchment management strategy.

The correct utilisation of the various components of the treatment train is a vital design consideration and requires a holistic approach to their performance specifications and positions in the treatment train. An overview of common elements of the stormwater treatment train may be summarised as follows:-

Source Controls

- Community awareness
- Land use planning and regulation
- Street cleaning
- Sewer leakage management
- Isolation of high pollutant source areas
- Construction site management
- Landfill management
- Litter traps
- On-site detention basins
- Stormwater infiltration systems
- Buffer strips

In-transit Controls

- Gross pollutant traps
- Swale drains
- Detention basins
- Ponds and wetlands

End-of-pipe Controls

- Gross pollutant traps
- Lakes
- Floating booms
- Ponds and wetlands
- Receiving water management

The list of stormwater management measures is by no means exhaustive and they serve to outline common techniques currently used in the industry.

The proper utilisation of the various components of the treatment train should be based on the general philosophy of:-

1. avoiding pollution whenever possible through source control measures;
2. controlling and minimising pollution by means of in-transit and end-of-pipe control methods where pollutant generation cannot be feasibly avoided; and
3. managing the impacts of stormwater pollution by managing receiving waters and their appropriate utilisation as a last resort.

3.2.2 Developing ABC Waters Strategy

ABC Waters strategy allows for the integration of all ABC Waters Design features within the development to ensure that the site complies with established sustainability objectives. The strategy needs to consider site specific environmental conditions that influence implementation of ABC Waters Design features, such as rainfall, topography, soils, creeks and receiving waters. The nature of the proposed development will also influence the implementation of ABC Waters Design Features. Therefore project-specific ABC Waters strategy is important to ensure that it leads to the best outcomes for each project, and can be integrated with the urban design masterplan or structure plan.

Key issues for the implementation of stormwater quality initiatives are outlined as follows:-

- Use ABC Waters Design Features in the urban landscape to maximise the visual and recreational amenity of developments.
- Size ABC Waters Design Features relative to the contributing catchment area and impervious fraction, as demonstrated in the sizing curves presented in this section.
- Generally, ABC Waters Design Features is most effective on slopes of 1-4%.
- Where slopes exceed 4%, either discrete treatment systems such as bioretention street planters or additional flow control features (such as check dams in swales and bioretention swales) can be used.
- Use ABC Waters Design Features such as wetlands and bioretention raingardens in open space areas where practical.
- Use ABC Waters Design Features such as bioretention swales in streets on the high-side verge reserve if there is one, or in the centre median of dual travel-way streets. ABC Waters Design Features such as bioretention raingardens can also be incorporated between parking bays or in traffic-calming features.

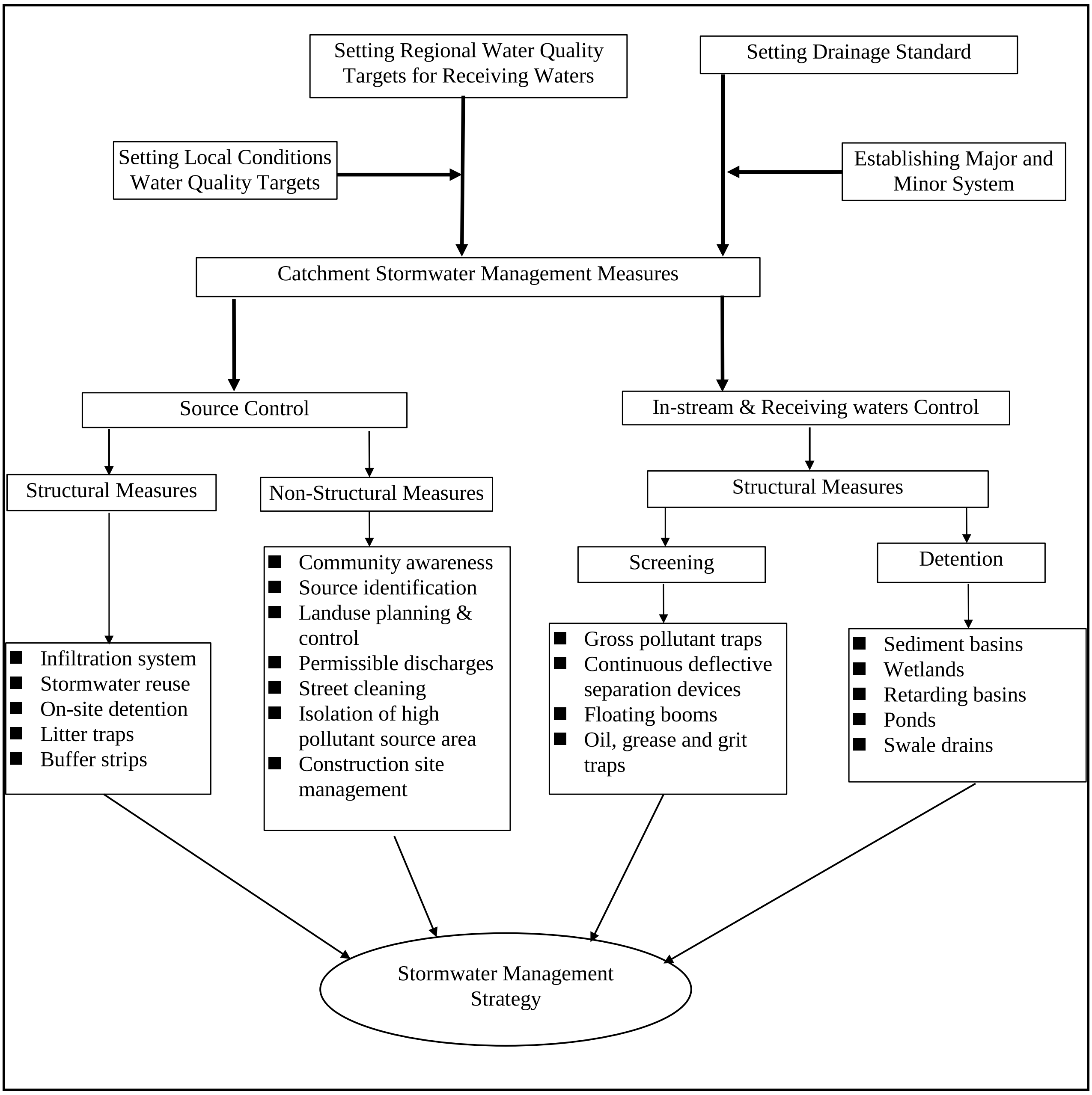


Figure 3.6 Structural and Non-structural Stormwater Management Measures

3.2.3 Best Planning Practice

The layout of the combination of Best Management Practice (BMPs) included within a 'treatment train' may be viewed as Best Planning Practice (BPP), although the two are not mutually exclusive as indicated by Figure 3. The selection of appropriate BMPs to include within a treatment train involves an assessment made within a variety of disciplines in order to account for site specific characteristic and limitations. This procedure is illustrated in Figure 3. and Figure 3.

Stormwater characteristics are highly varied and the effectiveness of individual BMPs and the treatment train as a whole will differ from one event to another. A statistical approach is probably the most appropriate method of evaluating the performance of the treatment train. A number of approaches can be adopted in evaluating the effectiveness of the stormwater management strategy ranging from detailed continuous model simulations to simplified flow frequency/mean event pollutant concentrations.

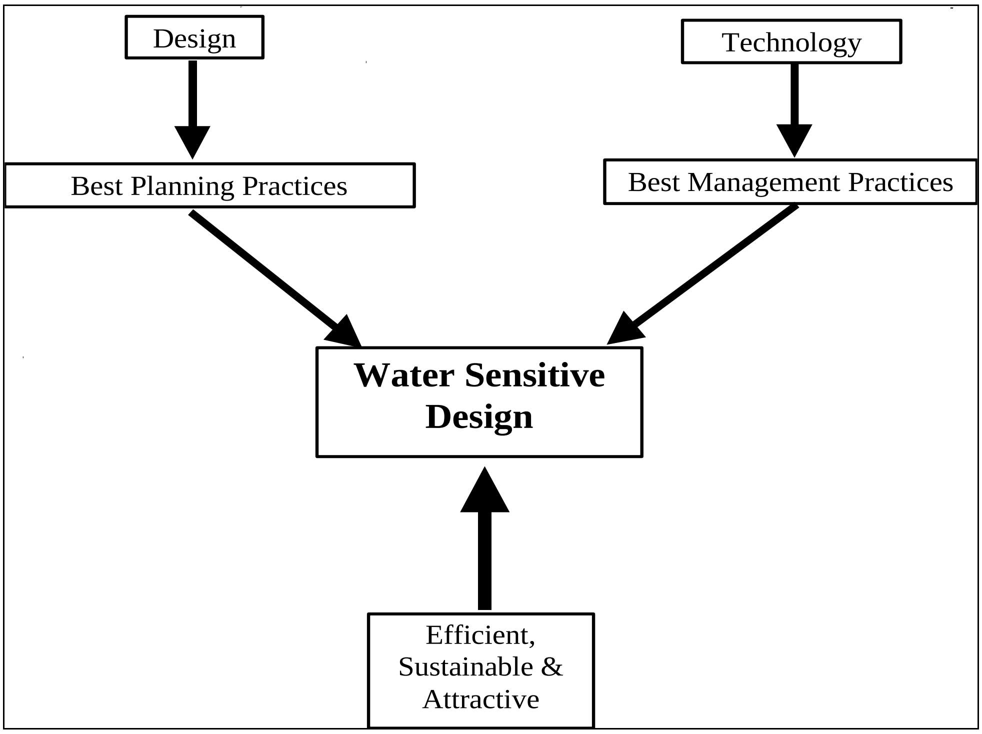


Figure 3.7 Incorporation of Best Management Practices and Best Planning Practices in Water Sensitive Urban Design (ABC Waters Design)

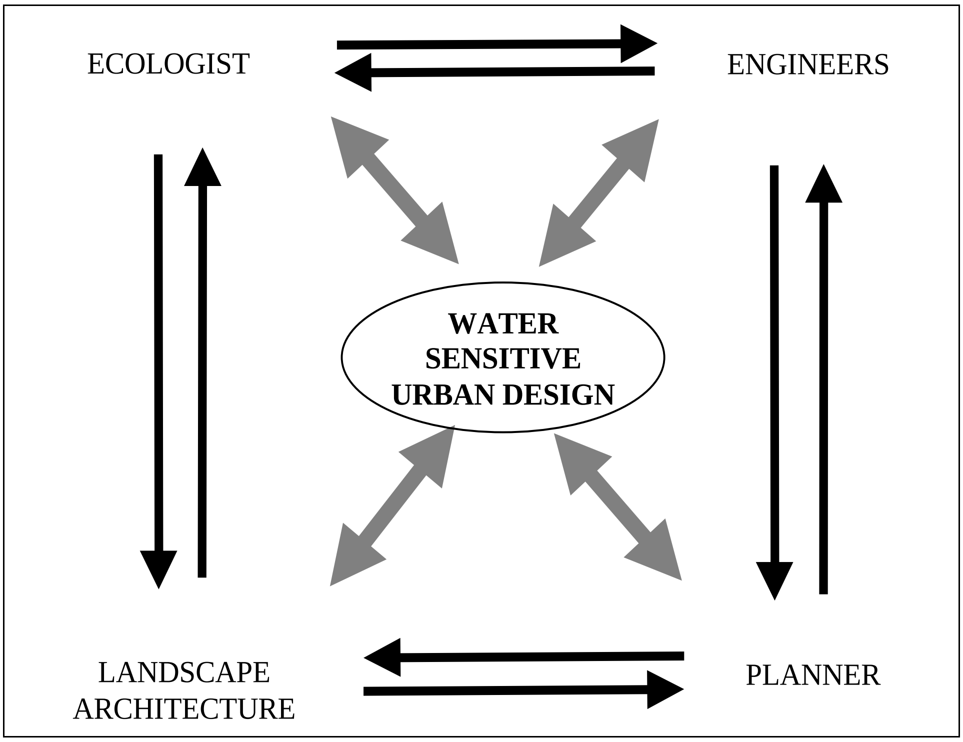


Figure 3.8 Study Teams Involved in Water Sensitive Urban Design (ABC Waters Design)

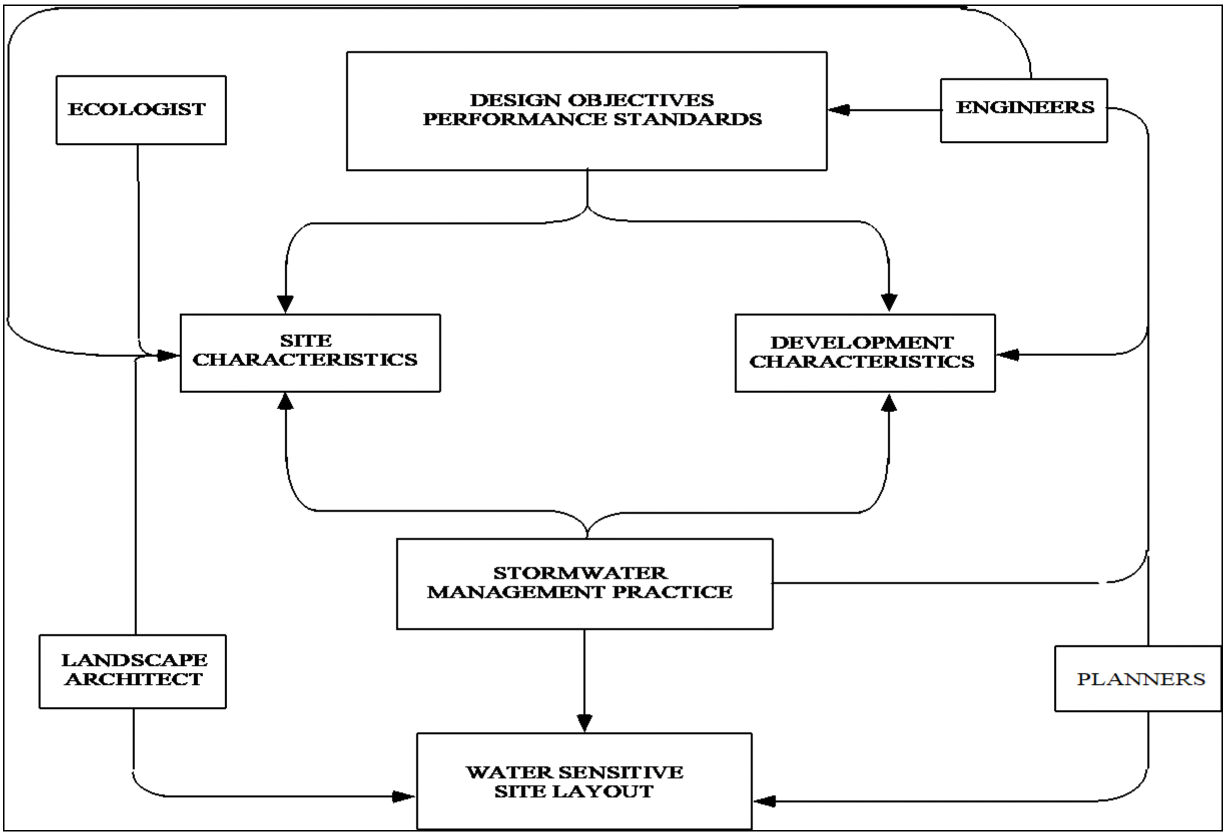


Figure 3.9 Steps to Developing the Site Layout for Integrated Stormwater Management

While WSUD elements or ABC Waters Design Features (in Singapore context) may have a landtake of up to 5% of the site area, implementing them into the urban design can optimise outcomes, due to:

- ABC Waters Design Features being integrated into streetscapes and not requiring larger areas at the outlet of developments
- ABC Waters Design Features acting as an interface between the development and the riparian zone.

The integration of ABC Waters Design Features into development projects is an iterative process involving the masterplanner and the project team. Ideally, this integration will be facilitated through close communication with the masterplanner and through a series of workshops to present preliminary thoughts and analyses of possible ABC Waters Design options that meets the design objectives of the project. The expected outcome from this process is the preferred lay-out of the site to meet the range of urban design objectives.

3.2.4 Public Open Space (POS) Layout

Integration of public open space (POS) with conservation corridors, stormwater management systems and recreational facilities is a fundamental objective of ABC Waters Design Strategy. POS areas can potentially incorporate stormwater conveyance and treatment systems as landscape features within a multiple use corridor. This can provide a recreation focus (such as a linear park with bike path or an urban forest) as well as enhancing community understanding and regard of stormwater as a valuable resource. The key principles to be considered in locating POS areas:

- Align POS along natural drainage lines.
- Protect/enhance areas containing natural water features (such as creeks and wetlands) and other environmental values by locating them within POS.
- Utilise POS to provide links between public and private areas and community activity nodes.

For water sensitive or ABC Waters landscape design the following natural landscape values should be considered:

- retention of natural features – watercourses, landforms and other water features should be retained or restored
- use of indigenous species – existing native vegetation needs to be retained or restored. Vegetated links should be provided with native vegetation on adjoining land
- planting – should be limited to locally indigenous species (or specifically appropriate other species) and exclude groups that can cause weed problems
- fauna habitat – provision should be made for fauna habitat measures such as wetlands, ponds, shrubs and nest boxes.

3.2.5 Road Layouts and Streetscaping

Roads account for a significant percentage of the overall impervious area created within a typical urban development and therefore can significantly change the way water is transported through an area. These areas also generate a number of water borne stormwater contaminants that can adversely impact on receiving waterway health (e.g. metals and hydrocarbons). Consequently, it is important to mitigate the impact of stormwater runoff generated from road surfaces. By carefully planning road alignments and streetscapes, ABC Waters Design features such as bioretention systems and vegetated swales can be used to collect, attenuate, convey and treat the runoff before discharge to receiving waterways.

Key principles in selecting road alignments and streetscapes for ABC Waters Design depend on the natural topography and overall masterplan for the development. Some general considerations include:

- Generally, ABC Waters Design Features in the streetscape are most effective on slopes of 1-4%, i.e. where road grades are 1-4%.
- Where slopes exceed 4%, either discrete treatment systems such as bioretention street tree planters or additional flow control features (such as check dams with swales and linear bioretention systems – see picture to right) can be used.
- Use ABC Waters Design Features such as bioretention swales on the high-side verge reserve if there is one.
- Where the street runs perpendicular to the contours, use either verge for bioretention systems.
- Where practical, incorporate ABC Waters Design Features in the centre medial of dual travel-way streets.

- Ensure street or driveway crossovers of bioretention swales are either at grade or incorporate a culvert crossing. If this is not possible, use discrete ABC Waters Design Features separated by driveway crossovers.
- Street-scale ABC Waters Design Features should be part of an overall ABC Waters Design strategy for a development.
- It is not necessary to provide ABC Waters Design Features on all streets, however streetscape may form an important part of an ABC Waters Design strategy for a development.
- Parking areas can be located adjacent to ABC Waters Design Features, but should be designed to prevent vehicles damaging these systems. Bollards or kerbs with regular breaks are required to allow distributed flow to the ABC Waters Design Features.
- Parking areas may be interspersed between ABC Waters Design Features, such as parking bays between raingardens.

Below are several examples of ABC Waters Design Features in streetscapes of varying scales.

3.3 Sizing Stormwater Treatment Systems

3.3.1 General

Successful environmental management of urban stormwater requires understanding of:

- relationships between rainfall and runoff in the urban context,
- pollutant generation from differing land uses and catchment characteristics,
- performance of stormwater treatment measures, and how it may vary with design specifications,
- long-term performance of proposed stormwater strategies against water quality standards,
- resultant impacts on receiving ecosystems, before and after implementation of the proposed stormwater strategy.

The performance of a stormwater quality management strategy is not determined by any individual event or dry spell, but is the aggregate of a continuous period of typical climatic condition. Modelling using well-established computer models of urban water systems is a recognised method for determining the long-term performance of water management strategies.

Stormwater quality management systems are often highly complex and difficult to understand without tools such as models. Examples of this include large catchments with varying land uses and a convoluted drainage network that delivers urban stormwater and runoff from other land uses in the catchment at different times and rates of flow. Furthermore, stormwater systems are highly non-linear and exhibit characteristics that are probabilistic or depend on antecedent conditions in some cases. This requires modelling to enable an adequate understanding and assessment to be undertaken.

Modelling will involve the use of historical or synthesised long-term rainfall and evapotranspiration information, expected water consumption data, algorithms that simulate the operation of alternative water source systems, and algorithms that simulate the performance of stormwater treatment measures to determine water conservation, pollution control and flow management outcomes.

As discussed earlier, stormwater-based pollutant exports have been shown to be variable, i.e. highly stochastic in manner. Patterns of stormwater quality vary highly both between, and within, individual storm events. The concept of a ‘design storm’ is of little use in stormwater quality modelling. A continuous modelling approach is more appropriate with simulation period of one or many years.

The performance of stormwater quality management measures can be highly variable during and between individual storm events. Issues such as antecedent rainfall, individual storm intensities and magnitudes and the time of year can all affect the performance of a stormwater quality management measure. Again, these processes are typically assessed with a continuous modelling approach, incorporating the inherent variability of rainfall and streamflow occurrences and associated operation of individual stormwater quality management facilities.

Often the performance of a proposed water management strategy will need to be benchmarked against current conventional design. Modelling techniques allow for this comparison by simulating the likely performances of a range of water management scenarios, based on a ABC Waters Design approach, and benchmarking against the performance of a conventional urban water cycle management design approach.

3.3.2 Rainfall in Singapore

Figure 3.10 shows the location of Singapore rainfall stations and annual rainfall isohyet (cm) in 2006. The distribution of the 2006 annual rainfall shows a significantly high variability, ranging from 2200 mm to 3800 mm.

An updated annual rainfall isohyet for Singapore (based on 30-year climatological reference period of 1991-2020) is shown in Figure 3.11 (2022 Annual Climate Assessment Report, Meteorological Service Singapore). The report also states that while the annual total rainfall for Singapore has a gradual increasing trend of 78mm per decade from 1980 to 2022, the trend is not statistically significant.

The monthly rainfall charts (Figure 3.12a and 3.12b) illustrate the variation of average monthly rainfall for Singapore and the Changi climate station for the same 30 year period (2022 Annual Climate Assessment Report, Meteorological Service Singapore). November, December and January are generally the wettest months and there is a tendency for lower rainfall during the middle months of the year. This is due to Singapore’s climate which is characterised by two monsoon seasons (Northeast Monsoon from December to early March, and Southwest Monsoon from June to September), separated by inter-monsoonal periods.

Monthly rainfall patterns can also vary from year to year, as can be seen from the comparison of the 2022 monthly total rainfall for Singapore and the Changi climate station against the corresponding 30-year average. The characteristics are symptomatic of climatic conditions where rainfall events are dominated by spatially random local thunderstorm activities.

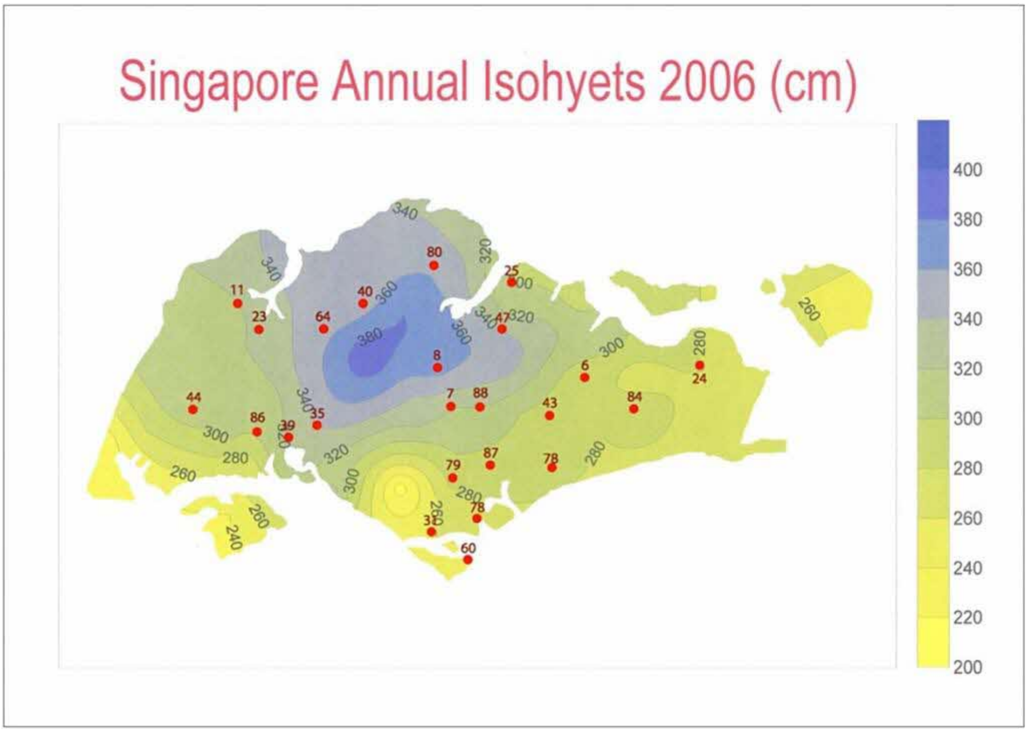


Figure 3.10 Distribution of Singapore rainfall stations and annual rainfall isohyets (cm) in 2006

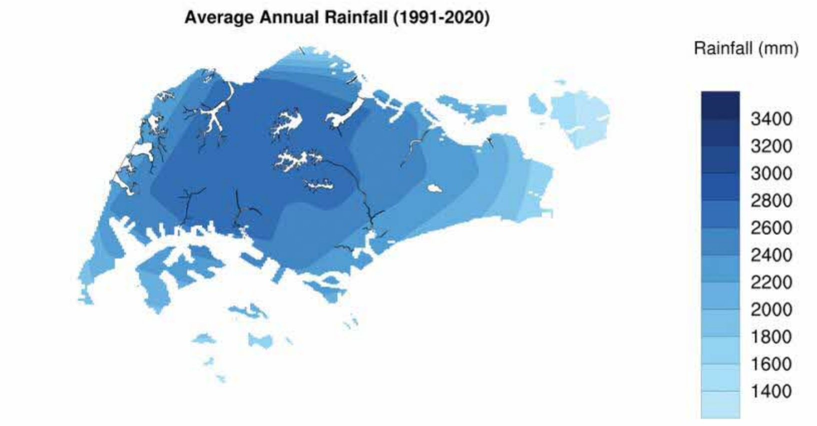


Figure 3.11 Average annual rainfall isohyets (1991-2020)

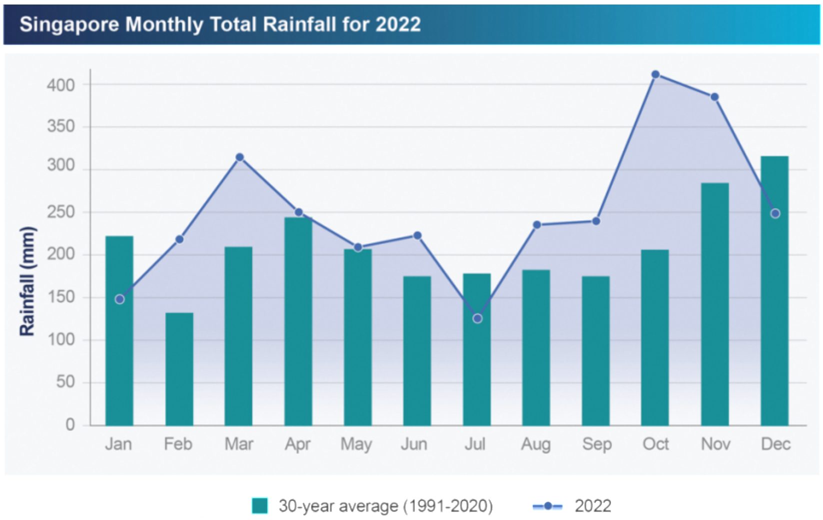


Figure 3.12a Singapore monthly total rainfall for 2022 (solid line) and long term average (bars, 1991-2020)

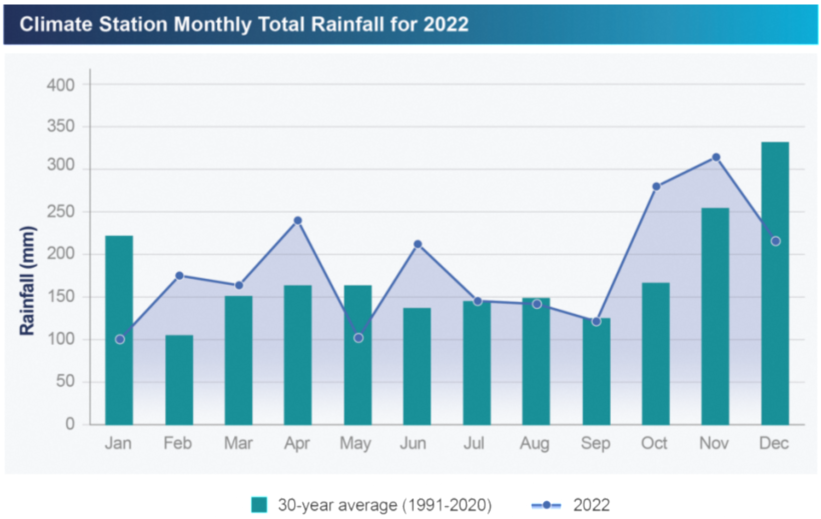


Figure 3.12b Changi climate station monthly total rainfall for 2022 (solid line) and long term average (bars, 1991-2020)

3.3.3 Performance of Treatment Systems in Singapore

The significance in variation in performances of stormwater treatment throughout Singapore in response to the observed variability in annual rainfall and monthly patterns were tested. 10 minute rainfall data for year 2006 from 22 stations across Singapore (Figure 3.10) were used to provide good geographic spread of rainfall data across Singapore. The data was aggregated into 30 minute time steps so that is in a suitable format for MUSIC modelling. While dated, the 2006 data covered the range of annual rainfall based on the long term 30-year average (1991-2020) and was deemed to be suitable for the study.

MUSIC modelling of the available 2006 meteorological data was undertaken to assess the effectiveness of bioretention systems throughout Singapore as a basis for examining treatment performances in general. The results are shown Figure 3.13.

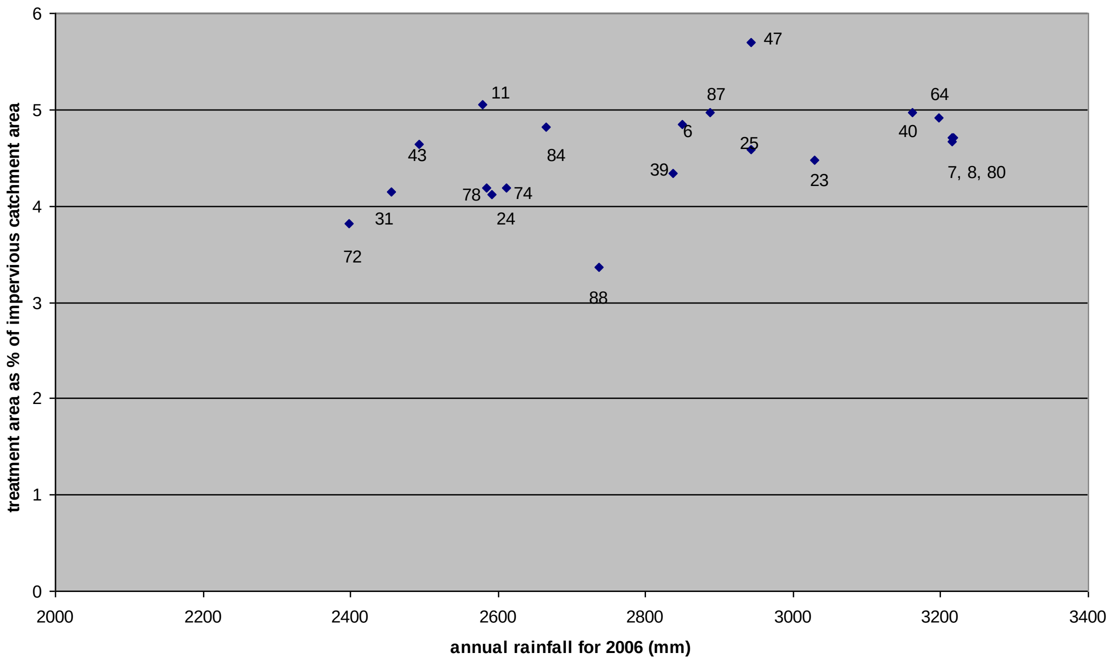


Figure 3.13 Comparison of bioretention size required to achieve 45% TN load reduction against mean annual rainfall for Singapore rainfall stations

From Figure 3.13, it is evident that despite the significant range in annual rainfall of the stations tested, there was only a weak trend of increasing treatment area with rainfall observed. The sizes of bioretention area necessary to deliver a 45% reduction in mean annual TN load falls generally between 4% and 5% (with the exception of Station 88 and Station 47, both stations having near-average annual rainfall). Given this result, there appears little benefit in creating different design zones for Singapore.

Thus it was considered reasonable to develop a single set of design curves based around the upper limit (ie. 5% for bioretention systems) that applicable for all regions in Singapore for each treatment measure. This would for a simple sizing guide for the treatment measures and the guideline would recommend that users adopt a modelling approach with local rainfall data should they want to refine the sizing further, which would generally lead to a reduction in required area.

The performance curves used as a checking guide in subsequent chapters in this document have been based on adopting a factoring the performance of the reference station factored by 1.1 to reflect this conservative simple approach to sizing stormwater treatment systems.

3.4 Model for Urban Stormwater Improvement Conceptualisation

The Model for Urban Stormwater Improvement Conceptualisation (MUSIC) developed by the Cooperative Research Centre for Catchment Hydrology in Australia has been adapted for use in Singapore climatic conditions to aid designers in modelling the performance of stormwater quality treatment measures. Rainfall files for the 22 rainfall stations (see Figure) have been formatted for MUSIC application.

MUSIC enables urban catchment managers to (a) determine the likely water quality emanating from specific catchments, (b) predict the performance of specific stormwater treatment measures in protecting receiving water quality, (c) design an integrated stormwater management plan for a catchment, (d) evaluate the success of a treatment node or treatment train against a range of water quality standards, and (e) analyse the life cycle costs of a treatment node or treatment train.

MUSIC was developed in a modular form to allow the incorporation of refinements and additions as a result of further research by the CRCCH, eWater CRC and others.

3.4.1 Model spatial and temporal resolution

MUSIC is designed to operate at a range of temporal and spatial scales, suitable for catchment areas up to 100 km^2. The modelling approach is based on continuous simulation, operating at time steps from six minutes to 24 hours, to match the spatial scale of the catchment.

The accuracy of MUSIC's simulation of treatment performance depends on selection of an appropriate time step, matched to the catchment area and detention time of the specified treatment measure.

3.4.2 Data requirements

3.4.2.1 Climate data

MUSIC simulations are based on a 'meteorological template' which can be of any duration, and be based on a time step ranging from six minutes to 24 hours. Climate templates can be created from local rainfall and evapotranspiration files, which can be supplied by the Bureau of Meteorology. MUSIC comes pre-loaded with rainfall and evapotranspiration files for a range of Australian locations.

3.4.2.2 Source node properties

Creating a source node from meteorological data requires the user to specify:

- Catchment area and impervious area
- Soil properties (where possible)
- Event mean and dry weather pollutant concentrations (default values are provided from the worldwide literature (Duncan, 2006).

Alternatively, the entire source node simulation may be bypassed by importing a file of flow and concentration data appropriate to the site.

3.4.2.3 Treatment node properties

MUSIC users specify the design properties of a given treatment node, such as the inlet, storage and outlet properties. Advanced parameters can also be accessed, to modify the default modelling parameters (such as the k and C* values in the Universal Stormwater Treatment Model). MUSIC also allows users to create a 'Generic Treatment Node', to simulate the performance of a stormwater treatment measure (structural or non-structural) that is not simulated by the USTM. The Generic Treatment Node provides a graphical transfer-function editor.

3.4.2.4 Drainage link properties

The links connecting source, treatment, and junction nodes may represent pipes, open channels, or natural watercourses. To enable more accurate simulation, the routing properties (using the Muskingum-Cunge routing method) of each link may be specified by the user.

3.4.3 Recommended uses for the model

MUSIC should be viewed as a conceptual design tool, not a detailed design tool; it does not contain the algorithms necessary for detailed sizing of structural stormwater quantity and/or quality facilities. MUSIC does not incorporate all aspects of stormwater management that decision-makers must consider. Hydraulic analysis for stormwater drainage, indicators of ecosystem health, and the integration of urban stormwater management facilities into the urban landscape are currently omitted from the model. Many of these are the subject of further research in the CRCCH and eWater CRC, and will be incorporated into future versions of MUSIC.

3.4.4 Links to further information/user groups etc.

MUSIC is part of the CRC Catchment Hydrology and eWater CRC Catchment Modelling Toolkit. Further details about MUSIC, and the Toolkit, can be found at:

www.toolkit.net.au/music

3.5 References

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Sedimentation Basins 4

4

Chapter 4 Sedimentation Basins

4.1 Introduction

Reducing sediment loads is an important way to improve stormwater quality. Sedimentation basins can form an integral component of a stormwater treatment train and are specifically employed to remove (by settling) coarse to medium-sized sediments from the water column. Sedimentation basins can take various forms and can be used as permanent systems integrated into an urban design or temporary measures to control sediment discharge during construction.

Sedimentation basins promote settling of particles by providing temporary detention and reducing flow velocities. They are designed to capture 70 to 90 percent of sediment above a target size (typically 125µm), whilst ensuring that the clean out frequency is consistent with the maintenance regime (typically annually to once every 3 years).

The desired capture efficiency and clean out frequency are influenced by design elements including the location of the inlet and outlet structures, the size of the settling pond, and the high flow structures. The settling pond consists of two sections: the permanent pool sediment settling zone and the sediment storage zone. Access for maintenance (for example, sediment dewatering) must also be considered. The layout and design considerations of these key design elements are shown schematically in Figure 4.1and Figure 4.2.

This chapter describes the design, construction and maintenance of permanent sedimentation basins designed as part of a treatment train.

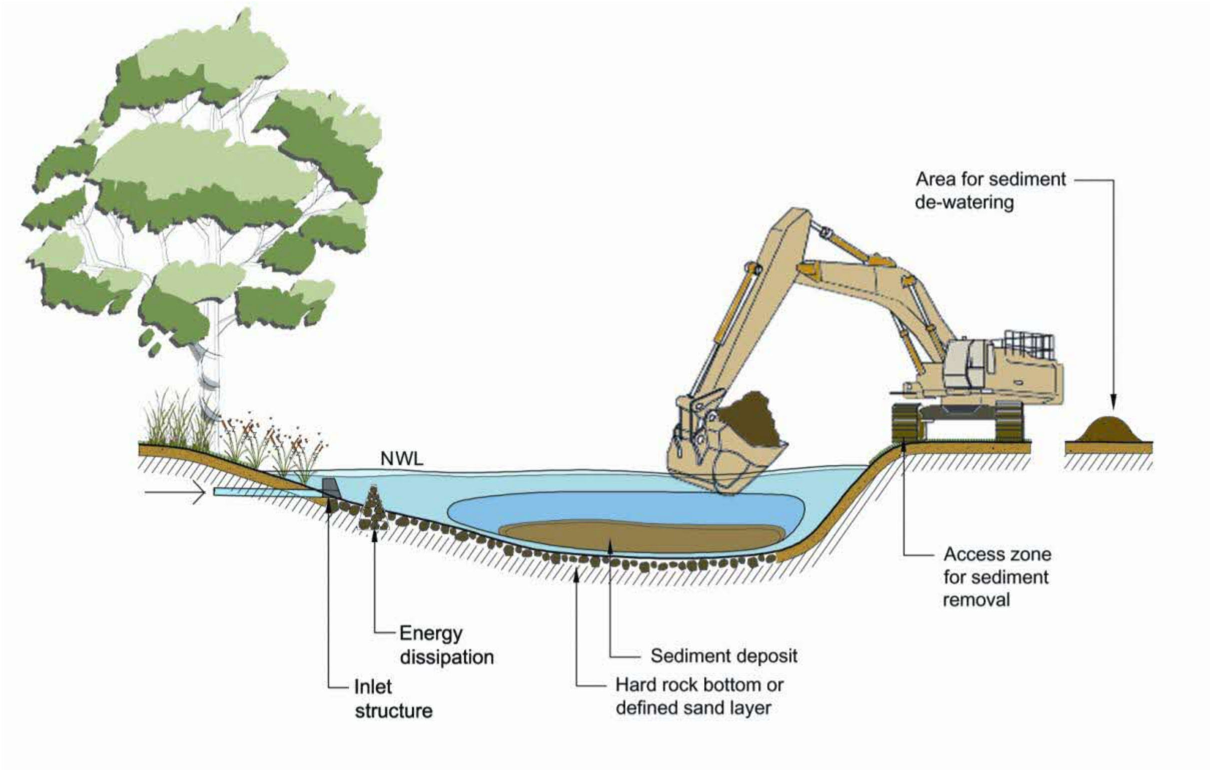


Figure 4.1 Layout of a typical sedimentation basin

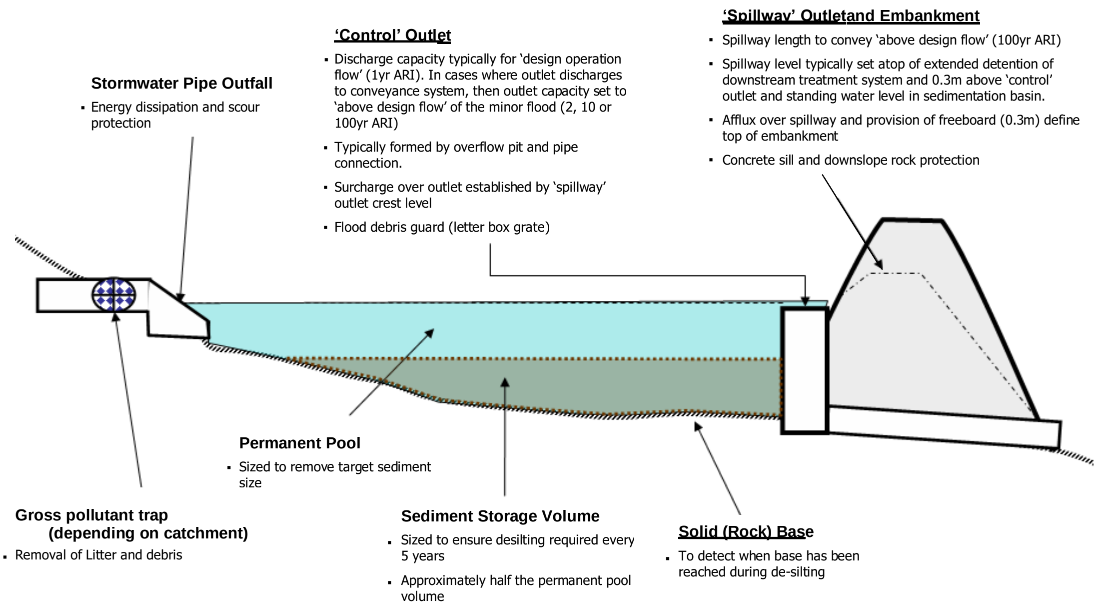


Figure 4.2 Design considerations for sedimentation basins

4.2 Design Considerations

4.2.1 Role of sedimentation basins in the stormwater treatment train

Sedimentation basins have two key roles when designed as part of a stormwater treatment train. Its primary function is to capture coarse to medium sized sediment as pre-treatment to waters entering a downstream treatment system (e.g. macrophyte zone of a constructed wetland or a bioretention basin) configured for removal of finer particulates and soluble pollutants.

The pre-treatment ensures that downstream treatment systems are not smothered by coarse sediment which may hamper their effectiveness to target finer particulates, nutrients and other pollutants.

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

4.2.2 Sizing a sedimentation basin

The size of a sedimentation basin is typically calculated to match the settling velocity of a target sediment size for a given design flow. In urban stormwater management, this design flow typically corresponds to the 1 year ARI peak flow. As a pretreatment facility, it is recommended that particles of 125µm or larger be the selected target sediment size.

This recommendation is based on the following considerations:-

- Analysis of typical catchment sediment loads suggest that between 50 – 80 % of suspended solids conveyed in urban stormwater are 125µm or larger. Almost all sediment bed loads are larger than this target sediment size. By targeting this particle size for pre-treatment, containment of a significant proportion of sediment inflow is within the sedimentation basin.

- Analysis of the characteristics of particulate nutrients and metals indicates that coarse to medium sized sediments (i.e. > 125µm) have low concentrations of attached pollutants (e.g. nutrients, heavy metals) when compared to finer sediment and colloidal particles. Basins sized to target coarse to medium sized sediment are therefore expected to contain sediment that has low levels of contamination and are unlikely to require special handling and disposal.

- Removal of particles smaller than 125µm is best undertaken by treatment measures other than sedimentation basins (e.g. constructed wetlands and bioretention systems).

Where the sedimentation basin forms part of a treatment train (i.e. inlet zone of a constructed wetland) and when available space is constrained, it is important to ensure that the size of the sedimentation basin is not reduced. If the sedimentation basin is not sized adequately, larger sediments will not be trapped effectively and the downstream treatment system is at risk of becoming smothered.

Conversely, a sedimentation basin should not be grossly oversized, as smaller particles may be allowed to settle (due to longer residence times) and special clean-out and disposal procedures would be required. Experiences have also shown that grossly oversized sedimentation systems may also be subject to poor water quality outcomes including the occurrence of algal blooms.

4.2.3 Sediment Storage

A further consideration in the design of a sedimentation basin is the provision of adequate storage for settled sediment to prevent the need for frequent desilting. Typically, a basin is designed to have frequency of desilting (clean-out frequency) ranging from annually to once every five years (generally triggered when sediment accumulates to half the basin pool depth).

The volume of accumulated sediment can be estimated from regular monitoring of sediment levels with a measuring post and reference against the top water level. Review of global literature suggests that a developing catchment can typically be expected to discharge between $50m^3$ and $200m^3$ of sediment per hectare each year. In a built-up catchment, the annual sediment export is generally one to two orders of magnitude lower.

No data is available to help estimate the expected sediment load generated from Singapore urban catchments and data from other sources have been used in the interim to estimate the required sediment storage. In Australia, an expected mean annual rate of $1.60m^3$/ha has been suggested (Engineers Australia, 2006).

Sediment loading rate for Singapore catchments may be higher than typically observed in Australian catchments, owing to higher intensity and magnitude of rainfall. Preliminary modeling suggests a sediment loading rate as high as 3 m^3/ha/year may be more appropriate for Singapore conditions.

4.2.4 Outlet Design

An outlet structure of a sedimentation basin can be configured in many ways and is generally dependent on its intended operation. In most cases, the outlet design of a sedimentation basin will consist of a 'control' outlet structure and a 'spillway' outlet structure:

- The 'control' outlet can either be an overflow pit with pipe connection or a weir. This structure conveys flows up to the 'design operation flow' (Section 4.3.1) to the downstream treatment system(s).
- The 'spillway' outlet weir structure configured to ensure that flows above the 'design operation flow' (Section 4.3.1) are discharged to a channel or pipe system that by-passes the downstream treatment system(s). The 'spillway' bypass weir level is set above the 'control' outlet structure and typically at the top of the extended detention depth of the downstream treatment system.

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

4.2.5 Landscape Design

Sedimentation basins are often located within open space zones areas and can be landscaped to create a focal point for passive recreation. The suitability of sedimentation basins as water features will vary depending on catchment type. It should be borne in mind that, unlike ponds located further down the stormwater treatment train (see Chapter 10); sedimentation basins bear the first brunt of urban stormwater pollution. Thus accumulation of gross pollutants, hydrocarbon (particularly during dry weather flow conditions) and generally poor water quality can be expected. The introduction of gross pollutant traps may be required if these basins have important water feature functions.

Landscape design treatments for sedimentation basins generally focus on dense and tall littoral vegetation planting to shield unsightly sections of the basins, restrict non-designated access to the open water zone, and therefore increase public safety, but can also include designated pathways, viewing platforms (preferably located at the downstream end of the basin), and information signs. Landscape design must also consider access to the sedimentation basin for maintenance (e.g. excavator).

4.2.6 Vegetation Specification

The role of vegetation in sedimentation basin design is to provide scour and erosion protection to the basin batters and littoral zone (i.e. the area around the shallow margin of the sedimentation basin). Terrestrial planting may also be recommended to screen areas and provide a barrier to steeper batters.

Plant species should be selected based on the water level regime, soil types of the region, and the life histories, physiological and structural characteristics, natural distribution, and community groups of the plants. A list of suggested plant species suitable for sedimentation basins will be developed in consultation with National Parks Board of Singapore and form a separate guideline.

Only the waters edge and batters of sedimentation basins should be planted and care needs to be taken in species selection to ensure vegetative growth will not spread to cover the deeper water zones. Similarly, floating or submerged macrophytes should be avoided.

4.2.7 Maintenance

Sedimentation basins are designed with a sediment storage capacity to ensure sediment removal frequency is acceptable (i.e. typically between once per year to once every five years, refer to Section 4.2.3).

Maintenance is focus on ensuring inlet erosion protection is operating as designed, monitoring sediment accumulation and ensuring that the outlet is not blocked with debris. Cleaning of the sedimentation basin is typically triggered when sediment accumulates to half the basin depth, determined from regular monitoring of sediment depth with a measuring post during maintenance visits.

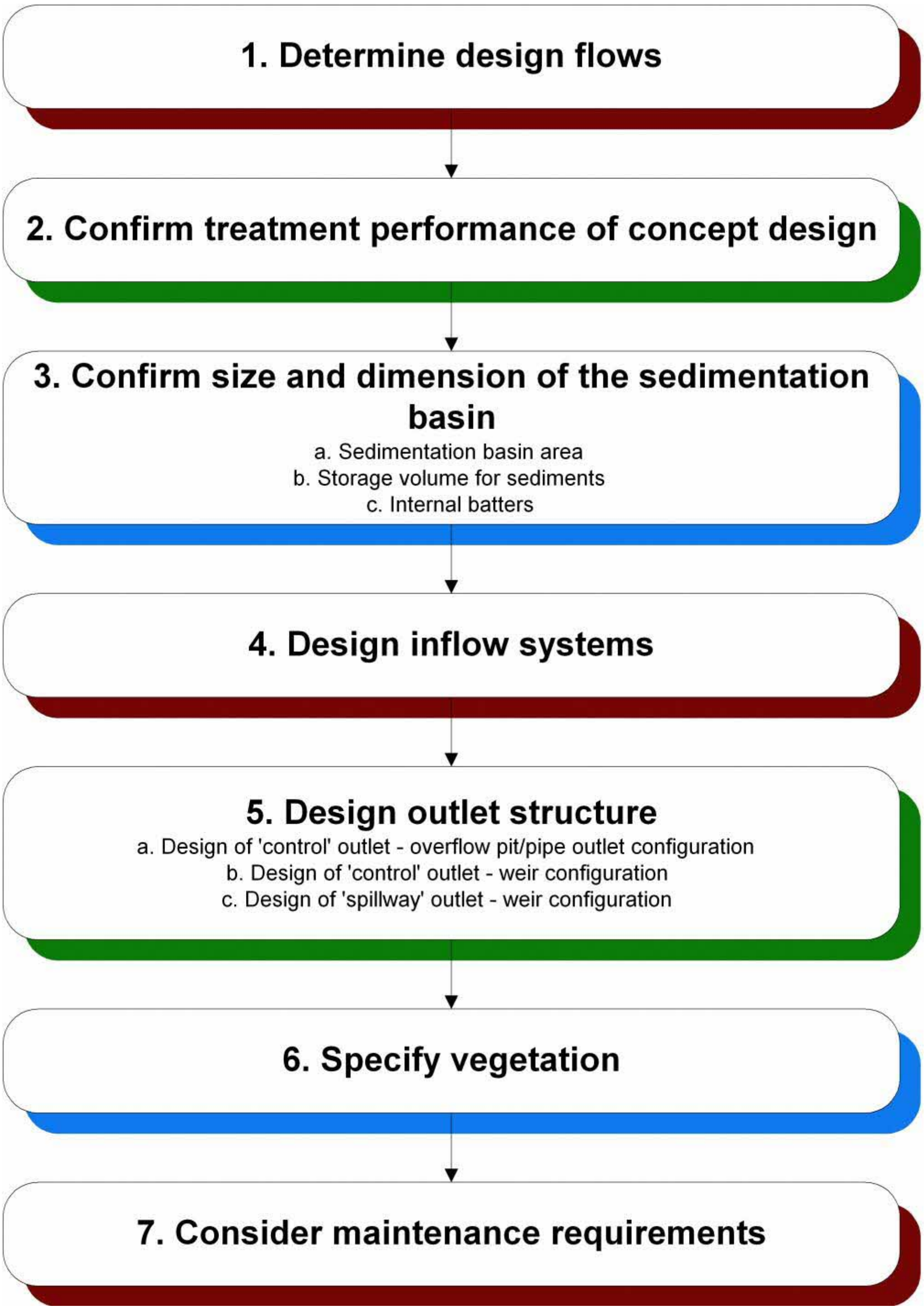
Accessibility for maintenance is an important design consideration. If an excavator is able to reach all parts of the basin from the top of the batter then an access ramp may not be required; however, an access track around the perimeter of the basin will be required and will need to be accommodate in the overall landscape design. If sediment collection requires earthmoving equipment to enter the basin, a stable ramp will be required into the base of the sedimentation basin (maximum slope 1:10).

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

Provision to drain the sedimentation basin of water for maintenance must be considered (which may involve the use of a pump which is either permanently installed on site or a portable unit). Approvals must be obtained to discharge flows downstream receiving waters or to sewer.

4.3 Design Process

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



4.3.1 Step 1: Determine Design Flows

A range of hydrologic methods can be applied to estimate design flows, ranging from detailed catchment runoff routing models to the simple Rational Method use for typical drainage design in Singapore. For catchment areas that are relatively small (< 50 ha), the Rational Method design procedure is considered adequate. For Sedimentation Basins with large catchments (> 50 ha), a runoff routing model should be used.

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

- ‘***Design Operation Flow***’ for sizing the basin area and to size a ‘control’ outlet structure when discharging directly into a treatment system (e.g. wetland or bioretention system). The 1 year ARI peak discharge is recommended as the Design Operation Flow.
- ‘***Above Design Flow’*** for design of the ‘spillway’ outlet structure to allow for bypass of high flows around a downstream treatment system. This is defined by either the:
  - Minor design flow (2 to 10 year ARI) corresponding to the discharge capacity of downstream drainage infrastructure. The required design event for the minor design flow in Singapore is the 5 year ARI peak discharge (Code of Practice on Surface Water Drainage). The adoption of the 5 year ARI peak discharge is appropriate for situations where only the minor drainage system is directed to the sedimentation basin.
  - Major flood flow (50 to 100 year ARI) is conveyed by major canal and waterways and/or designated overland flow paths or floodways within the urban area that is engaged when the capacity of the local drainage infrastructure is exceeded. In Singapore, this is either the 50 year or 100 year ARI peak discharge depending on the catchment land use. This design flow should be adopted as the Above Design Flow for the sedimentation basin where both the minor and major drainage systems discharge into the basin.

4.3.2 Step 2: Confirm Treatment Performance of Concept Design

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

Figure 4.3 shows the relationship between a required basin area and design discharge for 125μm sediment capture efficiencies of 70%, 80% and 90%. The sizing curves are based on a typical shape and configuration, having a hydraulic efficiency (λ) of 0.5 (see Section 4.3.3.1; Figure 4.4).

An upper and lower limit is given for the three target capture efficiencies, set by the absence and presence of a permanent pool, respectively. 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) and defines the lower limit of required basin area. The permanent pool has a typical depth of two metres (for ease of maintenance).

The performance of a typical sedimentation basin design can be expected to fall within the shaded areas shown in Figure 4.3. As the design charts relate the size of a required sedimentation basin to a design flow, they are applicable in all regions within Singapore and can be used to verify the selected size of a proposed sedimentation basin anywhere in Singapore.

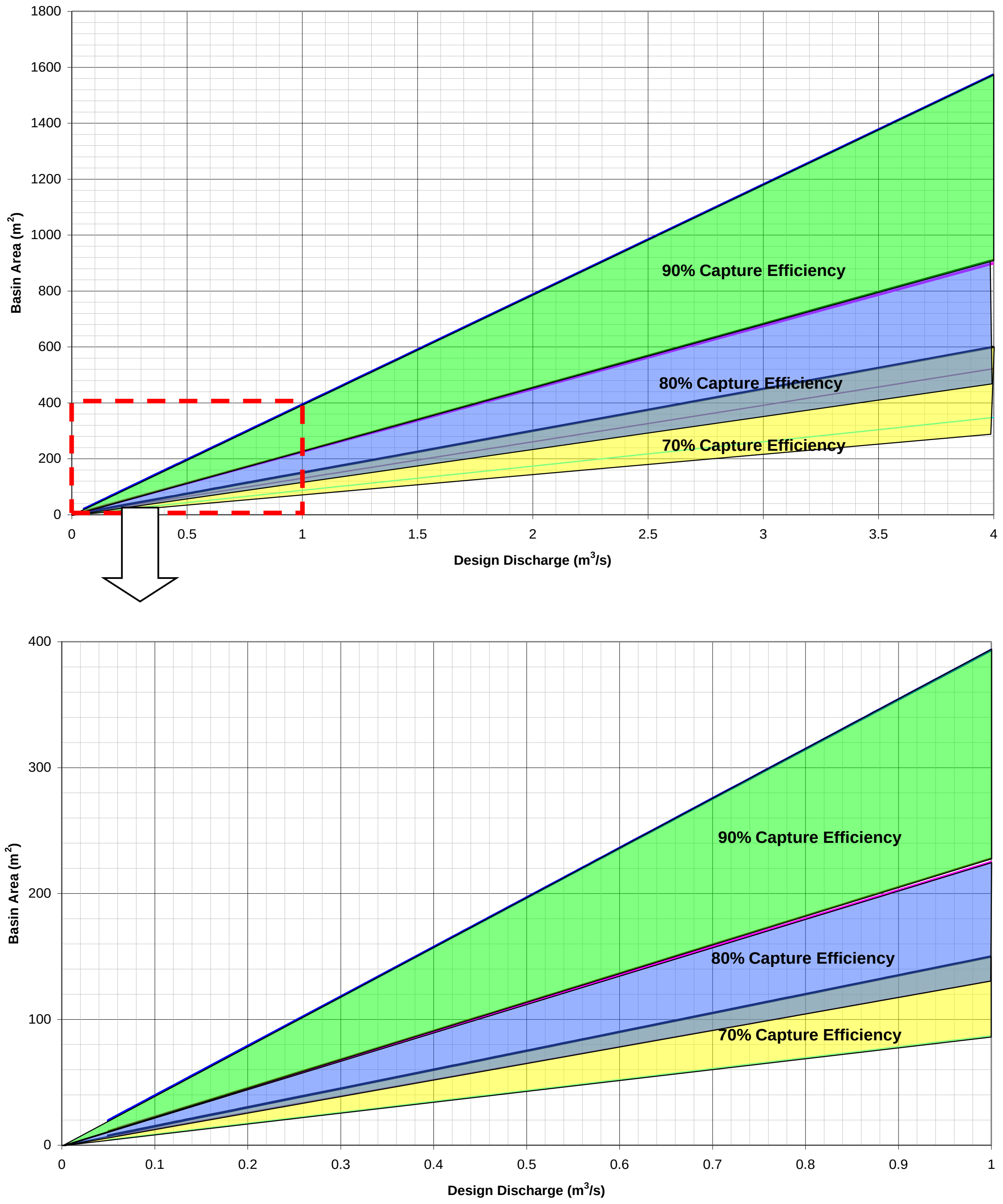


Figure 4.3 Sedimentation Basin Area Vs Design Discharges for varying capture efficiencies of 125µm sediment

4.3.3 Step 3: Confirm Size and Dimensions of the Sedimentation Basin

4.3.3.1 Sedimentation Basin Configuration

The configuration of a sedimentation basin, defined by its shape and locations of inlet and outlet structures, has a large impact on the effectiveness of the basin to retain sediment. The effectiveness of the basin at retaining sediment is described by the hydraulic efficiency (λ).

The hydraulic efficiency is greatly influenced by the length to width ratio of the basin, the relative position of the inlet and the outlet, and the inclusion and placement of any baffles, islands or flow spreaders. Hydraulic efficiency has a range from 0 to 1, with 1 representing the most efficient configuration for sedimentation. Basins should not be designed to have a hydraulic efficiency less than 0.5.

Guidance on estimating hydraulic efficiency is given in Figure 4.4. The shape designed as 'o' in diagrams O and P represent islands in the waterbody and the double line in diagram Q represents a weir structure to distribute flows evenly (Persson *et al.* 1999).

Consideration of maintenance access to a basin is also required when developing the shape, as this can impact the allowable width (if access is from the banks) or the shape if access ramps into a basin are required.

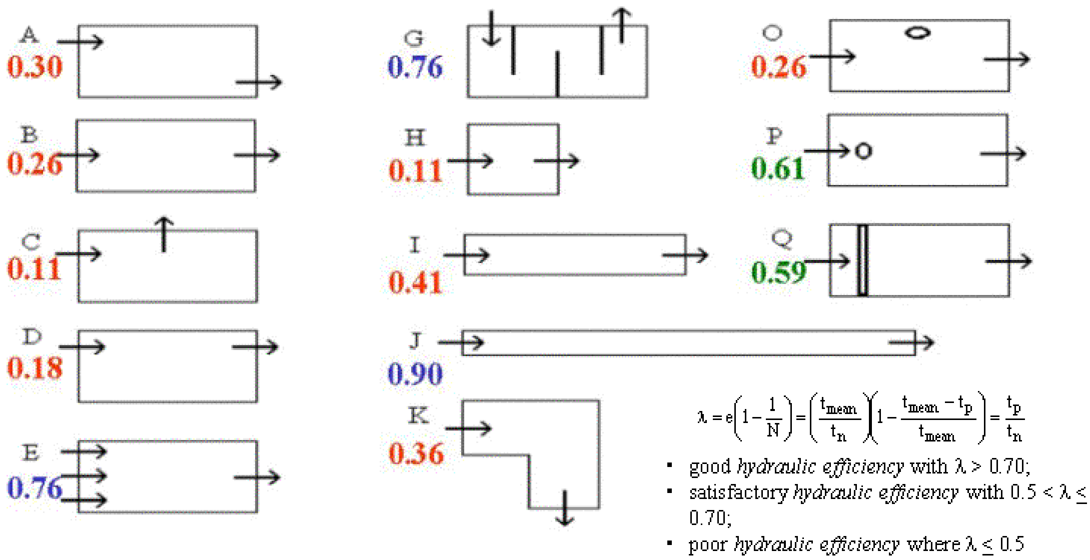


Figure 4.4 Hydraulic Efficiency, λ

4.3.3.2 Internal Batters

Batter slopes above and immediately below the water line of a basin should be configured with consideration of public safety and landscape integration. Both hard and soft edge treatments can be applied to compliment the landscape of a surrounding area. Soft edge treatments involve using gentle slopes to the waters edge (e.g. 1:8 to 1:10), extending below the water line for a distance of between 2 to 3 m before batter slopes steepen into deeper areas. Hard edge treatments typically have a large vertical drop from the waters edge to the water line. Such systems may require a handrail for public safety. In both hard and soft edge treatments, it is recommended to line the bottom of the basin with rock to prevent vegetation (particularly weed) growth and to guide extraction depths during sediment removal.

The safety requirements for individual basins will vary from site to site, and it is recommended that developers engage an independent safety audit of each design. At present no guidelines exist in Singapore for the design of Sedimentation Basins. In their absence it is recommended that the following be adopted:

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

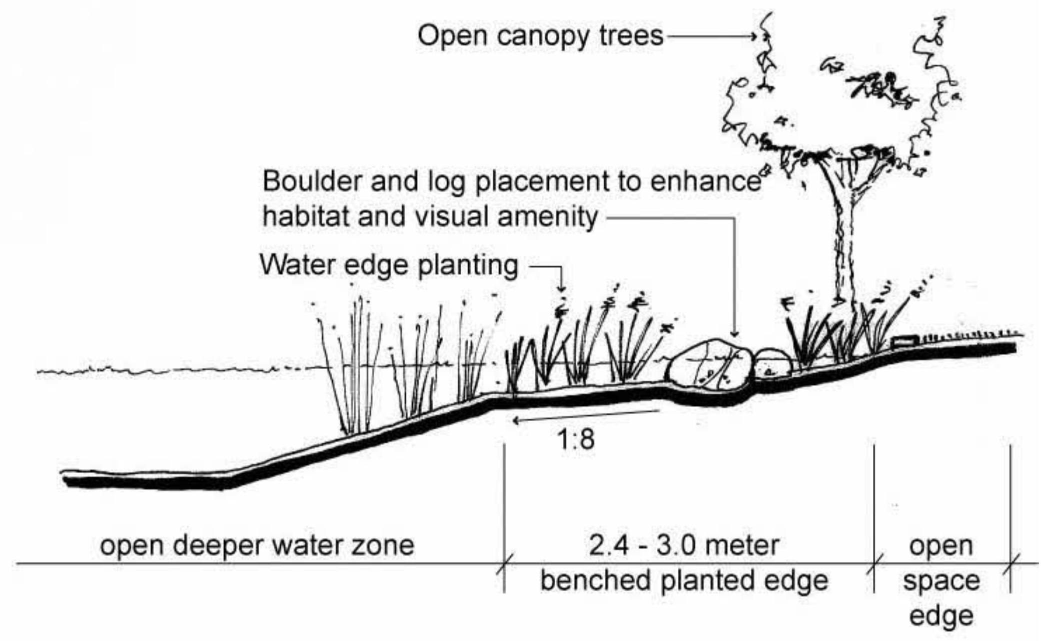


Figure 4.5 Design considerations for soft edge treatment for open waterbodies (Source: GBLA 2004)

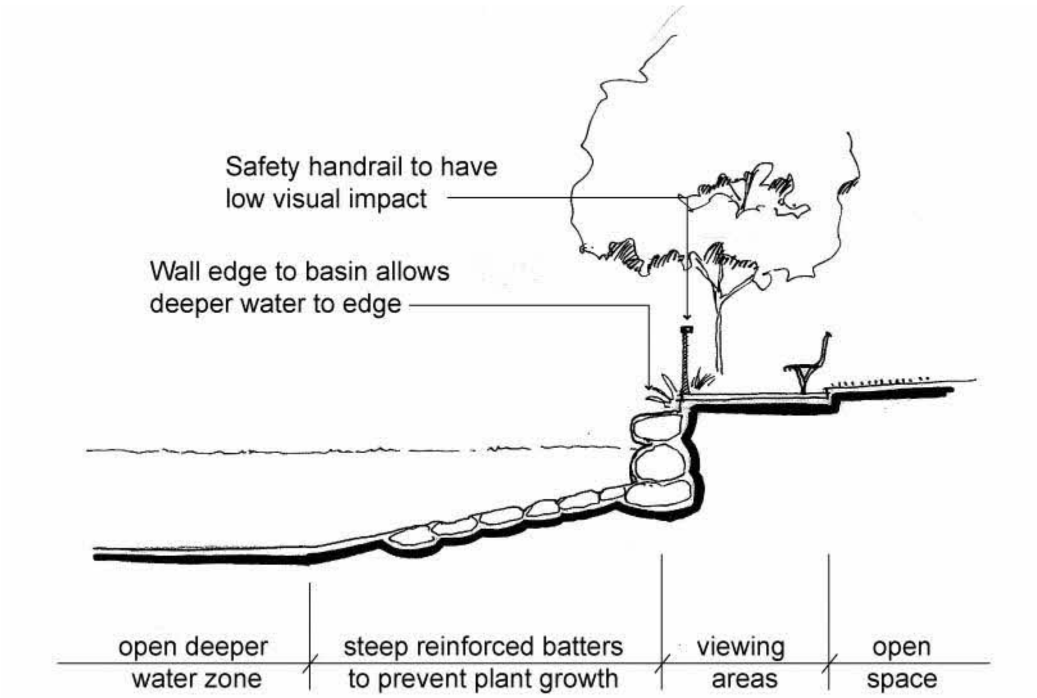


Figure 4.6 Design considerations for hard edge treatment for open waterbodies (Source: GBLA 2004)

4.3.3.3 Sedimentation Basin Area

The required area (A) of a sedimentation basin can be defined through the use of the (modified) Fair and Geyer (1954) expression of the sedimentation equation, i.e.:

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

Where

R = fraction of target sediment removed

v_s = settling velocity of target sediment

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

n = turbulence or short-circuiting parameter

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

d_p = depth (m) of the permanent pool

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

Good practice in the design of sedimentation basins is to include a permanent pool to reduce flow velocities and provide storage of settled sediment. The presence of a permanent pool reduces flow velocities in the sedimentation basin and thus increases detention times. With the outlet structure being located some distance above the bed of a sedimentation basin, it is also not necessary for sediment particles to settle all the way to the bed of the basin to be effectively retained. It is envisaged that sediments need only settle to an effective depth (d*) which is less than the depth to the bed of the sedimentation basin. This depth is considered to be approximately 1.0 m below the permanent pool level.

The turbulence parameter, *n*, is related to hydraulic efficiency (λ) described in Section 4.3.3.1. A value of *n* is estimated using the following relationship:

$$\lambda = 1 - \frac{1}{n}$$

$$\therefore n = \frac{1}{1 - \lambda}$$ Equation 4.2

The concept design stage will generally guide the selection of the fraction of target sediment removed (R) and permanent pool depth (d_p) depending on water quality objectives and the nature of local catchment geology. The selection of the target sediment size will lead to the determination of the theoretical settling velocity of the target particle size for use in Equation 4.1 to compute removal efficiency for a given size basin.

Table 4.1 lists the typical settling velocities (v_s) of sediments under 'ideal conditions' (velocity in standing water).

Table 4.1: Settling Velocities (v_s) under Ideal Conditions

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

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

4.3.3.4 Storage Volume for Sediments

A further consideration in the design of a sedimentation basin is the provision of adequate storage for settled sediment to prevent the need for frequent desilting. Basin desilting is triggered when accumulated sediment reaches half of the permanent pool volume. To ensure this storage zone is appropriate the following must be met:

The sedimentation basin storage volume (V_s) is defined as the storage available in the bottom half of the sedimentation basin permanent pool depth. The sedimentation basin storage volume can be determined by applying the following equation:

$$V = \frac{d_p}{2} \times \frac{(A_b + A_T)}{2} \qquad \text{Equation 4.3}$$

Where

A_b = Area of the basin at the base

A_T = Area of the basin at half the permanent pool depth

d_p = Depth (m) of the permanent pool

The basin areas are determined based on the surface dimensions and the batter slopes.

The volume of accumulated sediments over period before the basin is desilted (V_s) is established by gaining an understanding of the sediment loads entering the sedimentation basin and applying the fraction of target sediment removed (R):

$$V_s = A_c \cdot R \cdot L_o \cdot F_c \qquad \text{Equation 4.4}$$

Where

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

A_c = contributing catchment area (ha)

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

L_o = sediment loading rate (m^3/ha/year) – preliminary MUSIC modelling suggests a sediment loading rate of 3 m^3/ha/year may be appropriate for Singapore conditions.

F_c = desired cleanout frequency (years)

4.3.4 Step 4: Design Inflow Systems

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

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

If conceptual design of the stormwater system identified the need to remove anthropogenic litter (i.e. industrial or commercial situations) then some form of gross pollutant trap (GPT) may be required as part of an inlet structure. The provision of a GPT will depend on catchment activities as well as any upstream measures in place. There are a number of proprietary products available for removing gross pollutants. While there are no suitable references related to gross pollutant traps in Singapore, the reader is referred to Chapter 7 of Australian Runoff Quality (Engineers Australia 2006). The storage capacity of gross pollutant traps should be sized to ensure that maintenance (cleanout) frequency is not greater than once every 3 months.

4.3.5 Step 5: Design Outlet Systems

As outlined in Section 4.2.4, the outlet of a sedimentation basin will consist of a ‘control’ outlet structure and a ‘spillway’ outlet structure:

4.3.5.1 Design of ‘Control’ Outlet - Overflow Pit and Pipe Outlet Configuration

For a sedimentation basin that operates as a pre-treatment within a treatment train configuration, the ‘control’ outlet structure discharging to the downstream treatment system (e.g. constructed wetland) is an overflow pit and pipe with the following design criteria:

- Ensure that the crest of the overflow pit is set at the permanent pool level of the sedimentation basin.
- The overflow pit is sized to convey the design operational flow (e.g. the 1 year ARI peak discharge from the catchment). The dimension of the outlet pit is determined by considering two flow conditions: weir and orifice flow as expressed in Equation 4.5 and Equation 4.6.
- Provide protection against blockage of the overflow pit by flood debris by installation of debris screening (see Figure 4.7).

Figure 4.7 Examples of debris screens

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

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

$$P = \frac{Q_{des}}{B \cdot C_w \cdot h^{3/2}} \qquad \text{Equation 4.5}$$

Where

P = Perimeter of the outlet pit (m)

B = Blockage factor (0.5)

h = Depth of water above the crest of the outlet pit (m)

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

C_W = Weir coefficient (1.7)

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

$$A_o = \frac{Q_{des}}{B \cdot C_d \cdot \sqrt{2 \cdot g \cdot h}} \qquad \text{Equation 4.6}$$

Where

C_d = Orifice discharge coefficient (0.6)

B = Blockage factor (0.5)

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

A_o = Orifice area (m^2)

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

The pipe that connects the sedimentation basin to the downstream treatment system (e.g. macrophyte zone of a constructed wetland or bioretention system) should have sufficient capacity to convey the design operational flow (i.e. the 1 year ARI peak flow) when downstream water level is at the permanent pool level. This ensures the majority of flows have the opportunity to enter the downstream treatment system before the bypass system is engaged. As downstream water level increases due to the filling of the extended detention of the downstream treatment system, the capacity of the connecting pipe may reduce and ultimately triggering a by-pass from the sedimentation basin (see Chapter 9 Constructed Wetlands).

An energy dissipater is usually required at the end of the pipes to reduce velocities and distribute flows into the downstream treatment system.

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

$$h = \frac{k \cdot V^2}{2 \cdot g} \qquad \text{Equation 4.7}$$

Where:

h = head level driving flow through the pipe (defined as the ‘spillway’ outlet level minus the normal water level in the downstream treatment system)

k = head loss coefficient – assume k = 2, as a conservative estimate of the sum of entry and exit loss coefficients (K_{in} + K_{out})

V = pipe velocity (m/s)

g = gravity (9.81 m/s^2)

The area of pipe required to convey the ‘design operation flow’ (1 year ARI) is then calculated by dividing the above ‘design operation flow’ by the velocity. Alternatively, if the pipe outlet is not fully submerged, the orifice equation should be used (Equation 4.6) to estimate the size of the connection pipe.

4.3.5.2 Design of ‘Control’ Outlet – Weir Configuration

If a weir outlet structure is to be used instead of an overflow pit and pipe configuration, the required length of the weir for ‘control’ outlet operation can be computed using the weir flow equation (Equation 4.5) and the ‘design operation flow’ (Section 4.3.1). Depending on the width of the weir, a weir blockage factor may still be required in which case a factor of 0.5 is recommended.

4.3.5.3 Design of ‘Spillway’ Outlet – Weir Configuration

For operation under above-design conditions, a ‘spillway’ outlet weir will be required to safely convey above-design flows.

For sedimentation basins serving as pre-treatment to downstream systems, this spillway will form part of the high flow bypass system, which protects the downstream treatment system from scouring during ‘above design’ storm flows. The ‘spillway’ outlet weir level should ideally be set at the top of the extended detention level of the downstream treatment system.

The length of the ‘spillway’ outlet weir is to be sized to safely pass the maximum flow discharged into the downstream treatment system (as defined by the ‘above design flow’ in Section 4.3.1). The water level above the crest of the bypass weir plus freeboard provision (typically 0.3 m) sets the elevation of the embankment crest of the sedimentation basin.

The required length of the ‘spillway’ outlet weir can be computed using the weir flow equation (Equation 4.5) and the ‘above design flow’ (Section 4.3.1). No provision for blockage is necessary (i.e. blockage factor of 1.0). The 'spillway' outlet weir should be designed using standard methods to avoid scour and erosion. Typically, a concrete sill is required with rock protection on the downslope sides of the sill. Figure 4.8 shows typical spillway structures of sedimentation basins providing a means of bypassing above design flows around downstream constructed wetlands.

Figure 4.8 Example overflow spillway structures of inlet zones (sedimentation basins) of constructed wetlands.

4.3.6 Step 6: Specify Vegetation

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 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 sedimentation basin littoral zones will be developed for Singapore in consultation with National Parks Board of Singapore.

4.3.7 Step 7: Maintenance Plan and Schedule

Consider how maintenance is to be performed on the Sedimentation Basin (e.g. how and where is access available, where is litter likely to collect etc.). A specific maintenance plan and schedule should be developed for the basin, either as part of a maintenance plan for the whole treatment train, or for each individual asset. Guidance on maintenance plans is provided in Section 4.7.

4.3.8 Design Calculation Summary

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

Sedimentation Basin	CALCULATION CHECKSHEET	
CALCULATION TASK	OUTCOME	CHECK
Catchment characteristics		
- Land Uses		
Residential	Ha	
Commercial	Ha	
Roads	Ha	
- Fraction Impervious		
Residential	-	
Commercial	-	
Roads	-	
Weighted average	-	☐
Conceptual Design		
Basin Area	m^2	
Notional permanent pool depth	m	
Permanent pool level of sedimentation basin	m	
Basin extended detention	m	
Overflow level	m	
Identify design criteria		
Design operation flow	year	
Above design flow	year	☐
1. 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 design operation flow	mm/hr	
Design Rainfall Intensity for above design flow	mm/hr	☐
Design runoff coefficient		
Design operation flow		
Above design flow		
(refer to the Singapore Code of Practice on Surface Water Drainage(2000))		☐
Peak design flows		
Design operation flow	m^3/s	
Above design flow	m^3/s	☐
2. Confirm treatment performance and concept design		
Capture efficincy of sedimentation basin	**%**	☐
3. Confirm size and dimensions of sedimentation basin		
- Inlet design		
Area of sedimentation basin	m^2	
Aspect Ratio	L:W (1)	
Hydraulic Efficiency		
Depth of permanent pool	m	☐
- Internal batters		
Cross Section Batter Slope (below permanent pool depth)	V:H (1)	☐
- Sediment Storage Volume		
Sediment storage volume, V_s	m^3	
Volume of accumulated sediment over 5years ($V_{s,5yr}$)	m^3	
$V_s > V_{s,5year}$	-	
Sediment clean-out frequency, given V_s	years	☐
4. Design inflow systems		
Scour protection and/or energy dissipation provided		☐
5. Design outlet structures		
- Overflow pit		
Pit dimension	L x B	
Overflow crest level	m	
Provision of debris trap	-	☐
- Connection Pipe		
Connection pipe dimension	mm dia	
Connection pipe invert level	m	☐
- Control outlet weir (Spillway)		
Weir crest level	m	
Weir length	m	
Afflux	m	
Freeboard to top of embankment	m	☐
6. Vegetation Specification		☐

4.4 Checking tools

The following sections provide a number of checking aids for designers and referral authorities.

Checklists have been provided for:

- Design assessments
- Construction (during and post)
- Maintenance and inspections

4.5 Design assessment checklist

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

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

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

Checklist 1: Sedimentation basin design checklist

Sedimentation Basin Design Assessment Checklist				
Basin Location:				
Hydraulics:	Design operational flow (m^3/s):	Above design flow (m^3/s):		
Area:	Catchment Area (ha):	Basin Area (ha):		
TREATMENT			**Y**	**N**
Treatment performance verified from sizing 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 located upstream of treatment system (i.e. macrophyte zone of wetland)?				
Configuration of basin (aspect, depth and flows) allows settling of particles >125 µm?				
Basin capacity sufficient for desilting period (i.e. >= twice sedimentation accumulation over clean out frequency?)				
Maintenance access allowed for into base of Sedimentation Basin?				
Public access to basin prevented through dense vegetation or other means?				
Gross pollutant protection measures provided on inlet structures where required?				
Freeboard provided to top of embankment?				
Public safety design considerations included in design and safety audit of publicly accessible areas undertaken?				
Overall shape, form, edge treatment and planting integrate well (visually) with host landscape?				
HYDRAULIC STRUCTURES			**Y**	**N**
'Control' outlet structure required?				
'Control' outlet structure sized to convey the design operation flow?				
Designed to prevent clogging of outlet structures (i.e. provision of appropriate grate structures)?				
'Spillway' outlet control (weir) sufficient to convey 'above design flow'?				
'Spillway' outlet has sufficient scour protection?				
Visual impact of outlet structures has been considered?				
COMMENTS				

4.6 Construction Advice

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

4.6.1 Building phase damage

It is important to have protection from upstream flows during construction of a Sedimentation Basin. A mechanism to divert flows around a construction site, protection from litter and debris is required.

To overcome the challenges associated within delivering sedimentation basins a staged approach to construction and establishment should be adopted (Leinster, 2006):

Stage 1: Functional Installation. The functional elements of the sedimentation basin are constructed as part of civil works. The basin is allowed to form part of the sediment and erosion control strategy.

Stage 2: Sediment and Erosion Control. During the Building Phase the sedimentation basin will form part of the sediment and erosion control strategy to protect downstream aquatic ecosystems.

Stage 3: Operational Establishment. At the completion of the Building Phase, the sedimentation basins can be desilted (to establish the design bathymetry) and landscaped.

Protection from vehicular impact during construction should be provided by traffic controlling devices (for example, bollards).

4.6.2 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.

4.6.3 Maintenance access

An important component of a Sedimentation 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 is required regardless. If sediment is collected using earthmoving equipment, then a stable ramp will be required into the base of the inlet zone (maximum slope 1:10).

4.6.4 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.

4.6.5 Dewatering removed sediments

An area should be constructed that allows for dewatering of removed sediments from a Sedimentation 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.

4.6.6 Inlet checks

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

4.6.7 Timing for Planting

Timing of vegetation planting is dependent on a suitable time of year (and potential irrigation requirements) as well as timing in relation to the phases of development. Consideration should be made for:

- Growth period (relative to planting). It is recommended that plants be planted within their growth period to allow the plants to go through a growth period soon after planting
- Establishment of root zone prior to wet season

Further advice from the National Parks Board of Singapore should be sought when considering the suitable timing for planting.

4.6.8 Weed Control

Weed control along the littoral zone of a sedimentation basin is best undertaken through a combination of high planting density and applying suitable biodegradable erosion control matting. Organic mulch is generally not recommended for the littoral zone affected by frequent inundation. If the use of mulch on the littoral zones is preferred, it must be secured in place with appropriate mesh or netting (e.g. jute mesh).

4.6.9 Construction Inspection Checklist

The following checklist presents the key items to be reviewed when inspecting the sedimentation basin during and at the completion of construction. The checklist is to be used by Construction Site Supervisors, local authority, Compliance Inspectors and safety officer/ inspector to ensure all the elements of the sedimentation basin have been constructed in accordance with the design and safety measures. If an item is ticked as unsatisfactory, appropriate actions must be specified and delivered to rectify the construction issue before final inspection sign-off is given.

Checklist 2: Construction inspection checklist: Sedimentation Basin

Sedimentation Basin Construction Inspection Checklist		
Site: ______________________		**Inspected by:**
		Date:
Constructed by: ______________________		**Time:**
		Weather:
		Contact during site visit:

Items inspected	**Checked**		**Adequate**		**Items inspected**	**Checked**		**Adequate**	
	Y	**N**	**Y**	**N**		**Y**	**N**	**Y**	**N**
DURING CONSTRUCTION									
Preliminary works					**Structural components (continued)**				
1. Erosion and sediment control plan adopted					19. No seepage through banks				
2. Limit public access					20. Inlet energy dissipation installed				
3. Location same as plans					21. No seepage through banks				
4. Site protection from existing flows					22. Ensure spillway is level				
Earthworks					23. Provision of maintenance drain				
5. Integrity of banks					24. Collar installed on pipes				
6. Batter slopes as plans					**Vegetation**				
7. Impermeable (e.g. clay) base installed					25. Stabilisation immediately following earthworks and planting of terrestrial landscape around basin				
8. Maintenance access (e.g. ramp) installed					26. Weed removal prior to planting				
9. Compaction process as designed					27. Planting as designed (species and densities)				
10. Level of base, banks/ spillway as designed					28. Vegetation layout and densities as designed				
11. Check for groundwater intrusion					**Sediment and erosion control**				
12. Stabilisation with sterile grass					29. Sedimentation Basins to be used during construction				
Structural components					30. Silt fences and traffic control in place				
13. Location and levels of outlet as designed									
14. Safety protection provided									

15. Pipe joints and connections as designed									
16. Concrete and reinforcement as designed									
17. Inlets appropriately installed									
18. Inlet energy dissipation installed									
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					8. Check for uneven settling of banks				
2. Confirm structural element sizes					9. Evidence of stagnant water, short circuiting or vegetation scouring				
3. Check batter slopes					10. Evidence of litter or excessive debris				
4. Vegetation plantings as designed					11. Inlet erosion protection working				
5. Erosion protection measures working					12. Maintenance access provided				
6. Maintenance access provided					13. Construction generated sediment removed (including desilting of sedimentation basin if used during construction)				
7. Public safety adequate					14. Provision of removed sediment drainage area				
COMMENTS ON INSPECTION									
ACTIONS REQUIRED									
1.									
2.									
3.									
4.									
Inspection officer signature:									

4.7 Maintenance Requirements

Sedimentation basins treat runoff by slowing flow velocities and promoting settlement of coarse to medium sized sediments. Maintenance is focus on 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 and weed control 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. Weed management in sedimentation basins is important to ensure that weeds do not out-compete the species planted for the particular design requirements.

4.7.1 Operation & Maintenance Inspection Form

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

Checklist 3: Sedimentation Basin maintenance checklist

Sedimentation Basin Maintenance Checklist			
Inspection Frequency:	1 to 6 monthly	**Date of Visit:**	
Location:			
Description:			
Site Visit by:			

INSPECTION ITEMS	**Y**	**N**	**Action Required (details)**
Litter accumulation?			
Sediment accumulation at inflow points?			
Sediment requires removal (record depth, remove if >50%)?			
All structures in satisfactory condition (pits, pipes, ramps etc)?			
Evidence of dumping (building waste, oils etc)?			
Littoral vegetation condition satisfactory (density, weeds etc)?			
Replanting required?			
Weeds require removal from within basin?			
Settling or erosion of bunds/batters present?			
Damage/vandalism to structures present?			
Outlet structure free of debris?			
Maintenance drain operational (check)?			
Resetting of system required?			
COMMENTS			

4.8 Sedimentation Basin Design Worked Example

4.8.1 Worked example introduction

A sedimentation basin and wetland system are proposed to treat runoff from a freeway located in Singapore. This worked example focuses on the sedimentation basin (inlet zone) component of the system. A typical photograph of such a system is shown in Figure 4.9.

<u>Catchment Description</u>

The sedimentation basin receives stormwater from road runoff. 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 sedimentation basin. Each outfall services about 500m length of the 40 m wide freeway, giving a total contributing area of 2Ha (100% impervious) to each outfall.

<u>Design Objectives</u>

All stormwater runoff will be subjected to primary treatment, by sedimentation of coarse to medium size sediment.

As the sedimentation basins form part of a treatment train the design requirements of the sedimentation basin system are to:

- Promote sedimentation of particles larger than 125µm with a 90% capture efficiency for flows up to the 1-year ARI (unattenuated) peak discharge.
- Provide for connection to the downstream macrophyte zone with discharge capacity corresponding to the 1-year ARI (unattenuated) peak discharge.
- Provide for by-pass 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.

<u>Site Constraints and Concept Design</u>

The site is triangular in shape with a surface area of 500 m^2 as shown in Figure 4.10. The site of the sedimentation basin has a fall of approximately 2m (from RL 5 m to RL 3 m) towards a degraded watercourse.

The conceptual design process established the following key design elements to ensure effective operation of the constructed wetland and sedimentation basin:

- Notional permanent pool depth of sedimentation basin of 2m
- Wetland macrophyte zone extended detention depth of 0.5m (permanent water level of RL 3.4m)
- Sedimentation basin permanent pool level ('control' outlet pit level) 0.3m (RL 3.7m) above the permanent pool level of the wetland
- 'Spillway' outlet weir set 0.3 m above the permanent pool of the sedimentation basin at RL 4.0m such that the spillway is aligned with the top of the extended detention for the wetland (RL 4.0m) and triggering a by-pass when the water level in the wetland reaches then top of extended detention.

<u>Landscape Requirements</u>

Landscape design will be required and this will include the following:

- Littoral zone vegetation
- Terrestrial vegetation

Figure 4.9 Sedimentation Basin for Treatment of Freeway Runoff

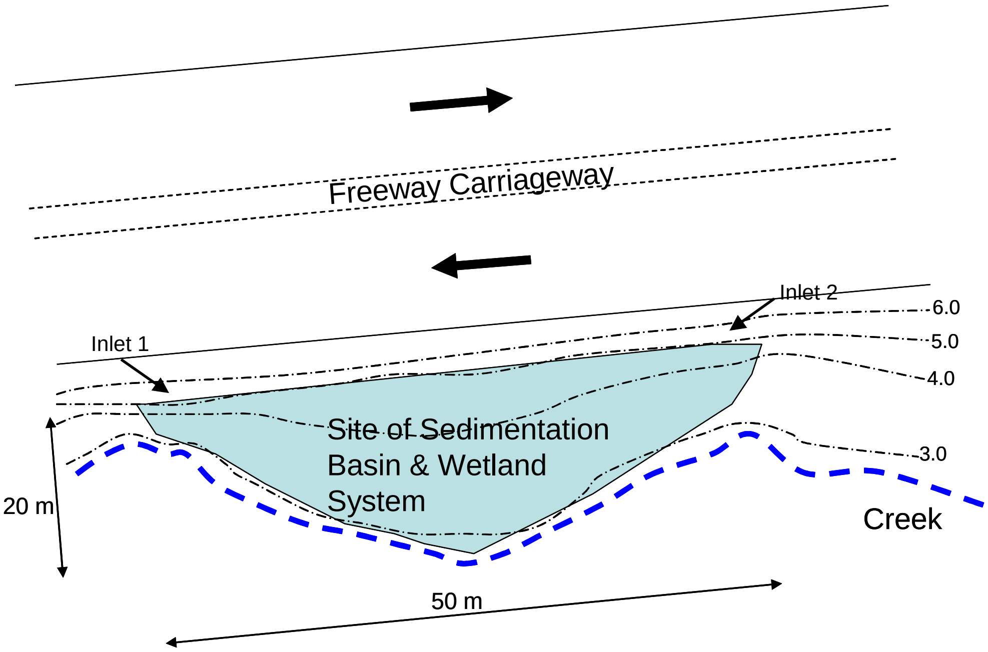


Figure 4.10 Layout of Proposed site for Sedimentation Basin

4.8.2 Calculation Steps

The design of the sedimentation basin has been divided into the following 6 calculations steps:

Step 1 Determine Design Flows
Step 2 Confirm Treatment Performance of Concept Design
Step 3 Confirm Size and Dimensions of the Sedimentation Basin
Step 4 Design inflow systems
Step 5 Design outlet structures
Step 6 Vegetation Specification

Details for each calculation step are provided below. A design calculation summary has been completed for the worked example and is given at the conclusion of the calculation steps.

Step 1 Determine Design Flows

The procedures in Singapore's Code of Practice on Surface Water Drainage (Part II) (the Rational Method) are used to determine design flow rates. The coefficients prescribed in the code of practice are based on land use within the catchment.

The site has two contributing catchments, each catchment being 2Ha in area, 500m long (along the freeway) and drained by culverts. The time of concentration (t_c) of the catchment consists of the overland flow time (t_o) plus the drain flow time from the most remote drainage inlet to the point of design (t_d), viz. $t_c = t_o + t_d$.

Overland flow time has been estimated to be relatively short (~ 4 min). A drain flow velocity of 2m/s was assumed for the purposes of estimating the time of concentration (t_c).

$$t_c = 4 + \left(\frac{500m}{2m/s} \times \frac{1\min}{60s} \right) \approx 8\min$$

Rainfall intensities for Singapore (for the 1yr and 100yr average recurrence intervals) are estimated using the IDF curves[1] for Singapore, with the time of concentration equaling 8 minutes. The 1 year ARI rainfall intensity was extrapolated using a log normal probability scale from the IDF data available:

I_1 ~ 110 mm/hr $\qquad$ I_{100} ~ 283 mm/hr

The runoff coefficients for the 1 year and 100 year ARI events were assumed to be 1.0 as given in the Code of Practice for Surface Water Drainage for roads and freeways.

The rational method is described by

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

Given the parameters for *C*, *I* and *A* described above:

$Q_1 = 1.2\ m^3/s$ $\qquad$ $Q_{100} = 3.1 m^3/s$

In summary, the design flow rates for the sedimentation basin are

Operation Design Discharge	=	$1.2 m^3/s$
Design discharge for connection to macrophyte zone	=	$1.2 m^3/s$
Spillway Design Discharge	=	$3.1 m^3/s$

Step 2 Confirm Treatment Performance of Concept Design

An initial estimate of the sedimentation basin area can be established using the curves provided in Figure 4.3. Assuming a notional permanent pool depth of 2m, a sedimentation basin area of approximately $260m^2$ is required to capture 90% of the 125µm particles for flows up to the design operation flow of $1.2m^3/s$.

[1]Please Refer to Code of Practice for Surface Water Drainage

Step 3 Confirm Size and Dimensions of the Sedimentation Basin

Sedimentation Basin Area

Confirmation of the sedimentation basin area is provided by using Equation 4.1:

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

Given

v_s = 0.011m/s

Q/A_S = 0.0046m^3/s/m^2

d_e = 0.3m

d_p = 2m

d^* = 1m (as d_P is not less than 1m)

An aspect ratio of 1 (W) to 4 (L) is adopted based on the available space (Figure 4.10). Using Figure 4.4 (configuration I), the hydraulic efficiency (λ) is estimated to be approximately 0.4. This value is less than desirable; however, site constraints prevent any other configuration. The turbulence factor (*n*) is computed from Equation 4.2 to be 1.67 and the corresponding removal efficiency to be 88%. This is reasonably close to the design removal efficiency of 90% but it may be necessary to increase the size of the basin to compensate for the lower than desired hydraulic efficiency. To achieve 90% capture efficiency, the required basin area would be approximately 300 m^2.

Sedimentation Basin storage

The sedimentation basin storage volume (V_s) is defined as the storage available in the bottom half of the sedimentation basin permanent pool depth.

Considering the relatively small size of the sedimentation basin (8m width), it is not possible to achieve the notional permanent pond depth of 2m using the 5:1 (H:V) required for public safety (Section 4.3.3.2). Therefore 4:1 (H:V) batter is to be adopted for the ground above the permanent pool level and to 0.2m below permanent pool level. A 2:1 (H:V) internal batter slope is to be adopted for 0.2m to 2m below the permanent pool level. The sedimentation basin will be fenced around most of its perimeter to ensure public safety.

Given a 2:1 (H:V) internal batter slope below the permanent water level, the area of the basin at 1m depth (i.e. half the permanent pool depth) is 115m^2 and at 2m depth (base of basin) is 2m^2.

The sedimentation basin storage volume V_s calculated using Equation 4.3 is approximately 58m^3 and corresponds to the approximately 11 years of accumulated sediment (adopting a sediment generation rate of 3 m^3/ha/yr and a capture efficiency of 87%)

The dimensions for the sedimentation basin are summarised below.

Open water area	=	260m^2
Basin length	=	32m
Basin width	=	8m
Depth of permanent pool	=	2m

Step 4 Design inflow systems

To prevent scour of deposited sediments from piped inflows, rock protection and benching is to be placed at the pipe headwall as shown in Figure 4.11.

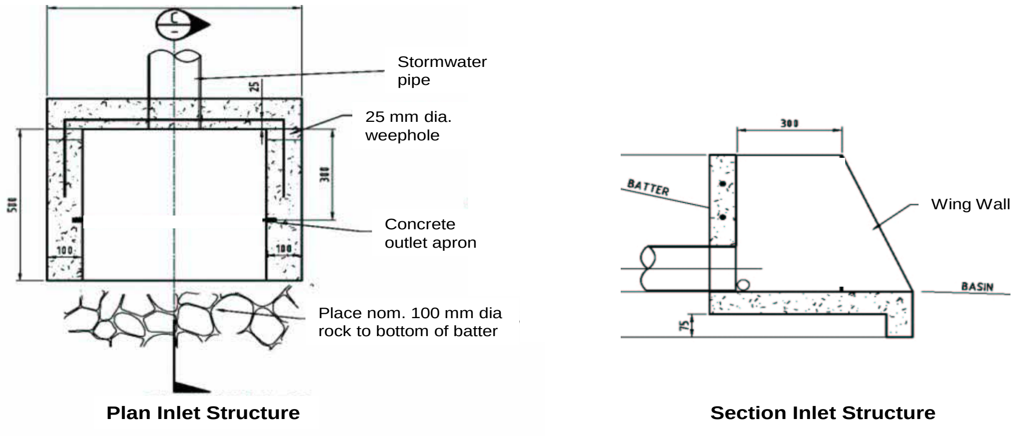


Figure 4.11: Conceptual Inlet Structure with Rock Benching

Step 5 Design outlet structures

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

The 'control' outlet structure is to consist of an outlet pit with the crest of the pit set at the permanent pool level of the sedimentation basin (RL 3.7m). The overflow pit is sized to convey the design operational flow (1 year ARI).

According to Section 4.3.5.1, two possible flow conditions need to be checked, i.e. weir flow conditions (with extended detention of 0.3m) and orifice flow conditions.

a. Weir Flow Conditions

From Equation 4.5, the required perimeter of the outlet pit to pass 1.2m^3/s with an afflux of 0.3m can be calculated assuming 50% blockage:

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

$$P = \frac{1.2}{0.5 \times 1.7 \times 0.3^{3/2}}$$

$$P = 8.6m$$

An overflow pit typically of 1.5m by 3m will be required.

b. Orifice Flow Conditions

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

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

$$A_o = \frac{1.2}{0.5 \times 0.6 \times \sqrt{2 \times 9.81 \times 0.3}}$$

$$A_o = 1.7m^2$$

A 1.5m by 3m overflow pit would have an opening area of 4.5 m^2. In this case, the orifice flow condition with a 1.7 m^2 area would be sufficient to convey the design discharge.

The top of the pit is to be fitted with a grate.

The size of the outlet pipe or connection pipe to the wetland macrophyte zone can be calculated by firstly estimating the velocity in the outlet/connection pipe using the following (Equation 4.7):

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

Where

h = head level driving flow through the pipe (defined as the 'spillway' outlet level minus the higher of the normal water level in the downstream treatment system or the obvert of the pipe)

= RL 4.0m – RL 3.4m = 0.6m

g = gravity (9.81 m/s^2)

The above equation gives a pipe flow velocity of 2.4 m/s, giving a required pipe area of 0.5m^2 to convey a flow of 1.2 m^3/s. This area is equivalent to an 800mm diameter pipe. To accommodate this pipe diameter, a pit dimension of 1.5 m x 1.5 m will be required.

If the sedimentation basin is the inlet zone of a wetland system, the obvert of the pipe is to be set just below the permanent water level in the wetland macrophyte zone (RL 3.4m) meaning the invert is at RL 2.6m.

In summary, the control outlet structure will be an overflow pit, 1.5m by 1.5m with the crest level at RL 3.7m and a raised grated cover set at RL 3.8m. The outlet/connection pipe to the wetland will be 800mm in diameter, the invert set at RL 2.6m.

Design of 'Spillway' Outlet - Weir Outlet

The 'above design flow' controlled discharge will be provided by a 'spillway' outlet weir designed to convey the 'above design flow' (100 year ARI). The crest of the spillway is set at 0.3 m above the permanent pool of the sedimentation basin.

The length of the 'spillway' outlet weir determines the afflux for the 100 year ARI peak discharge and sets the top of embankment of the sedimentation basin. It is common practice to allow for 0.3 m of freeboard above the afflux level when setting the top of embankment elevation.

An afflux of 0.3 m has been adopted in defining the length of the spillway weir. This value was adopted as a tradeoff between the bank height and the width of the weir. A bank height of 0.9 m (0.3 m afflux and 0.3 m freeboard plus 0.3 m extended detention) above the normal water level was deemed acceptable. The elevation of the crest of the spillway is RL 4.0m. The weir length is calculated using the weir flow equation (Equation 4.5) substituting outlet perimeter P with weir length L and blockage factor B=1 (no blockage):

$$L = \frac{Q_{100yr}}{C_w \cdot h^{3/2}}$$

$$L = \frac{3.1}{1.7 \times 0.3^{3/2}}$$

$$L = 11m$$

The 'spillway' outlet is located adjacent to the inflow culvert to minimise risk of sediment scour.

Step 6 Vegetation Specification

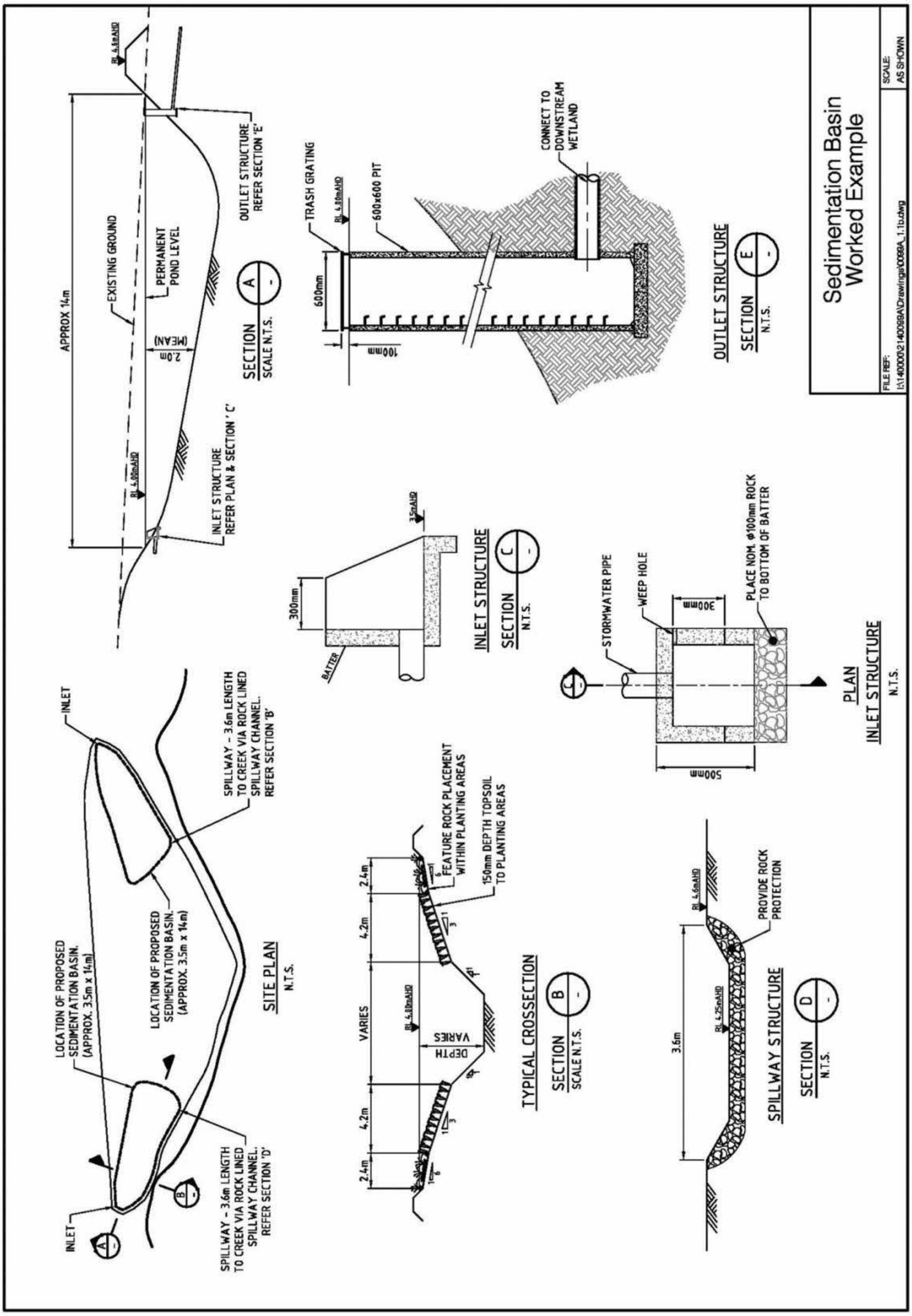
The vegetation specification for the littoral zone of a sedimentation basin will be advised once the list of recommended plantings has been established by National Parks Board of Singapore.

4.8.3 Design Calculation Summary

The sheet below summarises the results of the design calculations.

Sedimentation Basin	CALCULATION CHECKSHEET		
CALCULATION TASK	OUTCOME		CHECK
Catchment characteristics			
- Land Uses			
Residential	0	Ha	
Commercial	0	Ha	
Roads	4	Ha	
- Fraction Impervious			
Residential	0	-	
Commercial	0	-	
Roads	1	-	
Weighted average	1	-	✓
Conceptual Design			
Basin Area	260	m^2	
Notional permanent pool depth	2	m	
Permanent pool level of sedimentation basin	RL 3.8	m	
Basin extended detention	0.3	m	
Overflow level	RL 4.1	m	
Identify design criteria			
Design operation flow	1	year	
Above design flow	100	year	✓
1. Estimate design flow rates			
Time of concentration			
estimate from flow path length and velocities	8	minutes	✓
Identify rainfall intensities			
station used for IFD data:	Singapore		
Design Rainfall Intensity for design operation flow	110	mm/hr	
Design Rainfall Intensity for above design flow	283	mm/hr	✓
Design runoff coefficient			
Design operation flow	1		
Above design flow	1		
(refer to the Singapore Code of Practice on Surface Water Drainage(2000))			✓
Peak design flows			
Design operation flow	1.2	m^3/s	
Above design flow	3.1	m^3/s	✓
2. Confirm treatment performance and concept design			
Capture efficincy of sedimentation basin	87%	**%**	✓
3. Confirm size and dimensions of sedimentation basin			
- Inlet design			
Area of sedimentation basin	260	m^2	
Aspect Ratio	4	L:W (1)	
Hydraulic Efficiency	0.4		
Depth of permanent pool	2	m	✓
- Internal batters			
Cross Section Batter Slope (below permanent pool depth)	2	V:H (1)	✓
- Sediment Storage Volume			
Sediment storage volume, V_s	86	m^3	
Volume of accumulated sediment over 5years ($V_{s,5yr}$)	52	m^3	
$V_s > V_{s,5year}$	yes - OK	-	
Sediment clean-out frequency, given V_s	8	years	✓
4. Design inflow systems			
Scour protection and/or energy dissipation provided	yes		✓
5. Design outlet structures			
- Overflow pit			
Pit dimension	1.5m x 3m	L x B	
Overflow crest level	RL 3.8	m	
Provision of debris trap	yes	-	✓
- Connection Pipe			
Connection pipe dimension	802	mm dia	
Connection pipe invert level	RL 2.7	m	✓
- Control outlet weir (Spillway)			
Weir crest level	RL 4.1	m	
Weir length	11	m	
Afflux	0.3	m	
Freeboard to top of embankment	0.3	m	✓
6. Vegetation Specification	HOLD		✓

4.8.4 Construction drawings



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Swales and Buffer Strips 5

5

Chapter 5 Swales and Buffer Strips

5.1 Introduction

Vegetated swales are used to remove coarse and medium sediments and convey stormwater in lieu of or with underground pipe drainage systems. They are commonly combined with buffer strips and bioretention systems (refer Chapter 6 - Bioretention Swales). Swales utilise overland flow and mild slopes to convey water slowly downstream. They protect waterways from damage by erosive flows from frequent storm events because swale flow velocities are slower than concrete drains.

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

Swales alone cannot provide sufficient treatment to meet current stormwater treatment/ water quality objectives but can enable water quality objectives to be met by providing an important pretreatment function for other ABC Waters Design Features in a treatment train. Swales are particularly good at coarse sediment removal and can provide the necessary pretreatment for downstream treatment systems such as wetlands and bioretention basins. Some examples of swales are provided in Figure 5.1.

Figure 5.1 Swales in Singapore

Buffer strips (or buffers) are areas of vegetation through which runoff flows (as overland flow) to a discharge point. Sediment is deposited as flow passes through vegetation over a shallow depth. Effective treatment relies upon well distributed sheet flow. Vegetation slows flow velocities, encouraging coarse sediments to settle out of the water column. With the requirement for uniformly distributed flow, buffer strips are suited to treat road runoff in situations where road runoff is discharged via flush kerbs or through regular kerb ‘cut-outs’ or slotted kerbs. In these situations, buffer strips (located in the swale batter) can form part of a roadside swale system that receives the distributed inflows from the adjoining road pavement. The coverage of buffer strips in this chapter is limited to their application as part of a roadside swale system only. The reader is referred to *Australian Runoff Quality* (Engineers Australia 2006) for additional discussion on buffer strip design and for worked examples.

5.2 Design Considerations for Swales

5.2.1 Landscape Design

Swales may be located within parkland areas, easements, car parks or along road verges or centre medians. Landscape design of swales and buffer strips along the road edge can assist in defining the boundary of road or street corridors as well as enhancing landscape character. The landscape design of swales and buffers must address stormwater quality objectives whilst also incorporating landscape functions. As such, it is important that swales and buffers are carefully designed to integrate with the surrounding landscape character.

5.2.2 Hydraulic Design

Typically, swales are applicable for smaller scale contributing catchments. For larger catchments, dimension of swales may become too big for most urban areas in Singapore. Also, flow depths and velocities are such that the water quality improvement function of the swale, and its long-term function may be compromised. For water quality improvement, swales need only focus on ensuring frequent storm flows (typically up to the 3 month ARI flow) are conveyed within the swale profile. In most cases, however, a swale will also be required to provide a flow conveyance function as part of a minor drainage and/or major drainage system. In particular, swales located within road reserves must also allow for safe use of adjoining roadway, footpaths and bike paths by providing sufficient conveyance capacity to satisfy current engineering infrastructure design requirements as defined by PUB's Code of Practice on Surface Water Drainage. It may also be necessary to augment the capacity of the swale with underground drainage to satisfy the drainage requirements. This can be achieved by locating overflow pits (field inlet pits) along the invert of the swale that discharge into an underlying pipe drainage system. Careful attention should be given to the design of overflow pits to ensure issues of public safety (particularly when raised grates are being used) and aesthetic amenity are taken into account.

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

A rule of thumb for locating check dams is for the crest of a downstream check dam to be at 4 % grade from 100 mm below the toe of an upstream check dam (refer Figure 5.2). The impact of check dams on the hydraulic capacity of the swale must be assessed as part of the design process.

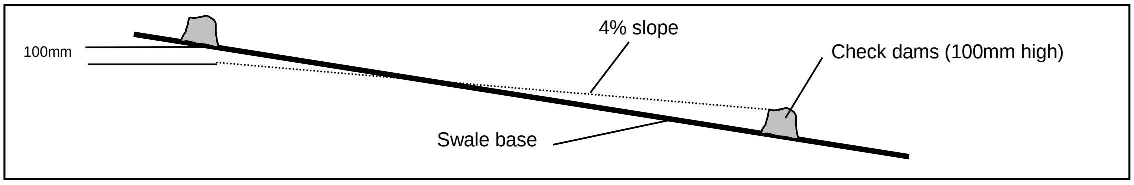


Figure 5.2 Location of Check Dams in Swales

Velocities within swales must be kept low to avoid scouring of collected pollutants and vegetation, preferably less than 0.5 m/s for minor flood flows (up to 10 year ARI events) and not more than 2.0 m/s for major flood flows (up to 100 year ARI events). Similar checks should also be undertaken to assess depth x velocity within the swale, at crossings and adjacent to pedestrian and bicycle pathways to ensure public safety criteria are satisfied. These are:

- depth x velocity < 0.6 m^2/s for low risk locations and 0.4 m^2/s for high risk locations (e.g. where pedestrian traffic is expected to be high)
- maximum flow depth on driveway crossings = 0.3 m.

5.2.3 Vegetation Types

Swales can use a variety of vegetation types including turf, sedges and tufted grasses. Vegetation is required to cover the whole width of the swale, be capable of withstanding design flows and be of sufficient density to prevent preferred flow paths and scour of deposited sediments (Figure 5.3).

Figure 5.3 Swale systems: heavily vegetated (left), use of check dams (centre), grass swale with elevated crossings (right)

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

Figure 5.4 Swale incorporated into road reserve

The reader should consult the National Parks Board of Singapore for more specific guidance on the selection of appropriate vegetation for swales and buffers located within road reserves.

5.2.4 Driveway Crossings

A key consideration when designing swales along roadways is the requirement for provision of driveway crossings (or crossovers). 'Elevated' crossings are common in Singapore and raised above the invert of the swale (e.g. like a bridge deck or culvert, see Figure 5.5).

Figure 5.5 Elevated driveway crossings to allow vehicle access across swales (right)

'Elevated' crossings are applicable in Singapore. Where appropriate, they can be designed as streetscape features. They also provide an opportunity for locating check dams (to distribute flows) or to provide temporary ponding above a bioretention system (refer Chapter 6 – Bioretention Swales). A major limitation with 'elevated' crossings can be their high life cycle costs due to the need for on-going maintenance. Safety concerns with traffic movement adjacent to 'elevated' crossings and the potential for blockages of small culvert systems beneath the crossing are other possible limitations. These limitations can be overcome by careful design through the use of spanning crossings rather than using small culverts and through the use of durable decking materials in place of treated timber.

5.2.5 Traffic Controls

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

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

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

5.2.6 Roof Water Discharge

Roof water should be discharged onto the surface of the swale for subsequent conveyance and treatment by the swale (and downstream treatment measures) before being discharged to receiving aquatic environments. Depending on the depth of the roof water drainage system and the finished levels of the swale, this may require the use of a small surcharge pit located within the invert of the swale to allow the roof water to surcharge to the swale. Any residual water in the surcharge pit can be discharged to the underlying subsoil drainage by providing perforations in the base and sides of the surcharge pit. If a surcharge pit is used, an inspection chamber along the roof water drainage line is to be provided within the property boundary. Surcharge pits are discussed further in Section 5.3.4.3.

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

5.2.7 Services

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

5.3 Swale Design Process

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

The key design steps are:

1. Confirm treatment performance of concept design

2. Determine design flows

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

4. Determine design of inflow systems

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

6. Size overflow pits (field inlet pits)

7. Make allowances to preclude traffic on measures

8. Specify plant species and planting densities

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

Each of these design steps is discussed in the following sections. A worked example illustrating application of the design process on a case study site is presented in Section 5.7.

5.3.1 Step 1: Confirm Treatment Performance of Concept Design

Before commencing detailed design, the designer should first undertake a preliminary check to confirm the swale outlined on the concept design is adequate to deliver the level of stormwater quality improvement inferred within the concept design documentation. The swale treatment performance curves shown in Figure 5.6 to Figure 5.8 can be used to undertake this verification check.

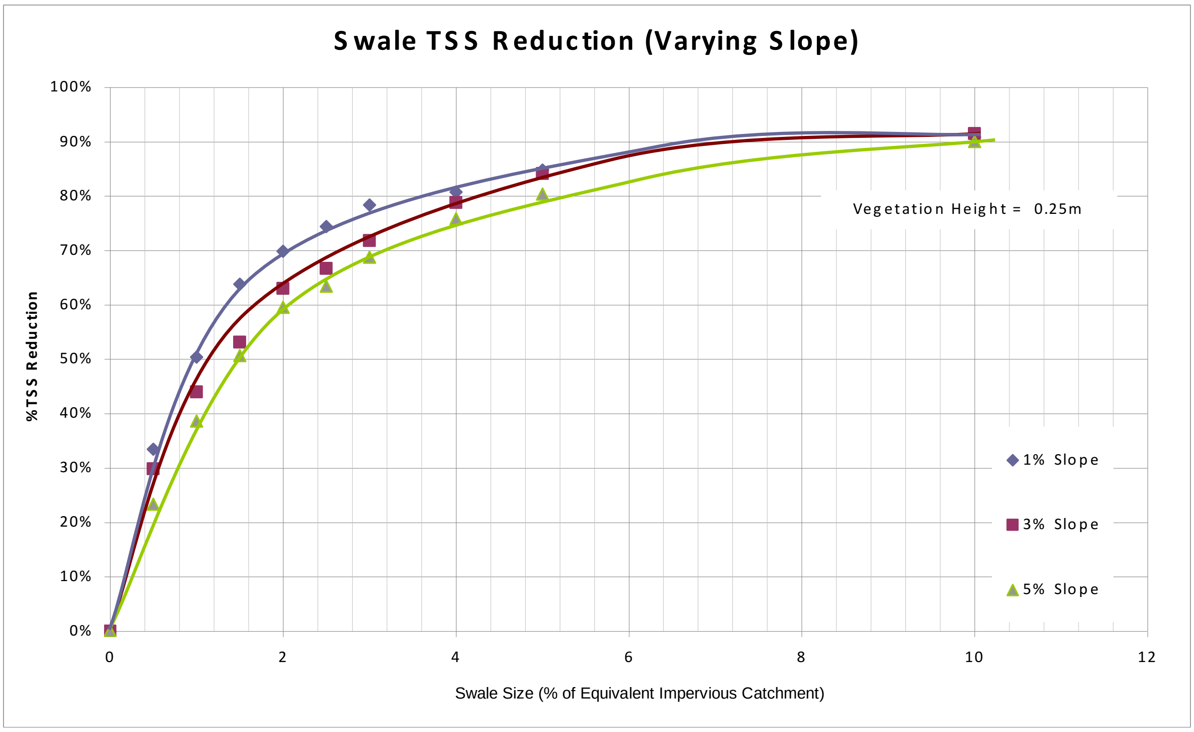
The curves in Figure 5.6 to Figure 5.8 were derived using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC), assuming the swale is a stand alone system (i.e. not part of a treatment train). The curves show the total suspended solid (TSS), total phosphorus (TP) and total nitrogen (TN) removal performance for a typical swale design, being:

- Top width 4.5 m
- Base width 1 m
- Side slopes 1 in 9

The curves in Figure 5.6 to Figure 5.8 are generally applicable to swale applications within residential, industrial and commercial land uses.

If the configuration of the swale concept design is significantly different to that described above, then a stormwater quality model such as MUSIC or equivalent should be used in preference to the curves in Figure 5.6 to Figure 5.8. The detailed designer should also use the stormwater quality model to verify swale concept designs that are part of a “treatment train”.

Swales should form part of the stormwater ‘treatment train’ as they will not achieve load-based pollutant reduction objectives on their own. Therefore, other stormwater quality best management practices should be incorporated into the surrounding catchment to augment the stormwater treatment performance of any proposed swale system.



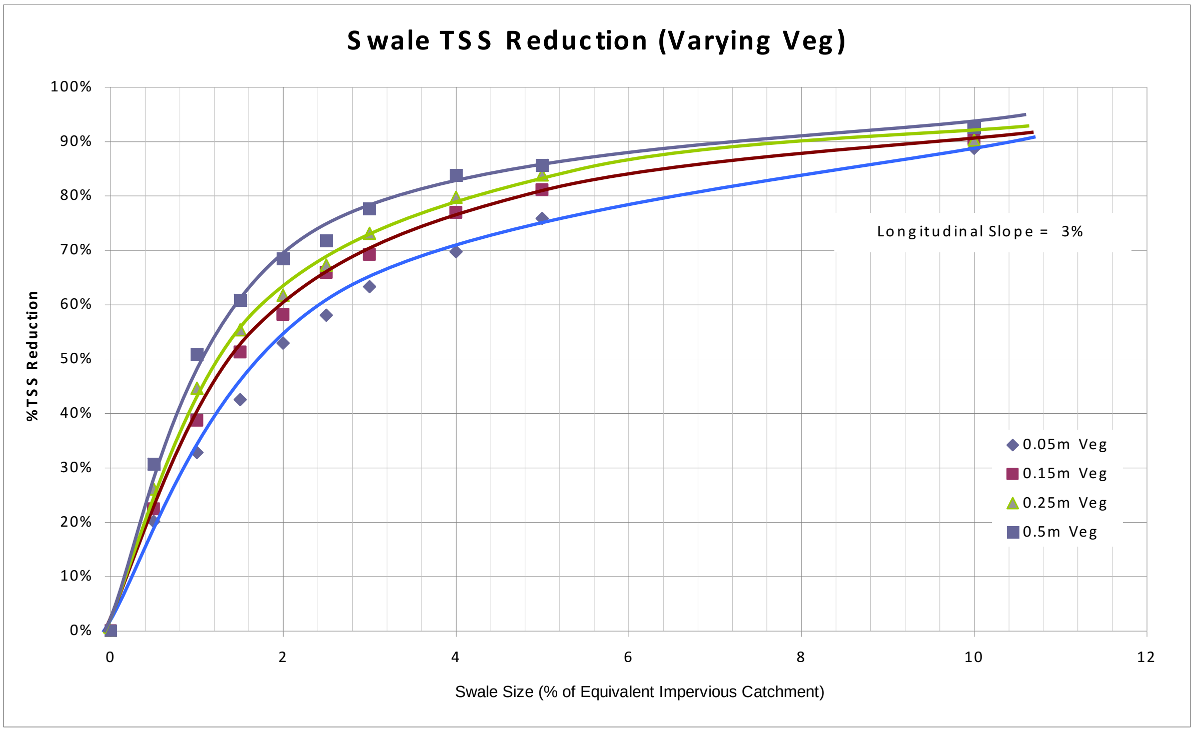
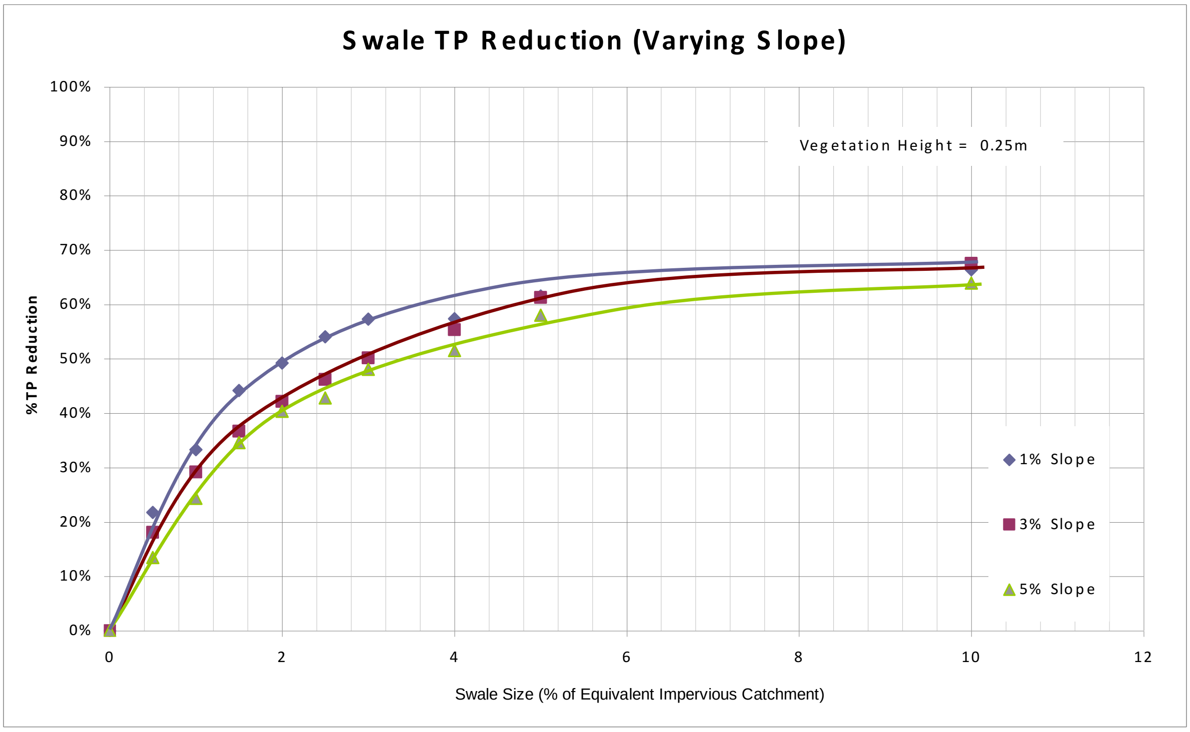


Figure 5.6 Swale TSS Removal Performance



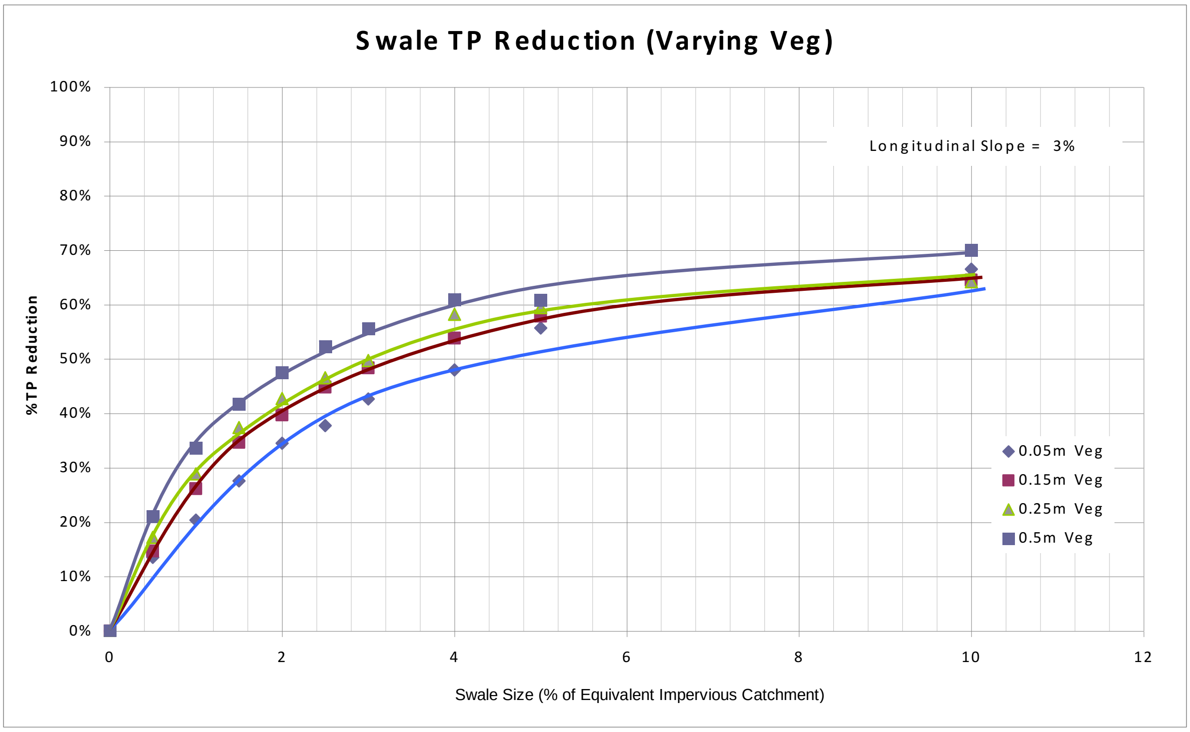
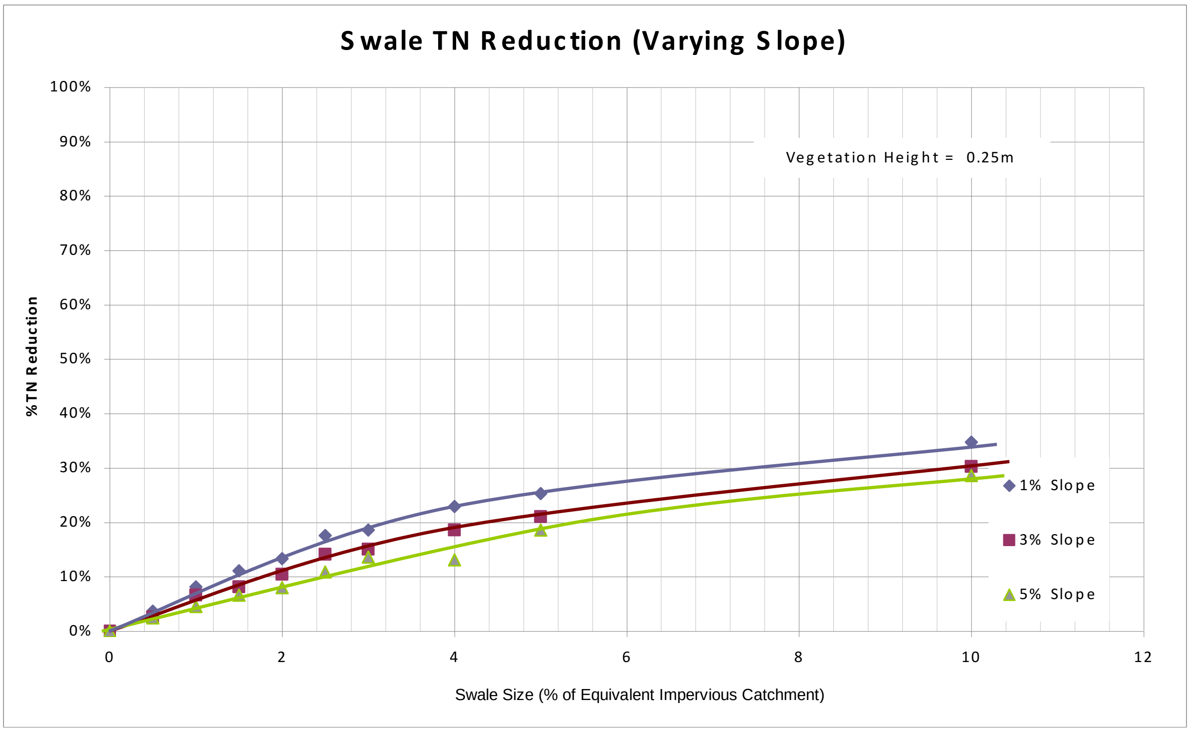


Figure 5.7 Swale TP Removal Performance



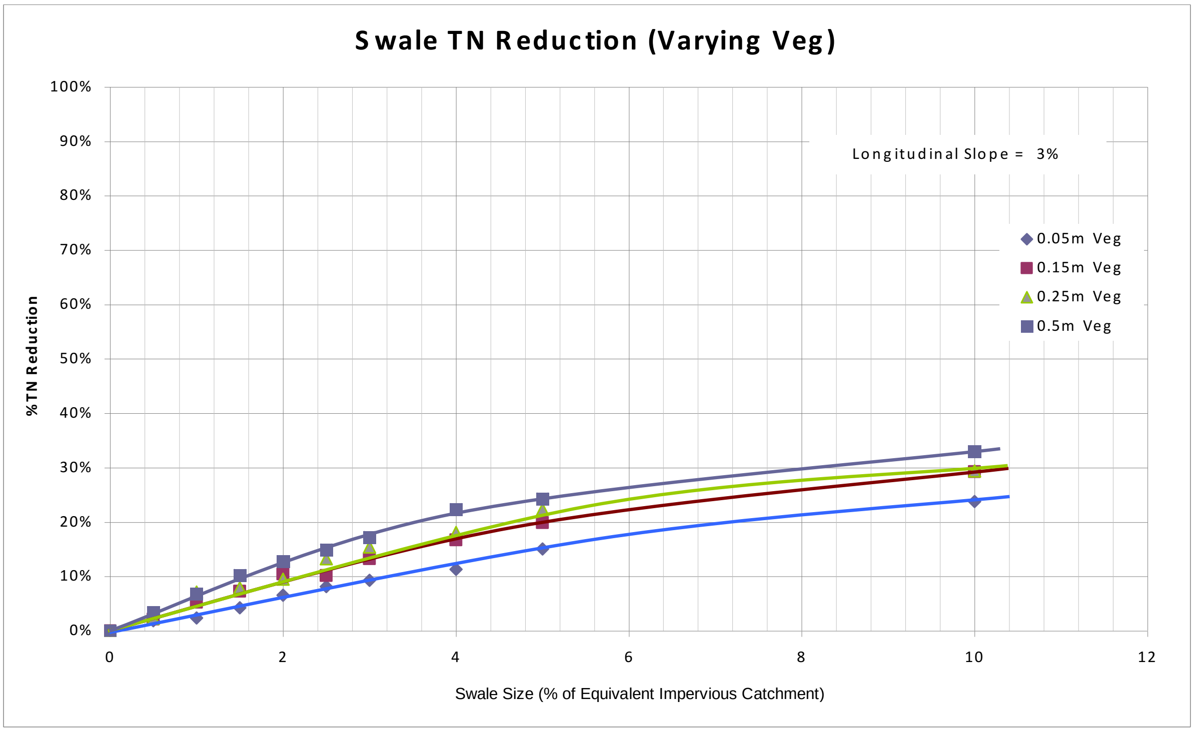


Figure 5.8 Swale TN Removal Performance

5.3.2 Step 2: Determine Design Flows

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

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

The *Code of Practice on Surface Water Drainage* (PUB 2018) identifies the Rational Method as the procedure most commonly used to estimate peak flows from small catchments in Singapore.

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

Factors to consider are:

- Contributing catchment area
- Allowable width given the proposed road reserve and/ or urban layout
- How flows will be delivered into a swale (e.g. cover requirements for pipes or kerb details)
- Vegetation height
- Longitudinal slope
- Maximum side slopes and base width
- Provision of crossings
- Other requirements in accordance with the latest version of Code of Practice on Surface Water Drainage (PUB).

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

5.3.3.1 Swale Width and Side Slopes

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

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

The maximum swale side slopes will be established from ease of maintenance and public safety considerations. Where ‘elevated’ crossings are used, swale side slopes would typically be between 1 in 6 and 1 in 4. ‘Elevated’ crossings will require provision for drainage under the crossings with a culvert or similar.

5.3.3.2 Maximum Length of a Swale

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

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

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

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

$$Q = \frac{A \cdot R^{2/3} \cdot S^{1/2}}{n^{n}}$$ Equation 5.1

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

A = cross section area (m^2)

R = hydraulic radius (m)

S = channel slope (m/m)

n = roughness factor (Manning's n)

Manning's *n* relates to the roughness of the channel and is a critical variable in Manning's equation. It varies with flow depth, channel dimensions and the vegetation type. For constructed swale systems, recommended values are between 0.15 and 0.3 for flow depths shallower than the vegetation height (preferable for treatment) and significantly lower for flows with depth greater than the vegetation (e.g. 0.03 – 0.05 at more than twice the vegetation depth i.e. 50-100 year ARI). It is considered reasonable for Manning's *n* to have a maximum at the vegetation height and then to sharply reduce as depths increase.

Figure 5.9 shows a plot of Manning's *n* versus flow depth for a grass swale with longitudinal grade of 5 %. It is reasonable to expect the shape of the Manning's *n* relation with flow depth to be consistent with other swale configurations, with the vegetation height at the boundary between low flows and intermediate flows on the top axis of the diagram. The bottom axis of the plot has been modified from Barling and Moore (1993) to express flow depth as a percentage of vegetation height.

Further discussion on selecting an appropriate Manning's *n* for a swale is provided in Appendix F of the *MUSIC User Guide* (eWater Ltd 2014).

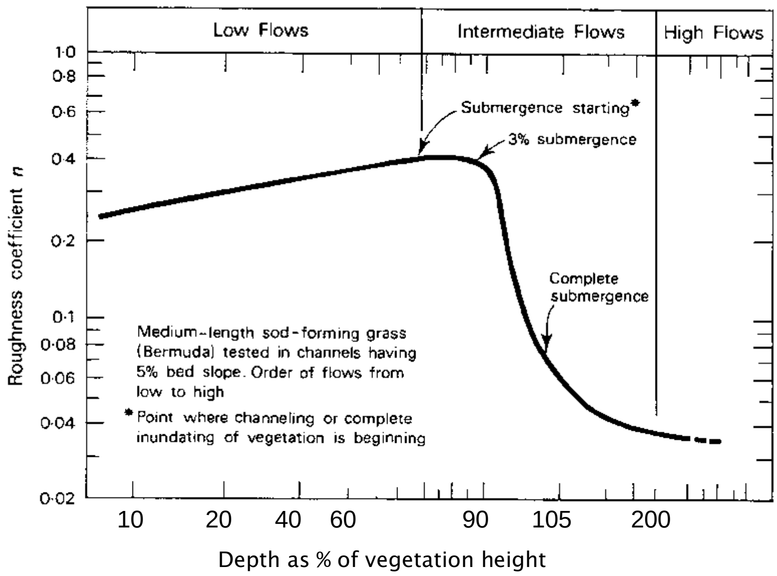


Figure 5.9 Impact of Flow Depth on Hydraulic Roughness (adapted from Barling & Moore (1993))

5.3.4 Step 4: Determine Design of Inflow Systems

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

5.3.4.1 Distributed Inflow

An advantage of flows entering a swale system in a distributed manner (i.e. entering perpendicular to the direction of the swale) is that flow depths are kept as shallow sheet flow, which maximises contact with the swale vegetation on the batter receiving the distributed inflows. This swale batter is often referred to as a buffer. To ensure the function of the buffer, flow depths must be shallow (below the vegetation height) and erosion must be avoided. The buffer provides good pre-treatment through coarse sediment removal prior to flows being conveyed along the swale.

Distributed inflows can be achieved either by having a flush kerb or by using kerbs with regular breaks in them to allow for even flows across the buffer surface (Figure 5.10).

Figure 5.10 Kerb arrangements to promote distributed flow into swales

5.3.4.2 Buffer Requirements

There are several design guides that may to be applied to ensure buffers operate to improve water quality and provide a pretreatment role. Key design parameters of buffer systems are:

- Providing distributed rather than concentrated flows onto a buffer to avoid erosion and channelled flows
- Maintaining flow depths less than vegetation heights. This may require flow spreaders, or check dams.
- Minimising the slope of the buffer. It is best if slopes can be kept below 5 %, however buffers can still perform well with slopes up to 20 % provided flows are well distributed. The steeper the buffer the more likely flow spreaders will be required to avoid rill erosion.

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

It is important to ensure coarse sediments accumulate off the road surface at the start of the buffer or green verge. To avoid accumulation of sediments on the carriageway or just before the kerb openings, slotted kerbs with a level drop should be used so that the top of the vegetation is set 60 mm below the edge of pavement. This requires the finished topsoil surface of the swale (i.e. before turf is placed) to be approximately 100 mm below the edge of pavement level. Sediments can then accumulate off any trafficable surface.

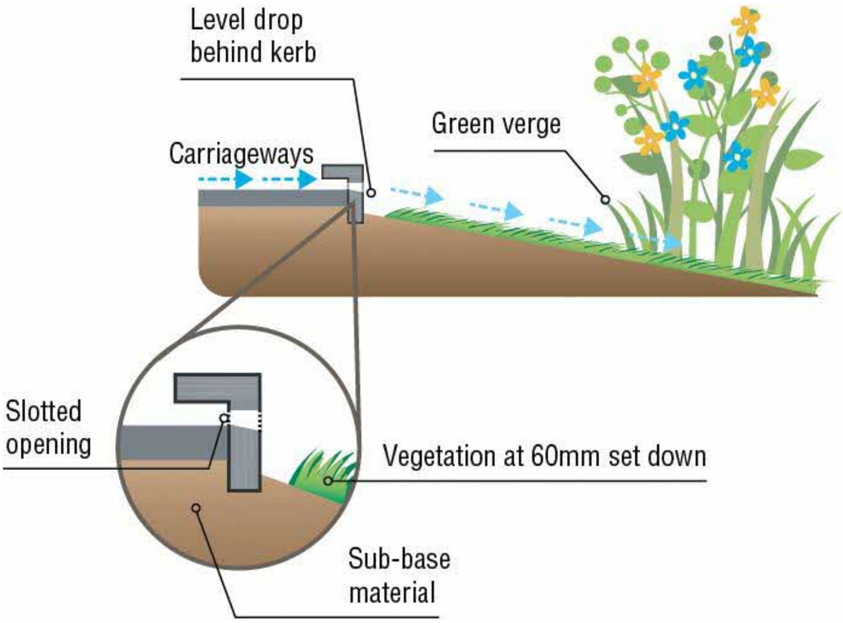


Figure 5.11 Slotted kerb with set-down to allow sediment to flow into the vegetated area

5.3.4.3 Concentrated Inflow

Concentrated inflows to a swale can be in the form of a concentrated overland flow or a discharge from a pipe drainage system. For all concentrated inflows, energy dissipation at the swale inflow location is an important consideration to minimise any erosion. This can usually be achieved with rock benching and/ or dense vegetation (Figure 5.12).

Figure 5.12 Energy Dissipator at swale inlet

The most common constraint on pipe systems discharging to swales is bringing the pipe flows to the surface of a swale. In situations where the swale geometry does not permit the pipe to achieve 'free' discharge to the surface of the swale, a 'surcharge' pit may need to be used. Surcharge pits should be designed so that they are as shallow as possible and have pervious bases to avoid long term ponding in the pits (this may require under-drains to ensure it drains, depending on local soil conditions). The pits need to be accessible so that any build up of coarse sediment and debris can be monitored and removed if necessary.

Figure 5.13 shows an example of a typical surcharge pit discharging into a swale. Surcharge pits are not considered good practice, due to additional maintenance issues and mosquito breeding potential and should therefore be avoided where possible. The design of surcharge pits shown here is for reference only. The actual design needs to be approved by the relevant agencies and the party that will take over the maintenance.

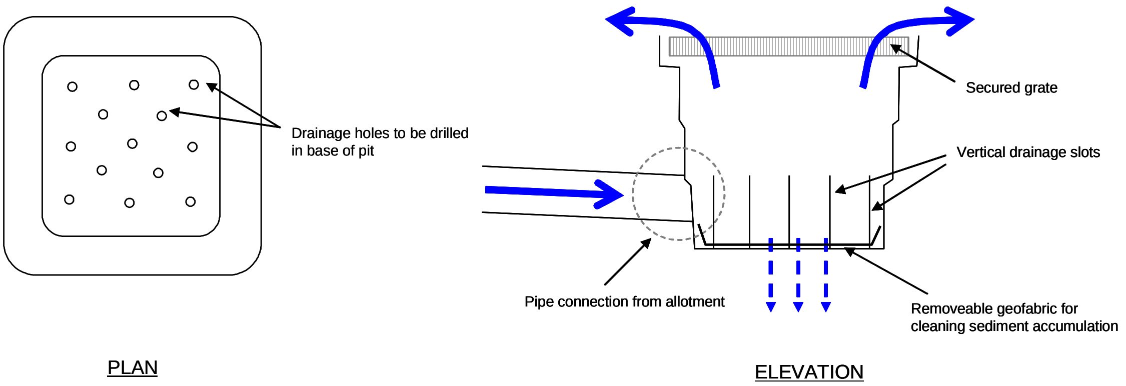


Figure 5.13 Example of Surcharge Pit for Discharging Concentrated Runoff into a Swale

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

5.3.5 Step 5: Verify Design

5.3.5.1 Vegetation Scour Velocity Check

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

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

5.3.5.2 Velocity and Depth Check – Safety

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

- depth x velocity of < 0.4 m^2/s is not exceeded for all flows up to the major design event, as defined in relevant local government guidelines
- maximum depth of flow over 'at-grade' crossings = 0.3 m

5.3.5.3 Confirm Treatment Performance

If the previous two checks are satisfactory then the swale design is adequate from a conveyance function perspective and it is now necessary to reconfirm the treatment performance of the swale by reference back to the information presented in Section 5.3.1.

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

To size a swale field inlet pit, two checks should be made to test for either drowned or free flowing conditions. A broad crested weir equation can be used to determine the length of weir required (assuming free flowing conditions) and an orifice equation used to estimate the area between openings required in the grate cover (assuming drowned outlet conditions). The smaller of the two pit configurations would normally suffice although other consideration such as the required pit to fit the stormwater pipe conveying overflows to the receiving waters need also to be considered. In addition, a blockage factor is to be used, that assumes the field inlet is 50 % blocked.

For free overfall conditions (weir equation):

$$Q_{weir} = B \cdot C_w \cdot L \cdot h^{3/2} \qquad \text{Equation 5.2}$$

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

B = blockage factor (0.5)

C_w = weir coefficient (1.66)

L = length of weir (m)

h = depth of water above weir crest (m)

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

For drowned outlet conditions (orifice equation):

$$Q_{orifice} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h} \qquad \text{Equation 5.3}$$

Where	$Q_{orifice}$	= flow into drowned pit (m^3/s)
	B	= blockage factor (0.5)
	C_d	= discharge coefficient (0.6)
	A	= total area of orifice (openings) (m^2)
	g	= 9.80665 m/s^2
	h	= depth of water above centre of orifice (m)

When designing grated field inlet pits reference should be made to the procedure described in the latest version of Code of Practice on Surface Water Drainage (PUB)

5.3.7 Step 7: Make Allowances to Preclude Traffic on Measures

Refer to Section 5.2.5 for discussion on traffic control options.

5.3.8 Step 8: Specify Plant Species and Planting Densities

For planting within road verge, the National Parks Board should be consulted for guidance of appropriate plant species and planting densities applicable for roadside swales in Singapore.

5.3.9 Step 9: Consider Maintenance Requirements

Consider how maintenance is to be performed on the swale (e.g. how and where is access available, where is litter likely to collect etc.). A specific maintenance plan and schedule should be developed for the swale, either as part of a maintenance plan for the whole treatment train, or for each individual asset. Guidance on maintenance plans is provided in Section 5.5.

5.3.10 Design Calculation Summary

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

SWALES – DESIGN CALCULATION SUMMARY SHEET

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

5.3.10.1 Typical Design Parameters

Table 5.1 provides typical values for a number of key swale design parameters.

Table 5.1: Typical Design Parameters

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

5.4 Construction advice

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

5.4.1 Building phase damage

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

5.4.2 Traffic and deliveries

Ensure traffic and deliveries do not access swales during construction. Traffic can compact the soil and cause preferential flow paths, deliveries can smother vegetation. Wash down wastes (e.g. silt, concrete) can disturb vegetation and cause uneven slopes along a swale. Swales should be protected during construction phase and controls implemented to avoid wash down wastes.

5.4.3 Inlet erosion checks

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

5.4.4 Timing for planting

Timing of vegetation is typically after completion of construction activities in the surrounding area and dependent on timing in relation to the phases of development too. For example temporary planting during construction for sediment control (e.g. with turf) then remove and plant out with long term vegetation upon completion of construction.

5.5 Maintenance Requirements

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

The most intensive period of maintenance is during the plant establishment period (first two years) when weed removal and replanting may be required. It is also the time when large loads of sediments may impact on plant growth, particularly in developing catchments with an inadequate level of erosion and sediment control.

Typical maintenance of swale elements will involve:

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

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

- inspection frequency
- maintenance frequency
- data collection/ data storage requirements
- detailed cleanout procedures (main element of the plans) including:
    - equipment needs

    - maintenance techniques
    - occupational health and safety
    - public safety
    - environmental management considerations
    - disposal requirements (of material removed)
    - access issues
    - stakeholder notification requirements
    - data collection requirements (if any)
- design details

An example of an operation and maintenance inspection form is provided in the checking tools provided in Section 5.6.3.

5.6 Checking tools

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

Checklists are provided for:

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

5.6.1 Design assessment checklist

The Design Assessment Checklist on the following page presents the key design features that are to be reviewed when assessing a design of a swale. These considerations include configuration, safety, maintenance and operational issues that need to be addressed during the design phase. If an item receives an 'N' when reviewing the design, referral is made back to the design procedure to determine the impact of the omission or error. In addition to the checklist, a proposed design is to have all necessary permits for installation.

5.6.2 Construction Checklist

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

5.6.3 Operation and Maintenance Inspection Form

The Operation and Maintenance forms on the following pages should be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time. Inspections should occur every 1 to 6 months depending on the size and complexity of the swale system, and the stage of development (i.e. inspections should be more frequent during building phase and until the swale landform has stabilised).

SWALE DESIGN ASSESSMENT CHECKLIST			
Asset I.D.		**Assessed by:**	Date:
Swale Location:			
Hydraulics:	Minor Flood (m^3/s):	Major Flood (m^3/s):	
Area:	Catchment Area (ha):	Swale Area (m^2):	

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

SWALE CONSTRUCTION INSPECTION CHECKLIST

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

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

COMMENTS ON INSPECTION

ACTIONS REQUIRED:

Inspection officer signature:

SWALE (AND BUFFER) MAINTENANCE CHECKLIST			
Asset I.D.:			
Inspection Frequency:	Weekly to monthly	**Date of Visit:**	
Location:			
Description:			
Site Visit by:			

INSPECTION ITEMS	FREQUENCY	Y	N	ACTION REQUIRED (DETAILS)
Sediment accumulation at inflow points?	Weekly			
Litter within swale?	Weekly			
Erosion at inlet or other key structures (e.g. crossovers)?	Weekly			
Traffic damage present?	Weekly			
Evidence of dumping (e.g. building waste)?	Weekly			
Vegetation condition satisfactory (density, weeds etc)?	Monthly			
Replanting required?	Monthly			
Mowing required?	Fortnightly			
Sediment accumulation at outlets?	Weekly			
Clogging of drainage points (sediment or debris)?	Weekly			
Evidence of ponding?	Weekly			
Set down from kerb still present?	Monthly			
Soil additives or amendments required?	Monthly			
Pruning and/ or removal of dead or diseased vegetation required?	Monthly			
Inspect swale cross-section profile according to Drawing No. XXXXX				
Inspect swale longitudinal profile according to Drawing No. YYYYY				
COMMENTS				

1) Cross-Section Plan – Drawing No. XXXXX
2) Longitudinal-Section Plan – Drawing No. YYYYY
3) Location Plan of Swale No. – Drawing No. ZZZZZ

ASSET TRANSFER CHECKLIST		
Asset Description:		
Asset I.D.:		
Asset Location:		
Construction by:		
DLP Period:		
TREATMENT	**Y**	**N**
System appears visually to be working as designed?		
No obvious signs of under-performance?		
MAINTENANCE	**Y**	**N**
Maintenance plans and indicative maintenance costs provided for each asset?		
Vegetation establishment period completed?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
Asset inspected for defects?		
ASSET INFORMATION	**Y**	**N**
Design Assessment Checklist provided?		
As constructed plans provided?		
Copies of all required permits (both construction and operational) submitted?		
Proprietary information provided (if applicable)?		
Digital files (e.g. drawings, survey, models) provided?		
Asset listed on asset register or database?		
COMMENTS		

5.7 Swale Worked Example

5.7.1 Worked example introduction

As part of a development, runoff from allotments and a street surface is to be collected and conveyed in a vegetated swale system to downstream treatments. The swale will be vegetated with turf (100mm tall). An additional exercise in this worked example is to investigate the consequences on flow capacity of using a taller species such as sedges in the swale (vegetation height equal to 300mm).

A concept design for the development proposed this system as part of a treatment train. The street will have a one-way cross fall (to the high side) with flush kerbs, to allow for distributed flows into the swale system across a buffer zone.

The swale is to convey minor flood events, including all flows up to a ten-year ARI storm. However, the width of the swale is fixed at 5.0 m and there will be a maximum catchment area the swale can accommodate, above which an underground pipe will be required to preserve the conveyance properties of the downstream swale.

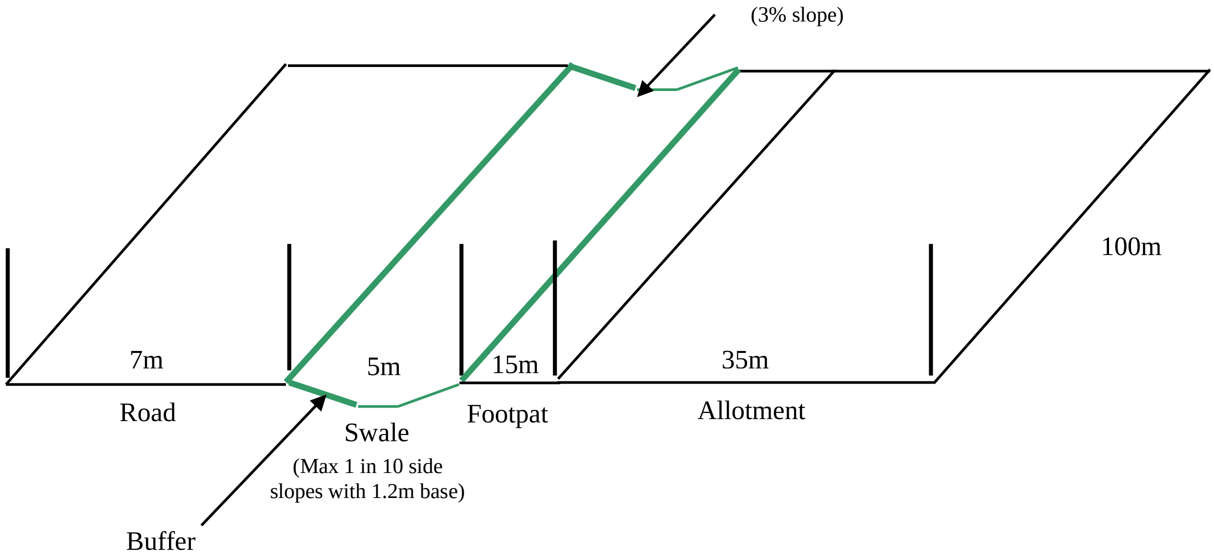


Figure 5.14 Cross section of proposed buffer/swale system

The contributing catchment area includes 35 m width and 100m length residential allotments on one side, a 7m wide road pavement surface and a 1.5 m footpath and 5.0 m swale and services easement (depicted in Figure 5.14, examples of similar systems are illustrated in Figure 5.15). The area is 100 m long with a 3 % slope.

Allotment runoff is to be discharged under a footpath via a conventional stormwater pipe directly into the swale system with appropriate erosion control.

Figure 5.15 Similar buffer swale system for conveying runoff

Design criteria for the buffer/ swale system are to:

- Promote sedimentation of coarse particles through the buffer by providing for an even flow distribution and areas for sediment accumulation (i.e. set down at kerb edge);
- Provide traffic management measures that will preclude traffic damage (or parking) within the buffer or swale (e.g. bollards or parking bays);
- Provide check dams to control velocities and spread flows (potentially using crossings);
- Provide driveway access to lots within side slope limits and
- Convey 10-year ARI flows within the swale and underground pipe system.

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

- Design the swale system to accommodate driveway crossovers and check dams where required
- Select vegetation such that the hydraulic capacity of the swale is sufficient
- Determine maximum length of swale to convey 10-year flows before an underground pipe is required
- Check velocities are maintained to acceptable levels
- Overflow structure from swale to underground pipe (if required).

Additional design elements will be required, including:

- Configure the street kerb details such that sheet flow is achieved through the buffer strip
- Configure house lot drainage so that erosion control is provided
- Buffer strip vegetation
- Swale vegetation (integral with hydraulic design of the system).

5.7.1.1 Design Objectives

The design objectives are summarised as follows:

- Swale shall convey at least all flows up to the peak 10-year ARI storm event.

- Sedimentation of coarse particles will be promoted within the buffer by providing an even flow distribution.
- Prevent traffic damage to the buffer swale system.
- Flow velocities to be controlled to prevent erosion.

5.7.1.2 Site Characteristics

Catchment area:	3,500 m^2	(lots)
	850 m^2	(roads and concrete footpath)
	500 m^2	(swale and services easement)

Total = 4,850 m^2

Land use/surface type Residential lots, roads/concrete footpaths, swale and service easement.

Overland flow slope:

Total main flow path length = 100m @ 3% slope

Soil type: Clay

Fraction impervious:

- lots f = 0.65
- roads/footpath f = 1.00
- swale/service easement f = 1.0

Vegetation height of 100 mm

5.7.2 Step 1: Confirm Treatment Performance of Concept Design

Interpretation of Figure 5.6 to Figure 5.8 with the input parameters below is used to estimate the reduction performance of the swale system to ensure the design will achieve target pollutant reductions. To interpret the graphs the area of swale base to the impervious catchment needs to be estimated. For a base width of 1.2 m, the area of swale base as percentage of the contributing impervious catchment area:

$$1.2 \times 100 / [(0.65 \times 3500) + (1.0 \times 850) + (1.0 \times 500)] = 3.3\ \%$$

From the figures using an equivalent area in the reference site, it is estimated that, depending on the height of the vegetation, pollutant reductions are between 68% and 80% for TSS, 45% to 57% for TP and 10% to 20% for TN respectively.

5.7.3 Step 2: Determine Design Flows

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

5.7.3.1 Major and minor design flows

<u>Time of concentration (t_c)</u>

The time of concentration is estimated assuming overland flow across the allotments and along the swale and is determined to be 10 minutes.

Design rainfall intensities

Adopt from IDF table[1] for Singapore for a time of concentration (t_c) of 10 minutes

ARI	Intensity
10yr	190 mm/hr
100yr	275 mm/hr

Design runoff coefficient

Apply the Rational Formula method outlined in Code of Practice on Surface Water Drainage (PUB).

$$Q = 0.002788 \times C \times I \times A$$

$$C_{10} = 0.65$$

$$C_{100} = 0.65$$

Peak design flows

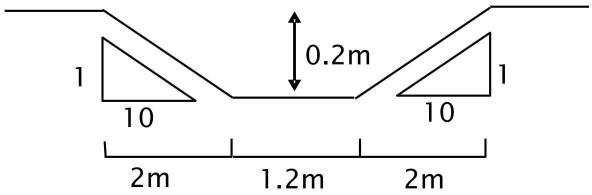
$$Q_{10} = 0.002788 \times 0.65 \times 190 \times 0.485 = 0.17 \text{ m}^3/\text{s}$$

$$Q_{100} = 0.002788 \times 0.65 \times 275 \times 0.485 = 0.24 \text{ m}^3/\text{s}$$

5.7.4 Step 3: Configuring the Swale

5.7.4.1 Swale Width and Side Slopes

The following cross section is proposed:



5.7.4.2 Maximum Length of Swale

The capacity of the swale is firstly estimated at the most downstream point. It is considered to be the critical point in the swale as it has the largest catchment and has the mildest slope. Flow velocities will also need to be checked at the downstream end of the steep section of swale.

The worked example firstly considers the swale capacity using a turf grass surface with a vegetation height of 100 mm. An extension of the worked example is to investigate the consequence of using 300mm tall vegetation (e.g. sedges) instead of grass.

A range of Manning's *n* values are selected for different flow depths appropriate for grass. It is firstly assumed that the flow height for a 10-year ARI storm will be above

[1] Please refer to Code of Practice for Surface Water Drainage

the vegetation and therefore Manning's *n* is quite low. A figure of 0.04 is adopted. The flow depth will need to be checked to ensure it is above the vegetation.

- Adopt slope 3% (minimum longitudinal slope)
- Manning's *n* = 0.04 (at 0.2m depth)
- Side slopes 1(v):10(h)

From Manning's equation: $Q = (AR^{2/3}S_o^{1/2})/n$

Q_{cap}= 0.683 m^3/s >> Q_{10} (0.17 m^3/s) ………………OK

The nominated swale has sufficient capacity to convey the required peak Q_{10} flow without any requirement for an additional piped drainage system (i.e. slope = 3%, n = 0.07, Q_{10} = 0.17 m^3/s), solving Manning's equation for depth, $d_{10\text{-year}}$ = 0.13 m.

The capacity of the swale (Q_{cap} = 0.683m^3/s) is also sufficient to convey the entire peak Q_{100} flow of 0.24m^3/s without impacting on the adjacent road and footpath (i.e. slope = 3%, n = 0.04, Q_{100} = 0.2425 m^3/s) and solving Manning's equation for depth gives $d_{100\text{-year}}$ = 0.143 m.

The flow depths of both the minor (0.13 m) and major (0.143 m) event flows are less than the depth of the swale (0.2 m), indicating that all flow is contained within the swales.

Based on this result, the maximum permissible length of swale is also much longer than the 'actual' length of the swale (i.e. 100 m) and as such no overflow pits are required except at the downstream end of the swale to facilitate discharge to the trunk underground pipe drainage system (see Chapter 6 for design of overflow pits).

To investigate flow rates at depths lower than the height of vegetation, Manning's *n* is varied according to the flow depth relating to the vegetation height. This can be performed simply in a spreadsheet application. The values adopted here are:

Table 5.2 Manning's n and flow capacity variation with flow depth – turf

Flow Depth (m)	Manning's n	Flow (m^3/s)
0.05	0.30	0.006
0.1	0.08	0.149
0.15	0.06	0.252
0.2	0.04	0.674

From the table of Manning's equation output (Table 5.2) it can be seen that the 10-year ARI flow depth is above the vegetation height and therefore the adopted Manning's *n* value of 0.07 is reasonable. The boundary layer effect created by the turf significantly decreases between a flow depth of 0.05 m and 0.1 m with Manning's *n* decreasing from 0.3 to 0.08. This is due to the weight of the water flowing over the grass causing it to 'yield over' creating a 'smoother' surface with less resistance to flow. Once the water depth has reached twice the vegetation height (0.2 m), the Manning's n roughness coefficient has been further reduced to 0.04.

For the purposes of this worked example, the capacity of the swale is also estimated when using 300mm tall vegetation (e.g. sedges). The taller vegetation will increase the roughness of the swale (as flow depths will be below the vegetation height) and therefore a higher Manning's *n* should be adopted. The table below presents the adopted Manning's *n* values and the corresponding flow capacity of the swale for different flow depths.

Table 5.3 Manning's n and flow capacity variation with flow depth – sedges

Flow Depth (m)	Manning's n	Flow (m^3/s)
0.05	0.35	0.004
0.1	0.32	0.002
0.15	0.30	0.05
0.2	0.30	0.09

It can be seen in Table 5.3 that the swale with current dimensions is not capable of conveying a 10-year discharge of 0.17 m^3/s if sedges are to be planted. Either the swale depth would need to be increased or overflow pits provided to allow excess water to bypass the swale.

This worked example continues using 100mm turf for the remainder.

5.7.5 Step 4: Design Inflow Systems

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

Direct runoff from the road enters the swale via a buffer (the grass edge of the swale). The pavement surface is set 60 mm higher than the start of the swale and has a taper that will allow sediments to accumulate off the pavement surface in the first section of the buffer. Flows from allotments will discharge into the base of the swale and localised erosion protection is provided with grouted rock at the outlet point of the pipe.

5.7.6 Step 5: Verification Checks

5.7.6.1 Vegetation scour velocity checks

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

Velocities are estimated using Manning's equation:

Firstly, velocities are checked at the most downstream location for the 10-year ARI (i.e. slope = 3%, n = 0.07, Q_{10} = 0.17 m^3/s)

$d_{10\text{-year}} = 0.13$ m

$V_{10\text{-year}} = 0.46$ m/s < 0.5 m/s therefore OK

Secondly, velocities are checked at the most downstream location for the 100-year ARI (i.e. slope = 3%, n = 0.04, Q_{100} = 0.24 m^3/s)

$d_{100\text{-year}} = 0.143$ m

$V_{100\text{-year}} = 0.645$ m/s < 2.0 m/s therefore OK

5.7.6.2 Velocity and Depth Checks - Safety

Check at critical points (bottom of entire swale) that velocity depth product is less than 0.4 during a 100-year ARI flow.

At bottom of swale:

V= 0.645 m/s, d= 0.143m; therefore V.d = 0.092 m^2/s <0.4 therefore OK.

5.7.6.3 Confirm Treatment Performance

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

5.7.7 Step 6: Size Overflow Pits

As the swale can carry a ten-year ARI discharge, overflow structures are not required for this worked example. See Chapter 6 for an example including the design of an overflow pit.

5.7.8 Step 7: Traffic Control

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

5.7.9 Step 8: Vegetation specification

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

5.7.10 Calculation summary

The sheet overleaf shows the results of the design calculations.

SWALES – DESIGN CALCULATION SUMMARY SHEET

	Calculation Task		CALCULATION SUMMARY Outcome		Check
	Catchment Characteristics (Swale 1)				
		Catchment Area	0.485	ha	✓
		Catchment Land Use (i.e. residential, Commercial etc.)	Res		
		Catchment Slope	3	%	
	Conceptual Design				
		Swale Top Width	5	m	✓
		Swale Length	100	m	
		Swale Location (road reserve/ park/other)	Road res		
		Road Reserve Width	13.5	m	
1	**Confirm Treatment Performance of Concept Design**				
		Swale Area	125	m^2	✓
		TSS Removal	68	%	
		TP Removal	45	%	
		TN Removal	10	%	
2	**Determine Design Flows**				
	Time of concentration				
	Swale 1				
		10 year ARI	10	minutes	✓
		100 year ARI	10	minutes	
	Swale 2				
		10 year ARI		minutes	✓
		100 year ARI		minutes	
	Identify Rainfall intensities				
	Swale 1				
		$I_{10\ year\ ARI}$	190	mm/hr	✓
		$I_{100\ year\ ARI}$	275	mm/hr	
	Swale 2				
		$I_{10\ year\ ARI}$		mm/hr	✓
		$I_{100\ year\ ARI}$		mm/hr	
	Design Runoff Coefficient				
		$C_{10\ year\ ARI}$	0.65		✓
		$C_{100\ year\ ARI}$	0.65		
	Peak Design Flows				
		10 year ARI	0.17	m^3/s	✓
		100 year ARI	0.24	m^3/s	
3	**Dimension the Swale**				
	Swale Width and Side Slopes				
		Base Width	1.0	m	✓
		Side Slopes – 1 in	10		
		Longitudinal Slope	3	%	
		Vegetation Height	100	mm	
	Maximum Length of Swale				
		Manning's *n*	0.04		✓
		Swale Capacity	0.63	m^3/s	
		Maximum Length of Swale	<100	m	
4	**Design Inflow Systems**				
		Swale Kerb Type	Flush		✓
		60 mm set down to Buffer/ Swale Vegetation	Yes	Yes/ No	
		Adequate Erosion and Scour Protection (where required)	N/A		
5	**Verification Checks**				
		Velocity for 10 year ARI flow (< 0.5 m/s)	0.46	m/s	✓
		Velocity for 100 year ARI flow (< 2 m/s)	0.65	m/s	
		Velocity x Depth for 100 year ARI (< 0.4 m^2/s)	0.09	m^2/s	
		Depth of Flow for 100 year ARI (< 0.3 m)	0.143	m	
		Treatment Performance consistent with Step 1	Yes		
6	**Size Overflow Pits (Field Inlet Pits)**				
		System to convey minor floods – Swale 1		L x W	✓
		System to convey minor floods – Swale 2		L x W	

5.8 References

Barling RD & Moore ID 1993, 'The Role of Buffer Strips in the Management of Waterway Pollution', in Woodfull J *et al*. (eds), *The Role of Buffer Strips in the Management of Waterway Pollution from Diffuse Urban and Rural Sources*, LWRRDC Occasional Paper No. 01/93, Canberra

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Engineers Australia 2006, *Australian Runoff Quality*, Engineers Australia, ACT,http://www.arq.org.au/

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Weibel SR, Weidner RB, Cohen JM & Christianson AG, 1996, 'Pesticides and Other Contaminants in Rainfall and Runoff', *Journal American Water Works Association*, vol. 58, no. 8, August 1966, pp. 1075-1084

Bioretention Swales 6

6

Chapter 6 Bioretention Swales

6.1 Introduction

Bioretention swales provide both stormwater treatment and conveyance functions. These systems consist of both components of a vegetated swale and a bioretention system. These components are subtly different in functions. The main function of the swale element is for conveyance of stormwater, while the primary function of the bioretention component is the promotion of soil filtration of stormwater. Typically, a bioretention swale would consist of a vegetated swale when the bioretention system is installed in the base of a swale. The swale may have a discharge capacity to convey stormwater flow for design events (i.e. up to the 10 year ARI event in accordance to the Singapore Code of Practice on Surface Water Drainage) or have overflow provision sized to by-pass design events to a drain with sufficient capacity.

The swale component provides pretreatment of stormwater to remove coarse to medium sediments while the bioretention system underneath removes finer particulates and associated contaminants. Figure 6.1 shows the cross-section of a bioretention swale. Bioretention swales provide flow retardation for frequent storm events and are particularly efficient at removing nutrients.

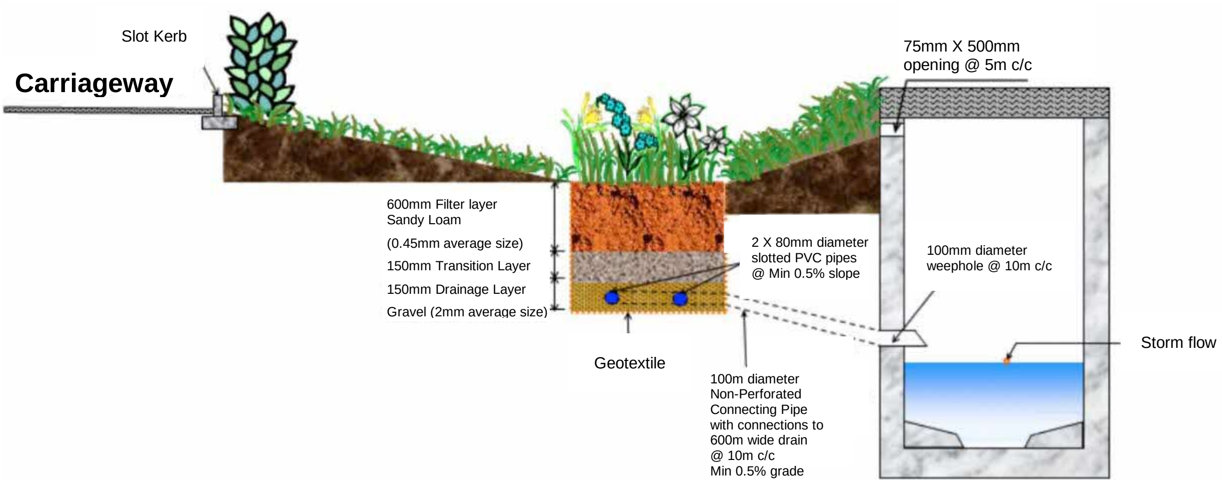


Figure 6.1 A typical Bioretention swale

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

Bioretention swales also act to reduce flow velocities compared with concrete drains and thus provide protection to natural receiving waterways from frequent storm events. The bioretention component is typically located at the downstream end of the overlying swale 'cell' (i.e. immediately upstream of the swale overflow pit(s) as shown in Figure 6.2 or can be provided as a continuous "trench" along the full length of a swale).

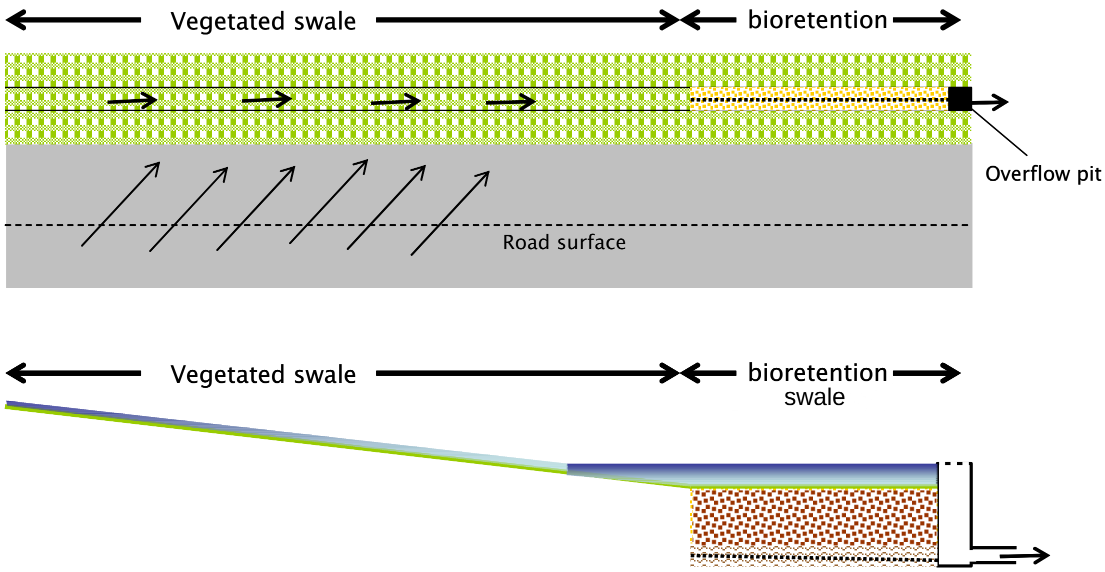


Figure 6.2 Bioretention Swale used downstream of vegetated swale

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

Bioretention swales should not be used as an 'infiltration' system to prevent excessive stormwater exfiltrate from the bioretention filter media to the surrounding in-situ soils. Rather, the typical design intent is to recover the percolated stormwater runoff at the base of the filter media, within perforated under-drains, for subsequent discharge to receiving waterways or to a storage facility for potential reuse. Thus these systems are suited even when close to structures as long as steps are taken to prevent exfiltration to surround soils through the use of a impervious liner where necessary.

In some circumstances however, where the in-situ soils are appropriate (i.e. have suitable permeability to avoid water stagnation) and there is a particular design intention to recharge local groundwater, it may be desirable to permit the percolated stormwater runoff to exfiltrate from the base of the filter media to the underlying in-situ soils.

6.2 Design Considerations for Bioretention Swales

This section outlines some of the key design considerations for bioretention swales that the designer should be familiar with. Standard design considerations for the swale component of bioretention swales are discussed in detail in Chapter 5 (Swales and Buffers) and are not reproduced here. However, swale design considerations that relate specifically to the interactions between the swale and bioretention components are presented in this chapter so as to provide sufficient clarity of these interactions with design considerations that are specifically related to the bioretention component.

Design considerations for the bioretention system are similar to that presented in Chapter 7 Bioretention Basins and are presented in both chapters for ease of reference with the exception of submerged zones which may be incorporated in bioretention swales to maximise treatment performance. Refer to Chapter 7.2 Key design configurations for further detail.

6.2.1 Landscape Design

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

6.2.2 Hydraulic Design

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

It is important to ensure that velocities in the bioretention swale are kept below 0.5 m/s for frequent runoff events (10 year ARI) and below 2.0 m/s for major (100 year ARI) runoff events to avoid scouring. This can be achieved by ensuring the slope and hydraulic roughness of the overlying swale reduce flow velocities by creating shallow temporary ponding (i.e. extended detention) over the surface of the bioretention filter media via the use of a check dam. This may also increase the overall volume of stormwater runoff that can be treated by the bioretention filter media.

6.2.3 Preventing Exfiltration to In-situ Soils

Bioretention swales can be designed to generally preclude exfiltration of treated stormwater to the surrounding in-situ soils. The amount of water potentially lost from bioretention trenches to surrounding in-situ soils is largely dependent on the characteristics of the surrounding soils and the saturated hydraulic conductivity of the bioretention filter media (see Section 6.2.5).

If the saturated hydraulic conductivity of the filter media is one to two orders of magnitude (i.e. 10 to 100 times) greater than that of the surrounding soil profile, the preferred flow path for stormwater runoff will be effectively contained within the bioretention filter media and into the perforated under-drains at the base of the filter media. As such, there will be little exfiltration to the surrounding soils.

If the selected saturated hydraulic conductivity of the bioretention filter media is less than 10 times that of the surrounding soils, it may be necessary to provide an impermeable liner. Flexible membranes or a concrete casting are commonly used to prevent excessive exfiltration. The greatest pathway of exfiltration is through the base of a bioretention trench. If lining is required, it is likely that only the base and the sides of the *drainage layer* (refer Section 6.2.5) will need to be lined.

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

Bioretention system built on highly porous landscape may suitably promote exfiltration to surrounding soils. In such circumstances, the designer must consider site terrain, hydraulic conductivity of the in-situ soil, soil salinity, groundwater and building setback.

6.2.4 Vegetation Types

Bioretention swales can use a variety of vegetation types including turf (swale component only), sedges and tufted grasses. Vegetation is required to cover the whole width of the swale and bioretention filter media surface, be capable of withstanding design flows and be of sufficient density to prevent preferred flow paths and scour of the media surface. Grass species that do not have fibrous or shallow roots should ideally be avoided as shallow rooted systems with inadequate penetration to the full depth of the filter media will not help to keep the permeability of filter media. Therefore it is preferred that the vegetation for the bioretention component of bioretention swales is sedges. A list of plants for use in the filtration area of bioretention systems is in 6.7 for reference. A CUGE (NParks) publication on “A selection of plants for bioretention systems in the tropics” can also be consulted for plant selection. The publication can be downloaded at https://www.cuge.com.sg/research/download.php?product=47.

Dense vegetation planted along the swale component can also offer improved sediment retention by reducing flow velocity and providing enhanced sedimentation for deeper flows. However, densely vegetated swales have higher hydraulic roughness and this will need to be considered in assessing their discharge capacity. Densely vegetated bioretention swales can become features of an urban landscape and once established, require minimal maintenance and can help to maintain soil porosity.

6.2.5 Bioretention Filter Media

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

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

The high rainfall intensities experienced in Singapore is expected to result in bioretention treatment areas being larger in Singapore than comparable systems overseas in Australia and the United States. The area available for bioretention swales in an urban layout is often constrained by factors such as the available area within the footpaths of standard road reserves.

Selecting bioretention filter media for bioretention swale applications in Singapore will often require careful consideration of saturated hydraulic conductivity and extended detention depth to ensure the desired minimum volume of stormwater runoff receives treatment. This must also be balanced with the requirement to ensure the saturated hydraulic conductivity does not become too high such that it can no longer sustain healthy vegetation growth.

The maximum saturated hydraulic conductivity (k_f) should not exceed 500 mm/hr (and preferably be between 100 - 300 mm/hr) in order to sustain vegetation growth. k_f less than 100 mm/hr (>50 mm/hr) could be accepted with caution.

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

As shown in Figure 6.3, a bioretention system can consist of three layers. The filter media is the primary soil layer consisting typically of sandy-loam material. In addition to the filter media, a drainage layer is also required to convey treated water from the base of the filter media to the outlet via a perforated under-drains unless the design intent is to allow the filtered water to discharge (exfiltrate) into insitu soil. The drainage layer surrounds perforated under-drains and consist typically of fine gravel of 2-5 mm particle size. In between the filter media layer and the drainage layer is the transition layer consisting of clean sand (1mm) to prevent migration of the base filter media into the drainage layer and into the perforated under-drains.

[Refer to the Bioretention Media Guidelines produced by FAWB[1] (2009) for more information.]

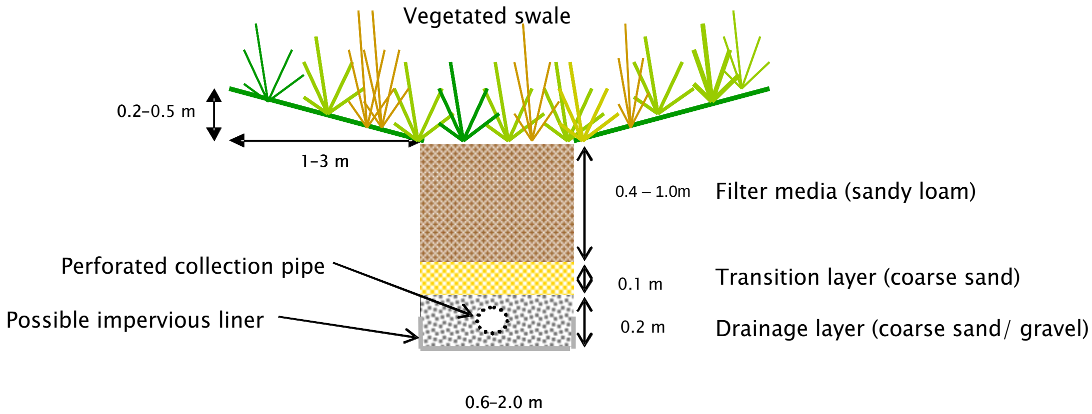


Figure 6.3 Typical Section of a Bioretention Swale

6.2.6 Traffic Controls

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

A staged construction and establishment method (see Section 6.4.2) affords protection to the sub-surface elements of a bioretention swale from heavily sediment laden runoff during the subsequent construction phases. However, to prevent vehicles driving on bioretention swales and inadvertent placement of building materials, it is necessary to consider appropriate traffic control solutions as part of the system design. These can include temporary fencing of the swale during the construction and allotment building

[1] Facility for Advancing Water Biofiltration – http://www.monash.edu.au/fawb/

phases with signage erected to alert builders and contractors of the purpose and function of the swales. Management of traffic near swales can be achieved in a number of ways such as planting the interface to the road carriageway with dense vegetation that will discourage the movement of vehicles onto the swale or, if dense vegetation cannot be used, by providing physical barriers such as kerb and channel (with breaks to allow distributed water entry to the swale) or bollards and/ or street tree planting.

Kerb with slots or drop inlet chambers should be used to convey road runoff to the bioretention swales. The transition from barrier type kerb to flush kerbs and vice versa is to be done in a way that avoids creation of low points that cause ponding onto the road pavement.

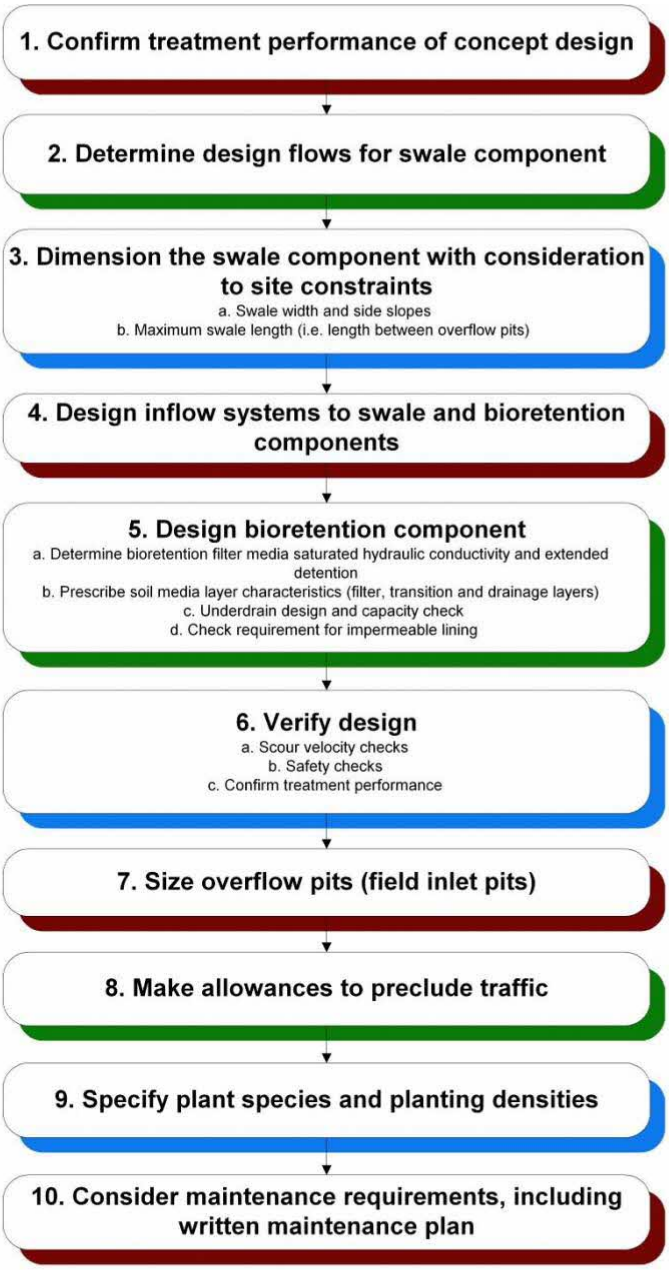
Where bioretention systems are used in road verge, the use of bioretention basins can allow for tree planting in between them.

6.2.7 Services

It is good to have bioretention systems not affected by any services. However, if this is not possible, selected services could locat beneath the batter of the bioretention swale, without affecting the filter layers and sub-soil pipes.

6.3 Bioretention Swale Design Process

To create bioretention swales, separate calculations are performed to design the swale and the bioretention system, with iterations to ensure appropriate criteria are met in each section. The calculations and decisions required to design the swale component are presented in detail in Chapter 5 Swales and Buffers and are reproduced in this chapter. This is to allow designers and assessors to consult with this chapter only for designing and checking bioretention swale designs. The key design steps are:



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

6.3.1 Step 1: Confirm Treatment Performance of Concept Design

Before commencing detailed design, the designer should first undertake a preliminary check to confirm the bioretention swale treatment area from the concept design is adequate to deliver the required level of stormwater quality improvement. A conceptual design of a bioretention basin is normally typically undertaken prior to detailed design. The performance of the concept design must be checked to ensure that stormwater treatment objectives will be satisfied.

The treatment performance curves shown in Figure 6.4 to Figure 6.6 reflect the treatment performance of the bioretention component only and will be conservative as they preclude the sediment and nutrient removal performance of the overlying swale component. Notwithstanding this, the performance of the swale component for nitrogen removal is typically only minor and thus the sizing of the bioretention component will typically be driven by achieving compliance with best practice load reduction targets for Total Nitrogen. Therefore, by using the performance curves below, the designer can be confident that the combined performance of the swale and bioretention components of a bioretention swale will be similar to that shown in the curves for total Nitrogen and will exceed that shown for Total Suspended Sediment and total Phosphorus.

These curves are intended to provide an indication only of appropriate sizing and do not substitute the need for a thorough conceptual design process. Nevertheless, it is a useful visual guide to illustrate the sensitivity of bioretention treatment performance to the ratio of bioretention treatment area and contributing catchment area. The curves allow the designer to make a rapid assessment as to whether the bioretention trench component size falls within the “optimal size range” or if it is potentially under or over-sized.

The curves in Figure 6.4 to Figure 6.6 show the total suspended solid (TSS), total phosphorus (TP) and total nitrogen (TN) removal performance for a typical bioretention basin design with the following configurations:

- Filter media saturated hydraulic conductivity (k) = 180 mm/hr (0.5 x 10^{-4} m/s) and 360mm/hr (1 x 10^{-4} m/s)
- Filter Media average particle size = 0.5mm
- Filter Media Depth = 0.6m
- Extended Detention Depth = from 0 mm to 300 mm

The curves in Figure 6.4 to Figure 6.6 are generally applicable to bioretention swale applications within residential, industrial and commercial land uses.

If the characteristics of the bioretention component of the bioretention swale concept design are significantly different to that described above, then the curves in Figure 6.4 to Figure 6.6 may not provide an accurate indication of treatment performance**.** In these cases, the detailed designer should use MUSIC or equivalent software to verify the performance of the bioretention swale.

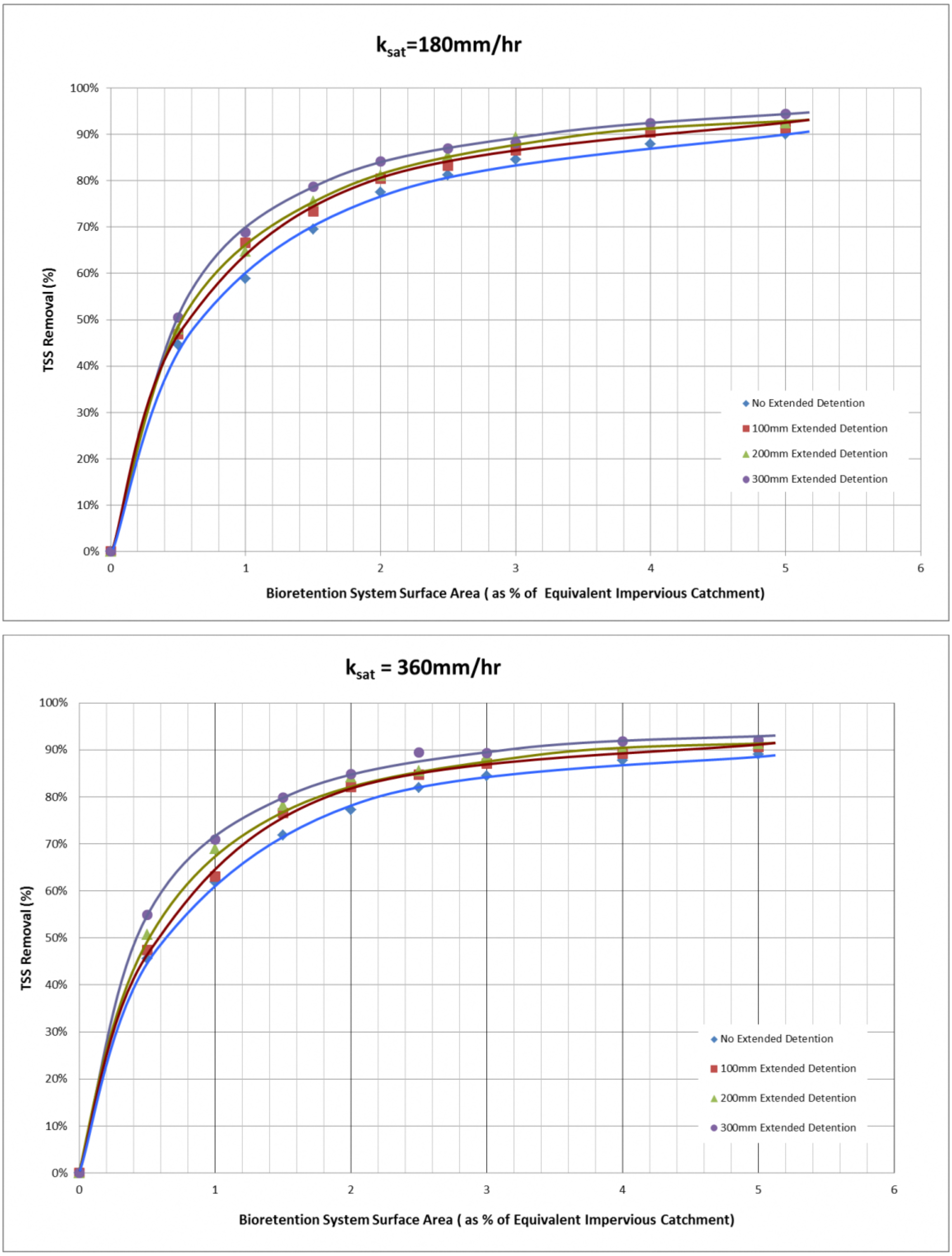
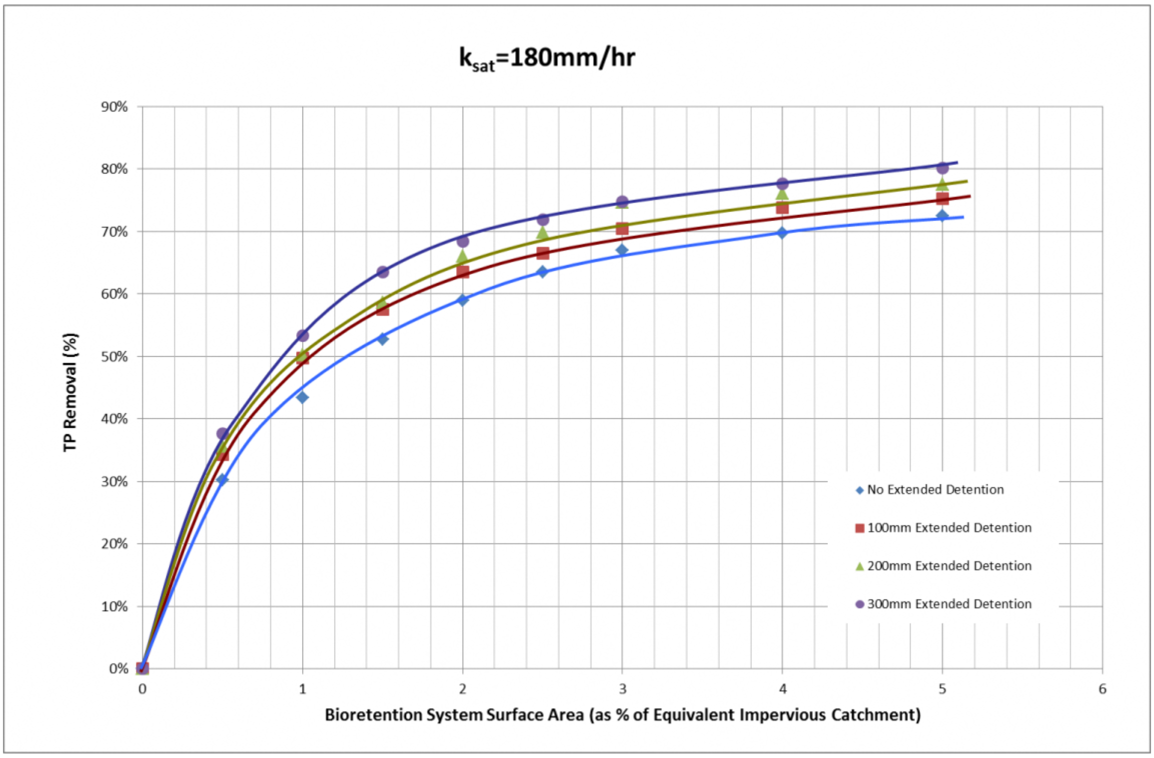


Figure 6.4 Bioretention system TSS removal performance (Reference: Station 43)



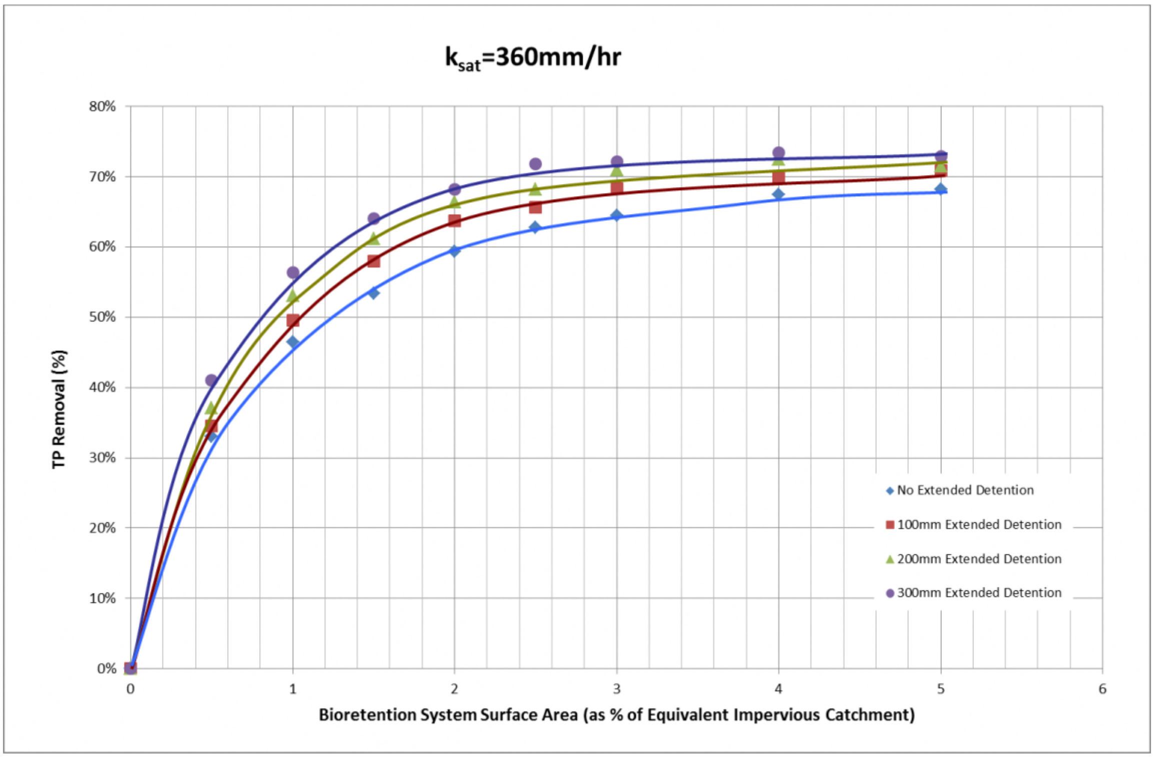
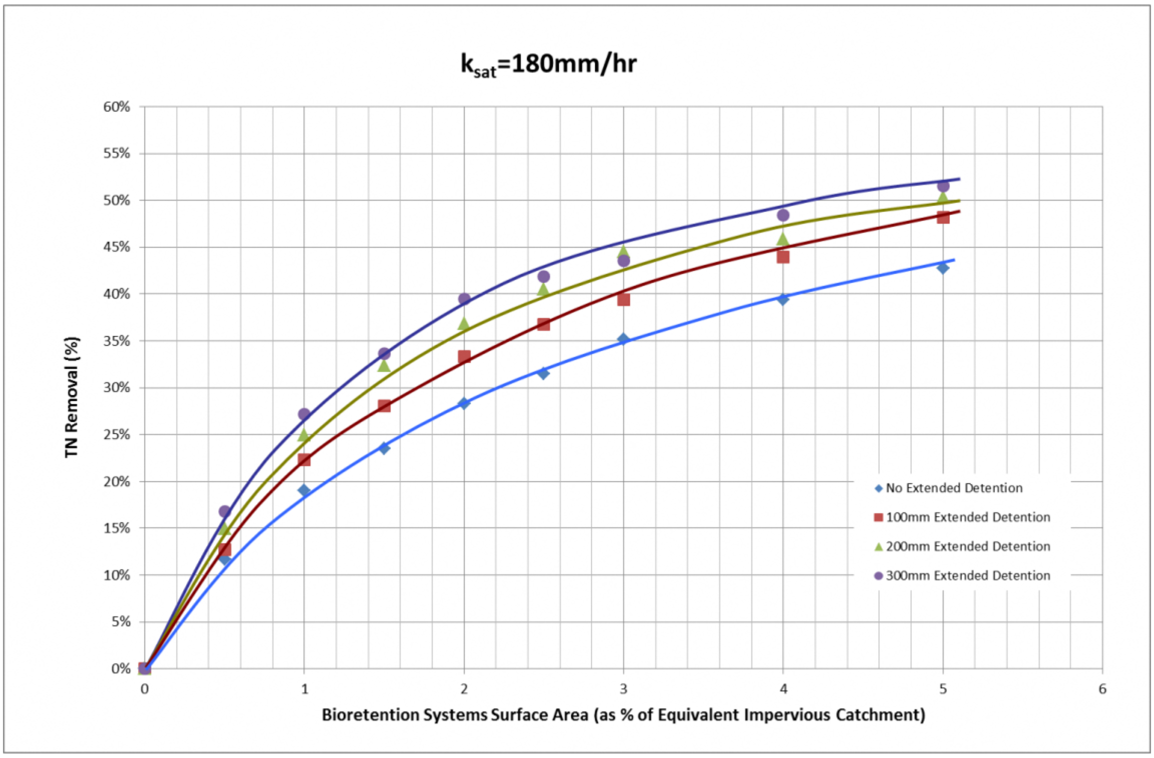


Figure 6.5 Bioretention system TP removal performance (Reference: Station 43)



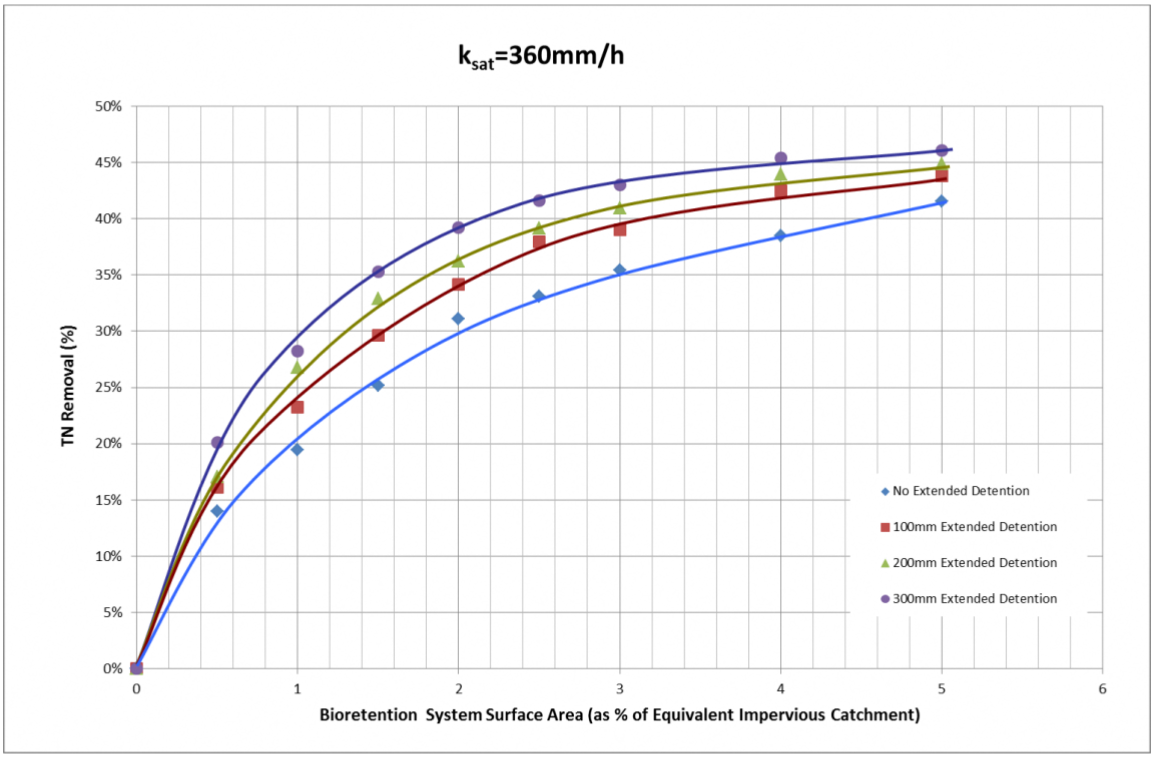


Figure 6.6 Bioretention system TN removal performance (Reference: Station 43)

6.3.2 Step 2: Determine Design Flows for the Swale Component

6.3.2.1 Design Flows

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

- Minor (frequent) storm conditions (typically 10 year ARI) to size the hydraulic structures to safely convey storm flows of frequent/minor events within the swale and not increase any flooding risk compared to conventional stormwater systems
- Major flood flow (100 year ARI) to check flow velocities, velocity depth criteria, conveyance within road reserve, and freeboard to adjoining property.

6.3.2.2 Design Flow Estimation

A range of hydrologic methods can be applied to estimate design flows. As the typical catchment area should be relatively small (<50 ha) the Rational Method design procedure is considered to be a suitable method for estimating design peak flows.

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

Factors to consider in defining the dimensions of the bioretention swale are:

- allowable width given the proposed road reserve and/ or urban layout
- how flows are delivered into a swale (e.g. cover requirements for pipes or kerb details)
- vegetation height
- longitudinal slope
- maximum side slopes and base width
- provision of crossings (elevated or at grade)
- requirements of the Public Utilities Board Code of Practice on Surface Water Drainage (latest edition).

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

6.3.3.1 Swale Width and Side Slopes

The maximum width of swale is usually determined from an urban layout and at the concept design stage, and should be in accordance with relevant local guidelines or standards of the Public Utilities Board. Where the swale width is not constrained by an urban layout (e.g. when located within a large parkland area) then the width of the swale can be selected based on consideration of landscape objectives, maximum side slopes for ease of maintenance and public safety, hydraulic capacity required to convey the desired design flow, and treatment performance requirements. Swale side slopes are typically between 1 in 10 and 1 in 4. The maximum swale width needs to be identified early in the design process as it dictates the remaining steps in the swale design process.

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

6.3.3.2 Maximum Length of a Swale

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

The maximum length of a swale located along a roadway is calculated as the distance along the swale to the point where flow on the adjoining road pavement (or road reserve) no longer complies with the local standards for road drainage (for both the minor and major flood flows). This is often related to the discharge capacity of the swale and is calculated as the distance along the swale to the point where the flow in the swale (for the specific design flood frequency) exceeds the bank full capacity of the swale. For example, if the swale is to convey the minor flood flow (typically the 10 year ARI event in accordance to the Singapore Code of Practice for Surface Drainage) without overflowing, then the maximum swale length would be determined as the distance along the swale to the point where the 10 year ARI flow from the contributing catchment is equivalent to the bank full flow capacity of the swale (bank full flow capacity is determined using Manning's equation as discussed section 6.3.3.3).

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

The flow capacity of a swale can be calculated using Manning's equation. This allows the flow rate (and flood levels) to be determined for variations in swale dimensions, vegetation type and longitudinal slope.

$$Q = \frac{A \cdot R^{2/3} \cdot S^{1/2}}{n} \qquad \text{Equation 6.1}$$

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

R = hydraulic radius (m)

S = channel slope (m/m)

n = roughness factor (Manning's n)

Q = flow (m^3/s)

Manning's *n* is a critical variable in Manning's equation relating to roughness of the channel. It varies with flow depth, channel dimensions and vegetation type. For constructed swale systems, typical Manning's n values are between 0.15 and 0.4 for flow depths shallower than the vegetation height (preferable for treatment) and significantly lower for flows with greater depth than the vegetation (e.g. 0.03 for flow depth more than twice the vegetation height).

Figure 6.7 shows a plot of Manning's *n* versus flow depth for a grass swale with longitudinal grade of 5 % which is also applicable for other swale configurations. The bottom axis of the plot has been modified from Barling and Moore (1993) to express flow depth as a percentage of vegetation height. Further discussion on selecting an appropriate Manning's *n* for a swale is provided in Appendix F of the *MUSIC User Guide* (eWater Ltd 2014).

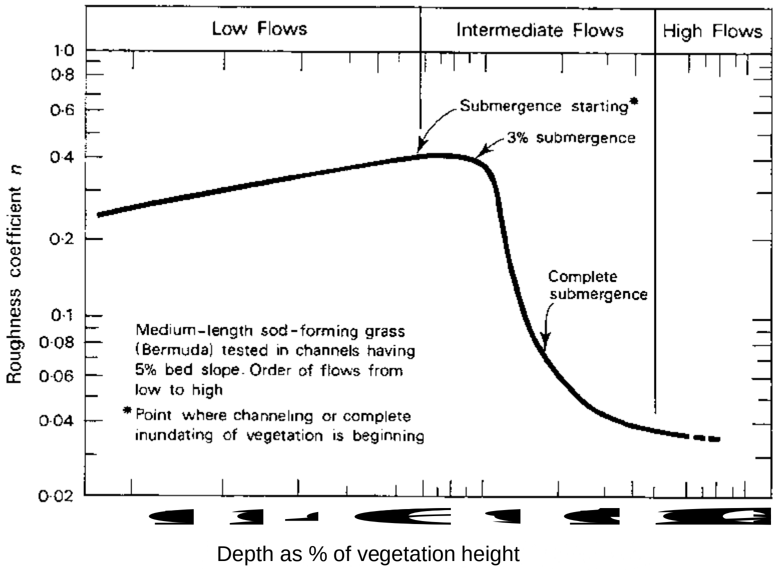


Figure 6.7 Impact of Flow Depth on Hydraulic Roughness (adapted from Barling and Moore (1993)

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

Inflows to bioretention swales can be via distributed runoff (e.g. from flush kerbs on a road) or point outlets such as pipe outfalls. Combinations of these inflow pathways can also be used. Uniform distribution of inflow would generally provide better operating conditions of bioretention swales owing to their long linear configuration.

6.3.4.1 Distributed Inflow

An advantage of flows entering a bioretention swale system in a distributed manner (i.e. entering perpendicular to the direction of the swale) is that flow depths are kept as shallow owing to sheet flow conditions. This maximises contact with the swale and bioretention vegetation, particularly on the batter (buffer strip) receiving the distributed inflows (see Figure 6.8). The buffer strip provides good pretreatment (i.e. significant coarse sediment removal) prior to flows being conveyed along the swale.

Distributed inflows can be achieved either by having a flush kerb or by using kerbs with regular breaks in them to allow for even flows across the buffer surface (Figure 6.9).

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

- providing distributed flows into a buffer (potentially spreading stormwater flows to achieve this)
- avoiding rilling or channelised flows
- maintaining flow heights lower than vegetation heights (this may require flow spreaders, or check dams)
- minimising the slope of buffer, best if slopes can be kept below 5 %, however buffers can still perform well with slopes up to 20 % provided flows are well distributed. The steeper the buffer the more likely flow spreaders will be required to avoid rill erosion (i.e. the removal of soil by concentrated water flow, and it occurs when the water forms small channels in the soil as it flows off site).

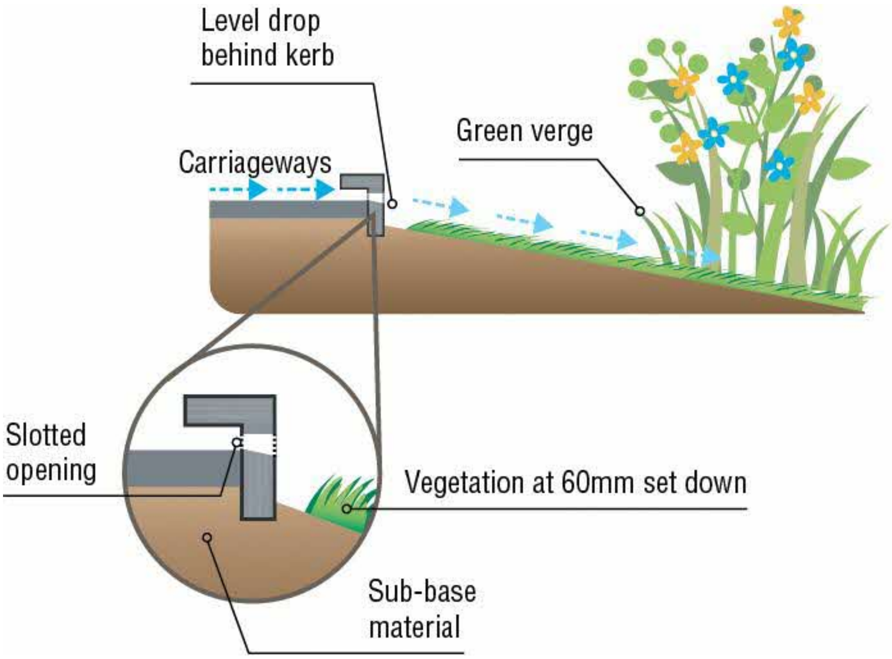


Figure 6.8 Slotted Kerbs with level drop or set-down allow Sediments to Flow into Vegetated Area

Figure 6.9 Kerb Arrangements with Breaks to Distribute Inflows on to Bioretention Swales and Prevent Vehicle Access

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

It is important to ensure coarse sediments accumulate off the road surface at the start of the buffer. Figure 6.10 shows sediment accumulating on a street surface where the vegetation is the same level as the road. To avoid this accumulation, a tapered flush kerb must be used that sets the top of the vegetation at approximately 60 mm below the road surface, which requires the top of the ground surface (before turf is placed) to be approximately 100 mm below the road surface. This allows sediments to accumulate off any trafficable surface.

Figure 6.10 Flush Kerb without Setdown, showing Sediment Accumulation on Road

6.3.4.2 Concentrated Inflow

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

The most common constraint on pipe systems discharging to bioretention swales is bringing the pipe flows to the surface of a swale. In situations where the swale geometry does not allow the pipe to achieve 'free' discharge to the surface of the swale, a 'surcharge' pit may need to be used. Surcharge pits should be designed so that they are as shallow as possible and have pervious bases or weep-holes to avoid long term ponding in the pits (this may require under-drains to ensure it drains, depending on local soil conditions). The pits need to be accessible so that any build up of coarse sediment and debris can be monitored and removed if necessary. Surcharge pits are not considered good practice due to additional maintenance issues and mosquito breeding potential and should therefore be avoided where possible.

Surcharge pit systems are most frequently used when allotment runoff is required to cross a road into a swale on the opposite side of the road or for allotment runoff discharging into shallow profile swales. Where allotment runoff needs to cross under a road to discharge to a swale, it is preferable to combine the runoff from more than one allotment to reduce the number of crossings required under the road pavement. Figure 6.11 illustrates a typical surcharge pit discharging into a swale. The design of the surcharge pit is for reference only. The actual design needs to be approved by relevant agencies and the party who will take over the maintenance.

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

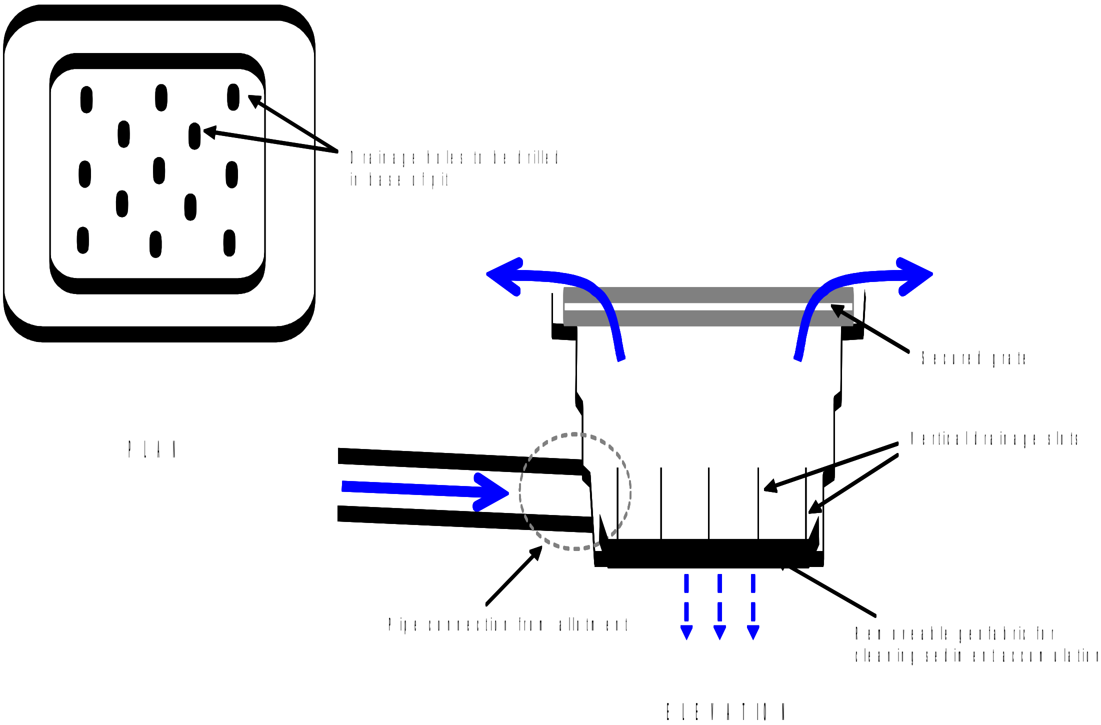


Figure 6.11 Example of Surcharge Pit for Discharging Allotment Runoff into a Swale

6.3.5 Step 5: Design Bioretention Component

6.3.5.1 Specify the Bioretention Filter Media Characteristics

Generally, three types of media are required in the bioretention component of bioretention swales (refer Figure 6.3 in Section 6.2.5).

<u>Filter Media</u>

The filter media layer provides the majority of the pollutant treatment function, through fine filtration and also by supporting vegetation. The vegetation enhances filtration, keeps the filter media permeable, provides substrate for biofilm formation that is important for the uptake and removal of nutrients and other stormwater pollutants. It is important to have a good plant density on filter media. As a minimum, the filter media is required to have sufficient depth to support vegetation. Typical depths are between 600-1000 mm with a minimum depth of 400mm accepted in depth constrained situations. It is important to note that if deep rooted plants such as trees are to be planted in bioretention swales, the filter media must have a minimum depth of 1000 mm to provide sufficient plant anchoring depth.

Saturated hydraulic conductivity should be between 100-300 mm/hr (and should not be greater than 500 mm/hr). Saturated hydraulic conductity between 50 and 100 mm/hr can be accepted with caution. The following procedure is recommended in determining the appropriate soil filter media to match the design saturated hydraulic conductivity:

- Identify available sources of a suitable base soil (i.e. topsoil) capable of supporting vegetation growth such as a sandy loam or sandy clay loam. In-situ topsoil should be considered first before importing soil. Any soil found to contain high levels of salt (see last bullet point), extremely low levels of organic carbon (< 3%), or other extremes considered retardant to plant growth and microbial activity should be rejected. The base soil must also be free from pollutants like heavy metals, excessive nutrient and organic pollutants that may affect water quality of the filtrate.

- Using laboratory analysis, determine the saturated hydraulic conductivity of the base soil using standard testing procedures. (In Australia, reference is made to AS 4419-2003 Appendix H Soil Permeability or refer to Constant head method BS1377-5:1990 for Singapore). A minimum of five samples of the base soil should be tested. Any occurrence of structural collapse during laboratory testing must be noted and an alternative base soil sourced.

- To amend the base soil to achieve the desired design saturated hydraulic conductivity either mix in a loose non-angular sand (to increase saturated hydraulic conductivity) or conversely a loose loam (to reduce saturated hydraulic conductivity).

- The required content of sand or clay (by weight) to be mixed to the base soil will need to be established in the laboratory by incrementally increasing the content of sand or clay until the desired saturated hydraulic conductivity is achieved. The sand or clay content (by weight) that achieves the desired saturated hydraulic conductivity should then be adopted on-site. A minimum of five samples of the selected base soil and sand (or clay) content mix must be tested in the laboratory to ensure saturated hydraulic conductivity is consistent across all samples. If the average saturated hydraulic conductivity of the final filter media mix is within ± 20% of the design saturated hydraulic conductivity then the filter media can be adopted and installed in the bioretention system. Otherwise, further amendment of the filter media must occur through the addition of sand (or clay) and retested until the design saturated hydraulic conductivity is achieved.

- The filter media must be structurally sound and not prone to structural collapse as this can result in a significant reduction in saturated hydraulic conductivity. The risk of structural collapse can be reduced by ensuring the soil has a well graded particle size distribution with a combined clay and silt fraction of < 12%.

- The base soil must have sufficient organic content to establish vegetation on the surface of the bioretention system. If the proportion of base soil in the final mix is less than 3%, it may be necessary to add organic material. This should not result in more than 10% organic content and should not alter the saturated hydraulic conductivity of the final filter media mix.

- The pH of the final filter media is to be amended (if required) to between 5.5 and 7.5. If the filter media mix is being prepared off-site, this amendment should be undertaken before delivery to the site.

- The salt content of the final filter media (as measured by EC1:5) must be less than 0.63 dS/m for low clay content soils like sandy loam. (EC1:5 is the electrical conductivity of a 1:5 soil/ water suspension).

- Testing of this soil property should be undertaken prior to their placement during construction. It should also be noted that soil hydraulic conductivity will vary after placement and is expected to initially decrease due to hydraulic compaction during operation. With maturity of plant growth, the soil hydraulic conductivty canbe expected to recover to asymptote to an equilibrium level comparable to its original value.

The selection of suitable soil filter media is a topic of continuing research. Further information can also be obtained from “Guidelines for Filter Media for Biofiltration System by FAWB (Facility for Advancing Water Biofiltration).

Transition Layer

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

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

The transition layer is recommended to be 100 mm thick.

The addition of a transition layer increases the overall depth of the bioretention system and may be an important consideration for some sites where total depth of the bioretention system may be constrained. In such cases, two options are available to reduce the overall depth of the system, ie.

- the use of a sand drainage layer and/or perforated pipes with smaller slot sized may need to be considered (Section **Error! Reference source not found.**).
- use a geotextile layer with a mesh size specified to be between 0.7 to 1mm. (This option should be an option of last resort as the risk of installing inappropriate liner is high).

Drainage Layer

The drainage layer is used to convey treated flows to the outlet via a perforated under-drainage system. The composition of the drainage layer is to be considered in conjunction with the selection and design of the perforated under-drainage system (refer to Section **Error! Reference source not found.**) as the slot sizes in the perforated pipes may determine the minimum drainage layer particle size to avoid washout of the drainage layer into the perforated pipe system.

Gravel is the preferred media for the drainage layer to match with the typical slot size of typical perforated or slotted under-drains.

However, there may be circumstances where site conditions constraint the depth of the bioretention system. In such cases, it may be possible to use sand as the drainage layer media to avoid having to provide a transition layer between the filter media and the drainage layer. The drainage layer is to be a minimum of 200 mm thick and it is advisable that the drainage media is washed prior to placement in bioretention system to remove any fines.

6.3.5.2 Under-drain Design and Capacity Checks

The maximum spacing of the perforated pipes in wide bioretention trenches is 1.5 m (centre to centre) to ensure effective drainage of the bioretention system.

By installing parallel pipes, the capacity of the perforated pipe under-drain system can be increased. The recommended maximum diameter of the perforated pipes is 100 mm to minimise the required thickness of the drainage layer. Either flexible perforated pipe (e.g. agricultural pipe) or slotted PVC pipes can be used, however care needs to be taken to ensure that the slots in the pipes are not too large that sediment would freely flow into the pipes from the drainage layer. This is also a consideration when specifying the drainage layer media.

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

- Ensure perforations are adequate to pass the maximum filtration rate of the media.
- Ensure the pipe itself has capacity to convey the design flow (ie. the maximum filtration rate multiplied by the surface area).
- Ensure that the material in the drainage layer will not be washed into the perforated pipes.

6.3.5.3 Maximum filtration rate

The maximum filtration rate represents the maximum rate of flow through the bioretention filter media and is calculated by applying Darcy's equation (Equation 6.2) as follows:

$$Q_{max} = K_{sat} \cdot L \cdot W_{base} \cdot \frac{h_{max} + d}{d} \qquad \text{Equation 6.2}$$

Where

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

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

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

L = length of the bioretention zone (m)

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

d = depth of filter media (m)

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

To ensure the perforated under-drainage system has sufficient capacity to collect and convey the maximum infiltration rate, it is necessary to determine the inflow capacity of combined slotted area or perforation area of the under-drainage system. To do this, the sharp edged orifice equation can be used, i.e.

- the number and size of perforations is determined (typically from manufacturer's specifications)
- the maximum driving head (being the depth of the filtration media plus the depth of extended detention).
- it is conservative but reasonable to use a blockage factor to account for partial blockage of the perforations by the drainage layer media. A 50 % blockage of the perforation is recommended. The orifice equation is expressed as follows:-

$$Q_{perf} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h} \qquad \text{Equation 6.3}$$

Where

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

B = blockage factor (0.5)

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

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

g = gravity (9.81 m/s^2)

h = head above the perforated pipe (m)

It is essential that adequate inflow capacity is provided to enable the filtered water to drain freely into the drainage layer.

After confirming the capacity of the under-drainage system to collect the maximum filtration rate, it is then necessary to confirm the conveyance capacity of the under-drainage system is sufficient to convey the collected runoff. To do this, Manning's equation (Equation 6.1) can be used assuming pipe full flow conditions and a nominal friction slope of 0.5%. The Manning's roughness used will be dependent on the type of pipe used.

One end of the under-drains should be extended vertically to the surface of the bioretention system to allow inspection and maintenance when required. The vertical section of the under-drain should be a non-perforated or slotted pipe and capped to avoid short circuiting of flows directly to the drain.

6.3.5.4 Check Requirement for Impermeable Lining

The saturated hydraulic conductivity of the natural soil profile surrounding the bioretention system should be tested together with depth to groundwater, chemical composition and proximity to structures and other infrastructure. This is to establish if an impermeable liner is required at the base (only for systems designed to preclude exfiltration to in-situ soils) and/or sides of the bioretention basin (refer also to discussion in Section 6.2.5). If the saturated hydraulic conductivity of the filter media in the bioretention system is more than one order of magnitude (10 times) greater than that of the surrounding in-situ soil profile, no impermeable lining is required.

6.3.6 Step 6: Verify Design

6.3.6.1 Vegetation Scour Velocity Check

Potential scour velocities are checked by applying Manning's equation (Equation 6.1) to the bioretention swale design to ensure the following criteria are met:

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

6.3.6.2 Velocity and Depth Check – Safety

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

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[2] This is consistent with the recommendation in the Singapore Code of Practice for Surface Drainage which stipulates that the maximum velocity for an earth drain and concrete-lined drain should not exceed 1.5 m/s and 3 m/s respectively.

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

6.3.6.3 Confirm Treatment Performance

If the previous two checks are satisfactory then the bioretention swale design is satisfactory from a conveyance function perspective and it is now necessary to confirm the treatment performance of the bioretention swale by reference to the performance information presented in Section 6.3.1

6.3.7 Step 7: Size Overflow Pit

In a bioretention swale system, overflow pits are used to control innundation depth. The crest of the pit is set raised above the surface of the bioretention filter media to establish the design extended detention depth.

Grated pits are typically used and the allowable head for discharges into the pits is the difference in level between the pit crest and the maximum permissible water level to satisfy the minimum freeboard requirements of the Public Utilities Board. Depending on the location of the bioretention swale, the design flow to be used to size the overflow pit could be the maximum capacity of the swale, the minor flood flow (10 year ARI) or the major flood flow (100 year ARI).

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

For free overfall conditions (weir equation):

$$Q_{weir} = B \cdot C_w \cdot L \cdot h^{3/2} \qquad \text{Equation 6.4}$$

Where
Q_{weir} = Flow into pit (weir) under free overfall conditions (m^3/s)
B = Blockage factor (= 0.5)
C_w = Weir coefficient (= 1.7)
L = Length of weir (perimeter of pit) (m)
h = Flow depth above the weir (pit) (m)

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

For drowned outlet conditions (orifice equation):

$$Q_{orifice} = B \cdot C_d \cdot A\sqrt{2 \cdot g \cdot h} \qquad \text{Equation 6.5}$$

Where *B*, *g* and *h* have the same meaning as in Equation 6.4
$Q_{orifice}$ = flow rate into pit under drowned conditions (m^3/s)
C_d = discharge coefficient (drowned conditions = 0.6)
A = area of orifice (perforations in inlet grate) (m^2)

When designing grated field inlet pits, refer to relevant guidelines or standards for grate types for inlet pits.

6.3.8 Step 8: Make Allowances to Preclude Traffic on Swales

Refer to Section 6.2.6 for discussion on traffic control options.

6.3.9 Step 9: Specify Plant Species and Planting Densities

Refer to Section 6.2.4 and the National Parks Board of Singapore for advice on selecting suitable plant species for bioretention swales in Singapore. Consultation with landscape architects is recommended when selecting vegetation to ensure the treatment system compliments the landscape design of the area. It is also good to check with the party who will take over the landscape maintenance (e.g. Town Councils) regarding plant species selection.

6.3.10 Step 10: Consider Maintenance Requirements

Consider how maintenance is to be performed on the bioretention swale e.g. how and where is access available, where is litter likely to collect etc. A specific maintenance plan and schedule should be developed for the bioretention swale in accordance with Section 6.5, and hand over to the party who will take over the maintenance.

6.3.11 Design Calculation Summary

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

BIORETENTION SWALES DESIGN CALCULATION SUMMARY

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

6.3.12 Typical Design Parameters

Table 6.1 shows typical values for a number of key bioretention swale design parameters.

Table 6.1: Typical Design Parameters for Bioretention Swales

Design Parameter	Typical Values
Swale longitudinal slope	1% to 4 %
Swale side slope for trafficability (with ‘at grade’ vehicular crossover)	Maximum 1 in 9
Swale side slope	Maximum 1 in 3
Manning’s *n* (with flow depth lower than vegetation height)	0.15 to 0.3
Manning’s *n* (with flow depth greater than vegetation height)	0.03 to 0.05
Maximum velocity for scour in minor event (e.g. 10 yr ARI)	0.5 m/s
Maximum velocity for 100 yr ARI	2.0 m/s
Perforated pipe diameter	100 mm (maximum)
Drainage layer average material diameter (typically fine gravel or coarse sand)	2-5 mm diameter
Transition layer average material diameter typically sand to coarse sand	0.7 – 1.0 mm diameter

6.4 Construction advice and checking tools

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

Checklists are provided for:

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

6.4.1 Design Assessment Checklist

The checklist overleaf below 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, referral should be made back to the design procedure to determine the impact of the omission or error.

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

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

BIORETENTION SWALE DESIGN ASSESSMENT CHECKLIST			
Asset I.D.		Assessed by:	Date:
Bioretention Location:			
Hydraulics:	Minor Flood (m^2/s):	Major Flood (m^2/s):	
Area:	Catchment Area (ha):	Bioretention Area (m^2):	

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

6.4.2 Construction Advice

This section provides general advice for the construction of bioretention basins.

6.4.2.1 Clean filter media

Ensure sand and gravel media is washed to remove fines prior to placement.

6.4.2.2 Perforated Pipes

Suitable perforated pipes can be either a PVC pipe with slots cut into the length of it or a flexible corrugated HDPE pipe with holes or slots distributed across its surface. PVC pipes have the advantage of being stiffer with less surface roughness therefore greater flow capacity; however the slots are generally larger than 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 may have the disadvantage of roughness (therefore lower flow capacity) but 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.

6.4.2.3 Tolerances

It is importance to stress the importance of tolerances in the construction of bioretention swales (e.g base, longitudinal and batters) – having flat surfaces is particularly important for a well distributed flow path and even ponding over the surfaces. Generally, a tolerance of 50mm in surface levels is acceptable.

6.4.2.4 Building Phase Damage

Protection of filtration media and vegetation is important during the building phase. Uncontrolled building site runoff is likely to cause excessive sedimentation, introduce weeds and litter and require replanting following the building phase. Where possible, a staged implementation should be adopted, i.e. during the site development/construction phase, use geofabric and some soil and instant turf (lay perpendicular to flow path) to provide erosion control and sediment trapping. Following the building phase, temporary measures and sediments would be removed and bioretention swale is revegetated in accordance with design planting schedule. It is also possible to reuse the instant turf in the subsequent stages.

If these systems are not staged to be part of the sediment control system during construction, it is advisable that stormwater flow during the site construction phases be diverted around the bioretention swales to sediment controls system to avoid smothering of planted vegetation by sediment loads from the construction site.

6.4.2.5 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) that can block filtration media is delivered onto the surface of the bioretention filter media. Washdown wastes (e.g. concrete) can also cause blockage of filtration media and damage vegetation. Bioretention areas should be fenced off during building phase and controls implemented to avoid washdown wastes.

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

6.4.2.6 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 60mm to the top of vegetation (if turf) is adequate. Therefore, total set down to the base soil is approximately 100 mm (with approximately 40mm turf on top of base soil).

6.4.2.7 Inlet Erosion Checks

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

6.4.2.8 Erosion Control

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

6.4.2.9 Timing for Planting

Timing of vegetation is dependent on the timing in relation to the phases of development. For example, temporary planting during construction for sediment control (e.g. with turf) is removed and the bioretention system planted out with long term vegetation. Alternatively, temporary planting (eg. turf or sterile grass) can be used until a suitable season for appropriate long-term vegetation. Ideally, long term vegetation should be planted when the surrounding soil has been stabilised.

6.4.2.10 Weed Control

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

6.4.2.11 Watering

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

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

After this initial three-month period, watering may not be required anymore, except during dry period. Watering requirements to sustain healthy vegetation should be determined during ongoing maintenance site visits.

6.4.3 Construction checklist

BIORETENTION SWALE CONSTRUCTION INSPECTION CHECKLIST			
Asset I.D.		**Inspected by:**	
Site:		**Date:**	
		Time:	
Constructed by:		**Weather:**	
		Contact during site visit:	

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

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

COMMENTS ON INSPECTION

ACTIONS REQUIRED
1.
2.
Inspection officer signature:

6.4.4 Asset transfer checklist

BIORETENTION SWALE ASSET TRANSFER CHECKLIST		
Asset I.D.:		
Asset Location:		
Construction by:		
Defects Liability Period:		
TREATMENT	**Y**	**N**
System appears to be working as designed visually?		
No obvious signs of under-performance?		
MAINTENANCE	**Y**	**N**
Maintenance plans and indicative maintenance costs provided for each asset?		
Vegetation establishment period completed (as per LGA requirements)?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
ASSET INSPECTED FOR DEFECTS AND/OR MAINTENANCE ISSUES AT TIME OF ASSET TRANSFER		
Sediment accumulation at inflow points?		
Litter within swale?		
Erosion at inlet or other key structures?		
Traffic damage present?		
Evidence of dumping (e.g. building waste)?		
Vegetation condition satisfactory (density, weeds)?		
Watering of vegetation required?		
Replanting required?		
Mowing/slashing required?		
Clogging of drainage points (sediment or debris)?		
Evidence of ponding?		
Damage/vandalism to structures present?		
Surface clogging visible?		
Drainage system inspected?		
COMMENTS/ACTIONS REQUIRED FOR ASSET TRANSFER		
ASSET INFORMATION	**Y**	**N**
Design Assessment Checklist provided?		
As constructed plans provided?		
Copies of all required permits (both construction and operational) submitted?		
Proprietary information provided (if applicable)?		
Digital files (eg drawings, survey, models) provided?		
Asset listed on asset register or database?		

6.5 Maintenance Requirements

Bioretention swales have a flood conveyance role that needs to be maintained to ensure adequate flood protection for local properties. In this regard, a key maintenance requirement is ensuring that the shape of the swale is maintained and that the swale is not subject to erosion or excessive deposition of debris that may impede the passage of stormwater or increase its hydraulic roughness from that assumed.

Vegetation plays a key role in maintaining the porosity of the soil media of the bioretention system and a strong healthy growth of vegetation is critical to its performance.

The most intensive period of maintenance is during the plant establishment period (first 3-6 months) when weed removal and replanting may be required. It is also the time when large loads of sediments could impact on plant growth, particularly in developing catchments with an inadequate level of erosion and sediment control.

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

Typical maintenance of bioretention swale elements will involve:

- Routine inspection of the swale profile to identify any areas of obvious increased sediment deposition, scouring of the swale invert from storm flows, rill erosion of the swale batters from lateral inflows, damage to the swale profile from vehicles and clogging of the bioretention trench (evident by a ‘boggy’ swale invert).
- Routine inspection of inlet points (if the swale does not have distributed inflows), surcharge pits and field inlet pits to identify any areas of scour, litter build up and blockages.
- Removal of sediment where it is impeding the conveyance of the swale and/ or smothering the swale vegetation, and if necessary, reprofiling of the swale and revegetating to original design specification.
- Repairing any damage to the swale profile resulting from scour, rill erosion or vehicle damage.
- Tilling of the bioretention trench surface if there is evidence of clogging.
- Clearing of blockages to inlet or outlets.
- Regular watering/ irrigation of vegetation until plants are established and actively growing (see section 6.4.2.11).
- Mowing of turf or slashing of vegetation (if required) to preserve the optimal design height for the vegetation.
- Removal and management of invasive weeds.
- Removal of plants that have died and replacement with plants of equivalent size and species as detailed in the plant schedule.
- Pruning to remove dead or diseased vegetation material and to stimulate new growth.

- Litter and debris removal.
- Vegetation pest monitoring and control.

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

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

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

An example of operation and maintenance inspection form is included in the checking tools provided in Section 6.5.1.

6.5.1 Operation & Maintenance Inspection Form

The form below summarises the basic maintenance items and suggested frequencies for Bioretention Swales. The ABC Waters Professional should customise the items and frequencies according to their design and project requirements. The customised form should be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

BIORETENTION SWALE MAINTENANCE CHECKLIST				
Asset I.D.				
Inspection Frequency:	Weekly to monthly	**Date of Visit:**		
Location:				
Description:				
Site Visit by:				
INSPECTION ITEMS	**FREQUENCY**	**Y**	**N**	**ACTION REQUIRED (DETAILS)**
Sediment accumulation at inflow points?	Weekly			
Litter within swale?	Weekly			
Erosion at inlet or other key structures (eg crossovers)?	Weekly			
Traffic damage present?	Weekly			
Evidence of dumping (eg building waste)?	Weekly			
Clogging of drainage points (sediment or debris)?	Weekly			
Evidence of ponding?	Weekly			
Drainage system inspected?	Weekly			
Surface clogging visible?	Weekly			
Vegetation condition satisfactory (density, weeds etc)?	Monthly			
Replanting required?	Monthly			
Keeping the maximum plant height at _ mm.	Fortnightly			
Set down from kerb still present?	Monthly			
Damage/vandalism to structures present?	Monthly			
Soil additives or amendments required?	Monthly			
Pruning and/ or removal of dead or diseased vegetation required?	Monthly			
Flushing of sub-soil pipes.	Half-yearly			
Resetting of system required?	Monthly			
Maintaining the cross section profile and longitudinal profile/slope.	Monthly			
COMMENTS				

Name of ABC Waters Professional: _______________

Registration No. of ABC Waters Professional: _______________

Signature: _______________

Name of Maintenance Agency: _______________

Handing Over Date (TOP or Completion of DLP): _______________

Drawing No. for Location Plan and Sectional Plans (X-section and long section) for All Bioretention Swales:

1. _______________
2. _______________
3. _______________

6.6 Bioretention swale worked example

6.6.1 Worked Example Introduction

Modelling using MUSIC was undertaken in developing a stormwater quality treatment system for a residential estate. 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 6.12. A photograph of a similar bioretention swale in a median strip is shown in Figure 6.13 (although in that example the vegetation cover of the swale and bioretention system is all grass).

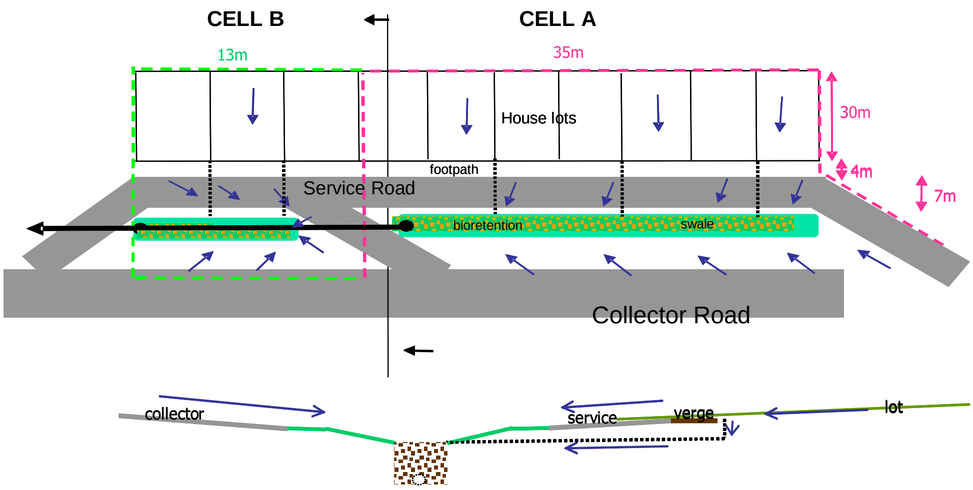


Figure 6.12 Catchment area layout and section for worked example

Figure 6.13 Photograph of bioretention swale

6.6.1.1 Site Description

The site comprised of the arterial road and a service road separated by a median of some 6m width. The median area offers the opportunity for a local treatment measure. The area available is relatively large in relation to the catchment and is elongated in shape. The catchment area for the swale and bioretention area includes the road reserve and the adjoining allotment (of approximately 30m 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 a 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 can be expected to be trapped by vegetation on the swale surface. Stormwater inflow exceeding the filtration rate of the soil media in the bioretention system will temporarily pond on 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 overflow into the piped drainage system at the downstream end of each bioretention cell.

Simulation using MUSIC found that the required area of bioretention system to meet a desired target of 80% reduction in TSS and 45% reduction in TP and TN 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.

6.6.1.2 Design Objectives

The design treatment objectives for the bioretention swale are as follows:-

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

6.6.1.3 Constraints and Concept Design Criteria

The constraints and design criteria are as follows:-

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

6.6.1.4 Site Characteristics

Key site characteristics are summarised as follows:-

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

	Allotments	Collector road	Local road	Footpath	Swale	Total
Cell A	35m x 30m	35m x 7m	35m x 7m	35m x 4m	103m x 7.5m	1680m^2
Cell B	13m x 30m	13m x 7m	13m x 7m	13m x 4m	44m x 7.5m	624m^2

6.6.2 Step 1: Confirm Treatment Performance of Concept Design

Nominated bioretention areas for Cell A and Cell B are 61m^2 and 22m^2 respectively. The equivalent imperious area for cell A is 1344m^2 (0.8 x 1680) and for Cell B is 499m^2 (0.8 x 624). Interpretation of Figure 6.4 to Figure 6.6 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.

- 200mm extended detention
- treatment area to impervious area ratio:
- Cell A - 61m^2/ 1344 m^2 = 4.5%
- Cell B - 22m^2/ 499 m^2 = 4.4%

From the graphs, the expected pollutant reductions are 93%, 77% and 49% for TSS, TP and TN respectively and exceed the design requirements of 80%, 45% and 45%.

6.6.3 Step 2: Estimate Design Flows for Swale Component

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

Time of concentration (t_c)

Cell A and Cell B are effectively separate elements for the purpose of sizing the swales for flow capacity and inlets to the piped drainage system 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 concluded the flow along the collector road was the longest and was adopted for t_c.
- Cell B – the t_c calculations include overland flow across the lots and road and swale/bioretention flow time.

The following t_c values are estimated:

t_c Cell A: 10 mins

t_c Cell B: 8 mins

Design rainfall intensities

Adopted from IDF Chart for Singapore

Design ARI	Cell A (10 min t_c)	Cell B (8 min t_c)
10	190 mm/hr	200 mm/hr
100	275 mm/hr	283 mm/hr

Runoff Coefficient

The runoff coefficients adopted were in accordance to those for a densely built-up urban area, as outlined in Code of Practice on Surface Water Drainage (Public Utilities Board 2013).

Design ARI	Cell A	Cell B
10	0.8	0.8
100	0.8	0.8

Design Flows

The design flows for the two cells, computed using the Rational Method (Q = 0.00278. C.I.A) are summarised below:

Design ARI	Cell A (m^3/s)	Cell B (m^3/s)
10	0.071	0.026
100	0.10	0.04

6.6.4 Step 3: Dimensions of Swale

The swales need to be sized such that they can convey the 10 year ARI peak discharge without water encroaching on the road. Manning's equation is used to compute the discharge capacity of the swale.

In determining the dimensions of the swale, the depth of the swale was determined by the requirement for it to enable allotment drainage to be discharged to the surface of the swale. Given the cover requirements of the allotment drainage pipes as they flow under the service road (600 mm minimum cover), it set the base of the bioretention systems at 0.76m below road surface. The following are the characteristics of the proposed swale:-

- Base width of 1m with 1:3 side slopes, max depth of 0.76m
- Grass vegetation mown to height of 0.1m (assume n = 0.045 for 10 year ARI with flows above grass height)
- 1.3% longitudinal 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 (Q_{cap}) is estimated adopting a 110mm freeboard[3] (i.e. maximum depth is 0.65m).

$$Q_{cap} = 2.19 \text{ m}^3/\text{s} >> 0.10 \text{ m}^3/\text{s}$$

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

With a base width of 1 m, the lengths of the bioretention system in Cells A and B will need to be 61 m and 22 m respectively to attain the required areas to meet the water quality objectives.

6.6.5 Step 4: Design of Swale Inlet

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

Flush kerbs with a 60 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.

6.6.6 Step 5: Design of bioretention component

6.6.6.1 Soil Media Specification

Three layers of soil media are to be used. A sandy loam filtration media (600mm), a medium to coarse sand transition layer (100mm) and a gravel drainage layer (200mm).

6.6.6.2 Filter Media Specifications

The filter media is to be a sandy loam with the following criteria (mainly from FAWB 2009):

The material shall meet the geotechnical requirements set out below:

Hydraulic conductivity between 100-300 mm/hr

Particle sizes of between**:** clay 2 – 4 %, silt 4 – 8 %, sand < 85 %

Organic content between 3% and 10%

pH 5.5 – 7.5

6.6.6.3 Transition Layer Specifications

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

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

6.6.6.4 Drainage Layer Specifications

The drainage layer is to be 2 - 5 mm screenings.

[3] The Singapore Code of Practice for Surface Drainage would normally stipulate a freeboard of 15% of the depth of the drain, ie. 0.15 x 760 = 110mm

6.6.6.5 Maximum Filtration Rate of Bioretention Media

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

Saturated hydraulic conductivity = 180 mm/hr = 5 x 10^{-5} m/s

Flow capacity of the filtration media = (1-ϒ) As k_h

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

$$Q_{\max} = 5 \cdot 10^{-5} \cdot L \cdot W_{base} \left(\frac{0.2 + 0.6}{0.6} \right)$$

where:

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

W_{base} = base width of the filtration area (m) – 1 m width adopted

L = length of the bioretention zone (m); 61 m (Cell A) and 22 m (Cell B)

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

d = depth of filter media

Maximum filtration rate Cell A = 0.0041 m^3/s

Maximum filtration rate Cell B = 0.0015 m^3/s

6.6.6.6 Sizing of Slotted Collection Pipes

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

Assuming drainage layer is saturated, driving head is half the depth of the drainage layer – H = 0.1m

Assume sub-surface drains with half of all pipes blocked

Product specification Clear Opening = 2100 mm^2/m

Assumed unblocked opening = 1050mm^2/m

Slot Width = 1.5 mm

Slot Length = 7.5 mm

Diameter = 100 mm

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

Assume orifice flow conditions – Q = CA $\sqrt{2gh}$

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

Inlet capacity /m of pipe = [0.61x (0.0015 x 0.0075) x $\sqrt{2 \text{x} 9.81 \text{x} 0.1}$] x 93.3

= 0.0009 m^3/s

Inlet capacity/m x total length =

Cell A = 0.0009 x 61 = 0.055 m^3/s >> 0.0041 m^3/s (max infiltration rate), hence 61 m of pipe has sufficient perforation capacity to pass flows into the perforated pipe.

Cell B = 0.0009 x 22 = 0.020 m^3/s >> 0.0015 m^3/s (max infiltration rate), hence 22m of pipe is sufficient.

6.6.6.7 Slotted Pipe Capacity

The Colebrook-White equation is applied to estimate the flow rate in the perforated pipe. A slope of 0.5%[4] is assumed and a 100mm perforated pipe (as above) was used. Should the capacity not be sufficient, additional pipes would be required. The capacity of this pipe needs to exceed the maximum filtration rate of the media.

Estimate applying the Colebrook-White Equation

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

Adopt

D = pipe internal diameter (0.10m)

S_f = slope (0.005m/m)

g = gravitational acceleration (9.81m^2/s)

k = hydraulic roughness (0.007m)

v = velocity (1.007 x 10^{-6} m/s)

A = cross-sectional area of pipe

Q_{cap} = 0.01 m^3/s[5] (for one pipe) > 0.0041 m^3/s (Cell 1); 0.0015 m^3/s (Cell 2), and hence 1 pipe is sufficient to convey the maximum infiltration rate for both Cell A and B.

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

6.6.6.8 Drainage Layer Hydraulic Conductivity

Typically flexible perforated pipes are installed using fine gravel media to surround them. In this case study, 2-5mm gravel is specified for the drainage layer. This media is much coarser than the filtration media (sandy loam) therefore to reduce the risk of washing the filtration layer into the perforated pipe, a transition layer is to be used. This is to be 100 mm of coarse sand.

6.6.6.9 Impervious Liner Requirement

In this catchment the surrounding soils are clay to silty clays with a saturated hydraulic conductivity of approximately 3.6 mm/hr. The sandy loam media that is proposed as the filter media has a hydraulic conductivity of 100 - 300 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.

6.6.7 Step 6: Verification checks

6.6.7.1 Vegetation Scour Velocity Check

Assume Q_{10} and Q_{100} will be conveyed through the swale/bioretention system. Check for scouring of the vegetation by checking that velocities are below 0.5m/s during Q_{10} and 2.0 m/s for Q_{100}.

[4] A slope of 0.5% is adopted simply for convenience. In reality, the discharge capacity is reached when the soil is saturated and water ponded to the full extended detention depth. Bioretention systems can operate equally effectively with the underdrain laid at near-zero (but positive) slopes.

[5] Per manufacturer data

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

Q_{10} = 0.071 m^3/s, depth = 0.12 (with n = 0.06), velocity = 0.38m/s < 0.5m/s – therefore, OK

Q_{100} = 0.103 m^3/s, depth = 0.14m (with n = 0.045), velocity = 0.52m/s < 2.0m/s – therefore, OK

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

6.6.7.2 Safety Velocity Check

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

V = 0.52m/s (calculated previously)

D = 0.14m

V.D = 0.52 x 0.14 = 0.07 < 0.4m^2/s

Therefore, velocities and depths are OK.

6.6.8 Step 7: Overflow pit 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.

There are standard pit sizes to accommodate connection to the underground stormwater pipe. For a minimum underground pipe of 300 mm diameter, a 450 mm x 450 mm pit will be required for both Cell A and Cell B.

To check the adequacy of this pit to convey the 10 year ARI peak discharge, two flow conditions need to be check. The assumed water level above the crest of the pit is the depth of water from the road surface, less freeboard and the extended detention (i.e. 0.76 – (0.11 + 0.2) = 0.45m).

First check using a weir equation

Q_{weir} = B.C.L.$H^{3/2}$ with B = 0.5, C = 1.7, L = 1.8 and H = 0.45

= 0.4 m^3/s > 0.071 m^3/s …….OK

Now check for drowned conditions:

$Q_{orifice}$ = B.C.A $\sqrt{2gh}$ with B = 0.5, C = 0.6, A = 0.20 and H = 0.45

= 0.17 m^3/s > 0.071 m^3/s……OK

6.6.9 Step 8: Allowances to preclude traffic on swales

Traffic control is achieved by using traffic bollards.

6.6.10 Step 9: Vegetation specification

Plants may be selected from the reference list in 6.7. Also check with the party who is going to take over the maintenance of the bioretention swales.

6.6.11 Step 10: Maintenance Plan

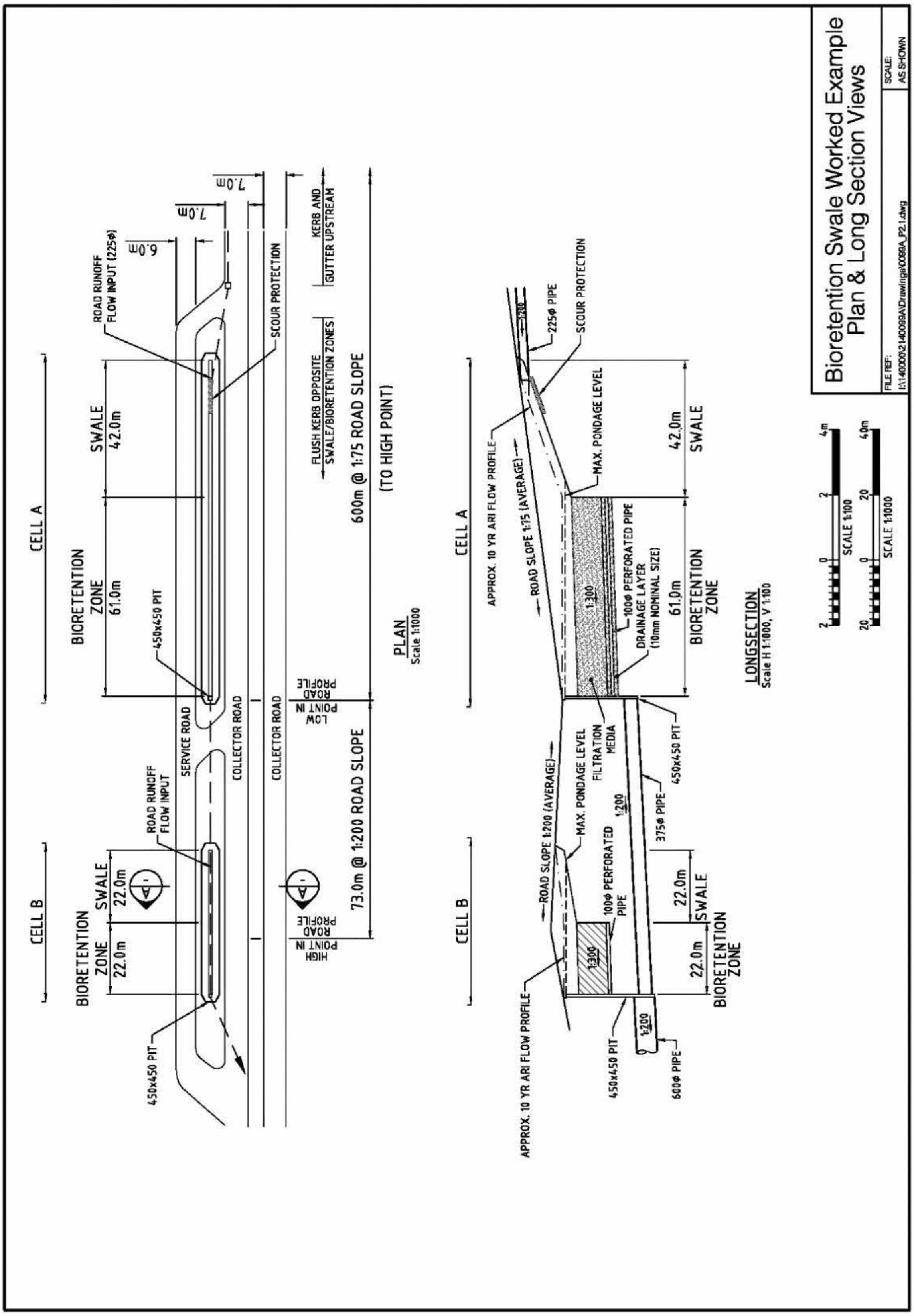
A maintenance plan for Swales 1 and 2 is to be prepared by the ABC Waters Professional in accordance with the requirements of the Code of Practice on Surface Water Drainage.

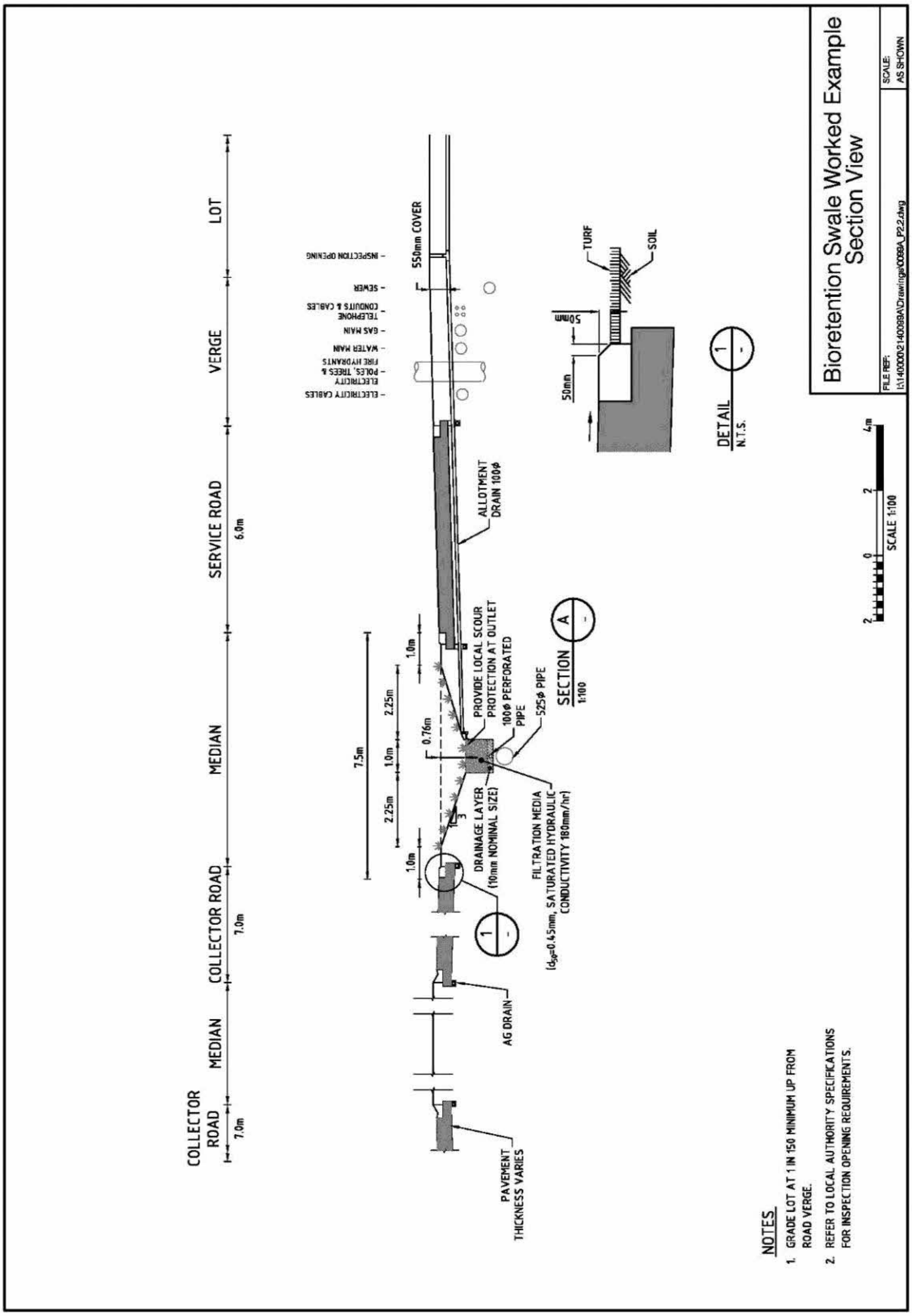
6.6.12 Calculation summary

The sheet below summarises 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**			✓
		1680	m^2	
		624	m^2	
	slope	1.3	%	
	Fraction impervious			✓
	Cell A	0.8		
	Cell B	0.8		
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:	Singapore		
	major flood - 100 year ARI	A - 275, B - 283	mm/hr	
	minor flood - 10 year ARI	A - 190, B - 200	mm/hr	
	Peak design flows			
	Q_{minor}	0.07 (A), 0.026 (B)	m^3/s	
	Q_{100}	0.10 (A), 0.04 (B)	m^3/s	
	Q_{infil}	0.0041 (A) 0.0015 (B)	m^3/s	✓
3	**Swale design**			
	appropriate Manning's n used?	yes		✓
4	**Inlet details**			
	adequate erosion and scour protection?	rock pitching		✓
5	**Velocities over vegetation**			
	velocity for 10 year flow (<0.5m/s)	0.38	m/s	
	velocity for 100 year flow (<1.0m/s)	0.52	m/s	
	safety: Vel x Depth (<0.4)	0.07	m^2/s	✓
6	**Slotted collection pipe capacity**			
	pipe diameter	100	mm	
	number of pipes	1		
	pipe capacity	0.01	m^3/s	
	capacity of perforations	0.055 (A); 0.020 (B)	m^3/s	
	soil media infiltration capacity	0.004, 0.001	m^3/s	✓
8	**Overflow system**			
	system to convey minor floods			✓
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**			
		Zoysia Matrella		✓

6.6.13 Construction drawings




Bioretention Swale Worked Example
Section View
SCALE:
AS SHOWN
FILE REF:
COLLECTOR ROAD
7.0m
MEDIAN
COLLECTOR ROAD
7.0m
MEDIAN
7.5m
SERVICE ROAD
6.0m
VERGE
LOT
550mm COVER
ALLOTMENT DRAIN 100ϕ
PROVIDE LOCAL SCOUR PROTECTION AT OUTLET
100ϕ PERFORATED PIPE
525ϕ PIPE
SECTION A
1:100
DRAINAGE LAYER (10mm NOMINAL SIZE)
FILTRATION MEDIA
AG DRAIN
PAVEMENT THICKNESS VARIES
DETAIL
N.T.S.
TURF
SOIL
50mm
SCALE 1:100
NOTES
1. GRADE LOT AT 1 IN 150 MINIMUM UP FROM ROAD VERGE.
2. REFER TO LOCAL AUTHORITY SPECIFICATIONS FOR INSPECTION OPENING REQUIREMENTS.

6.7 Reference List of Plants for Filtration Area in Bioretention System

Asystasia sp.

Calathea lutea

Canna generalis, hybrids

Canna indica

Cyathula prostrata

Cymbopogon citratus

Cyperus alternifolius

Cyperus papyrus

Dianella ensifolia

Dissotis rotundifolia

Excoecaria cochinchinensis

Galphimia glauca

Heliconia psittacorum

Leea indica

Leucophyllum frutescens

Loropetalum chinense

Murraya paniculata

Osmoxylum lineare

Pandanus amaryllifolius

Pandanus pygmaeus

Pennisetum alopecuroides

Pennisetum setaceum

Pennisetum x advena 'Rubrum'

Phyllanthus cochichinensis

Phyllanthus myrtifolius

Pogonantherum paniceum

Ruellia brittoniana

Russelia equisetiformis

Schefflera arboricola

Serissa japonica

Sphagneticola trilobata

Thalia geniculata

Zoysia matrella

6.8 References

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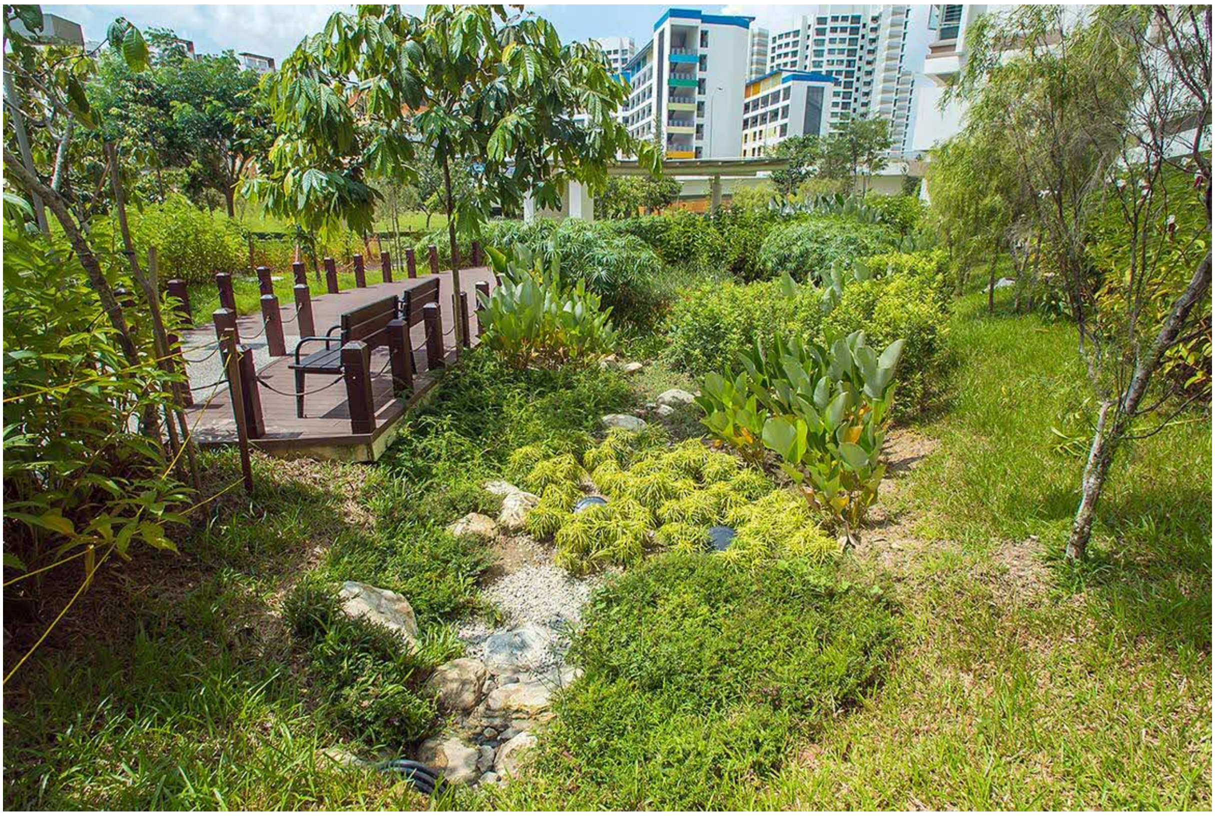
Bioretention Basins 7



7

Chapter 7 Bioretention Basins

7.1 Introduction

Bioretention basins use ponding above a bioretention surface to maximise the volume of runoff treated through the filtration media. Their treatment processes are the same as that for bioretention swales. However, they are predominantly detention systems designed for frequent storms (like 3 month ARI storms) with flood flows bypassing the filtration surface into stormwater drains or detention tanks.

Bioretention basins operate by filtering stormwater runoff through densely planted surface vegetation as a means of pre-treatment before they infiltrate/percolate through a prescribed filter media. During percolation, pollutants are retained through fine filtration, adsorption and some biological uptake. The vegetation in a bioretention system is a vital functional element of the system both in terms of maintaining the hydraulic conductivity of the filter media and the improving soil capacity for chemical and biological removal of stormwater contaminants. Vegetation facilitates the transport of oxygen to the soil and enhances soil microbial communities which enhance biological transformation of pollutants.

Bioretention basins are generally not intended to be 'infiltration' systems that discharge from the filter media to surrounding in-situ soils. Rather, the typical design intent is to recover stormwater at the base of the filter media in perforated under-drains and discharge to receiving waterways or to storages for potential reuse. In some circumstances however, where the in-situ soils allow infiltration or when there is a particular design intention to recharge local groundwater, it may be desirable to allow stormwater to infiltrate from the base of a filter media to underlying in-situ soils. This type of Bioretention basin is termed as Soak-away rain gardens.

Bioretention basins can be installed at various scales, ranging from planter boxes, to streetscape raingardens integrated with traffic calming measures, to system contained within retarding basins. 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 system this is not required. Example applications are given in Figure 7.1

This chapter describes the design, construction and maintenance of a bioretention basins.

Figure 7.1 Example of bioretention basin in Sungei Tampines

7.2 Key Design Configurations

There are many possible design variations for bioretention systems and these may be grouped into five core design configurations. The features of each of these configurations are described below.

It is strongly recommended that bioretention systems which include submerged zones should be used wherever possible. It has been shown that the treatment performance of bioretention systems is significantly reduced after extended dry periods. The presence of a submerged, permanent pool of water at the bottom of the systems acts as a buffer against drying and helps maintain a healthy plant community throughout long dry spells.

Illustrations in this section are for demonstration purposes only. Outlet structures may be any combination of raised pits or more complex outflow structures as described in chapter 7.4 Design Process.

7.2.1 Lined bioretention system

A standard lined bioretention system (Figure 7.2) prevents exfiltration and minimises losses through the system. This type of bioretention basin is optimal in the following situations.

- Sites where exfiltration is not possible. This may arise where there is a need to protect built infrastructure or whereby interactions with shallow groundwater are undesirable.
- Climates that do not experience long dry spells.
- If systems are designed for NO_x removal or if receiving waters are highly sensitive to Cu or Zn.

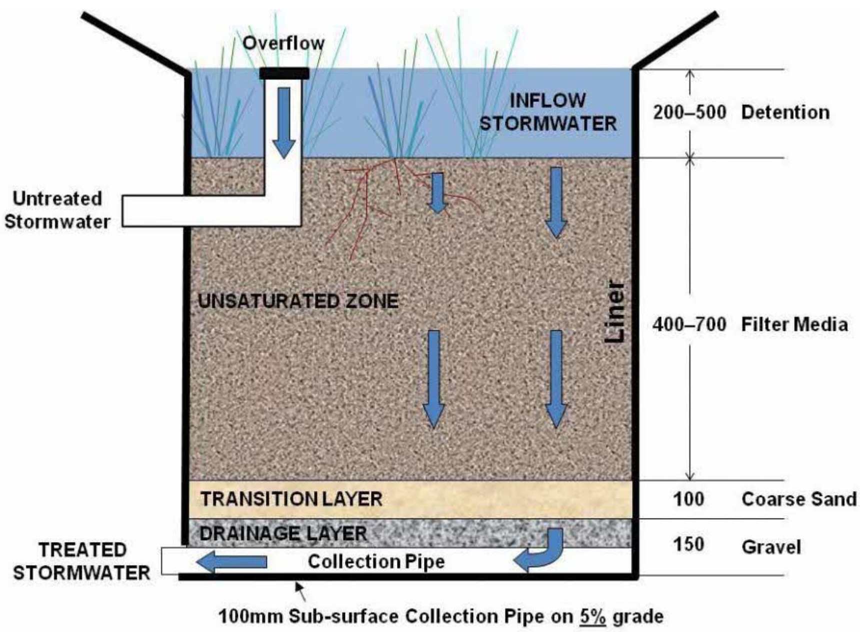


Figure 7.2 Lined standard bioretention system [source: FAWB, 2009]

A lined bioretention system may also be designed to include a submerged zone with the submerged zone comprising of sand (Figure 7.3) or gravel. This type of bioretention basin should be used for the following situations;

- Sites where exfiltration is not possible. This may arise where there is a need to protect built infrastructure or whereby interactions with shallow groundwater are undesirable.
- Climates that have very long dry spells. The submerged zone is able to act as a water source for up to five weeks, supporting the plants and microbial community.
- If systems are designed for NO_x removal or if receiving waters are highly sensitive to Cu or ZN.

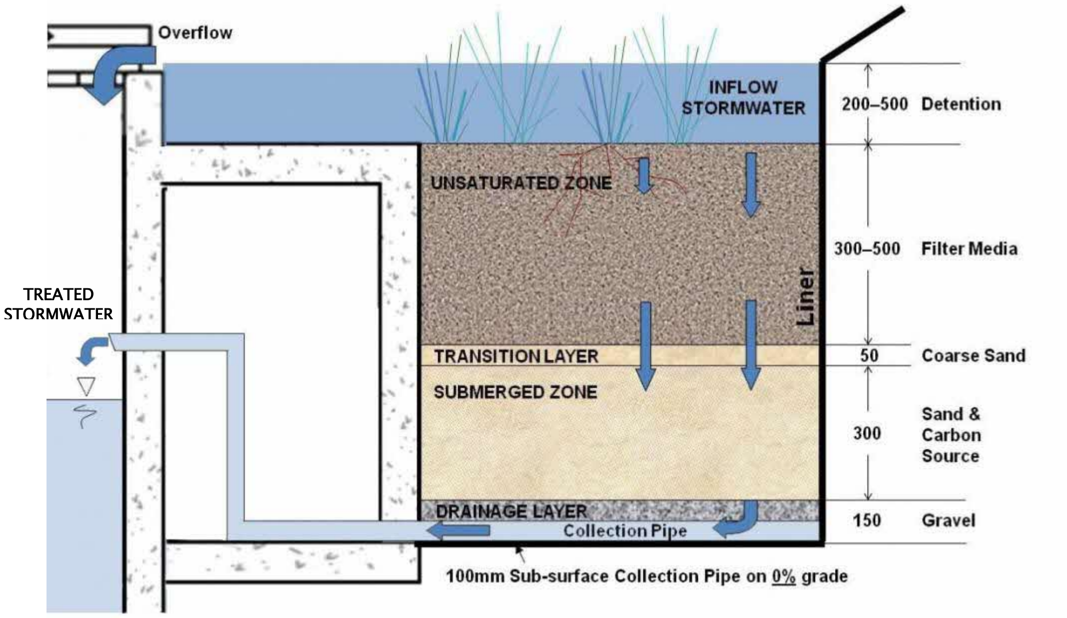


Figure 7.3 Lined bioretention system with submerged zone comprised of gravels and hardwood chips [modified from: FAWB, 2009]

7.2.2 Unlined bioretention system

A standard unlined bioretention system (Figure 7.4) is the simplest configuration of bioretention system to design and build. These systems are suited for

- Sites where minimal infiltration is allowed. (The hydraulic conductivity of the surrounding soils should be an order of magnitude lower than the filter media to ensure minimal infiltration.
- Climates that do not experience long dry spells.
- Systems that are not designed for stormwater harvesting.

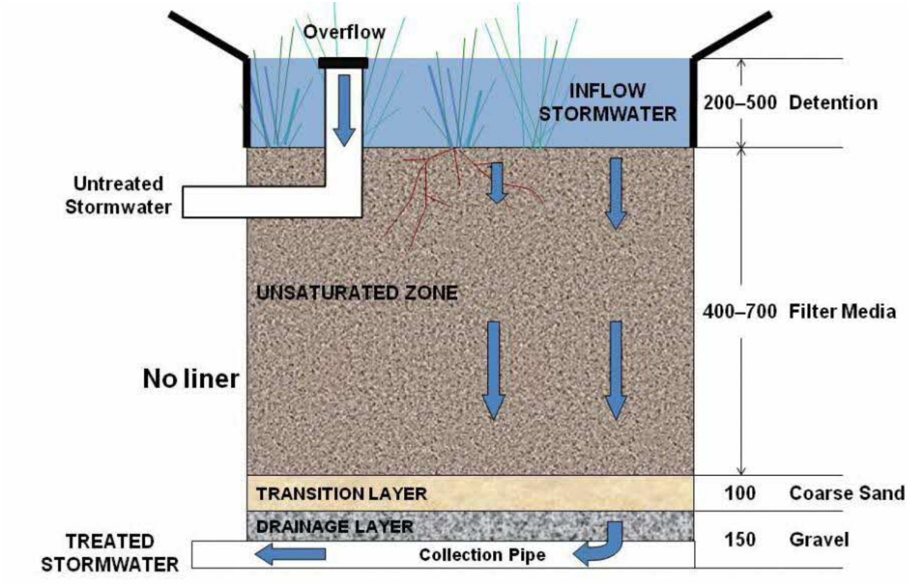


Figure 7.4 Unlined standard bioretention system [source: FAWB, 2009]

Unlined bioretention systems may also include a submerged zone (Figure 7.5). The addition of a submerged zone is appropriate whereby exfiltration is permissible and the local climate yields long dry spells. These systems have unlined sides, however the submerged zone must be lined to maintain saturation.

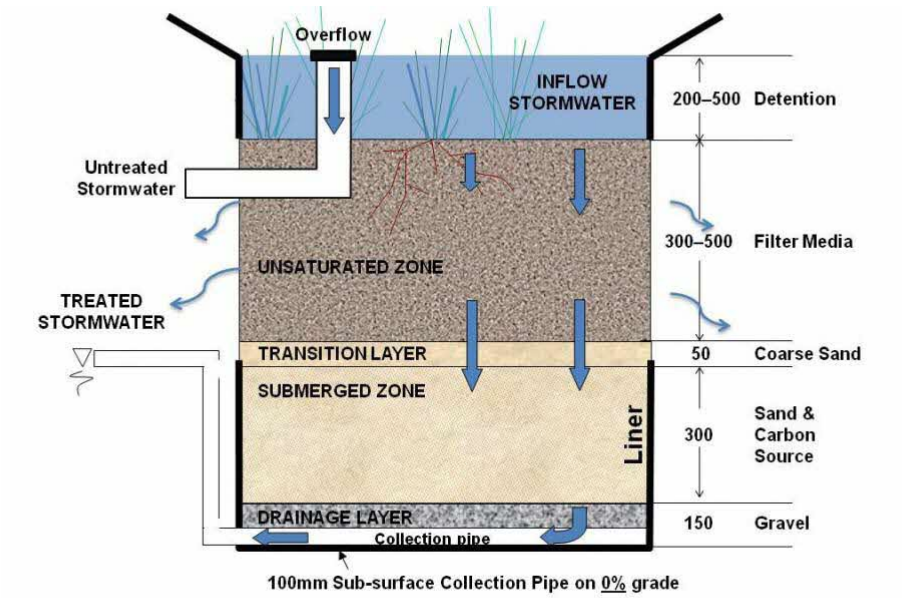


Figure 7.5 Unlined bioretention system with submerged zone [source: FAWB, 2009]

7.2.3 Unlined bio-infiltration system

An unlined bioretention system is a hybrid system, combining a standard bioretention system and an infiltration system, which is also referred to as a bio-infiltration system. Unlined bio-infiltration systems are recommended for;

- Sites where exfiltration is allowed.
- Whereby both water quality improvements and runoff reduction are required.
- Systems that are not designed for stormwater harvesting.

Unlined bioretention systems do not contain collection pipes in the drainage layer. Where possible, unlined bioretention systems are preferable to standard, non-vegetated infiltration systems due to the increased nutrient removal and are therefore highly recommended whereby appropriate.

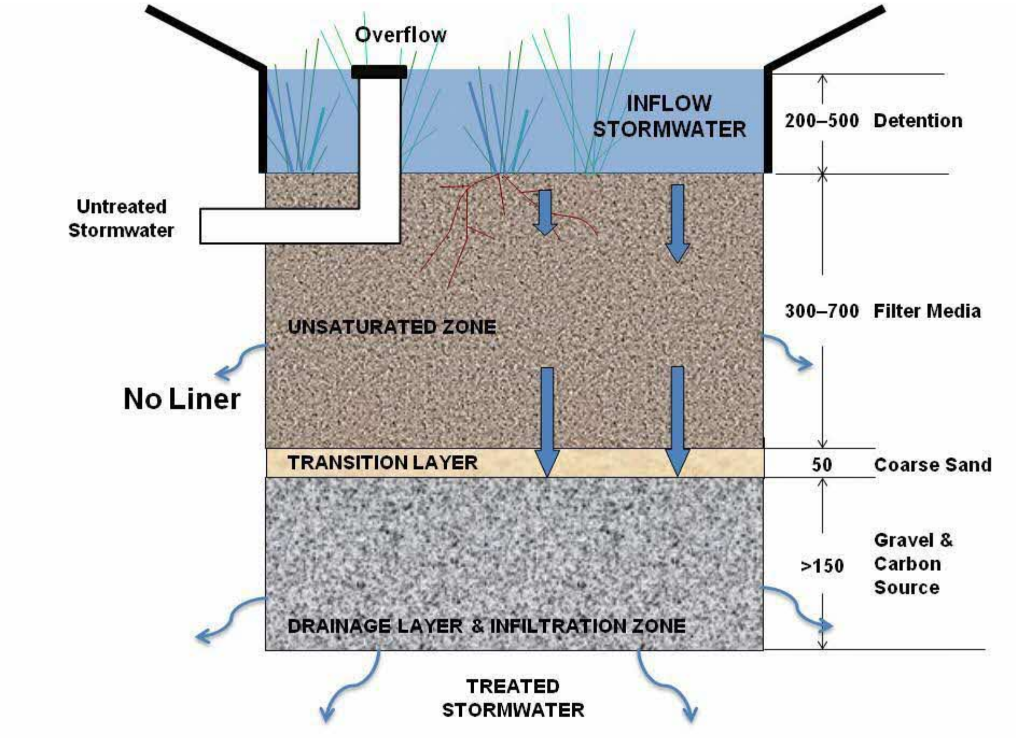


Figure 7.6 A hybrid bioretention and infiltration system [source: FAWB, 2009]

7.3 Design Considerations for Bioretention Basins

A typical design for a bioretention basin is given in Figure 7.7. Key to the design is the hydraulic operation, the filter media, the vegetation and the interaction of the basin within the urban space. These design considerations are discussed further in the following sections. Design considerations are similar to that presented in Chapter 6 Bioretention Swales and are presented in both chapters for ease of reference.

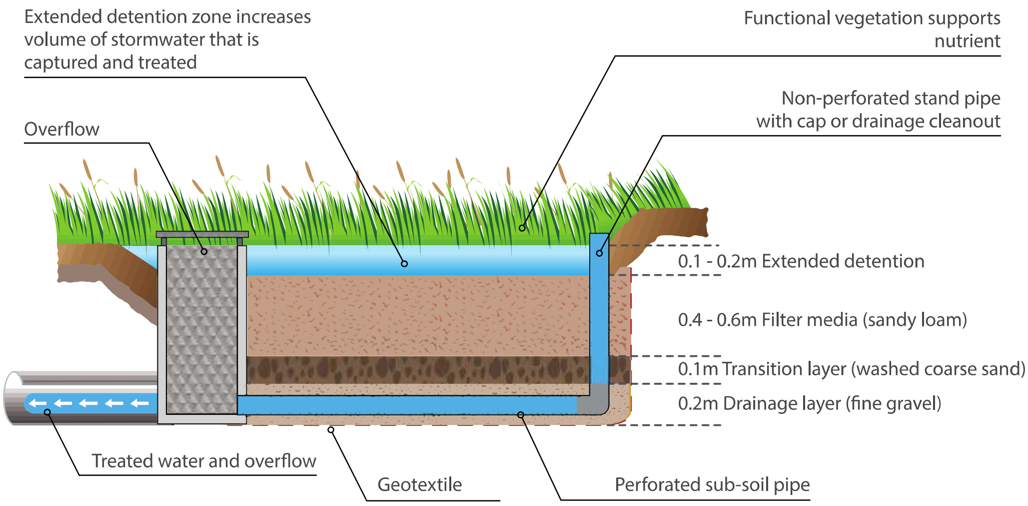


Figure 7.7 Typical cross section of a bioretention basin

7.3.1 Landscape Design

Bioretention basins, sometimes referred to as bioretention pods and rain gardens, may be located within parkland areas, easements, carparks or along roadway corridors as 'standalone' soil filtration systems. Landscape design of bioretention basins along the road edge can assist in defining traffic islands and intersections as well as providing landscape character and amenity. It is therefore important that the landscape design of bioretention basins addresses stormwater quality objectives and accommodates these other important landscape functions.

7.3.2 Hydraulic Design

The hydraulic design of bioretention basins is directed at ensuring effective stormwater treatment performance; minimize damage by storm flows, and to protect the hydraulic integrity and function of associated minor and major drainage systems. The following aspects are of key importance:

- The finished surface of the bioretention filter media must be horizontal (i.e. flat) to ensure full engagement of the filter media by stormwater flows and to prevent concentration of stormwater flows within depressions.
- Temporary ponding or extended detention, typically of up to 0.3m depth over the surface of the soil filter media created through the use of raised inlet pits (overflow pits) can assist in increasing the overall volume of stormwater runoff that can be treated by the bioretention filter media.
- Where possible, the overflow pit or bypass pathway should be located near the inflow zone to prevent high flows passing over the surface of the filter media. If this is not possible, then velocities during the minor (10 year ARI) and major (100 year ARI) floods should be maintained sufficiently low (preferably below values of 0.5 m/s and not more than 2.0 m/s for major flood) to avoid scouring of the filter media and vegetation.

- Where the inlet to a bioretention system is required to convey the minor storm flow (i.e. is part of the minor drainage system), the inlet must be designed to avoid blockage, flow conveyance and public safety issues.
- For streetscape applications, the design of the inflow to the bioretention basin must ensure the kerb and channel flow requirements are preserved.

7.3.3 Preventing Exfiltration to In-Situ Soils

Bioretention basins can be designed to generally preclude exfiltration of treated stormwater to the surrounding in-situ soils. The amount of water potentially lost from bioretention trenches to surrounding in-situ soils is largely dependent on the characteristics of the surrounding soils and the saturated hydraulic conductivity of the bioretention filter media (see Section 7.3.5).

If the saturated hydraulic conductivity of the filter media is one to two orders of magnitude (i.e. 10 to 100 times) greater than that of the insitu soil, the flow path of stormwater percolation will be effectively contained within the bioretention filter media and through to the drainage layer. As such, there will be little exfiltration to the surrounding soils.

If the selected saturated hydraulic conductivity of the bioretention filter media is less than 10 times that of the surrounding soils, it may be necessary to provide an impermeable liner. Flexible membranes or a concrete casting are commonly used to prevent excessive exfiltration.

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

7.3.4 Vegetation Specification

Vegetation is a key component of a bioretention basin, servicing the following processes:

- Scour protection
- Maintaining the porosity of filtration layer
- Enhancing pollutant adsorption to biofilms in roots within the filter media

Generally, the greater the density and height of vegetation planted in a bioretention basin the better will be the treatment especially when extended detention is provided in the design. When the extended detention is engaged, the contact between stormwater and vegetation results in enhanced sedimentation of suspended sediments and some adsorption of associated pollutants.

Bioretention basins should be planted to cover the whole bioretention filter media surface. Vegetation should be of sufficient density to prevent preferred flow paths, scour and re-suspension of deposited sediments. Turf grasses should ideally be avoided as these are shallow rooted systems with inadequate penetration to the full depth of the filter media and the turf stems inadequately prevent clogging at the surface of the filter media.

A list of commonly used plants for Bioretention Systems is in Section 6.7. A CUGE (NParks) publication on “A selection of plants for bioretention systems in the tropics” can also be consulted for plant selection. The publication can be downloaded at https://www.cuge.com.sg/research/download.php?product=47.

7.3.5 Bioretention Filter Media

Selection of an appropriate bioretention filter media is a key design step that involves consideration of the following three inter-related factors:

- Saturated hydraulic conductivity of the filter media.
- Depth of extended detention provided above the filter media.
- Suitability as a growing media to support vegetation (i.e. retains sufficient soil moisture and organic content).

The high rainfall intensities experienced in Singapore is expected to result in bioretention treatment areas being larger in Singapore than comparable systems overseas in Australia and the United States. The area available for bioretention basins in an urban layout is often constrained by factors such as the available area within the footpaths of standard road reserves.

Selecting bioretention filter media for bioretention basin applications in Singapore will often require careful consideration of saturated hydraulic conductivity and extended detention depth to ensure the desired minimum volume of stormwater runoff receives treatment. This must also be balanced with the requirement to also ensure the saturated hydraulic conductivity does not become too high such that it can no longer sustain healthy vegetation growth.

> The maximum saturated hydraulic conductivity (k_f) should not exceed 500 mm/hr (and preferably be between 100 - 300 mm/hr) in order to sustain vegetation growth. k_f less than 100 mm/hr (>50 mm/hr) could be accepted with caution.

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

As shown in Figure 7.7, a bioretention system can consist of three layers. The filter media is the primary soil layer consisting typically of sandy-loam material. In addition to the filter media, a drainage layer is also required to convey treated water from the base of the filter media to the outlet via a perforated under-drains, unless the design intent is to allow the filtered water to discharge (exfiltrate) into insitu soil. The drainage layer surrounds perforated under-drains and consists typically of fine gravel of 2-5 mm particle size. In between the filter media layer and the drainage layer is the transition layer consisting of clean sand (1mm) to prevent migration of the base filter media into the drainage layer and into the perforated under-drains.

[Refer to the Bioretention Media Guidelines produced by FAWB[1] (2009) for more information.]

7.3.6 Maintenance and Access

The performance of a bioretention system will be affected by impeded flow. Driving over or storing construction material on the bioretention basin can cause the filter media to become impacted (compacted) and the vegetation damaged. The design of a bioretention system should consider means of preventing or discouraging the bioretention basin as becoming a trafficable and/or storage area.

[1] Facility for Advancing Water Biofiltration – http://www.monash.edu.au/fawb/

7.4 Design Process

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

1. Confirm treatment performance of concept design

2. Determine design flows

3. Design inflow systems
a. Inlet scour protection
b. Coarse sediment forebay
c. Streetscape bioretention application

4. Specify the bioretention filter media characteristics
a. Filter media
b. Drainage layer
c. Transition layer

5. Design under-drain and undertake capacity checks

6. Check requirement for impermeable lining

7. Size overflow pit

8. Specify vegetation

9. Undertake verification checks
a. Vegetation scour velocity check
b. Confirm treatment performance

7.4.1 Step 1: Confirm treatment size given in conceptual design

A conceptual design of a bioretention basin is normally typically undertaken prior to detailed design. The performance of the concept design must be checked to ensure that stormwater treatment objectives will be satisfied.

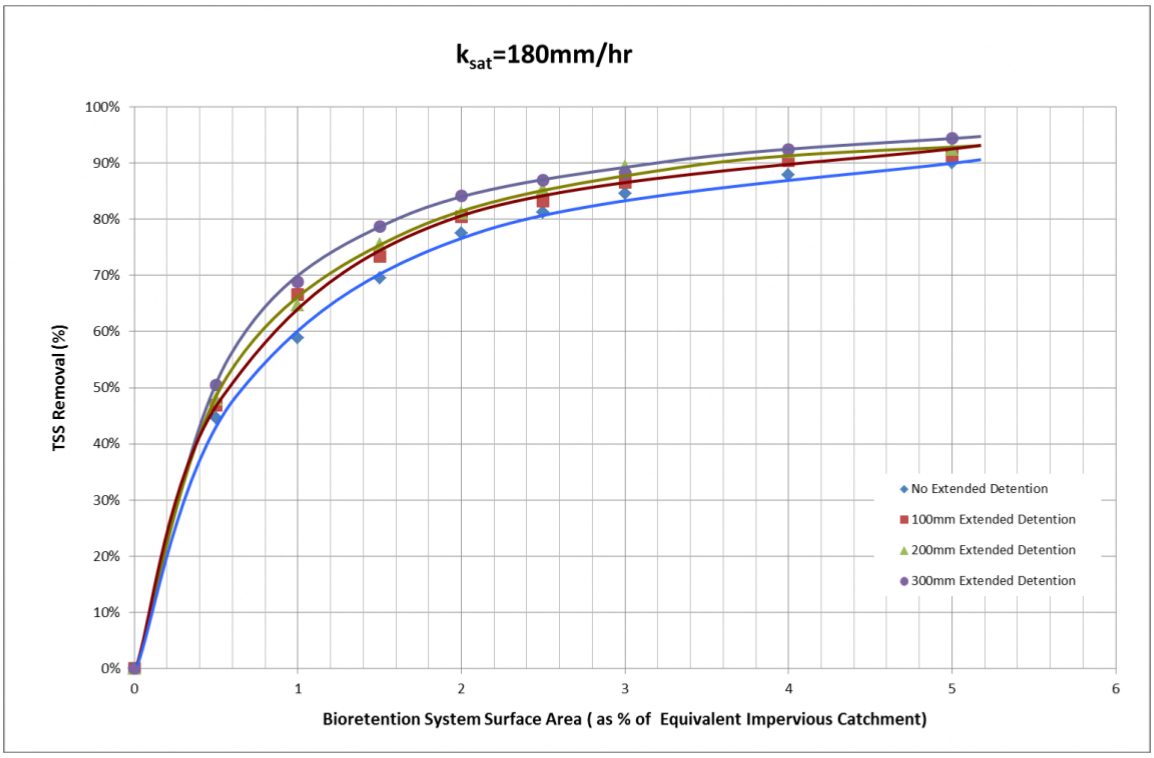
The treatment performance curves shown in Figure 7.8 to Figure 7.10 reflect the treatment performance of the bioretention basin. The performance curves provide an indication only of appropriate sizing and do not substitute the need for a thorough conceptual design process. Nevertheless, it is a useful visual guide to illustrate the relationship of bioretention treatment performance to the ratio of bioretention treatment area and contributing catchment area. The curves allow the designer to make a rapid assessment as to whether the bioretention basin size falls within the "optimal size range".

The curves in Figure 7.8 to Figure 7.10 show the total suspended solid (TSS), total phosphorus (TP) and total nitrogen (TN) removal performance for a typical bioretention basin design with the following configurations:

- Filter media saturated hydraulic conductivity (k) = 180 mm/hr and 360mm/hr or 0.5 x 10^{-4} m/s and 1 x 10^{-4} m/s
- Filter Media average particle size = 0.5mm
- Filter Media Depth = 0.6m
- Extended Detention Depth = from 0 mm to 300 mm

The curves in Figure 7.8 to Figure 7.10 are generally applicable to bioretention basin applications within residential, industrial and commercial land uses. Please take note that "Equivalent Imperious Catchment" is used in the curves. Equivalent Imperious Catchment is the product of total catchment area and the runoff coefficient (C).

If the characteristics of the bioretention component of the bioretention swale concept design are significantly different to that described above, then the curves in Figure 7.8 to Figure 7.10 may not provide an accurate indication of treatment performance. In these cases, the detailed designer should use MUSIC to verify the performance of the bioretention swale.



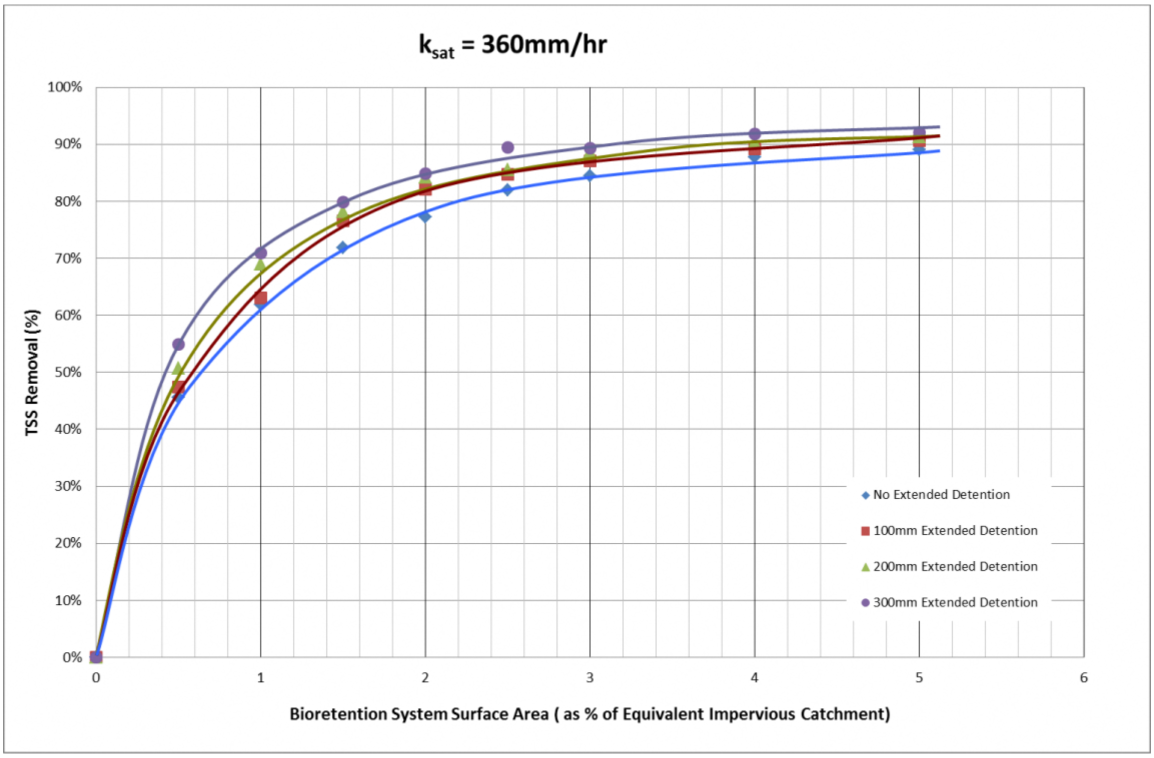
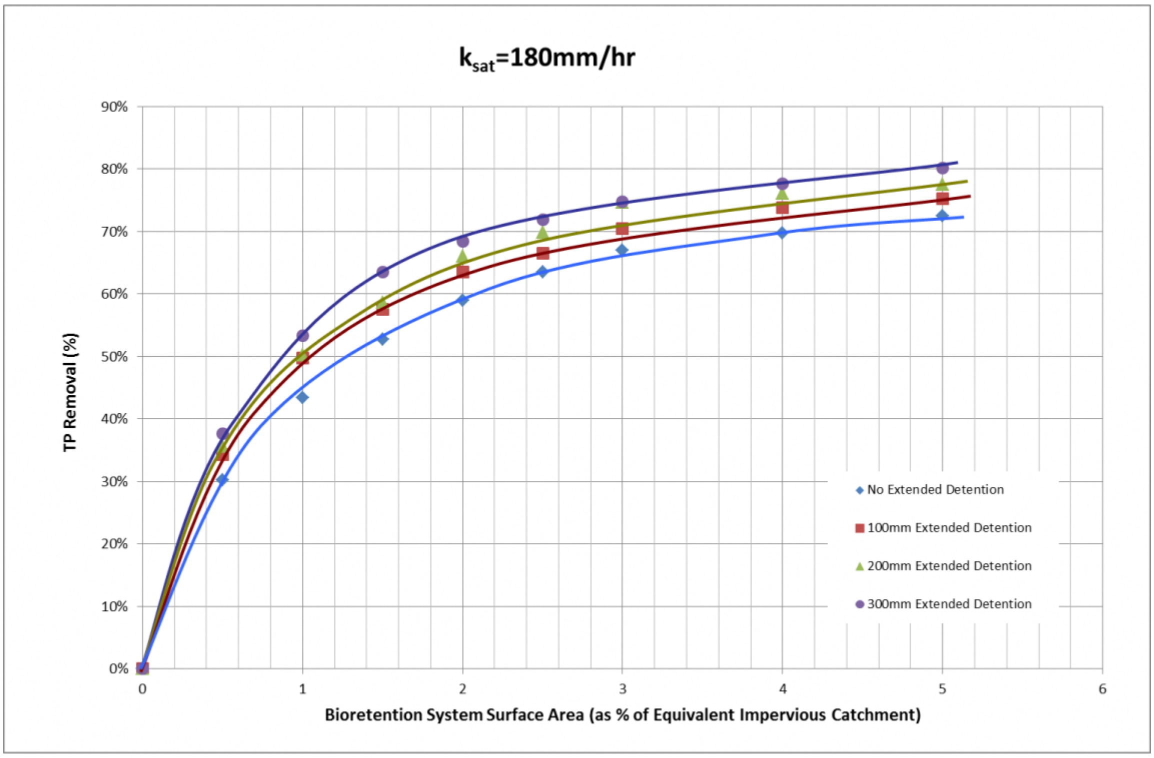


Figure 7.8 Bioretention system TSS removal performance (Reference: Station 43)



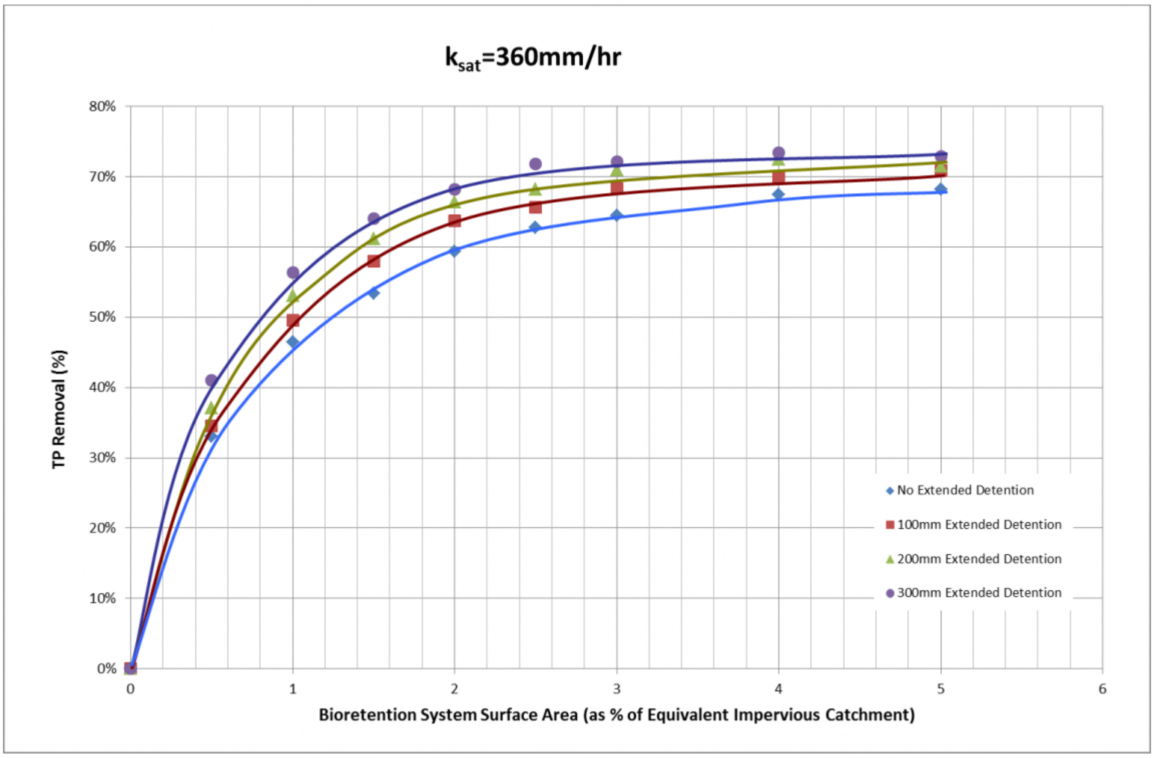
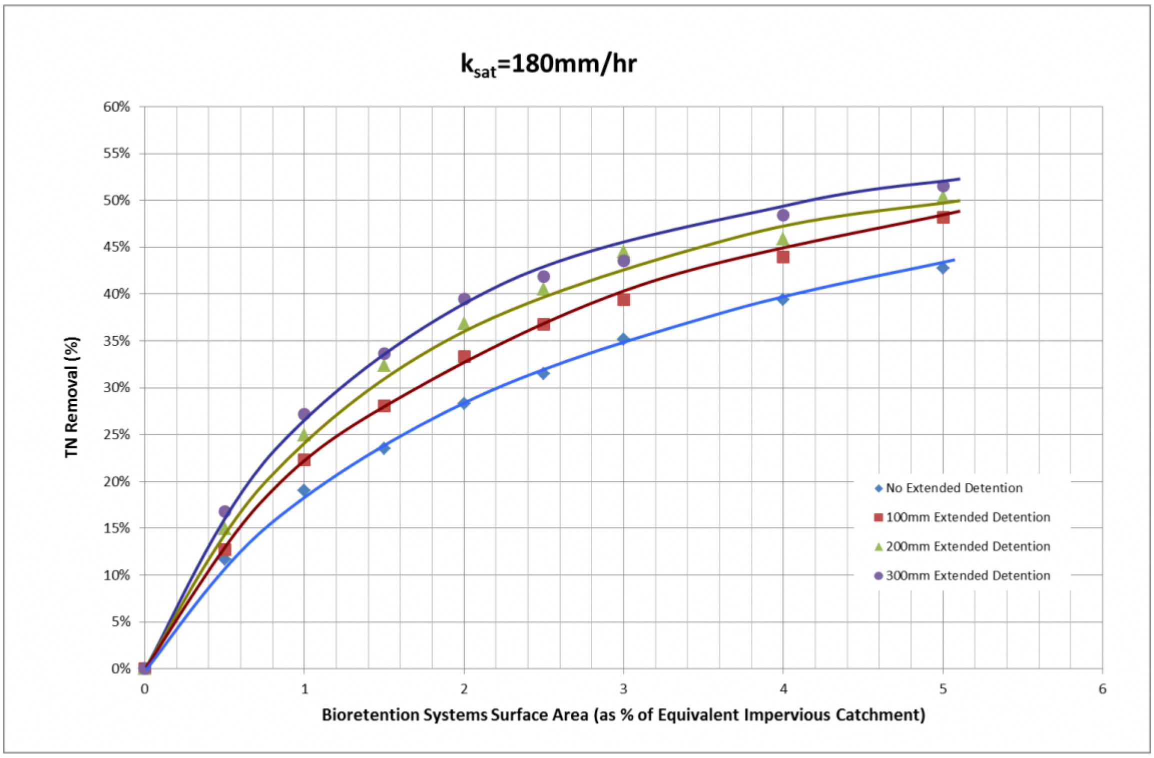


Figure 7.9 Bioretention system TP removal performance (Reference: Station 43)



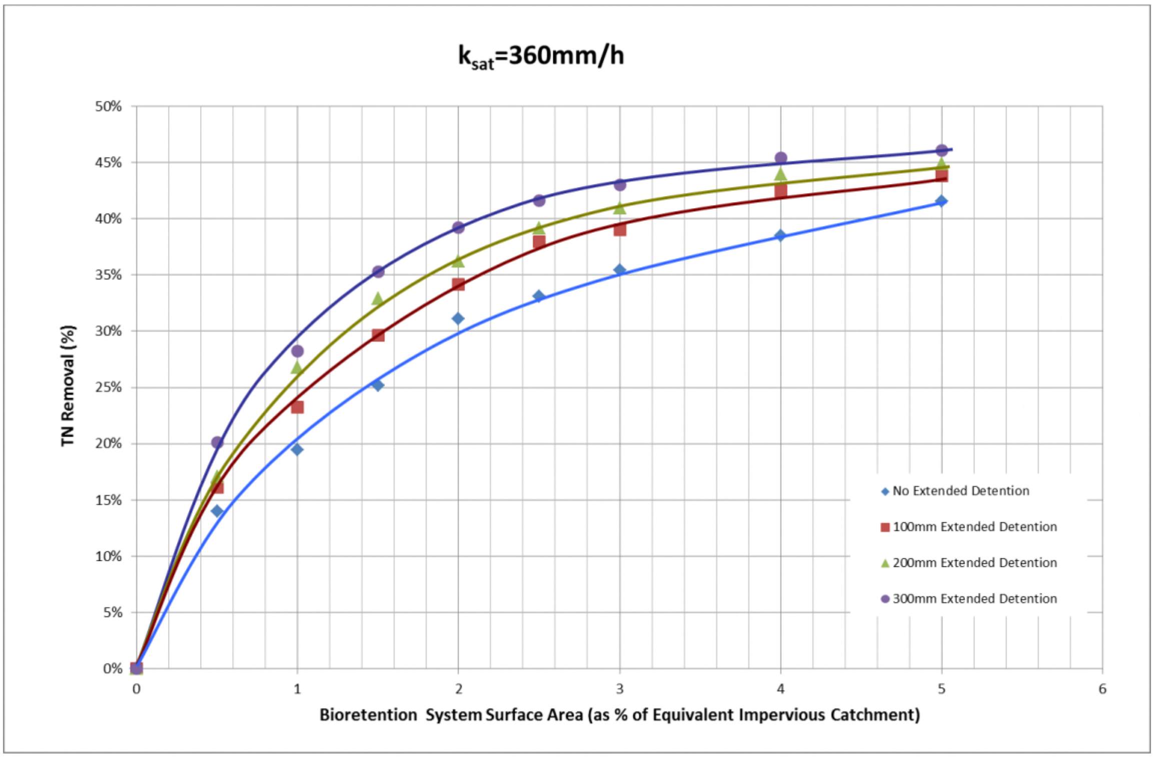


Figure 7.10 Bioretention system TN removal performance (Reference: Station 43)

7.4.2 Step 2: Determine design flows

7.4.2.1 Design Flow

Two design flows are required for bioretention basins:

- Minor (frequent) storm conditions (typically 10 year ARI) to size the overflows to allow minor floods to be safely conveyed and not increase any flooding risk compared to conventional stormwater systems
- Major flood conditions (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

7.4.2.2 Design Flow Estimation

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

7.4.3 Step 3: Design Inflow System

The design of the inflow systems to bioretention basins needs to consider a number of functions:

- Scour protection – In most bioretention applications stormwater flows will enter the bioretention basin as concentrated flow (piped, channel or open channel) and as such is it important to slow and spread flows using appropriate scour (rock) protection.
- Coarse sediment forebay – Where stormwater runoff from the catchment is delivered directly to the bioretention basin without any coarse sediment management (through vegetated swale or buffer treatment) a coarse sediment forebay is to be included in the design. The forebay is to remove coarse sediment from stormwater to minimise the risk of sediment smothering the vegetation in the bioretention basin.
- Street hydraulics (streetscape applications only) – In streetscape applications, where stormwater is delivered directly from the kerb and channel to the bioretention basin, it is important to ensure the location and width of the kerb opening is designed such that flows enter the bioretention basin without adversely affecting trafficability of the road.

Each of these functions and the appropriate design responses are described in the following sections.

7.4.3.1 Inlet Scour Protection

Erosion protection should be provided for concentrated inflows to a bioretention basin. Typically, flows will enter the bioretention basin from either a surface flow system (i.e. roadside kerb, open channel) or a piped drainage system. Rock beaching is a simple method for dissipating the energy of concentrated inflow. Where the bioretention basin is receiving stormwater flows from a piped system (i.e. from larger catchments), the use of impact type energy dissipation may be required to prevent scour of the filter media. In most cases this can be achieved with rock protection and by placing several large rocks in the flow path to reduce velocities and spread flows as depicted in Figure 7.11 (with the 'D' representing the pipe diameter dimension). The rocks can form part of the landscape design of the bioretention component.

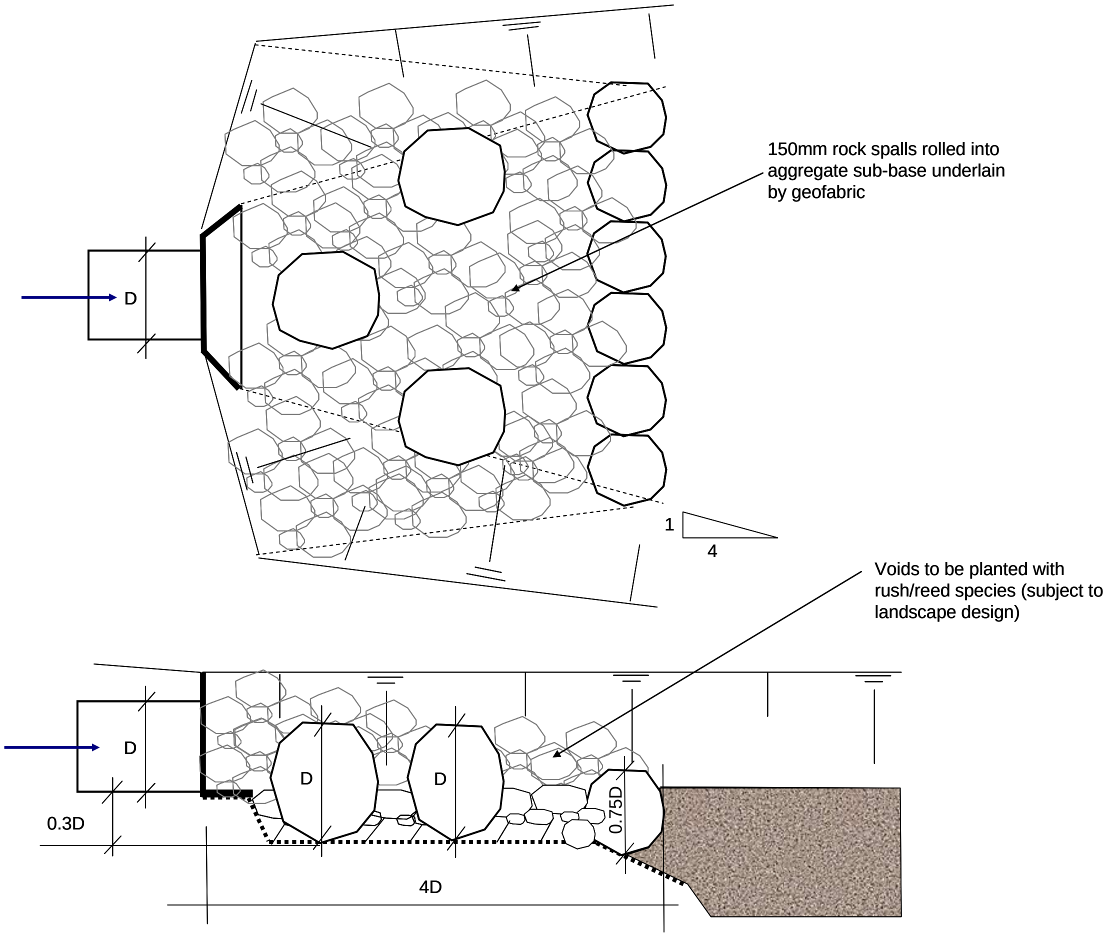


Figure 7.11 Typical Inlet Scour Protection Detail for Bioretention Basins Receiving Piped Flows

7.4.3.2 Coarse Sediment Forebay

Coarse sediment may accumulate near the basin inflow where stormwater runoff from the catchment is delivered directly to the bioretention basin without pre-treatment (through vegetated swale or buffer treatment). To mitigate these effects, it is recommended that a coarse sediment forebay be incorporated into the design of a bioretention basin. The forebay should be designed to:

- Remove particles that are 1mm or greater in diameter from the 3 month ARI storm event.

- Provide appropriate storage for coarse sediment to ensure desilting is required once every year.

The area of the sediment forebay (A_S) is calculated by solving the following expression (modified version of Fair and Geyer (1954)):

$$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A_S}\right]^{-n} \qquad \text{Equation 7.1}$$

Where

R	=	fraction of target sediment removed (adopt 80% or higher)
v_s	=	settling velocity of target sediment (100 mm/s or 0.1 m/s for 1 mm particle)
Q	=	Design flow (3 month ARI peak discharge calculate from the Rainfall Intensity – Duration – Frequency curve for Singapore in Figure 7.12.
n	=	turbulence or short-circuiting parameter (adopt 0.5)
A_S	=	The area of the sediment forebay

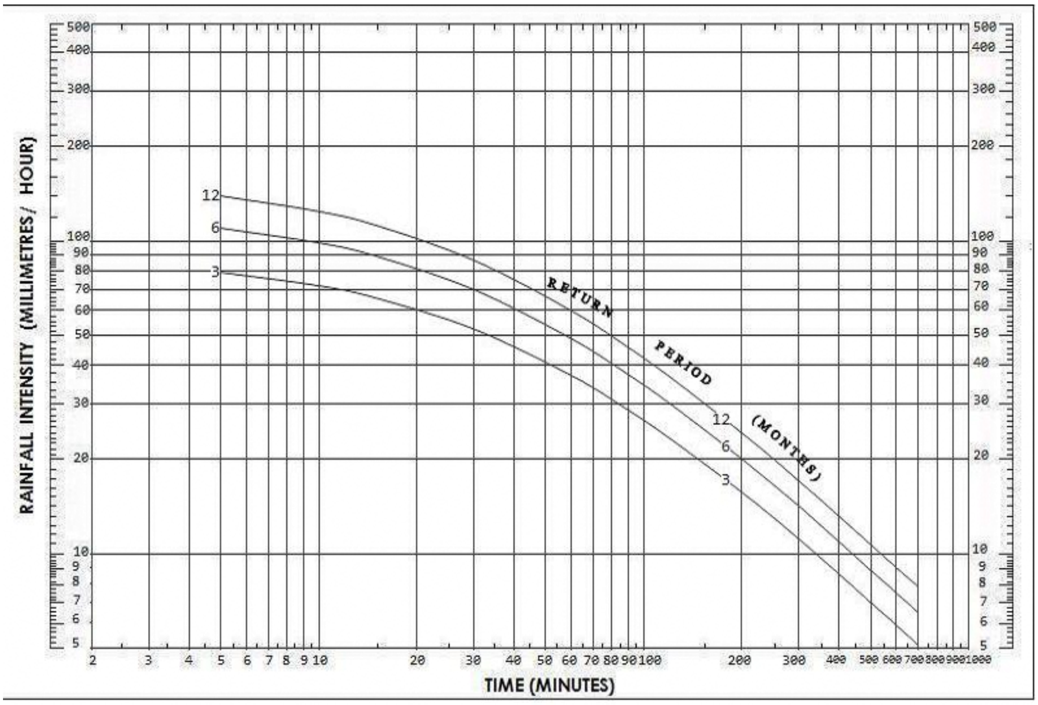


Figure 7.12 IDF Curves for 3-month, 6-month, & 12-month ARI storms

A catchment sediment loading rate (L_o) of 3m^3/ha/year for developed catchments in Singapore may be used to estimate the total sediment loads entering the basin (see Chapter 4 Sedimentation Basin). This volume represents the full range of sediment sizes. In the absence of local sediment particle size distribution, it is not possible to accurately estimate the volume captured at the forebay. A conservative approach is to multiply the capture efficiency (R) by the sediment load estimated applying the catchment loading rate (L_o).

The coarse sediment forebays will contain large rocks for energy dissipation and be underlain by filter material to promote drainage following storm events. The depth of the forebay should not be greater than 0.3m below the surface of the filter media. As the sediment forebay will be filled with rocks and gravels, a porosity factor (ρ) should be applied to estimate the volume of voids within the sediment forebay that is available for deposition of sediments.

7.4.3.3 Streetscape Applications

As bioretention pods are not continuous systems, streetscape applications need to carefully consider the locations of inlets to the bioretention pods so as not to increase the width of channel flow along the street leading to the inflow to these systems.

7.4.4 Step 4: Specify the bioretention media characteristics

7.4.4.1 Specify the Bioretention Filter Media Characteristics

Generally, three types of media are required in the bioretention component of bioretention swales (refer Figure 7.1 and Figure 7.7).

Filter Media

The filter media layer provides the majority of the pollutant treatment function, through fine filtration and also by supporting vegetation. The vegetation enhances filtration, keeps the filter media porous, provides substrate for biofilm formation that is important for the uptake and removal of nutrients and other stormwater pollutants. As a minimum, the filter media is required to have sufficient depth to support vegetation. Typical depths are between 600-1000 mm with a minimum depth of 400mm accepted in depth constrained situations. It is important to note that if deep rooted plants such as trees are to be planted in bioretention swales, the filter media must have a minimum depth of 1000 mm to provide sufficient plant anchoring depth.

Saturated hydraulic conductivity should remain between100-300 mm/hr (and should not be greater than 500 mm/hr. Saturated hydraulic conductity less than 100 mm/hr (but higher than 50 mm/hr) shall be accepted with caution. The following procedure is recommended in determine the appropriate soil filter media to match the design saturated hydraulic conductivity:

- Identify available sources of a suitable base soil (i.e. topsoil) capable of supporting vegetation growth such as a sandy loam or sandy clay loam. In-situ topsoil should be considered first before importing soil. Any soil found to contain high levels of salt (see last bullet point), extremely low levels of organic carbon (< 3%), or other extremes considered retardant to plant growth and microbial activity should be rejected. The base soil must be free from pollutants like heavy metals, excessive nutrient and organic pollutants that may affect water quality of the filtrate.
- The base soil must also be structurally sound and not prone to structural collapse as this can result in a significant reduction in saturated hydraulic conductivity. The risk of structural collapse can be reduced by ensuring the soil has a well graded particle size distribution with a combined clay and silt fraction of < 12%.
- Using laboratory analysis, determine the saturated hydraulic conductivity of the base soil using standard testing procedures. (In Australia, reference is made to AS 4419-2003 Appendix H Soil Permeability or refer to Constant head method BS1377-5:1990 for Singapore). A minimum of five samples of the base soil should be tested. Any occurrence of structural collapse during laboratory testing must be noted and an alternative base soil sourced.
- To amend the base soil to achieve the desired design saturated hydraulic conductivity either mix in a loose non-angular sand (to increase saturated

hydraulic conductivity) or conversely a loose loam (to reduce saturated hydraulic conductivity).

- The required content of sand or clay (by weight) to be mixed to the base soil will need to be established in the laboratory by incrementally increasing the content of sand or clay until the desired saturated hydraulic conductivity is achieved. The sand or clay content (by weight) that achieves the desired saturated hydraulic conductivity should then be adopted on-site. A minimum of five samples of the selected base soil and sand (or clay) content mix must be tested in the laboratory to ensure saturated hydraulic conductivity is consistent across all samples. If the average saturated hydraulic conductivity of the final filter media mix is within ± 20% of the design saturated hydraulic conductivity, then the filter media can be adopted and installed in the bioretention system. Otherwise, further amendment of the filter media must occur through the addition of sand (or clay) and retested until the design saturated hydraulic conductivity is achieved.
- The base soil must have sufficient organic content to establish vegetation on the surface of the bioretention system. If the proportion of base soil in the final mix is less than 3%, it may be necessary to add organic material. This should not result in more than 10% organic content and should not alter the saturated hydraulic conductivity of the final filter media mix.
- The pH of the final filter media is to be amended (if required) to between 5.5 and 7.5. If the filter media mix is being prepared off-site, this amendment should be undertaken before delivery to the site.
- The salt content of the final filter media (as measured by EC1:5) must be less than 0.63 dS/m for low clay content soils like sandy loam. (EC1:5 is the electrical conductivity of a 1:5 soil/ water suspension).
- Testing of this soil property should be undertaken prior to their placement during construction. It should also be noted that soil hydraulic conductivity will vary after placement and is expected to initially decrease due to hydraulic compaction during operation. With maturity of plant growth, the soil hydraulic conductivty can be expected to recover to asymptote to an equilibrium level comparable to its original value.

The selection of suitable soil filter media is a topic of continuing research. Further information can also be obtained from “Guidelines for Filter Media for Biofiltration System by FAWB (Facility for Advancing Water Biofiltration).

Transition Layer

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

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

The transition layer is recommended to be 100 mm thick.

The addition of a transition layer increases the overall depth of the bioretention system and may be an important consideration for some sites where total depth of the bioretention system may be constrained. In such cases, two options are available to reduce the overall depth of the system, ie.

- the use of a sand drainage layer and/or perforated pipes with smaller slot sized may need to be considered.
- use a geotextile layer with a mesh size specified to be between 0.7 to 1mm. (This option should be an option of last resort as the risk of installing inappropriate liner is high).

Drainage Layer

The drainage layer is used to convey treated flows to the outlet via a perforated under-drainage system. The composition of the drainage layer is to be considered in conjunction with the selection and design of the perforated under-drainage system (refer to Section 0) as the slot sizes in the perforated pipes may determine the minimum drainage layer particle size to avoid washout of the drainage layer into the perforated pipe system.

Gravel is the preferred media for the drainage layer to match with the typical slot size of typical perforated or slotted under-drains.

However, there may be circumstances where site conditions constraint the depth of the bioretention system. In such cases, it may be possible to use sand as the drainage layer media to avoid having to provide a transition layer between the filter media and the drainage layer. The drainage layer is to be a minimum of 200 mm thick and it is advisable that the drainage media is washed prior to placement in bioretention system to remove any fines.

Submerged Zone

The submerged zone should be comprised of a mix of medium to coarse sand and carbon, or a mix of fine gravel and carbon. The carbon source should be a mix of 5% mulch and 5% hardwood chips, by volume.

A depth of 450mm has been shown to be optimal (Zinger *et al*., 2007), however the feasibility of this will be determined by site conditions. A minimum of 300mm is required for this zone to be effective. A submerged zone of 300mm will protect against drying for up to five weeks of continuous null inflow. In climates where dry periods are likely to exceed five weeks, the submerged zone should be increased in depth by 120mm for every additional week of expected zero inflows. It is also important to note that a 50mm transition layer should separate the filter media and submerged zone. This will prevent the leaching of pollutant and nutrients by ensuring that the filter media does not become permanently saturated.

7.4.5 Step 5: Under-drain design and capacity checks

The slotted collection pipes at the base of bioretention filter media collect treated water for conveyance downstream. The collection pipes are sized to ensure flow through the filter media is not choked (or impeded) by the collection system.

The recommended maximum diameter of the perforated pipes is 100 mm to minimise the required thickness of the drainage layer. Either flexible perforated pipe (e.g. agricultural pipe) or slotted PVC pipes can be used, however care needs to be taken to ensure that the slots in the pipes are not too large that sediment would freely flow into the pipes from the drainage layer.

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

- Ensure the perforations are adequate to pass the maximum filtration rate of the media
- Ensure the pipe itself has sufficient capacity to convey the design flow (ie. the maximum filtration rate multiplied by the surface area).
- Ensure that the material in the drainage layer will not be washed into the perforated pipes.

7.4.5.1 Maximum filtration rate

The maximum filtration 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 filtration rate to ensure the filter media drains freely and does not 'choke' the system.

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

$$Q_{\max} = k \cdot L_b \cdot W_{base} \cdot \frac{h_{\max} + d}{d} \qquad \text{Equation 7.2}$$

Where

k	=	hydraulic conductivity of the soil filter (m/hr)
W_{base}	=	average width of the ponded cross section above the soil filter (m)
L_b	=	length of the bioretention zone (m)
h_{max}	=	depth of pondage above the sand filter (m)
d	=	depth of filter media (m)

There are two possible configurations for an underdrain in a bioretention system with a submerged zone:

1. Perforated collection pipe with riser outlet

In this configuration, the collection pipe(s) is placed in the drainage layer with an elbow to create a riser outlet to raise the invert. The collection pipe(s) does not need to be sloped as the outlet is elevated. Slotted PVC pipes are preferable to flexible perforated ag-pipe, as they are easier to clean and ribbed pipes are likely to retain moisture which may attract plant roots into pipes, however this necessitates a drainage layer to ensure that finer material from the filter media and transition layers are not washed into the collection pipe(s). The upstream end of the collection pipe should extend to the surface to allow inspection and maintenance; the vertical section(s) of the pipe should be unperforated and capped. Where more than one collection pipe is required, these should be spaced no further than 1.5 m apart.

The following need to be checked:

a) Perforations in pipe are adequate to pass the maximum infiltration rate.
b) Pipe has sufficient capacity to convey the treated water; this component should be oversized to ensure it does not become a choke in the system.
c) Material in the drainage layer will not wash into the perforated pipes.

2. Riser outlet only (no perforated pipe)

A collection pipe is not strictly necessary in a bioretention system with a submerged zone; inclusion of a riser outlet confines exit flow to be via this path and the drainage layer can act as a surrogate collection pipe. The riser outlet should extend to the surface to allow inspection and maintenance.

The following need to be checked:

a) Pipe has sufficient capacity to convey the treated water; this component should be oversized to ensure it does not become a choke in the system.
b) Material in the drainage layer will not wash into the riser outlet.

7.4.5.2 Spacing of perforated pipes

Spacing of perforated pipes should be sufficiently close together to ensure effective drainage of the bioretention system. With a smaller aspect ratio associated with bioretention basins compared with bioretention swales, the maximum spacing of the perforated pipes can be increased to 2.5 – 3 m, especially for large bioretention basins (> 100 m^2).

Where possible the perforated pipes are to grade at a minimum of 0.5% towards the overflow pit to ensure effective drainage. A slope of 0.5% is adopted simply for convenience. In reality, the discharge capacity of the perforated pipe is not dependent on this slope since maximum discharge condition is reached when the soil is saturated and water ponded to the full extended detention depth. Bioretention systems can operate equally effectively with the underdrain laid at near-zero (but positive) slope. Grading the base of the bioretention system towards the pit and placing the perforated pipes (and the drainage layer) on this grade is a recommended approach to ensuring that the pipes are laid on a positive slope.

Perforated pipes should not use a geofabric wrapping, as this is a potential location for blockage and would require a complete resetting of the bioretention system. Where perforated pipes are supplied with geofabric wrapping, it is to be removed before installation.

7.4.5.3 Perforations inflow check

To ensure the perforated under-drainage system has sufficient capacity to collect and convey the maximum filtration rate, it is necessary to determine the capacity for flows to enter the under-drainage system via the perforations in the pipes. If the capacity of the drainage system is unable to collect the maximum filtration rate additional under-drains will be required.

To calculate the flow through the perforations, orifice flow can be assumed and the sharp edged orifice equation used as given in the following equation.

$$Q_{perf} = B \cdot C_d \cdot Ao\sqrt{2 \cdot g \cdot h} \qquad \text{Equation 7.3}$$

Where

Q_{perf}	=	Flow rate through perforations (m^3/s)
B	=	Blockage factor (= 0.5)
C_d	=	Orifice discharge coefficient (= 0.6)
A_o	=	Total area of the orifices (m^2)
h	=	Assuming drainage layer is saturated, driving head is half the depth of the drainage layer (m)

The total area of the orifice (A_o) is a function of the number of perforations in the pipe. This information is typically provided in the manufacturer's specifications. The maximum driving head is equal to the depth of the filter media plus the extended detention depth, if extended detention is provided.

It is conservative, but reasonable to use a blockage factor to account for partial blockage of the perforations by the drainage layer media. A blockage factor of 0.5 is considered adequate.

7.4.5.4 Perforated pipe capacity

After confirming the capacity of the under-drainage system to collect the maximum filtration rate, it is necessary to confirm the conveyance capacity of the under-drainage system is sufficient to convey the collected runoff. The Colebrook-White equation can be applied to estimate the flow rate in the perforated pipe.

$$Q_{pipe} = A_p\left[-2\left(2gD_pS_f\right)^{0.5} log\left(\frac{k}{3.7D}+\frac{2.51v}{D_p\left(2gD_pS_f\right)^{0.5}}\right)\right] \quad \text{Equation 7.4}$$

Where

Q_{pipe}	=	Flow rate through the perforated pipe (m^3/s)
A_p	=	Pipe cross sectional area (m^2)
D_p	=	Pipe diameter (m)
S_f	=	Hydraulic gradient (m/m)
k	=	Hydraulic roughness
v	=	Kinematic viscosity of water (m^2/s)

One end of the under-drains should be extended vertically to the surface of the bioretention system to allow inspection and maintenance when required. The vertical section of the under-drain should be a non-perforated or slotted pipe and capped to avoid short-circuiting of flows directly to the drain.

7.4.6 Step 6: Check requirements for impermeable lining

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

It is important to note that for unlined bioretention systems with submerged zones, the bottom and sides of the submerged zone will need to be lined in order to maintain a permanent pool of water.

7.4.7 Step 7: Size overflow pit

The intention of the high flow design is to convey safely the minor floods (eg. 10-year ARI flows) with the same level of protection that a conventional stormwater system provides. Bioretention basins are typically served with either grated overflow pits or conventional side entry pits (located downstream of an inlet) to transfer flows into an underground pipe network (the same pipe network that collects treated flows).

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

Overflow pits can also be located external to bioretention basins, potentially in the form of convention side entry pits associated with the street kerb and gutter immediately downstream of the inlet to the basin. In this way the overflow pit can operate in the same way as a conventional drainage system, with flows entering the pit only when the bioretention system is at maximum ponding depth. This is illustrated in Figure 7.13.

Figure 7.13 A conventional side entry pit for overflow from Bioretention Pod. Once inundated to street level, stormwater will no longer enter the bioretention raingarden but will instead be conveyed to the adjoining side entry pit.

A grated overflow pit is sized based on the governing flow condition; weir flow or submerged flow conditions. A weir equation can be used to determine the length of weir required (assuming free overfall conditions). An orifice equation is used to estimate the required area between openings in the grate cover (assuming drowned outlet conditions). The larger of the resulting required dimensions to accommodate the two flow conditions should be adopted. In sizing the overflow pit for both drowned and free flowing conditions, it is recommended that a blockage factor that assumes the orifice is 50% blocked be used.

The weir equation for free flowing conditions is given by:

$$Q_{\min or} = Q_{weir} = B \cdot C_w \cdot L \cdot h_w^{\ 3/2} \qquad \text{Equation 7.5}$$

Where

Q_{weir}	=	Flow over weir pit (m^3/s)
B	=	Blocked factor (assumed to be 50%)
C_w	=	Weir coefficient (adopt 1.7)
L	=	Length of weir (m)
h_w	=	Flow depth above weir (m)

A standard sized pit can be selected with a perimeter at least the same length as the required weir length.

The orifice equation for drowned outlet conditions is given by:

$$Q_{\min or} = Q_{grate} = B.C_d.A_{grate}\sqrt{2gh_w} \qquad \text{Equation 7.6}$$

Where

Q_{grate}	=	Flow rate under drowned conditions (m^3/s)
A_{grate}	=	Area of perforations in inlet grate (m^2)
h_w	=	Flow depth above weir (m)
C_d	=	Discharge coefficient (0.6)

7.4.8 Step 8: Specify Vegetation

Advice from the party who will takeover the features for maintenance (e.g. NParks, Town Councils, MCST etc.) should be sought in determining the lists of plants suitable for bioretention basins. Consultation with landscape architects is recommended when selecting vegetation, to ensure the treatment system compliments the landscape design of the area.

7.4.9 Step 9: Verification Checks

Once the detailed design is complete, a final check should be undertaken to confirm that vegetation will be protected from scour during flood events and that the final design will achieve the required treatment performance.

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

Velocities should be kept below 0.5 m/s for flows up to the 10-year ARI peak discharges and less than 2.0 m/s for events up to the 100-year ARI discharges.

If the design of the bioretention basin (i.e. the treatment area) changes to ensure the above criteria are met, the performance of the bioretention system given the new treatment area should be checked against the sizing curves given in Figure 7.8 to Figure 7.10.

7.4.10 Design Calculation Summary

A summary of the key design elements of a bioretention basin are presented in the following table.

Bioretention basins

	CALCULATION TASK			
	Catchment Characteristics			
	- Land Uses			
	Residential		m^2	
	Commercial		m^2	
	Roads		m^2	
	- Fraction Impervious			
	Residential		-	
	Commercial		-	
	Roads		-	
	Weighted average			☐
	Conceptual Design			
	Basin Area		m^2	
	Maximum width		m	
	maximum ponding depth (extended detention)		m	
	Filter media type (hydraulic conductivity)		mm/hr	
	Identify design criteria			
	Minor flood		year ARI	
	Major flood		year ARI	
1	**Confirm treatment performance and concept design**			
	Reduction in TSS		**%**	
	Reduction in TP		**%**	
	Reduction in TN		**%**	☐
2	**Estimate design flow rates**			
	Time of concentration			
	Estimate from flow path length and velocities		minutes	☐
	Identify rainfall intensities			
	Design Rainfall Intensity for minor flow		mm/hr	
	Design Rainfall Intensity for major flow		mm/hr	☐
	Design runoff coefficient		-	
	refer to the Singapore Code of Practice on Surface Water Drainage(2000)			☐
	Peak design flows			
	Minor Storm (selected design storm ARI and flow)	ARI	m^3/s	
	Major Storm (selected design storm ARI and flow)	ARI	m^3/s	
	$Q_{\text{max infiltration}}$		m^3/s	☐
3	**Design inflow system**			
	Adequate erosion and scour protection?		y/n	☐
	Coarse Sediment Forebay Required?			
	Volume (V_s)		m^3	
	Area (A_s)		m^2	
	Depth (D)		m	
				☐
	Check flow widths in upstream channel			
	Minor storm flow width		m	
	CHECK ADEQUATE LANES TRAFFICABLE			
				☐
	Kerb opening width			
	Kerb opening length		m	
				☐
4	**Specify bioretention media characteristics**			
	Filter media hydraulic conductivity		mm/hr	
	Filter media depth		mm	
	Drainage layer media (sand or fine screenings)			
	Drainage layer depth		mm	
	Transition layer (sand) required			
	Transition layer depth		mm	☐
5	**Under-drain design and capacity check**			
	- Perforations inflow check			
	Pipe diameter		mm	
	Number of pipes			

	Capacity of perforations	m^3/s	
	CHECK PERFORATION CAPACITY > Filter media maximum infiltration rate		
6	**Check requirement for impermeable lining**		
	Soil hydraulic conductivity	mm/hr	
	Filter media hydraulic conductivity	mm/hr	
	MORE THAN 10 TIMES HIGHER THAN IN-SITU SOILS?		
7	**Size overflow pit**		
	System to convey minor floods	L x W	
	Flow capacity for the overflow pits	m^3/s	
8	**Vegetation Specification**		
9	**Verification Checks**		
	Velocity for Minor Storm (<0.5m/s)	m/s	
	Velocity for Major Storm (<2.0m/s)	m/s	

7.5 Checking Tools

The following sections provide a number of checking aids for designers and referral authorities. Additional advice on construction and maintenance is provided.

Checklists have been provided for:

- Design assessments
- Construction (during and post)
- Maintenance and inspections

7.5.1 Design Assessment Checklist

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

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

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

7.5.2 Construction Advice

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

7.5.2.1 Building Phase Damage

Protection of filtration media and vegetation is important during the building phase. Uncontrolled building site runoff is likely to cause excessive sedimentation, introduce weeds and litter and require replanting following the building phase.

To minimise the impact of construction activities on the site, it is recommended that the bioretention system be installed in stages. For example, temporary protection of a bioretention basin can be achieved by using a temporary arrangement of a suitable geofabric covered with shallow topsoil (e.g. 25mm) and instant turf (laid perpendicular to flow path) (Leinster, 2006). Such a system will provide erosion and sediment control. At the completion of construction activities, the temporary protection can be removed (along with the collected sediment) and the system planted in accordance with the planting schedule.

It is also recommended that a silt fence be installed around the periphery of the basin to exclude silt and restrict access. The silt fence is removed once construction is completed.

7.5.2.2 Traffic and Deliveries

It is important to ensure traffic and deliveries do not access bioretention basins during construction. Traffic can compact the filter media and cause preferential flow paths, while deliveries can block filtration media (if placed above media). Washdown wastes (eg. concrete) can also cause blockages in the filtration media.

Bioretention areas should be fenced off during building phase and controls implemented to avoid washdown wastes.

Checklist 1: Bioretention basin design assessment checklist

BIORETENTION BASIN DESIGN ASSESSMENT CHECKLIST				
Basin Location:				
Hydraulics:	Minor Flood (m^3/s):	Major Flood (m^3/s):		
Area:	Catchment Area (ha):	Bioretention Area (ha):		
TREATMENT			**Y**	**N**
Treatment performance verified from curves?				
BIORETENTION MEDIA AND UNDER-DRAINAGE			**Y**	**N**
Design documents bioretention area and extended detention depth as defined by treatment performance requirements.				
Overall flow conveyance system sufficient for design flood event(s)?				
Where required, bypass sufficient for conveyance of design flood event?				
Where required scour protection provided at inflow point to bioretention?				
Bioretention media specification includes details of filter media, drainage layer and transition layer (if required)?				
Design saturated hydraulic conductivity included in specification?				
Transition layer provided where drainage layer consists of gravel (rather than coarse sand)?				
Perforated pipe capacity > infiltration capacity of filter media?				
Selected filter media hydraulic conductivity > 10 x hydraulic conductivity of surrounding soil?				
Liner provided if selected filter media hydraulic conductivity < 10 x hydraulic conductivity of surrounding soil?				
Maximum spacing of collection pipes <1.5m?				
Collection pipes extended to surface to allow inspection and flushing?				
Maximum upstream flood conveyance complies with Singapore surface water drainage requirements?				
BASIN			**Y**	**N**
Bioretention area and extended detention depth documented to satisfy treatment requirements?				
Overflow pit crest set at top of extended detention?				
Maximum ponding depth will not impact on public safety?				
Maintenance access provided to surface of bioretention system (for larger systems)?				
Protection from coarse sediments provided (where required) with a sediment forebay?				
Protection from gross pollutants provided (where required)?				
LANDSCAPE			**Y**	**N**
Plant species selected can tolerate periodic inundation and design velocities?				
Bioretention design and plant species selected integrate with surrounding landscape or built environment design?				
COMMENTS				

7.5.2.3 Inlet Erosion Checks

It is good practice to check the operation of inlet erosion protection measures following the first few rainfall events to ensure the system would not be affected by scour. Should problems occur in these events the erosion protection should be enhanced.

7.5.2.4 Timing for Planting

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

7.5.2.5 Planting Strategy

A planting strategy for a development will depend on the timing of construction 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 control facilities (to be planted out following building).

7.5.2.6 Perforated Pipes

Corrugated perforated HDPE pipes are normally used as perforated subsoil pipes for Bioretention systems. Pipe fittings should be compatible in type and material. There should also be standpipes to facilitate flushing of subsoil pipes. The standpipes should be corrugated non-perforated HDPE pipes with removable end caps.

7.5.2.7 Inspection Openings

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

7.5.2.8 Clean Filter Media

Ensure sand, gravels and other material used in the filter, transition, drainage and other layers are washed prior to placement to remove fines.

7.5.2.9 Construction Inspection Checklist

The following checklist presents the key items to be reviewed when inspecting the bioretention basin during and at the completion of construction. The checklist is to be used by Construction Site Supervisors and local authority Compliance Inspectors to ensure all the elements of the bioretention basin have been constructed in accordance with the design. If an item is ticked as unsatisfactory, appropriate actions must be specified and delivered to rectify the construction issue before final inspection sign-off is given.

Checklist 2: Construction Inspection Checklist

BIORETENTION BASIN CONSTRUCTION INSPECTION CHECKLIST			
Site:		**Date:**	
		Time:	
Constructed By:		**Weather:**	
		Contact During Visit:	

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

FINAL INSPECTION									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of banks				
2. Confirm structural element sizes					7. Under-drainage working				
3. Check batter slopes					8. Inflow systems working				
4. Vegetation as designed					9. Maintenance access provided				
5. Bioretention filter media surface flat and free of clogging					10. Construction generated sediment removed				

COMMENTS ON INSPECTION
ACTIONS REQUIRED
Inspection officer signature:

7.5.3 Maintenance Requirements

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

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

Maintenance is primarily concerned with:

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

Vegetation maintenance will include:

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

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

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

7.5.3.1 Operation & Maintenance Inspection Form

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

Checklist 3: Bioretention basin maintenance checklist

BIORETENTION BASIN MAINTENANCE CHECKLIST				
Inspection Frequency:	Weekly to monthly (adjust according to site requirement)	**Date of Visit:**		
Location:				
Description:				
Site Visit by:				
INSPECTION ITEMS:	**FREQUENCY**	**Y**	**N**	**Action Required (details)**
Sediment accumulation at inflow points?				
Litter within basin?				
Erosion at inlet or other key structures?				
Evidence of dumping (e.g. building waste)?				
Watering of vegetation required?				
Clogging of drainage points (sediment or debris)?				
Evidence of ponding?	Weekly or After rain			
Surface clogging/ algae on filter media surface visible?	Bi-weekly			
Drainage system inspected?				
Vegetation condition satisfactory (density, weeds etc)?				
Trimming/thinning of overgrown vegetation as necessary and removal of weeds.	Monthly			
Damage/vandalism to structures present?				
Flushing of subsoil pipes.	Half-yearly			
Resetting of system required?				
COMMENTS				

Name of ABC Waters Professional: _______________

Registration No. of ABC Waters Professional: _______________

Signature: _______________

Name of Maintenance Agency: _______________

Handing Over Date (TOP or Completion of DLP): _______________

7.6 Bioretention Basin Worked Example

7.6.1 Worked example introduction

A series of bioretention basins (pods), designed as street traffic parking “out-stands” is to be retrofitted into a local street to treat road runoff. The local street is in Singapore. A proposed layout of the bioretention system is shown in Figure 7.14 and an image of a similar system to that proposed is shown in Figure 7.15.

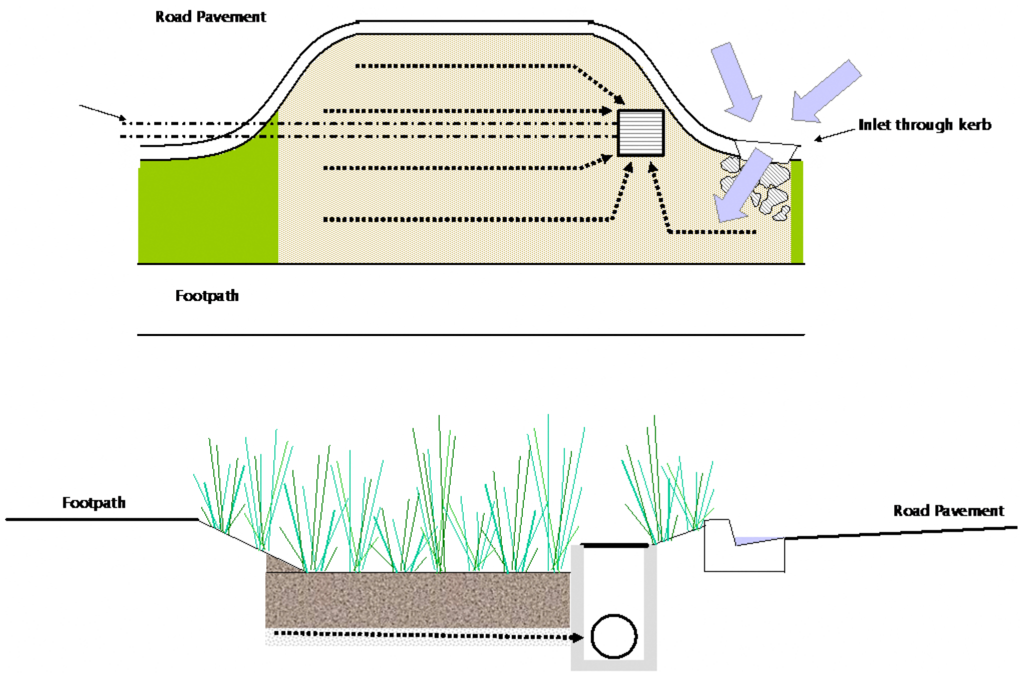


Figure 7.14 Layout of proposed bioretention system

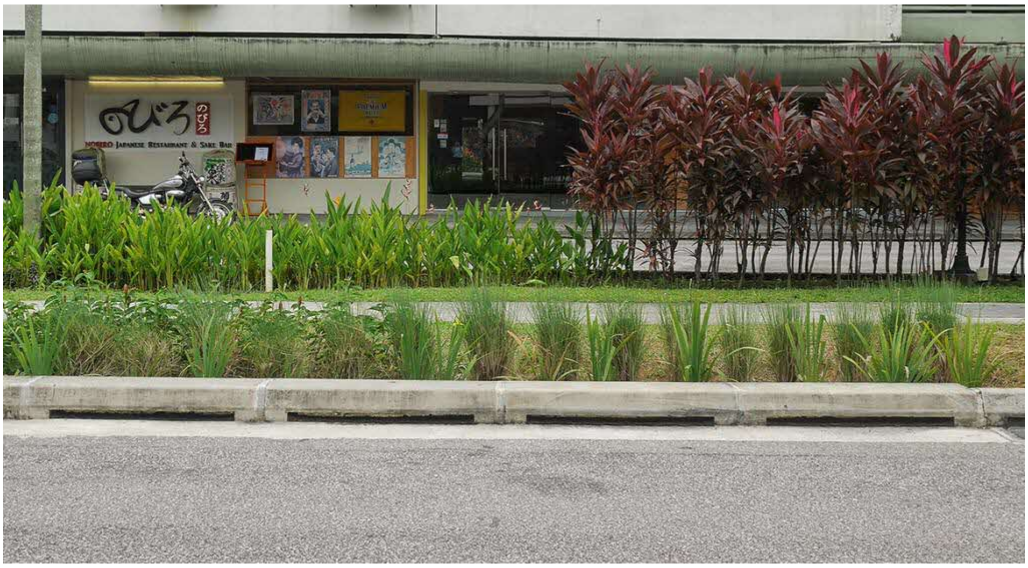


Figure 7.15 Example of a bioretention system in a street (Faber Hill Estate)

Catchment Description

Each of the individual bioretention basins (pods) has a contributing catchment of $100m^2$ road and footpath pavement and $300m^2$ of adjoining properties. Runoff from adjoining properties (approx. 60% impervious) is discharged into the road gutter and, together with road runoff, is conveyed along a conventional roadside gutter to the bioretention pod.

Table 7.1: Catchment properties

Catchment Land Uses	Area (m^2)	% Impervious
Car Park	100	0.9
Allotment	300	0.6
Total	400	0.68

Design Objectives

The aim of the design is to facilitate effective treatment of stormwater runoff while maintaining a level of flood protection for the local street during frequent storm events up to the 10yr ARI event. Effective stormwater quality treatment is described in terms of pollutant load reductions for total suspended solids (TSS), total phosphorous (TP) and total nitrogen (TN).

The key design elements for effective operation of the bioretention basins are:

- road and channel details to convey water into the basins
- detailing inlet conditions to provide for erosion protection
- configuring and designing a system for 'above design' operation that will provide the required 10year ARI flood protection for the local street
- detailing of the bioretention under-drainage system
- specification of the soil filter medium
- landscape layout and details of vegetation.

Constraints and Concept Design Criteria

The proposed site for the bioretention basin has the following characteristics:

Overland flow slope: 1% typical

Soil: clay

A preliminary design was completed for the site. The specifications for the bioretention basins were determined as follows:

- A bioretention basin area of $14m^2$ (minimum) is required to achieve the stipulated water quality objectives for this worked example (pollutant load reductions of 80%, 45% and 45% for TSS, TN and TP respectively)
- The maximum width of the bioretention basin is to be 2m.
- The extended detention depth is 200 mm.
- Filter media shall be a sandy loam.

7.6.2 Calculation Steps

The design of a bioretention system has been divided into the following 9 calculations steps:

Step 1 Confirm treatment size given in conceptual design
Step 2 Determine design flows
Step 3 Design inflow system
Step 4 Specify the bioretention media characteristics
Step 5 Under-drain design and capacity checks
Step 6 Check requirements for impermeable lining
Step 7 High flow route and by-pass design
Step 8 Vegetation Specification
Step 9 Verification Checks

Details for each calculation step are provided below. A design calculation summary has been completed for the worked example and is given at the conclusion of the calculation steps.

Step 1 Confirm treatment size given in conceptual design

The sizing of the bioretention system determined during conceptual design was verified using the sizing curves given in Figure 7.8 to Figure 7.10 The sizing curves, developed for Singapore conditions, give an estimate of the pollutant load reduction for a given treatment size (defined in terms of equivalent impervious treatment area). Verification using the sizing curves requires the following information:

- The extended detention depth (200mm)
- The ratio of treatment area to equivalent impervious area =

$$\frac{14m^2}{\left[(0.9\times 100)+(0.6\times 300)\right]}=5\%$$

The expected pollutant reductions given in the sizing curves for the above criteria are 93%, 77% and 49% for TSS, TP and TN respectively and exceed the design requirements of 80%, 45% and 45%.

Step 2 Determine design flows

<u>Minor and major flood estimation</u>

With a small catchment, the Rational Method is considered an appropriate approach to estimate the 10 and 100 year ARI peak flow rates. The calculation steps are given below.

a. Time of concentration (t_c)

The time of concentration is associated with overland flow and kerb and gutter travel times. In this worked example, the time of concentration is estimated to be approximately 10 minutes.

b. Design rainfall intensities

The rainfall intensity for the 10 year and 100 year ARI event is calculated from the Singapore IDF curve. Data for the 1year flow is extrapolated from the curve. Rainfall intensities for the 100year, 10year and 1year ARI event for 10min storm duration are given below.

	100yr	10yr	1yr
Intensity (mm/hr)	271	190	106

c. Design runoff coefficient

The runoff coefficients are taken from the Singapore Code of Practice on Surface Water Drainage (PUB, 2013). A runoff coefficient of 0.65 and 1.0 is recommended for residential areas (not densely built up) and road catchment, respectively. The weighted average for the catchment given the two land uses is 0.74.

d. Peak Design flows

Apply the Rational Method to determine the peak flows for 10year and 100year ARI events:

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

$$Q_{10} = \frac{0.74 \times 190 \times 400 \times 10^{-4}}{360}$$

$$= 0.016 m^3 / s$$

$$Q_{100} = \frac{0.74 \times 271 \times 400 \times 10^{-4}}{360}$$

$$= 0.02 m^3 / s$$

The Intensity – Duration – Frequency (IDF) curve for 3 month ARI storms is in Figure 7.12. The 3 month ARI flow is calculated as:

$$Q_{3mth} = \left[\frac{0.74 \times 71 \times 400 \times 10^{-4}}{360} \right]$$

$$= 0.006 m^3 / s$$

<u>Maximum filtration rate</u>

The maximum filtration rate, or the flow reaching the perforated pipe in the drainage layer, is estimated by applying Darcy's equation (Equation 7.2) at the saturated hydraulic conductivity of the filter media (assuming no blockage of the media) and the head above the base of the filter media:

$$Q_{\max} = k \cdot L_b \cdot W_{base} \cdot \frac{h_{\max} + d}{d} \quad = 3.4 \text{ m}^3\text{/hr or } 0.0009 \text{ m}^3\text{/s}$$

Given

k	=	hydraulic conductivity of the soil filter (0.18m/hr)
$L_b \cdot W_{base}$	=	average area of the ponded section above the sand filter (14m^2)
h_{max}	=	depth of pondage above the sand filter (0.2m)
d	=	Filter media depth (0.6m)

Step 3 Design inflow system

Inlet Scour Protection

Rock beaching is to be provided in the bioretention basins to manage flow velocities entering from the kerb opening.

Coarse Sediment Forebay

A bioretention system such as the one proposed here should incorporate a coarse sediment forebay to remove coarse sediment from stormwater prior to flowing across the surface of the filter media. The forebay should be designed to:

- Remove particles that are 1mm or greater in diameter from the 3mth ARI storm event.
- Provide appropriate storage for coarse sediment to ensure desilting is required once every year.

The size of the sediment forebay (A_s) is determined by solving Equation 7.1 for a capture efficiency of 80%, i.e.

$$A_s = \frac{nQ}{V_s}\left[(1-R)^{-1/n} - 1\right]$$

Where

R	=	fraction of target sediment removed (adopt 80% or higher)
V_s	=	settling velocity of target sediment (100 mm/s or 0.1 m/s for 1 mm particle)
Q	=	Design flow (3 month ARI peak discharge)
n	=	turbulence or short-circuiting parameter (adopt 0.5)
A_s	=	The area of the sediment forebay
ρ	=	Porosity (adopt 0.4 for gravel)

$$A_s = \frac{(0.5)(0.006)}{0.1}\left[(1-0.8)^{-1/0.5} - 1\right] = 0.72m^2$$

The volume of the sediment forebay is calculated by adopting a mean depth of 0.3 m, ie.

$$V_s = 0.72 \times 0.3 = 0.22m^3$$

Adopting a sediment loading rate of 3 m^3/ha/yr, the clean-out frequency of the sediment forebay is estimated to be 0.22 x 0.4(3 x 0.04) = 0.72 years.

Step 4 Specify the bioretention media characteristics

The bioretention system will have three layers:

- Sandy loam layer as the filter media (600mm)
- Coarse sand transition layer (100mm)
- Fine gravel drainage layer (200mm)

Filter Media Specifications

The filter media shall have the following properties:

- saturated hydraulic conductivity of approximately 180 mm/hr
- particle sizes ranging between: clay 5 – 15 %, silt <30 %, sand 50 – 70 %

- between 5% and 10% organic content
- pH neutral

Transition layer specifications

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

Particle Size	%Passing
1.4mm	100%
1.0mm	80%
0.7mm	44%
0.5mm	8.4%

Drainage layer specifications

The drainage layer is to be 200mm deep of 5mm screenings graded at 0.5% towards the overflow pit.

Step 5 Under-drain design and capacity checks

A single under-drain is to be installed in the drainage layer. The perforated pipe is to be laid on the base of the bioretention system which grades at 0.5 % towards the overflow pit. A standard perforated pipe has the following specifications:

Openings per metre of pipe = 2100mm^2

Slot (opening) width = 1.5mm

Slot length = 7.5mm

No. of rows = 6

Pipe diameter = 100mm

Number of perforations (n) = 186

The flow capacity of the perforations and the pipe need to be checked against the maximum filtration rate (0.0009 m^3/s – determined in Step 2) to ensure the flow through the media is not impeded by the drainage system.

Perforations inflow check

The inlet capacity of a sub-surface drainage system (perforated pipe) is estimated to ensure it is not a choke in the system. To build in conservatism, it is assumed that 50% of the holes are blocked.

The flow capacity of the perforations is calculated using equation 7.3

$$Q_{perf} = B.C_d.nA\sqrt{2gh}$$

When

C_d	=	Discharge coefficient (0.6)
h	=	Assuming drainage layer is saturated, driving head is half the depth of the drainage layer – H = 0.1m
A	=	1.125x10^{-5}m^2/hole
B	=	Blockage factor (adopt 0.5)
n	=	Numbers of holes

$$Q_{perf} = 0.5 \times 0.6 \times 186 \times 1.125 \times 10^{-5} \times \sqrt{2 \times 9.81 \times 0.1}$$
$$= 0.0009 m^3 / s / metre\ \ of\ \ pipe$$

Given the perforated pipe will be 3m in length, the perforated flow for the pipe system is 0.0027 m^3/s. As the perforation flow capacity of the pipe is greater than the maximum filtration rate the perforated pipe, it is adequate in transferring flows from the media.

Perforated pipe capacity

The Colebrook-White equation is applied to estimate the flow rate in the perforated pipe. A slope of 0.5% is assumed[2] and a 100mm perforated pipe (as above) was used. The capacity of this pipe needs to exceed the maximum infiltration rate.

Applying the Colebrook-White Equation (Equation 7.**4**) to calculate the capacity of the perforated pipe

$$Q = A_p\left[-2\left(2gD_pS_f\right)^{0.5} log\left(\frac{k}{3.7D_p} + \frac{2.51v}{Dp\left(2gD_pS_f\right)^{0.5}}\right)\right]$$

Where

D_p = 0.10m

S_f = 0.005m/m

g = 9.81m^2/s

k = 0.007m

v = 1.007 x 10^{-6}

A_p= 0.009m^2

The flow capacity of the pipe is 0.003 m^3/s, which is greater than the infiltration rate. Hence, the perforated pipe specified is adequate for the under-drainage system.

Step 6 Check requirements for impermeable lining

The soils found in Singapore are typically clay with the saturated hydraulic conductivity expected to be ~3.6mm/hr. The sandy loam media that is proposed as the filter media has a hydraulic conductivity of approximately 180 mm/hr. Therefore, the conductivity of the filter media is > 10times the conductivity of the surrounding soils and an impervious liner is not considered to be required.

Step 7 High flow route and by-pass design

The overflow pit (sump) is required to convey 10 year ARI flows safely from above the bioretention system into an underground pipe network. Grated pits are to be used at the upstream end of the bioretention system. There are standard pit sizes to accommodate connection to the underground stormwater pipe.

For the existing 450 mm diameter stormwater pipe, 600 x 600 mm pit will be required.

The size of the pit necessary to convey the overflow is computed assuming both free overfall weir flow and submerged flow conditions. For the free overflow condition, a weir equation is used with the maximum headwater depth (h) above the weir being set by the level difference between the crest of the overflow pit and the invert level of the inflow kerb opening (i.e. 100mm).

[2] A slope of 0.5% is adopted simply for convenience. In reality, the discharge capacity is reached when the soil is saturated and water ponded to the full extended detention depth. Bioretention systems can operate equally effectively with the underdrain laid at near-zero (but positive) slope.

The weir equation is

$$Q_{\min or} = Q_{weir} = B \cdot C_w \cdot L \cdot h_w^{3/2}$$

For the 10year ARI event, assuming a blockage factor (B) and weir coefficient (C) of 0.5 and 1.7, respectively, the weir length is

$$L = \frac{Q_{\min or}}{B.C_W.H^{\frac{3}{2}}} = \frac{0.016}{0.5 \times 1.7 \times 0.1^{\frac{3}{2}}} = 0.08m$$

A 0.08m weir length is equivalent to a 200mm by 200mm pit – smaller than the standard 600 mm by 600 mm pit.

For drowned outlet conditions, the orifice equation is used:

$$Q = B.C_d.A\sqrt{2gh}$$

For the minor flow event, given a discharge coefficient of 0.6, the required area of the pit is

$$A = \frac{0.016}{0.5 \times 0.6 \times \sqrt{2 \times 9.81 \times 0.1}}$$
$$= 0.048m^2$$

The equivalent pit dimensions for the drowned outlet condition are 200mm by 200mm – smaller than the standard 600 mm by 600 mm pit.

Hence, the 600mm by 600mm pit is to be adopted.

Step 8 Vegetation Specification

Consultation with the maintenance party is required in determining the list of suitable plant species for the proposed bioretention basin. A list of the commonly used plants in bioretention basin is in Section 6.7.

Step 9 Verification Checks

Flows for the 10yr ARI (Q_{10}) and 100yr ARI (Q_{100}) storm events may be conveyed through the bioretention system. A check for vegetation scouring is completed by checking those velocities through the bioretention system are below 0.5m/s and 2.0 m/s for the 10yr ARI and 100yr ARI event, respectively. The scour check is performed using Equation 7.6.

Given the width of the basin is 2m and the extended detention is 0.2m, the susceptible flow area is 0.4m^2. Hence,

$$V_{10\,year} = \frac{Q_{10}}{A} = 0.04m/s < 0.5m/s$$

$$V_{100\,year} = \frac{Q_{100}}{A} = 0.05m/s < 2.0m/s$$

Hence, bioretention system can satisfactorily convey the peak 10yr and 100yr ARI flood, minimising the potential for scour.

7.6.3 Calculation summary

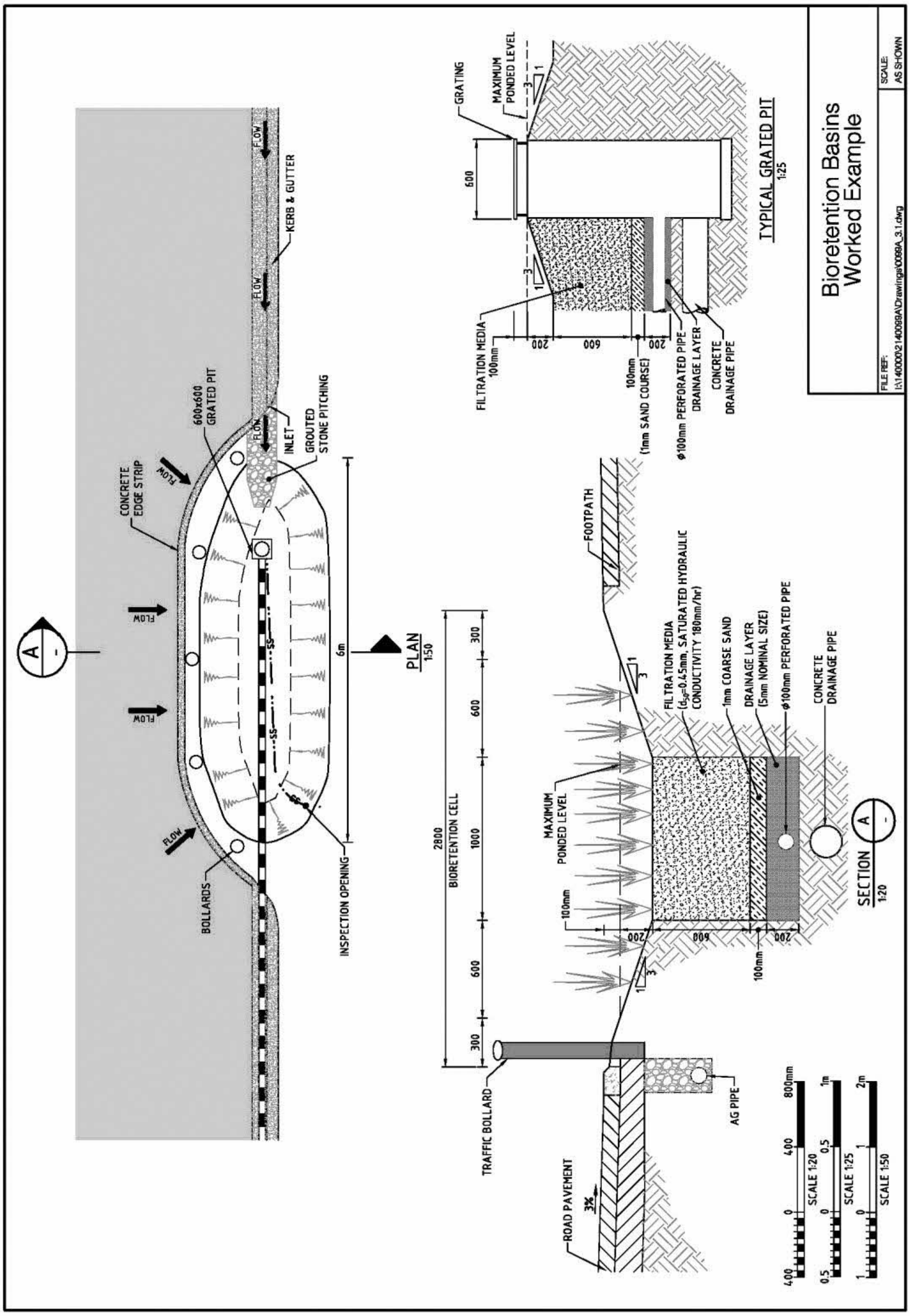
The sheet below shows the results of the design calculations.

Bioretention basins

CALCULATION TASK			
Catchment Characteristics			
- Land Uses			
Residential	300	m^2	
Commercial	0	m^2	
Roads	100	m^2	
- Fraction Impervious			
Residential	0.6	-	
Commercial	0	-	
Roads	0.9	-	
Weighted average	0.74		✓
Conceptual Design			
Basin Area	14	m^2	
Maximum width	2	m	
maximum ponding depth (extended detention)	0.2	m	
Filter media type (hydraulic conductivity)	180	mm/hr	
Identify design criteria			
Minor flood	10	year ARI	
Major flood	100	year ARI	
1. Confirm treatment performance and concept design			
Reduction in TSS	93	%	
Reduction in TP	77	%	
Reduction in TN	49	%	✓
2. Estimate design flow rates			
Time of concentration			
estimate from flow path length and velocities	10	minutes	✓
Identify rainfall intensities			
station used for IFD data:	Singapore		
Design Rainfall Intensity for minor flow	190	mm/hr	
			✓
Design runoff coefficient		-	
(refer to the Singapore Code of Practice on Surface Water Drainage(2000))	0.74		✓
Peak design flows			
Q_{minor}	0.016	m^3/s	
Q_{major}	0.022	m^3/s	
Q_{infil}	0.0009	m^3/s	✓
3. Design inflow system			
Adequate erosion and scour protection?	yes	y/n	✓
Coarse Sediment Forebay Required?	yes		
Volume (V_s)	0.22	m^3	
Area (A_s)	0.72	m^2	
Depth (D)	0.3	m	
			✓
Check flow widths in upstream channel			
Minor storm flow width	0.95	m	
CHECK ADEQUATE LANES TRAFFICABLE	OK		
			✓
Kerb opening width			
Kerb opening length	0.62	m	
			✓
4. Specify bioretention media characteristics			
Filter media Max. Filtration rate	180	mm/hr	
Filter media Max. Filtration rate	0.0009	m^3/s	
Filter media depth	600	mm	
Drainage layer media (sand or fine screenings)	gravel		
Drainage layer depth	200	mm	
Transition layer (sand) required	yes		
Transition layer depth	100	mm	✓
5. Under-drain design and capacity check			
pipe diameter	100	mm	
Number of pipes	1		
total pipe capacity	0.003	m^3/s	
Capacity of perforations	0.02	m^3/s	
CHECK PERFORATION CAPACITY > FILTER MEDIA CAPACITY	OK		✓
6. Check requirement for impermeable lining			
Soil hydraulic conductivity	10	mm/hr	
Filter media hydraulic conductivity	180	mm/hr	
MORE THAN 10 TIMES HIGHER THAN IN-SITU SOILS?	yes		✓
7. Size overflow pit			
System to convey minor floods	600x600	L x W	✓
8. Vegetation Specification			✓
9. Verification Checks			
Velocity for Minor Storm (<0.5m/s)	0.03	m/s	
Velocity for Major Storm (<2.0m/s)	0.06	m/s	
Treatment performance consistent with Step 1	yes		✓

7.6.4 Construction drawings

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



7.7 Case Study

NUS SCHOOL OF DESIGN & ENVIROMENT

The bioretention system at the NUS School of Design and Environment is a component part of a stormwater management strategy to treat, detain, and harvest rainwater, and to showcase ABC Waters design. The bioretention system consists of a 4 cascading bioretention basins. Excess water from a basin overflow via a weir to the adjacent basin downstream. The first basin (B1) receives water from Water Feature Pond 1 while the last basin (B4) discharges excess water to the detention tank through an overflow sump. Filtrate from all 4 basins enters Water Feature Pond 2. Water Feature Pond 1 and 2 are showcase features that demonstrate the results of treatment through the bioretention system. The schematic diagram is in Figure 7.17 and the design calculation by ABC Waters Professionals is also given in Page 45-47.

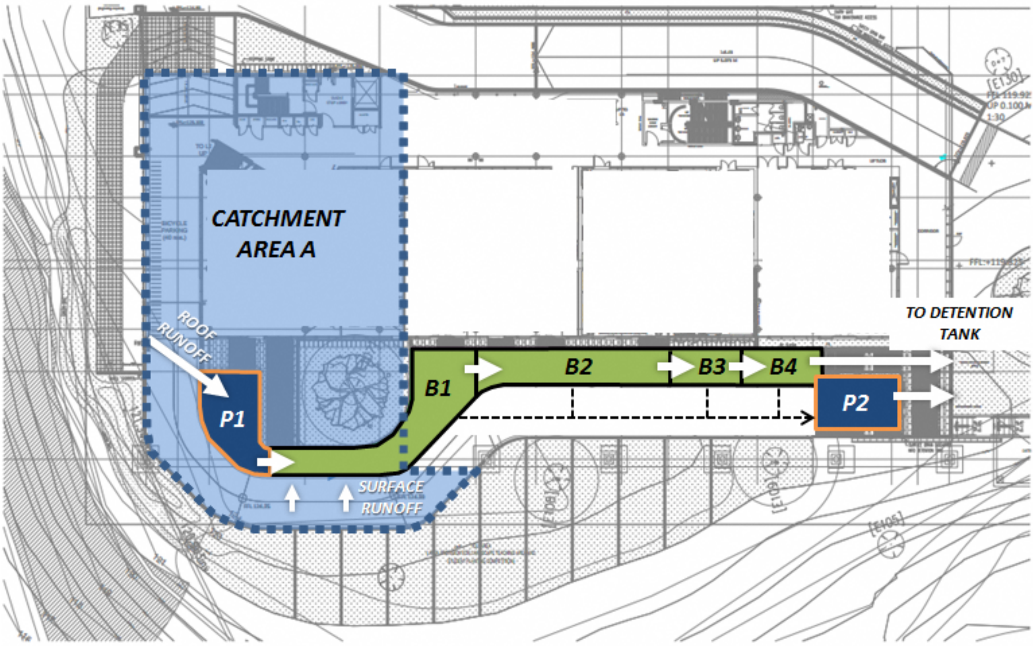


Figure 7.16 Site Plan

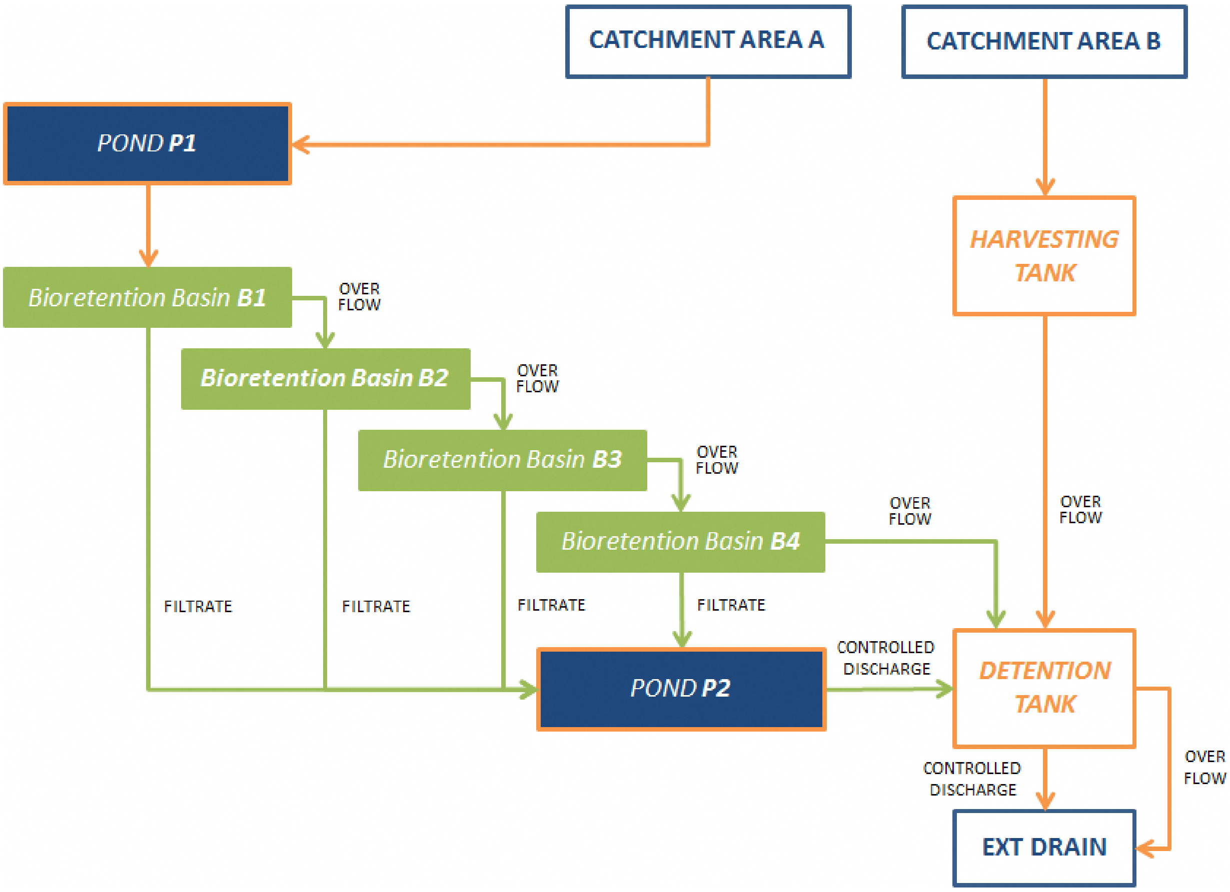


Figure 7.17 Schematic diagram of the bioretention and pond system

Credit is due to the following contributors for the case study:
NUS
Surbana Jurong Pte Ltd
Kajima Overseas Asia Pte Ltd
Netatech Engineering Ptd Ltd.
ABCWP (IES) Mr Sam Ko Luan Bock
ABCWP (SILA) Mr Koh Jiann Bin

Hydraulic Calculations for Bioretention Basin

Project: NUS SDE4

DESIGN FLOW:

(Refer to IDF Graph of the PUB Code of Practice on Surface Drainage 6th Edition)

ARI	Time of Concentration, t_c	Rainfall Intensity, I	Runoff Coefficient, C
	[min]	[mm/hr]	
3-mth	5	79.4	1
10-yr	5	217	1
100-yr	5	304	1

Bio-retention Basin	Catchment Area, A_c	Bio-retention Area, A_r	Ratio, A_r / A_c	Peak Flow at t_c = 5min, $Q = C . I . A_c / 360$		
				3-mth ARI	10-yr ARI	100-yr ARI
	[m^2]	[m^2]		[m^3/s]		
Section B1	-	83.20	-	0.024	0.065	0.090
Section B2	-	58.90	-	0.024	0.065	0.090
Section B3	-	21.40	-	0.024	0.065	0.090
Section B4	-	22.40	-	0.024	0.065	0.090
Total:	1069.50	185.90	17.38%	0.024	0.065	0.090

HIGH FLOW ROUTE & BY-PASS DESIGN:

(Refer to Section 7.4.7 of the PUB Engineering Procedures for ABC Waters Design Features)

Flow Condition	Bio-retention Basin	Discharge Coefficient, C_w	Headwater Depth, H	Blockage Factor, b	Required Weir Length, $L = Q / (b . C_w . H^{3/2})$			Weir Length Provided	Check	Overflow Sump Opening Dimensions	Sump Quantity
					3-mth ARI	10-yr ARI	100-yr ARI				
			[m]		[m]			[m]		[m] x [m]	
Weir	Section B1	1.7	0.1	0.5	0.88	2.40	3.36	2.50	OK	-	-
Weir	Section B2	1.7	0.1	0.5	0.88	2.40	3.36	3.20	OK	-	-
Weir	Section B3	1.7	0.1	0.5	0.88	2.40	3.36	3.20	OK	-	-
Weir	Section B4	1.7	0.1	0.5	0.88	2.40	3.36	3.20	OK	0.6 x 1.0	1

Flow Condition	Bio-retention Basin	Discharge Coefficient, C_o	Headwater Depth, H	Blockage Factor, b	Required Opening, $A_o = Q / (b . C_o . \sqrt{(2 \times 9.81 \times H)})$			Opening Provided	Check	Overflow Sump Opening Dimensions	Sump Quantity
					3-mth ARI	10-yr ARI	100-yr ARI				
			[m]		[m^2]			[m^2]		[m] x [m]	
Orifice	Section B4	0.6	0.1	0.5	0.06	0.15	0.21	0.60	OK	0.6 x 1.0	1

Hydraulic Calculations for Bioretention Basin

Project: NUS SDE4

VELOCITY CHECK:
(Refer to Section 5.7.6 of the PUB Engineering Procedures for ABC Waters Design Features)

Bio-retention Basin	Basin Width, W_{ch}	Extended Detention Depth, d	Channel Flow Area, $A_x = W_{ch} \cdot d$	Flow Velocity, $V = Q / A_x$		Scouring Check		Flow Depth, $d_{100\text{-}yr}$	$X = V_{100\text{-}yr} \cdot d_{100\text{-}yr}$	Velocity & Depth Check
				$V_{10\text{-}yr}$	$V_{100\text{-}yr}$	$V_{10\text{-}yr}$	$V_{100\text{-}yr}$			
	[m]	[m]	[m^2]	[m/s]				[m]	[m^2/s]	
Section B1	2.5	0.20	0.50	0.13	0.18	< 0.5 m/s, OK	< 1m/s, OK	0.3	0.054	X < 0.4, OK
Section B2	3.2	0.20	0.64	0.10	0.14	< 0.5 m/s, OK	< 1m/s, OK	0.3	0.042	X < 0.4, OK
Section B3	3.2	0.20	0.64	0.10	0.14	< 0.5 m/s, OK	< 1m/s, OK	0.3	0.042	X < 0.4, OK
Section B4	3.2	0.20	0.64	0.10	0.14	< 0.5 m/s, OK	< 1m/s, OK	0.3	0.042	X < 0.4, OK

OVERFLOW SUMP DISCHARGE PIPE FLOW CAPACITY:

Bio-retention Basin	Peak Discharge at 10yr ARI	Pipe Diameter, D_{op}	Pipe X-sectional Area, A_{op}	Pipe Slope, S_f	k	v	Flow Capacity of Single Pipe, $Q_{op} = A_{op} \cdot (-2(2g \cdot D_{op} \cdot S_f)^{0.5} \cdot \log(k/3.7D_{op} + (2.51v) / (D_{op} \cdot (2g \cdot D_{op} \cdot S_f)^{0.5}))$
	[m3/s]	[m]	[m^2]	[m/m]	[m]		[m^3/s]
Section B4	0.065	0.30	0.071	0.01	0.007	0.000001007	0.075

Bio-retention Basin	Pipe Quantity, X_{op}	Total Capacity, $TQ_2 = Q_{op} \cdot X_{op}$	Check against $Q_{peaktotal}$	Safety Factor
		[m^3/s]		
Section B4	2	0.151	> 0.065m3/s, OK	2.33 >= 1.5

SEDIMENT FOREBAY:
(Refer to Section 7.4.3.2 of the PUB Engineering Procedures for ABC Waters Design Features)

Bio-retention Basin	Catchment Area, A_c	Sediment Loading Rate, L_o	Desired Clean-out Frequency, F_c	Sediment Volume, $V_{sd1} = A_c \cdot L_o \cdot F_c$	Capture Efficiency, R	Settling Velocity, v_s	3mth ARI Design Flow, Q	Turbulence Parameter, n
	[m^2]	[m^3/ha/yr]	[yr]	[m^3]		[m/s]	[m^3/s]	
Section B1	1069.5	3	1	0.32	0.8	0.1	0.024	0.5

Bio-retention Basin	Min. Forebay Area Required, $A_s = n \cdot Q \cdot ((1-R)^{-1/n} - 1) / v_s)$	Forebay Depth Required, d_{fb}	Forebay Area Provided, A_{fb}	Forebay Depth Provided, D_{fb}	Assumed Porosity, P	Forebay Vol. Provided, $V_{fb} = P \cdot A_{fb} \cdot D_{fb}$	Check against V_{sd1}
	[m^2]	[m]	[m^2]	[m]		[m^3]	
Section B1	2.83	0.11	6	0.15	0.4	0.36	> 0.32m3, OK

Hydraulic Calculations for Bioretention Basin

Project: NUS SDE4

MAXIMUM INFILTRATION RATE:

(Refer to Section 7.4.5 of the PUB Engineering Procedures for ABC Waters Design Features)

Bio-retention Basin	Saturated Hydraulic Conductivity, K_{sat}		Basin Base Area, A_{base}	Extended Detention Depth, d	Filter Media Depth, d_m	Maximum Filtration Rate, $Q_{max} = K_{sat} \cdot A_{base} \cdot (d + d_m) / d_m$		Emptying Time, $T = K_{sat} / d$
	[mm/hr]	[m/s]	[m^2]	[m]	[m]	[m^3/s]	[m^3/hr]	[min]
Section B1	180.0	0.00005	83.20	0.20	0.4	0.0062	22.46	67
Section B2	180.0	0.00005	58.90	0.20	0.4	0.0044	15.90	67
Section B3	180.0	0.00005	21.40	0.20	0.4	0.0016	5.78	67
Section B4	180.0	0.00005	22.40	0.20	0.4	0.0017	6.05	67

SUB-SOIL PIPE PERFORATION INFLOW CHECK:

(Refer to Section 7.4.5 of the PUB Engineering Procedures for ABC Waters Design Features)

Bio-retention Basin	Blockage factor, B	Discharge Coefficient, C_d	Area per m Length, A*	Driving Head, h	Flow Capacity of Perforations, $Q_{perf} = B \cdot C_d \cdot nA \cdot \sqrt{2gh}$	Min. Total Length of Perforated Pipe, L	Perforated Flow, $TQ_3 = Q_{perf} \cdot L$	Check against Q_{max}	Safety Factor
			[m^2]	[m]	[m^3/s/m]	[m]	[m^3/s]		
Section B1	0.5	0.6	0.0050	0.075	0.002	6	0.011	> 0.0062m3/s, OK	1.75 >= 1.5
Section B2	0.5	0.6	0.0050	0.075	0.002	4	0.007	> 0.0044m3/s, OK	1.65 >= 1.5
Section B3	0.5	0.6	0.0050	0.075	0.002	4	0.007	> 0.0016m3/s, OK	4.53 >= 1.5
Section B4	0.5	0.6	0.0050	0.075	0.002	4	0.007	> 0.0017m3/s, OK	4.33 >= 1.5

***Typical corrugated fully perforated subsoil pipes with 50cm2 of perforation per metre length are used.**

SUB-SOIL PIPE FLOW CAPACITY:

(Refer to Section 7.4.5 of the PUB Engineering Procedures for ABC Waters Design Features)

Bio-retention Basin	Pipe Diameter, D_p	Pipe X-sectional Area, A_p	Pipe Slope, S_f	k	v	Flow Capacity of Single Pipe, $Q_{cap} = A_p \cdot (-2(2g \cdot D_p \cdot S_f)^{0.5} \cdot \log(k/3.7D_p + (2.51v) / (D_p \cdot (2g \cdot D_p \cdot S_f)^{0.5}))$	No. of Pipe Discharge Points, x	Total Flow Capacity, $TQ_4 = Q_{cap} \cdot x$	Check against Q_{max}
	[m]	[m^2]	[m/m]	[m]		[m^3/s]		[m^3/s]	
Section B1	0.10	0.008	0.005	0.007	0.000001007	0.002670401	3	0.008	> 0.0062m3/s, OK
Section B2	0.10	0.008	0.005	0.007	0.000001007	0.002670401	2	0.005	> 0.0044m3/s, OK
Section B3	0.10	0.008	0.005	0.007	0.000001007	0.002670401	1	0.003	> 0.0016m3/s, OK
Section B4	0.10	0.008	0.005	0.007	0.000001007	0.002670401	1	0.003	> 0.0017m3/s, OK

Endorsed by: **Koh Jiann Bin** (ABC-SILA001)
Sam Ko Luan Bock (ABC-E101)

7.8 References

CRCCH (Cooperative Research Centre for Catchment Hydrology), 2003, Model for Urban Stormwater Improvement Conceptualisation (MUSIC) User Guide, Version 2.0, December

Engineers Australia, 2003, Australian Runoff Quality Guidelines, DRAFT, June

FAWB (2009), Adoption Guidelines for Stormwater Biofiltration Systems, Facility for Advancing Water Biofiltration, Monash University, ISBN 978-0-9805831-1-3, June 2009.

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

Leinster, S 2006, Delivering the Final Product – Establishing Water Sensitive Urban Design Systems, 7th International Conference on Urban Drainage Modelling and 4th International Conference on Water Sensitive Urban Design Book of Proceedings, Volume 2, A Deletic and T Fletcher (eds), Melbourne.

Public Utilities Board (PUB), Code of Practice on Surface Water Drainage, Seventh Edition

Cleansing Biotopes 8

8

Chapter 8 Cleansing Biotopes

8.1 Introduction

General Description of Cleansing Biotope Systems

Cleansing biotopes, a type of vertical flow constructed wetland, have been implemented overseas and in Singapore. The natural water treatment feature could be an excellent tool for urban stormwater management and environmental protection, in addition to being a facility for environmental education, promoting biodiversity, and enhancing urban livability.

There are currently many cleansing biotopes in Singapore located in Bishan-Ang Mo Kio Park, Jurong Eco Garden, Jurong Lake Garden West and other locations.

Cleansing biotopes consist of nutrient-poor substrates that are planted with wetland plants which are known for their water cleansing capacity. Because of the high-performance potential of such natural cleansing systems they can be implemented in a variety of situations, such as the revitalization of lakes and the cleansing of urban water bodies. Water bodies that are slightly polluted can be treated especially efficiently with this type of system.

Inspired by nature, cleansing biotopes offer effective water treatment with a soft and natural aesthetic. Maintenance requirements are low and easy to carry out. These practical and visual aspects combined with their cleansing potential make them an attractive element of a water system.

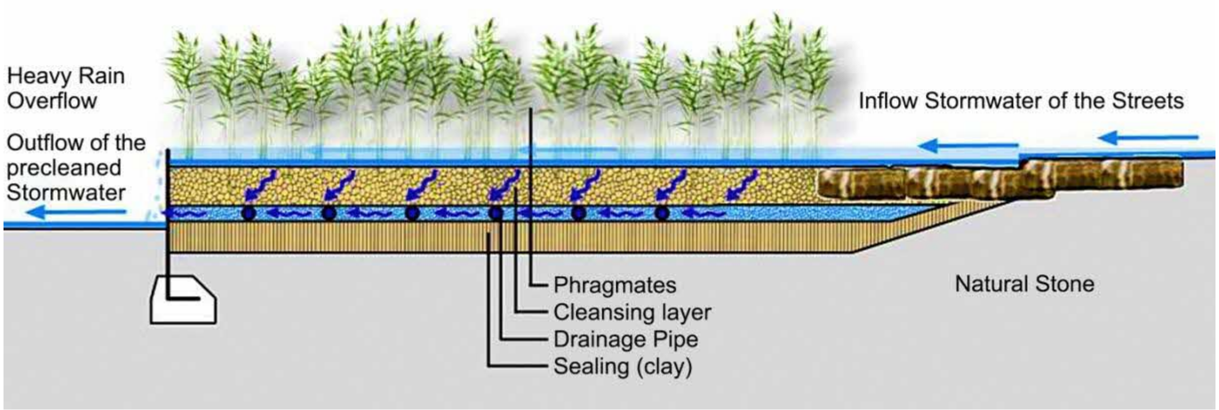


Figure 8.1a Typical Section of cleansing biotopes

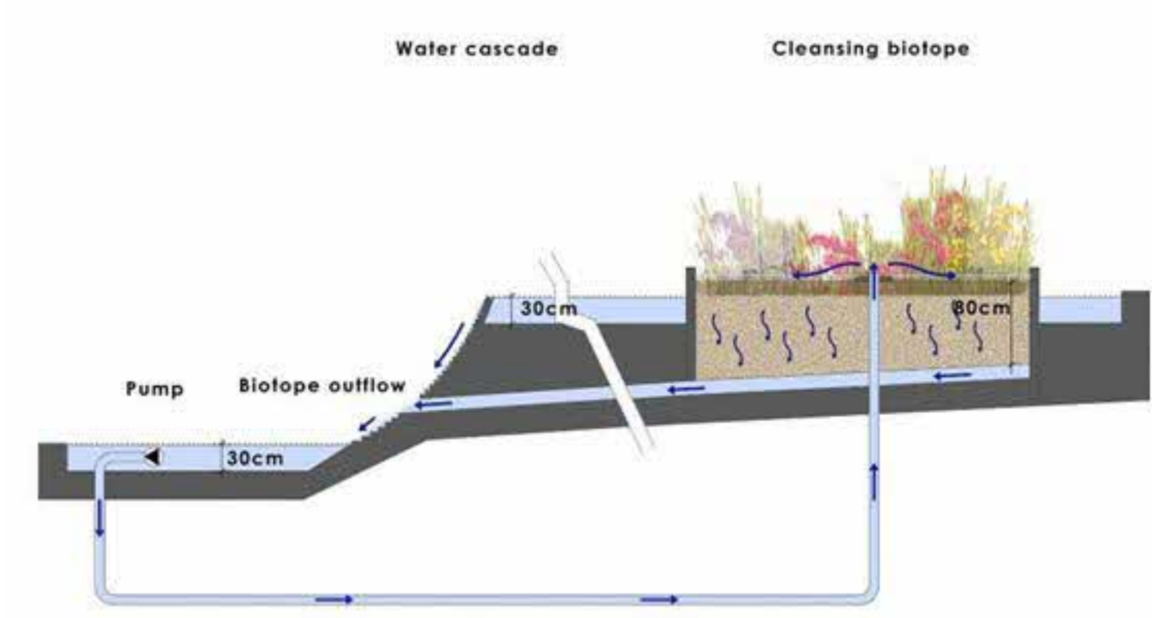


Figure 8.2b Recirculation of cleansing biotope

Cleansing Biotopes can be implemented in a variety of situations:

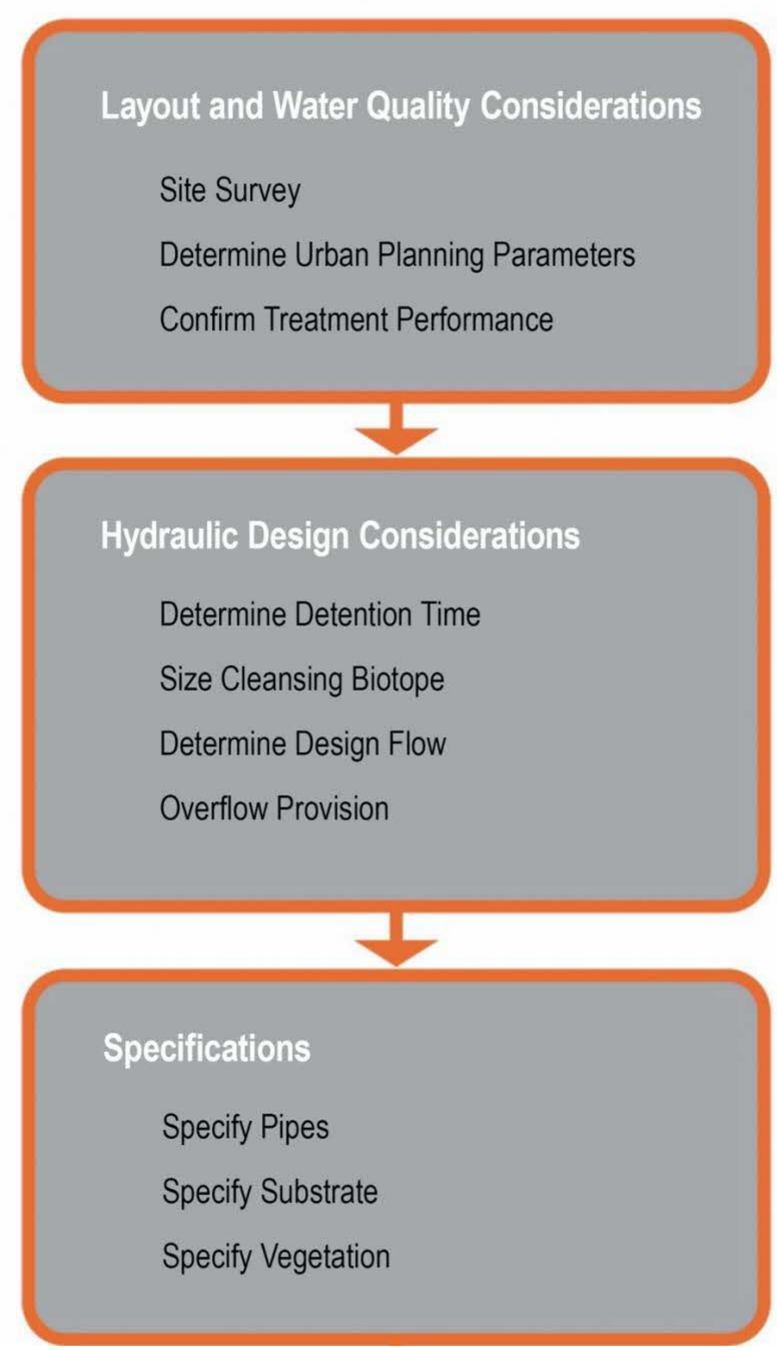
- outdoor areas, such as parks, open fields, to complement ponds and lakes;
- rooftop gardens, open plazas next to buildings or even under elevated structures;
- subdivided into smaller areas (such as small sky-gardens and planters) that work together in sequence for incremental cleaning.

BENEFITS

- can be constructed simply;
- completely flexible in form - can be subdivided into smaller biotope areas;
- versatile - suitable to be implemented in ecologically sensitive areas, public parks, urban open spaces and rural areas;
- highly ecological - water is cleansed naturally without the use of toxic chemicals such as chlorine or ozone (treatment processes, such as ultra-filtration, UV-treatment may be added, depending on the application, generally be done without;
- beautify surroundings and aesthetically unified - cleansing biotopes can blend seamlessly within landscape area or parks.

8.2 Design Process

This section outlines the general steps for designing a cleansing biotope, highlighting the different factors that have to be taken into considerations. Land availability, site topography, land use and urban planning parameters would present limits and opportunities for locating and sizing the cleansing biotope. On the other hand, the primary motivation for cleansing the water, whether it is for on-site reuse or to aid in achieving greater national stormwater management efficiency, would determine the desired level of water purification and, subsequently, the hydraulic flow, detention time, piping system, as well as the types of substrate and vegetation to be used.



8.2.1 Layout and Water Quality Considerations

8.2.1.1 Site Survey

A detailed land survey of the site should be undertaken to identify existing site conditions that may present constraints as well as naturally conducive opportunities for the optimum location and size of the cleansing biotope:

- Topography – Natural depression, existing slopes and changes in elevations that would advantage the desired flow of water, particularly in multi-tiered cleansing biotope systems.
- Area available for building cleansing Biotopes.
- Size and water quality of any water body that would link to Cleansing Biotopes.
- Soil content – availability of material suitable for substrate (refer to substrate specification). Variations to techniques of soil-compacting. Plasticity of soil would determine the types and rigidity of structures, such as concrete foundations, berms, to be designed as edges.
- Vegetation – Determine existing vegetations which can be transplanted for use in the cleansing biotope. Where comparable, indigenous plants are preferred over introducing new species into the site. This could be seen as an effort in conserving the ecological heritage of the site. More practically, this would minimize any adjustments that the rest of the local flora and fauna have to make, thus ensuring a continuity of the ecology. Diversity of plants rather than monoculture.
- Immediate Surrounding area – consider existing elements (streams, reservoirs ponds, lakes etc.), future changes and possible expansion.

8.2.1.2 Determine urban planning parameters

Precise information of the following must also be obtained from relevant governmental authorities, and verified against the site survey:

- Locations of the site boundary, green buffer and drainage discharge points.
- Locations of any drainage reserves and existing drains, including their invert levels and hydraulic capacities. This is especially important for designing the hydraulic flow of the cleansing biotope, particularly in its interface with the existing system (inflow and discharge points).
- Locations of any road reserves and setbacks – where the development is adjacent to a road, care must be taken to ensure that no structures are within any road reserves and setbacks.
- A thorough understanding of, and adherence to, the basic authority requirements is critical. It is imperative that the relevant authorities are consulted before the commencement of the detailed design process. These include, but are not limited to, the following:
- Obtaining planning approval from the Urban Redevelopment Authority (URA),
- Environmental protection consultation and public health consultation with the National Environment Agency (NEA), when the water will be reused for recreational purposes, especially with human contact.

8.2.1.3 Confirm Treatment Performance

A wide range of physical, chemical, and biological processes contribute to water quality improvement in a cleansing biotope. These processes include sedimentation, nutrient uptake by microorganisms and plants, binding, adsorption and precipitation, and volatilisation. Removal of contaminants may also be accomplished through storage in the soil media and vegetation, or through losses to the atmosphere. The following lists the types of pollutants that can be removed using cleansing biotopes -

Organic Load:

Cleansing biotopes are very efficient at breaking down both particulate and dissolved organic materials. The degradation of organic carbon composites occurs with the help of microorganisms. Micro-fauna lives in symbiosis with plant roots in the extremely large surface of the substrate matrix. The necessary oxygen input into the system occurs through the incoming water that is to be treated. The plants also contribute significantly to the oxygen supply through their root systems. Carbon is converted to carbon dioxide and extruded from the system.

Suspended Solids:

Cleansing biotopes are very efficient at removing suspended solids via sedimentation and adsorption.

Phosphate:

A specific chemical composition of the soil matrix can be achieved through a controlled mixing of substrates. Depending on the chemical composition of the soil matrix a good phosphate-bonding characteristic can be achieved. The addition of metallic oxides or red lava rock with an iron content of up to 15% can significantly increase the phosphate-binding capability. The year-round input of oxygen into the system prevents the re-release of phosphates. An effective binding capacity of the soil matrix can be assumed for many years.

Nitrogen composites:

In the presence of aerobic nitrification bacteria, ammonium will be converted to nitrates. The subsequent denitrification process, however, can only occur to a very small extent in the Cleansing Biotope conditions. The plants will partially feed off the nitrates.

Water Quality Standards

There are various reasons for including a cleansing biotope in a development.

It could be motivated by social responsibility: the desire to contribute to the efficiency of sustainable stormwater management and stormwater quality objectives (refer to ABC Waters Design Guidelines). The cleansed water from the cleansing biotope that is discharged into the public stormwater infrastructure would help to improve water quality of the downstream drains and waterways. Nevertheless, it is recommended that a sufficient level of runoff purification is accomplished to enhance the healthy growth of downstream ecology. Nevertheless, it is recommended that a sufficient level of runoff purification is accomplished to enhance the healthy growth of downstream ecology.

In some cases, the cleansing biotopes are meant to recycle the water for reuse on-site. Commonly, these local uses include irrigation and the flushing of toilets, or to supply and replenish the water in aesthetic ponds and water features. The possibility of human interaction (but not consumption) necessitates a higher level of purification and disinfection.

Cleansing biotopes have also been successfully implemented to purify water for water playgrounds, where children are likely to unintentionally consume mouthfuls of water. A more stringent standard of water purification and disinfection would, in such situations, have to be kept.

As explained above, standards of water quality are typically set according to its intended use. These different standards include benchmarks for different types of contaminants and water quality indicators.

These include:

WHO Drinking Water Quality Guidelines
https://www.who.int/publications/i/item/9789240045064
The water quality guidelines for drinking water by WHO are listed under Annex 3 – Table A3.3.

Local Standards
The following provides local standards that have been set by PUB and the National Environment Agency:

NEA – Environment Pollution Control Act 1999
https://sso.agc.gov.sg/Acts-Supp/9-1999/

NEA – Environmental Public Health (Quality of Piped Drinking Water) Regulations 2008
https://sso.agc.gov.sg/SL-Supp/S35-2008/Published/20080129?DocDate=20080129

NEA – Environment Public Health (Swimming Pools) Regulations
https://www.nea.gov.sg/docs/default-source/our-services/environmental-public-health-(swimming-pools)-regulations-(60-kb).pdf

NEA – Water Quality Guidelines for Popular Recreational Beaches
https://www.nea.gov.sg/our-services/pollution-control/water-quality/recreational-beaches

PUB – Requirements for Discharge of Trade Effluent into the Public Sewers
https://www.pub.gov.sg/Documents/requirements_UW.pdf

NEA – Allowable Limits for Trade Effluent Discharge to Watercourse or Controlled Watercourse
https://www.nea.gov.sg/our-services/pollution-control/water-quality/allowable-limits-for-trade-effluent-discharge-to-watercourse-or-controlled-watercourse

8.2.2 Hydraulic Design Considerations

8.2.2.1 Determine Detention Time

As explained above, the detention time within a cleansing biotope has immediate effects on the level of water purification.

Detention time is a measure of how long a particular molecule of water stays within a cleansing biotope from the time that it was introduced to the time that it is discharged. This is commonly measured using the **Plug-flow method**, which assumes constant velocity of water flow across any cross section. It is thus assumed that the first molecule entering the cleansing biotope would also be the first molecule exiting from it.

As a general gauge, wastewater that is highly polluted requires about 2-3 days of detention time, whereas water that is lightly moderately polluted (equivalent to lake water) would only require about 1-2 hrs.

A hydrologist should be engaged to determine the specific amount of detention time needed to achieve the minimum desired level of water purification.

8.2.2.2 Size Cleansing Biotope and Determine Design Flows

For a design flowrate, the detention time is proportional to the size of the cleansing biotope. As such, the most direct way of achieving a higher level of purification would be to increase the detention time by increasing the size of the cleansing biotope.

$$Q \; \alpha \; \frac{V}{t}$$

Q = flow rate (m^3 / hr)

V = Volume of water in cleansing biotope (m^3)

T = detention time (hr)

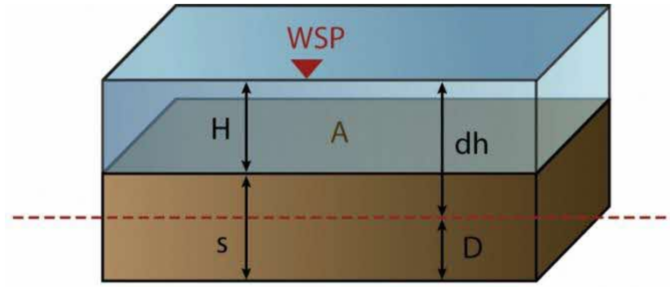
$$V \; \alpha \; AH$$

V = Volume of water in cleansing biotope (m^3)

A = filter surface / cleansing biotope area (m^2)

H = depth of water in cleansing biotope (m)

Where space is plentiful, the cleansing biotope can be sized to achieve the necessary detention time. Where there is insufficient space to implement the required size of cleansing biotope, other variables, such as filter depth and filter material, could also be adjusted accordingly.



$$Q = k_f \cdot \frac{dh}{s} \cdot A$$

A = filter surface (m)

Q = flow rate (m/s)

k_f = saturated hydraulic conductivity (m^3/m^2/day)

H = water depth (m)

s = filter depth (m)

dh = level deviation (m)

D = permanent water level (m)

Hydraulic detention time (HRT) = 1-3 hour
Total volume of water treated per HRT = Q x Hydraulic detention time

8.2.2.3 Flow configuration of Cleansing Biotope

A constant and evenly distributed flow of water in the Cleansing biotope is necessary for optimum cleansing effectiveness. Segmenting a cleansing biotope into several (usually three) areas has the benefit of extending its life cycle. Running the segments in turn allows each to consistently have a period of regeneration that guarantees the long-term permeability of the top layer of the filter substrate. For example, water could be fed to each area for a period of 4 days followed by a break of 2 days. As such, in a three-segment system, two areas are in operation while the third is regenerating.

It is therefore common to design a cleansing biotope system with a water body (e.g. an ecopond) and pumping / recirculation systems. To maintain the good water quality in the water body, a certain turn-over time of the water body is kept with the pumping/ recirculation system.

Figure 8.2 A multi-segment Cleansing Biotope system in Bishan-Ang Mo Kio Park

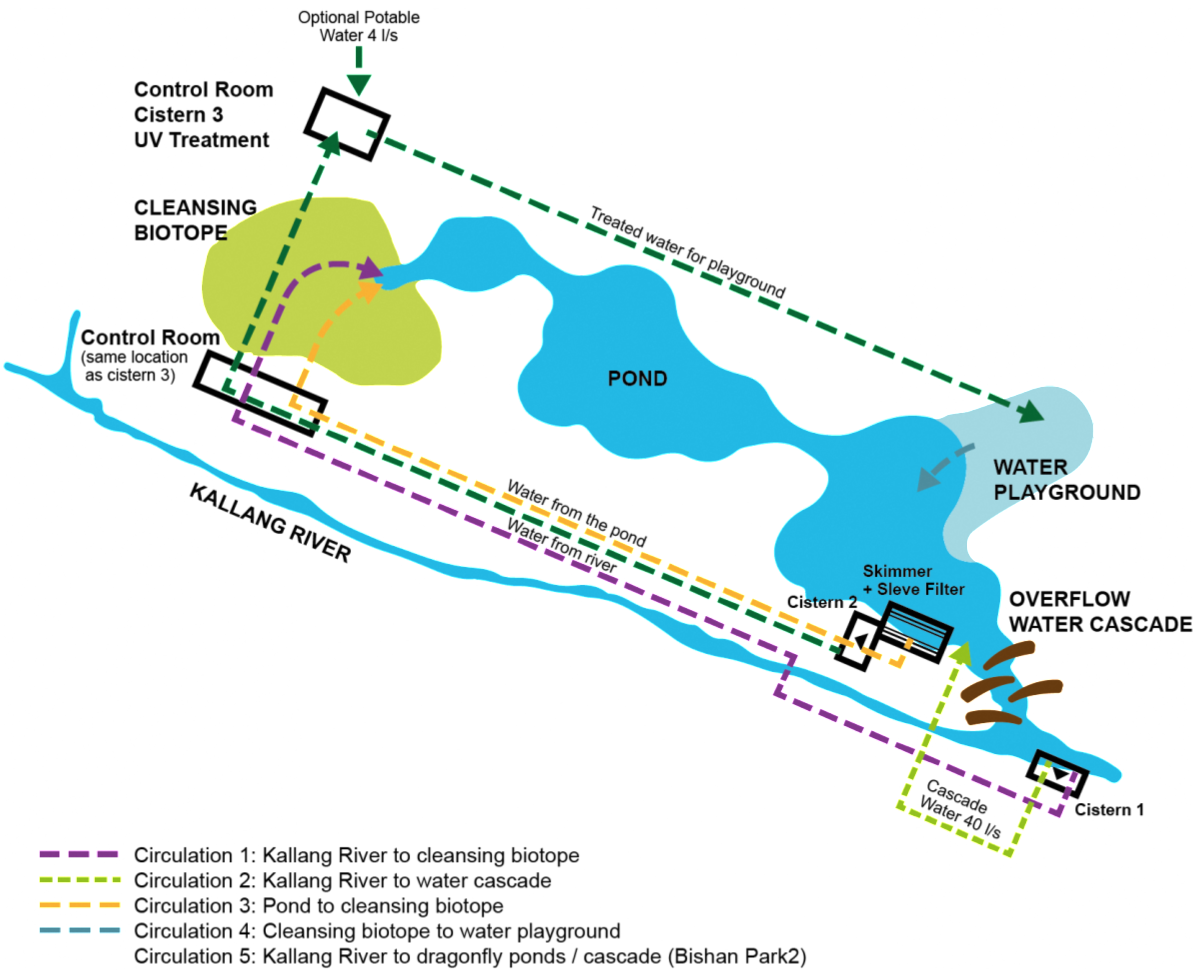


Figure 8.3 Water Recirculation overall system for the cleansing biotope in Bishan-Ang Mo Kio Park

Figure 8.4 Cleansing Biotope (left) and Eco pond (right) in Kampong Admiralty

A single cell cleansing biotope is also acceptable for small set-up. An example is the cleansing biotope at Kampong Admiralty. It provides natural treatment of runoff pumped up from the ecopond and the ecopond showcases cleansed runoff of the cleansing biotope.

Figure 8.5 Cleansing Biotope in Jurong Lake Garden West

A tiered arrangement with overflow weirs is common for cleansing biotope to suit site topography.

Figure 8.6 Overflow arrangement (like overflow sumps) at the lowest tier must be provided.

8.2.2.4 Determine Design Flow

8.2.2.4.1 Design Inflow

Regulating water volume

In specific instances, the velocity of inflow is fixed. For example, a cleansing biotope fed by a stream that has a bypass channel or flume would not have to worry about variable water inflow. In other cases where water level is susceptible to fluctuations, cleansing biotopes would have to be implemented in combination with regulating devices, such as holding ponds or overflow weirs/sumps, in order to maintain an even inflow of water. This is to ensure a constant detention time that has a direct effect on the level of water purification.

Inflow loading

Depending on various factors, polluted water can be introduced into the cleansing biotopes in 2 general ways:

"top-flow" – water is introduced from the top of the bed, either as an overflow from the higher terraces, or using distribution plates. A shallow film of water is visible above the biotope bed which then percolates down through the substrate.

Figure 8.7a Distribution plate

"Side-flow" – water is piped into the cleansing biotopes from one end of the side of the bed, such that the pipes are submerged in the substrate. In this case, the polluted water flows through the substrate at a slower rate and is not visible on the surface of the cleansing biotope as water level does not rise above it. This is especially effective for treating black water.

Adjusting Water Levels

Adjust water levels within each cell via the rotatable riser within the manhole by raising or lowering the riser accordingly.

Figure 8.7b Rotatable Riser Pipe

Dry Cell: When the valve is closed, surface water is expected to drain below the surface of the filter substrate within 30mins. No standing water is to be expected.

Wet Cell: When the valve is open, 70% - 100% water coverage is expected. Check to ensure that the water level does not go beyond the maximum water level as indicated at the manhole, as well as it does not go beyond any of the banks.

Figure 8.7c Dry Cell and Wet Cell

8.2.2.4.2. Design Discharge

In the "top-flow" model, treated water is collected using perforated pipes at the bottom of the substrate, and is then recycled back to the waterbody or conveyed to its desired discharge location, together with any overflow water. Its final discharge local can be moderately far away, up to several hundred metres or several kilometres, as long as there is sufficient pressure for its conveyance.

8.2.3 Specifications

8.2.3.1 Multi-Layered Liner

Multi-Layered Liner is to be comprised of the following (or equivalent) subject to compliance with requirements and manufacturer's instructions for installation:

1. Anchors: For membrane securement; steel rod, 6mm dia.
2. Stabilisation mat: 10mm thick, non-decaying, for stabilising slopes.
3. Waterproofing membrane (HDPE or EPDM).
4. Geotextile mat: levelling and protective layer.
5. Wire mesh blankets: Protection against rodents, plastic coated against corrosion mesh.

Figure 8.8 Installation and Testing of HDPE Liner (Photo Credit: Enviro Pro Green Innovations (S) Pte Ltd).

8.2.3.2 Pipes

<u>Perforated Pipes</u>

Figure 8.9 Perforated Pipes

Perforated Pipes on the floor of the cleansing biotope are to have the following Characteristics:

a) Corrugated Perforated Pipe with perforations < 3mm diameter all around

b) Material: High Density Polyethylene (HDPE)

Upon installation, each drainage pipe would have a non-perforated maintenance flush pipe with opening at one end, which would enable the piping to be flushed through if necessary.

Gravity Pipes

Gravity Pipes (including riser pipes) are to have the following Characteristics:

Figure 8.10 Gravity pipes (Photo Credit: Enviro Pro Green Innovations (S) Pte Ltd)

a) Material: Polypropylene (PP)

b) Dimensions: varies

Gravity pipes upon installation to be able to be flushed from outlet manholes. Pipes are to be laid with a gradient and be capable of conveying water without pressure.

8.2.3.3 Filter Substrate

Selection of the filter media plays a crucial role in ensuring good performance of a biofilter system. Cleansing Biotope substrates contributes to the purification process through multiple micro-functions:

- On the one hand, they trap and filter solid particles as water pass through their fine texture.
- Secondly, they support bacterial growth

They also remove contaminants through a process of surface absorption and complexation. In this process, there is an inverse relationship between the size of the particles and its purification effectiveness. As particle size decreases, total surface area increases, which effectively increases surface absorption rates.

This, however, has to be balanced with hydraulic conductivity, which has a direct relationship with the size of the substrate particle. As particle size decreases, substrate become less permeable and the flow of water through it become slower.

It is general practice in Singapore to design cleansing biotopes with hydraulic conductivities of about 1,000 to 2,000 mm/hr.

As a rule of thumb, the most suitable substrate would be comprised of the smallest particle that can still meet the minimum hydraulic conductivity. Empirically, coarse to medium sized (1-3 mm diameter) washed sand has proven to be very effective in achieving this balance. The chemical make-up of the substrate must also be taken into consideration.

Depending on availability, various combinations of materials can be used to achieve similar results. The following 3 formulas define the relationship between the flow rate of the water, permeability and particle size of the substrate.

Figure 8.11 Filter substrate

The following is an example of what has been specified for the Kallang River Bishan Park case study. This is just one example of what would be suitable for Singapore. Each site would present different requirements and a specialist should be engaged to determine the appropriate substrate composition.

It is important that the filter substrate is thoroughly washed before laying to prevent any stray foreign particles from clogging up / affecting the chemical composition of the substrate. The Filter substrate (about 700-800 mm thick) is not to be compacted during construction stage or post-construction. The Filter substrate is to be mixed off-site and brought onto site and should consist of the following:

a) Sand
   - I up to 85%
   - ii) particle size 1-3mm, well washed and without null particles
   - iii) rounded particles
   - iv) maximum chalk content 20%
   - v) hydraulic conductivity 1,000 to 2,000 mm/hr

b) Lava stone (red)
   - i) up to 15%
   - ii) particle size 2-4mm
   - iii) iron content about 10%

An R&D study completed in 2022 by PUB stated that soil amendments have been used to enhance treatment efficiency. Bioaugmentation can be applied to cleansing biotopes, if necessary. The study found that Biochar, which is produced via pyrolysis from raw materials like manure, forestry residues, wood chips, and agricultural wastes, is an effective soil amendment strategy due to its high capacity to adsorb nitrate, phosphate and remove organic pollutants as well as lower cost.

In general, the advantage of a higher contact time (HRT = 3 hrs) was demonstrated where removal rates for TP, PO4 and TN were higher. With low nutrient concentrations in typical stormwater runoff, Nitrate removal could be improved by using a higher HRT and adding biochar to base filter substrate (sand and lava stone).

8.2.3.4 Plants

The planting of the cleansing biotope is an important element of the overall system and ensures the long-term performance and the infiltration capacity of the substrate. Plant selection criteria include ability to remove pollutants in water, enhance biodiversity, improve aesthetic and withstand climate change.

The planting should be relatively dense and potted plants should be planted so as to balance out the initial low nutrient levels and compensate for any related loss of plant material. The plants selected should need little maintenance. The chief task is to remove dead and cut plant material. (Planting density illustrated in the photographs for plant species used in cleansing biotopes below).

Plant Species

- Diversity is preferred over monoculture.
- Where possible, indigenous plant should be included.
- Wetland species that are capable for adapting to fluctuating water levels has a higher chance of survival and robust growth.
- Plants with deep roots will aid in supporting the substrate.
- Planting density is important as weeds tend to take over quickly when there is exposed substrate.

Most plant species do not possess universal capabilities to treat all pollutants. For example, some species maybe good at removal of nutrients but not heavy metals. Thus, a mixture of vegetations should be incorporated in a Cleansing Biotope system to ensure optimal removal of various pollutants in the water.

Selected vegetation should ideally be deep-rooted plants that possess extensive root structure. Depending on site conditions and plant requirements, nearly all wetland plants can be used. Some of the plants used in Cleansing Biotopes are shown in the tables below. Designer should select based on the specific site and operating conditions. Other aquatic plants may also be appropriate but would have to be verified. A consultation with NParks is recommended to determine any other suitable species.

Plants used for the Cleansing Biotope:

Acrostichum aureum

Alocasia macrorrhizos

Costus speciosus

Cyperus alternifolius

Cyperus papyrus

Echinodorus palustris

Lepironia articulata

Melastoma malabathricum

Thalia dealbata

Typha angustifolia

Typha angustifolius

8.3 Construction Process

Stages of Cleansing Biotope Construction

a. Earthworks and fine grading;
b. Multi-Layer Liner Installation;
c. Collection/ Overflow Manhole;
d. Distribution and Drainage Pipe Installation;
e. Filling of Layered Media;
f. Testing and Commissioning of Pipes and Media (conductivity);
g. Planting Works.

Figure 8.12a Earthworks

Figure 8.12b Installation of Multi-layer liner

Figure 8.12c Collection/ Overflow manhole

Figure 8.12d Installation of perforated pipes

Figure 8.12e Laying of substrate layers

Figure 8.12f Planting work

8.4 Maintenance Requirements

Cleansing biotopes perform best when there are no suspended solids in the inflow, as they tend to clog the substrate. The polluted water can be passed through a settling tank or a skimmer to remove the suspended solids before being fed into the cleansing biotope.

A general management regime that includes the following steps:

8.4.1 Water quality and quantity monitoring

Maintenance regime should include periodic water quality checks to ensure that water stays within the set criteria. Water samples for analysis may be necessary. Water quantity control devices, such as weirs and valves, may have to be adjusted to ensure a constant velocity of flow into the cleansing biotope.

Figure 8.13 Water quality monitoring

8.4.2 Substrate Maintenance

The upper most layer of the filter substrate is particularly active during the biological break-down process. No exterior influences should disturb this process. Cleansing biotope systems generally function increasingly well over the years, as the biological system gets established. Unlike traditional sand filter systems, replacing the substrate or removing the upper layers is generally not necessary.

8.4.3 Plant Maintenance

Plant maintenance is minimal, and generally involves basic regimes, such as inspection for signs of insect damage or infestation, the removal of dead plants and pruning.

Certain species like Papyrus may require clearing and grubbing of rhizomes. This species can be propagated by division of rhizomes, however, with scheduled pruning, clearing may be necessary as well.

As plants used in Cleansing Biotope are pre-grown and pre-adapted to substrate conditions, fertilizing, both during and after plant establishment, is not necessary.

8.4.4 Mosquito Control

Mosquito control is not necessary as there is no stagnant water for mosquito breeding. In the “top-flow” model, water is constantly in motion and percolating to the bottom of the substrate. In the “side-flow” model, water is contained beneath the surface of the cleansing biotope, such that neither mosquito, not algae growth could occur.

8.5 The first Cleansing Biotope in Singapore – Bishan-Ang Mo Kio Park

8.5.1 Introduction

The decision to include a cleansing biotope in the new design of the Bishan-AMK Park reflects the philosophy of educating the public on water conservation as well as ecological conservation / sustainability.

On the more pragmatic side of things, the desire to facilitate increased recreational activities, such as kayaking, fishing and even just wadding in the river would require the water to be constantly cleansed in a manner that is cost-efficient and requires low-maintenance.

The cleansing biotope facility will be located upstream in Bishan-AMK Park 1, before the areas where the bulk of the recreational activities will take place. It is about 5400m^2 in area, and is comprised of 4 terraces of cleansing biotopes.

Figure 8.14 Cleaning biotope at Bishan-AMK Park, Singapore

8.5.2 Design Flow

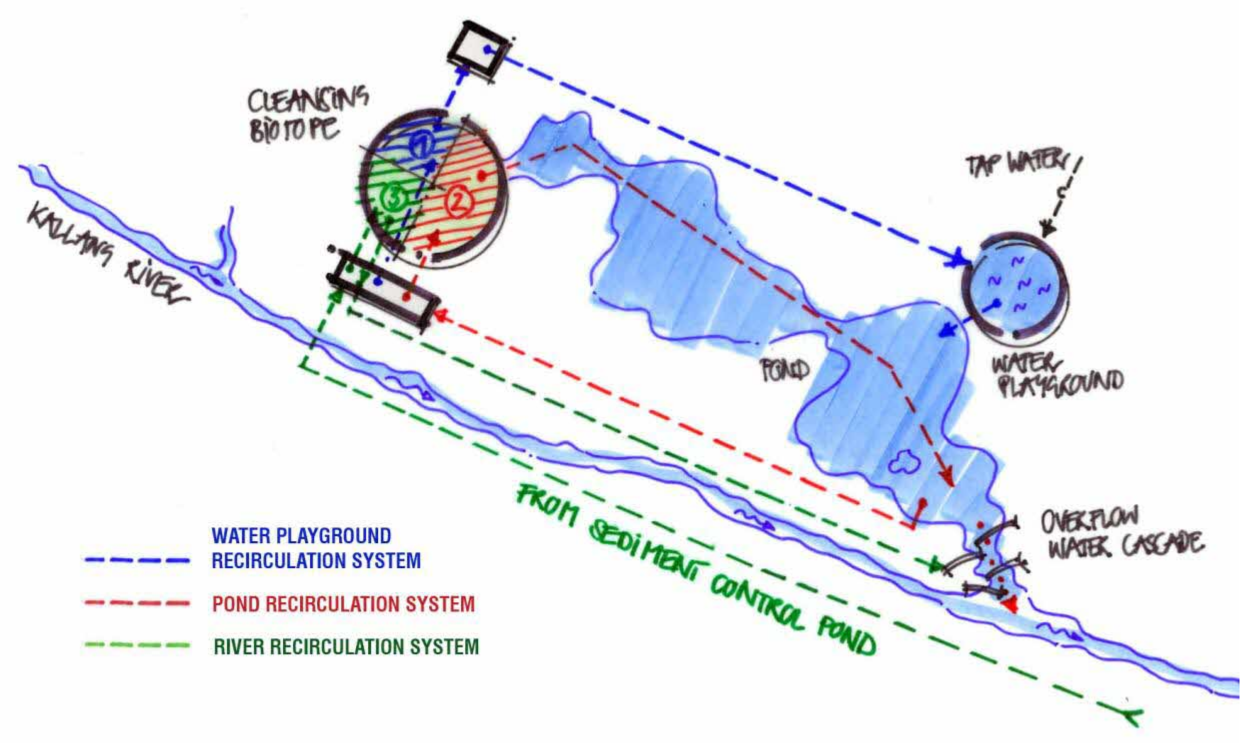


Figure 8.15 Design flow at Bishan-AMK Park

8.5.2.1 Design Inflow

Water to the cleansing biotope is mainly fed from the pond, through a pump room. Water from the end of the pond is also re-circulated back to the cleansing biotope.

8.5.2.2 Design Outflow

The cleansing biotope at Bishan-AMK Park is aimed at improving the quality of the water in three different locations:

- Water playground

  Located to the east side of the cleansing biotope, the water playground is intended for kids of all ages. As such, the quality of the water must meet potable drinking standards.

- Ponds

  The ponds are home to many of the aquatic fauna found in Bishan-AMK Park. The water in the lake should therefore meet environmental health standards. The series of ponds is also designed to end in a water cascade / overflow that will give visual prominence to moving water as part of a wider water recirculation system, thus potentially attract human people to wade and play in it. As such, a level of water quality sufficient for safe human interaction is also necessary.

- Kallang River

  Water discharged to the Kallang River would eventually flow downstream towards Marina Barrage, passing through channels and streams where diverse wildlife and ecology thrive. It is therefore designed to also meet environmental health standards.

Figure 8.16 Design outflow

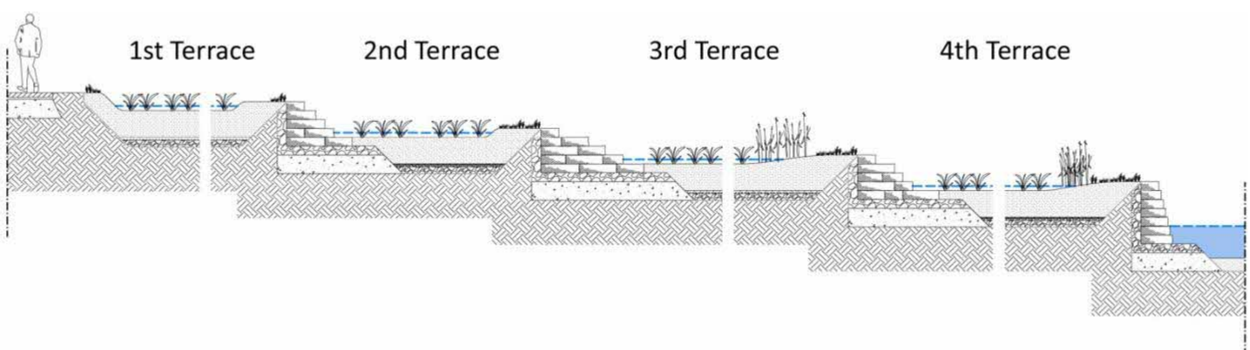
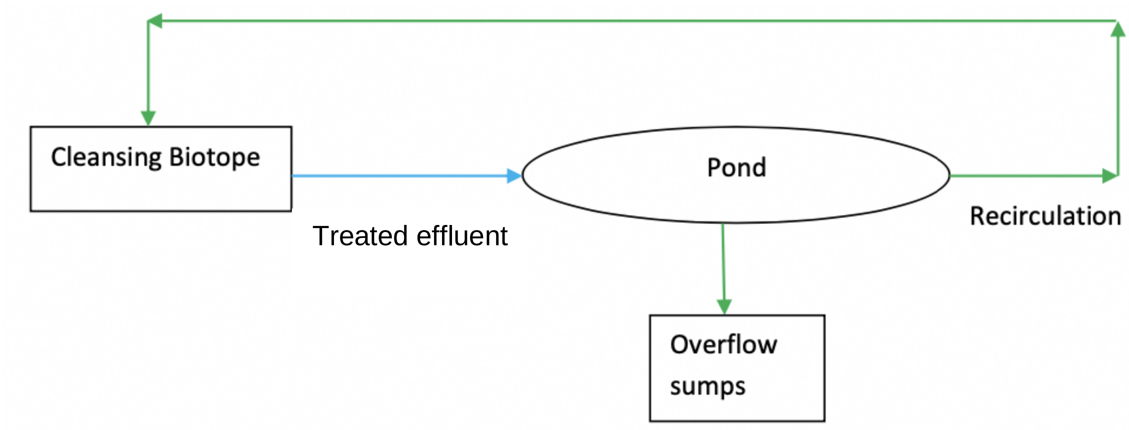


Figure 8.17 Water terrace

8.6 An Example of Cleansing Biotope Design

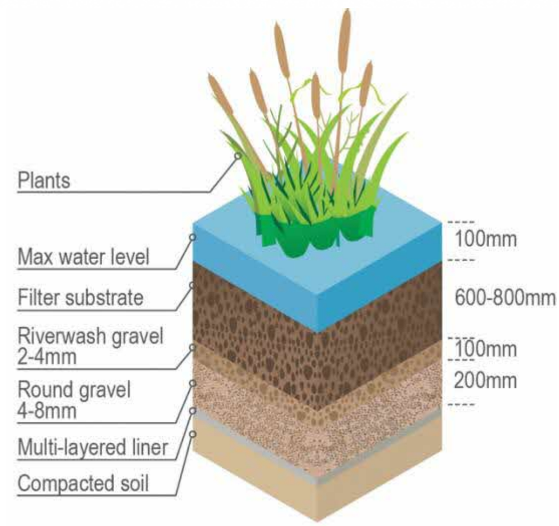
This worked example was based on the material shared by Mr Paul Nettleton of RSD in EU3, ABC Waters Professional Course, Nov 2021.



Biotope Filtration System Design

Key Component (From Top to Bottom):

- 600-800mm Filter Media
- 100mm Transition Layer
- 200mm Drainage Layer
- HDPE Perforated Pipes
- Geotextile and Liner
- Concrete Base/ True ground (compacted soil)



ENGINEERING DESIGN

Step 1: Data Collection → **Step 2: Design Pump flow rate**
- Operating hour
- Pond turn over time

→ **Step 3: Biotope design**
- Hydraulic Conductivity
- Infiltration rate
- HRT
- Perforated pipes
- Overflow

Data collection	
Pond Area	500 m^2
Pond depth	2 m
Cleansing Biotope surface area (A)	50 m^2

Design Pump Flowrate

RECIRCULATION SYSTEM

- Turnover time for waterbody ~ 3 - 5 days
- Related to reducing the symptoms that might promote algal blooms in our tropical weather
- Based on planned biotope (pump) operating hours, e.g. 14hrs a day
- Minimum pump flow rate required = $\frac{\textit{Total volume of Water Body}}{\textit{Turnover Time}} / \textit{Pump operating hours}$

Since recirculation will only operate during no-rain weather and for the purpose of pump rate calculation, effective pond depth is from base level up to low water level (LWL) only.

Total volume in the Cleansing Biotope-Pond system, V = V1 + V2
Where V1 = volume of water in the Cleansing Biotope = (A x H) + (A x filter substrate depth x 40%) and V2= [Pond Area x Depth]

Pump is designed to operate for at least 14 hours to maintain water level depth in the system
Water body turnover frequency (days) = V / (Q_{pump} x Operating hours)

Check that turn over time for water body (Pond) is within range and $Q_{pump} < Q_{max}$

Volume in the Cleansing Biotope (V1)	19 m^3
Extended detention (H)	0.1 m
Pond Volume (V2)	1000 m^3
Total volume in the Cleansing Biotope/ pond system (V)	1019 m^3
Cleansing Biotope operating hours per day	14
Pond turnover time, days	4
Pumping rate to achieve turnover time (Q_{pump}), m^3/hr	1019 ÷ (4 x 14) = 18.2

Biotope Design

DESIGNING THE RISER PIPE

Where at steady state, Qpump = Qo

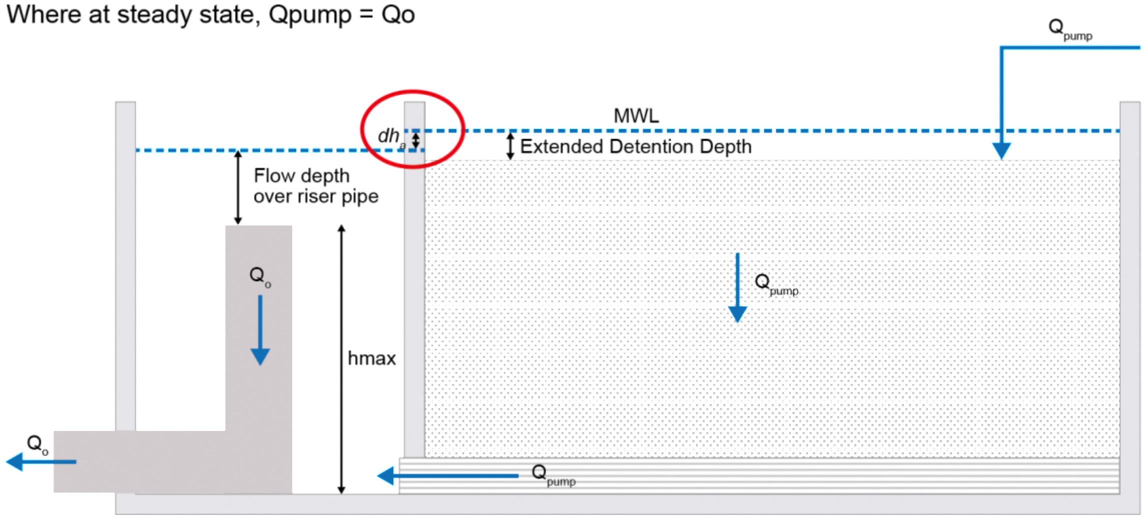


Figure 8.18 Flow depth over riser pipe: Sum of media layers + extended detention depth - riser pipe height - *dh*

To find *dh*, use the equation $Q = k_f \cdot \frac{dh}{s} \cdot A$

$Q = Q_{pump} = 18.2\ m^3/hr$
$2\ m/hr \times (dh \div 0.7) \times 50\ m^2 = 18.2\ m^3/hr \rightarrow dh = 0.13\ m$

Calculate Infiltration Rate (Q) with the design treatment area

The cleansing biotope is designed to cleanse a water body through recirculation. The pump flow rate is considered as the infiltration flow rate (Q) at steady state.

Checking Hydraulic Detention Time

Hydraulic Retention Time needed for treatment = 1 - 3 hours

Assuming porosity = 40%

Estimated volume of water in the Cleansing Biotope treatment zone, V1 = (A x H) + (A x filter media depth x 40%) = 50 x 0.1 + (50 x 0.7 x 0.4) = 19 m^3

Hydraulic Retention Time = V1 / Q = 19 ÷ 18.2 = 1 hr

PEFORATED PIPE DESIGN

Perforation Inflow Check

$$Q_{perf} = B \cdot C_d \cdot Ao\sqrt{2 \cdot g \cdot h}$$ Equation 7.1

Where

Q_{perf}	=	Flow rate through perforations (m^3/s)
B	=	Blockage factor (= 0.5)
C_d	=	Orifice discharge coefficient (= 0.6)
A_o	=	Total area of the orifices (m^2)
h	=	Assuming drainage layer is saturated, driving head is half the depth of the drainage layer (m)

Check that $Q_{perf} > Q_{pump}$

Capacity Check

$$Q_{pipe} = A_p\left[-2\left(2gD_pS_f\right)^{0.5} log\left(\frac{k}{3.7D} + \frac{2.51v}{D_p\left(2gD_pS_f\right)^{0.5}}\right)\right]$$ Equation 7.2

Where

Q_{pipe}	=	Flow rate through the perforated pipe (m^3/s)
A_p	=	Pipe cross sectional area (m^2)
D_p	=	Pipe diameter (m)
S_f	=	Hydraulic gradient (m/m)
k	=	Hydraulic roughness
v	=	Kinematic viscosity of water (m^2/s)

Check that $Q_{pipe} \times n > Q_{pump}$ and n = no. of perforated pipes

OVERFLOW PROVISION
Design for peak flow at 10-year ARI storm.

Overflow sump within biotope

- Use WEIR/ORIFICE equation, whichever more conservative

Overflow weir edge

- Use WEIR/ORIFICE equation

A grated overflow pit sump is sized based on the governing flow condition; weir flow or submerged flow conditions. A weir equation can be used to determine the length of weir required (assuming free overfall conditions). An orifice equation is used to estimate the required area between openings in the grate cover (assuming drowned outlet conditions). The larger of the resulting required dimensions to accommodate the two flow conditions should be adopted. In sizing the overflow pit for both drowned and free flowing conditions, it is recommended that a blockage factor that assumes the orifice is 50% blocked be used.

The weir equation for free flowing conditions is given by:

$$Q_{\min or} = Q_{weir} = B \cdot C_w \cdot L \cdot h_w^{3/2} \qquad \text{Equation 7.3}$$

Where

Q_{weir}	=	Flow over weir pit (m^3/s)
B	=	Blocked factor (assumed to be 50%)
C_w	=	Weir coefficient (adopt 1.7)
L	=	Length of weir (m)
h_w	=	Flow depth above weir (m)

A standard sized pit can be selected with a perimeter at least the same length as the required weir length.

The orifice equation for drowned outlet conditions is given by:

$$Q_{\min or} = Q_{grate} = B.C_d.A_{grate}\sqrt{2gh_w} \qquad \text{Equation 7.4}$$

Where

Q_{grate}	=	Flow rate under drowned conditions (m^3/s)
A_{grate}	=	Area of perforations in inlet grate (m^2)
h_w	=	Flow depth above weir (m)
C_d	=	Discharge coefficient (0.6)

8.7 References

Kadlec, R. R. Knight. 1996. Treatment Wetlands, Lewis Publishers, Chelsea, M.I.

INVESTIGATION OF CLEANSING BIOTOPES FOR SURFACE RUNOFF IN SINGAPORE'S CONTEXT - FINAL PROJECT REPORT
Prof. Adrian Law, Dr Angel Anisa Cokro et al, Nanyang Technological University Singapore, 2022

Soil Bioengineering 9

9

Chapter 9 Soil Bioengineering

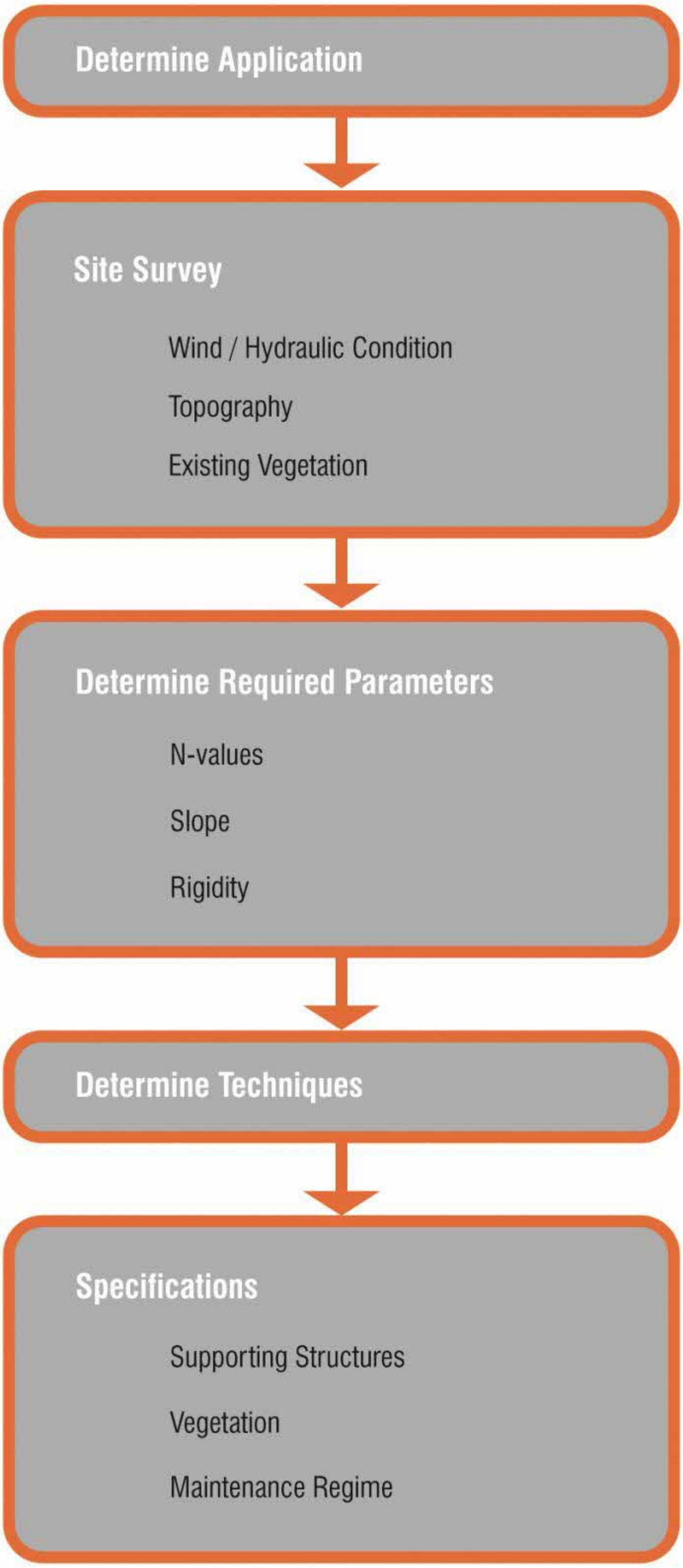
9.1 Introduction

Bioengineering refers to the use of vegetation and inert structures, such as geotextiles, as a way of forming and maintaining landforms. As a field, bioengineering straddles the disciplines of civil engineering, botany, and landscape architecture. It seeks to harness the inherent qualities and capabilities of organic matter (plants, seeds, branches, roots etc.) for the purpose of structural integrity, be it in a natural environment (such as stabilising a river embankment) or a constructed space (retaining walls supporting roads and buildings). Bioengineering aspires to come as close as possible to nature not only in the use of materials but also in the methods of construction. Bioengineering techniques can be employed to replace traditional civil engineering applications, but more often than not, they are used in combination as a complement to each other.

These could be broken down into various processes and techniques aimed at achieving various goals: restoration, reclamation, remediation and protection from degradation.

In order to accurately apply the knowledge of bioengineering, one needs to understand the fundamentals of land movements and mechanics, as well as related erosions caused by various natural elements, such as water and wind. It is also important to know the basic characteristics (resilience, hydrological affinity, root structure, mature size and growth period etc.) of different types of plants to appropriately use them in the design of structures for bio-engineering purposes.

9.2 Design Process



9.2.1 Determine application

9.2.1.1 Suitable Conditions

Since soil bioengineering can only be applied in regions where plants can grow easily and abundantly, they are often limited to only tropical, subtropical and temperate climates. While frosty and arid climates are equally hostile to ecological growth, the use of irrigation may compensate the latter. As a general rule of thumb, the more ecological diversity there is in a region, the more suitable it will be for the employment of soil bioengineering for slope and riverbank stabilisation.

9.2.1.2 Micro-functions

The general aim of soil-stabilization can be broken down into more specific functions that work hand-in-hand and can be achieved through a combination of soil bioengineering techniques. These functions include:

<u>Technical</u>

- Protecting the soil surface from erosion caused by climactic elements (snow, rain, flowing water, wind)
- Reduce the velocity of water flow
- Facilitating settlement and deposition of sand and sediments
- Maintaining soil texture as a prevention of landslides

<u>Ecological</u>

- Moderation of moisture level and temperature of the soil and its micro-region
- Enhancement of soil fertility through the retention of nutrients
- Noise barrier
- Beautification and camouflage of unpleasant structures
- Increasing soil integrity through root networks
- Improvement of air quality

9.2.1.3 Community Enhancement

Soil bioengineering also has added advantages thanks to its ecological nature. Pleasant and healthy green environments are created for the enjoyment of the community and long-term sustainability of ecosystem.

Social interaction is also enhanced in a beautiful and clean environment, creating communities that are vibrant and active.

9.2.2 Site Survey

It is important that a thorough site survey is carried out prior to construction commencement, as the information that is gathered would affect the decisions made during the design process. It would also highlight the opportunities and restrictions inherent in the site:

9.2.2.1 Topography

Soil bioengineering techniques can be employed to stabilise new earth formations, but more often, it is used to enhance existing landforms. Accurate information on the gradient of these slopes, banks and depressions, as well as their existing condition (to what degree they are vegetated and if there are existing support structures), must be obtained.

9.2.2.2 Geology

A soil survey should be carried out to determine the make-up of the soil on various parts of the site. This would help to identify area that are rocky and impervious, as well as the general plasticity and porosity of the soil. This would have effect on the general stability of the slopes, the level of stabilization that is needed, as well as the suitability of proposed new plant species.

9.2.2.3 Wind / Hydraulic Condition

Accurate measurements of the primary climactic element that causes erosion, such as water, snow or wind is necessary to determine the extent and type of soil bioengineering techniques to be employed.

9.2.2.4 Existing Vegetation

An inventory of the plants and “living” construction materials (including branches, cuttings, shoots, species of grass and shrubs) that exists on site should be compiled. Suitable material can be reused for the construction. Some may have to be removed and stored temporarily before being transplanted back. Endemic species are preferred because they are adapted to the site and have a higher change of propagation and a have a better likelihood of supporting existing fauna communities.

9.2.3 Determine Required Parameter

9.2.3.1 N-values

The Gauckler-Manning formula is commonly used for estimating water flow velocity in an open channel, and states that:

$$V = \frac{k}{n} R_h^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$$

where:

V is the cross-sectional average velocity (m/s)
k is a conversion constant equal to 1.0
n is the Gauckler-Manning coefficient
R_h is the hydraulic radius (m)
S is the slope of the water surface or the linear hydraulic head loss (m/m)

The **Gauckler-Manning coefficient**, often denoted as ***n***, is an empirically derived coefficient and is inversely proportionate to the velocity of the water flow.

Type of channel and description	Minimum	Normal	Maximum
Flood plains (pasture, short grass)	0.025	0.030	0.035
Minor streams (clean, straight, no rifts)	0.025	0.030	0.033
Excavated channel (stony bottom, weedy banks)	0.025	0.035	0.040

Table 9.1 Typical n-values for various types of channels

Each specific soil bioengineering technique would have a corresponding n-value and would affect the flow of water differently. **n-values** are a function of both the specific soil bioengineering techniques deployed and the type of vegetation used.

9.2.3.2 Slope and rigidity

Soil bioengineering techniques have varying rigidities and stabilization capabilities. The following is a rough grouping of the techniques shown in this chapter, according to their suitability in stabilizing slopes of varying gradients.

River Bed

- Rock gabion mat
- Rock sill

Gentle Slope

- Pegged geo-textile with fascines
- Pegged geo-textile with reed roll

Moderate Slope

- Brush Mattress
- Stone-wall / Rip-rap with cutting
- Wrapped soil lift

Steep Slope

- Vegetated reinforced soil
- Gabion wall

Fascines and reed rolls are designed to be applied to the interface between the embankment and the water. They prevent turbulence at these edges and thus protect the slope from erosion. They are typically used in combinations with geo-textile and brush mattresses.

9.2.4 Determine Techniques

Refer to Section 9.3 for a detailed construction sequence of different techniques.

9.2.5 Specifications

Refer to Section 9.3 for elements and material needed for specific techniques.

9.2.6 Maintenance

As long as appropriate conditions for plant growth are maintained, soil bioengineering applications are dynamically sustained, self-regulated, and enhanced without the need for excessive maintenance. The deepening of the roots over time improves soil stabilization, and the intensification of plant growth adds soil coverage and shade.

Unlike hard structures, however, soil bioengineered waterways comprise of live, dynamic and loose elements. Bedrock movements (e.g. stones and pebbles moved and carried along by high water velocity) and sedimentation (debris and silts generated from eroded rocks, plant damage and degeneration) is a natural part of the river. Periodically but infrequently, minor maintenance (replacement of displaced rocks, trimming of plants to prevent breakage etc.) is necessary. Maintenance regime should include the following:

Plant establishment period:

- It is important that cuttings are kept short in the beginning so that it does not get uprooted and washed away by strong river flows. Accompanying structures could also be damaged in the process.

- Watering of trees and shrubs immediately after planting. They should continue to be watered at least once a day for at least four to eight weeks, or until the plants are well established. Periodic watering after that may be necessary.

- Erection of temporary fencing, guards, barricades, supports and netting as necessary to protect the plants / trees / turf.

After plant establishment:

- Watering and replacement of dead, unhealthy, undesirable plants, trees, saplings, shrubs and turf

- Replacement of plant guards / stakes if any.

- Proper pruning, trimming, weeding and clearing of unwanted growth as required

In the unlikely event of slope failure:

- Survey and analyse reason for failure
- Determine a more suitable / more rigid soil bioengineering technique
- Apply Erosion Control Measures (ECM) to cordon off affected area if there is a threat of further erosion
- Apply necessary site remediation
- Remove ECM

9.3 Techniques

9.3.1 Rip Rap with Cuttings

Elements / Materials:

Plants

- Cuttings of appropriate length (80-100 cm), and diameter (5-10 cm)
  - Ends pointed for easy driving
  - Species:

Stones

- Natural stone or concrete rubble (average diameter 15 – 50 cm)
- Filter material under and in between stone layer: sand-gravel mixture (0-200 mm)

Installation:

1. Grade the slope smoothly to desired angle, as shown in site plan and/or section.
2. Spread an even layer of filter material (approx. 20 cm) on graded slope.
3. Place stones on the filter layer. Adjust stones by hand to ensure dense interlocking. Layer of stones must be at least 50cm thick.
4. Simultaneously drive the cuttings into the ground as stones are adjusted and laid, making sure that they are securely locked in between the stones. Approx. 5 cuttings should be driven every m^2 of stone. Cuttings should not protrude more than 10 cm above the stone layer.

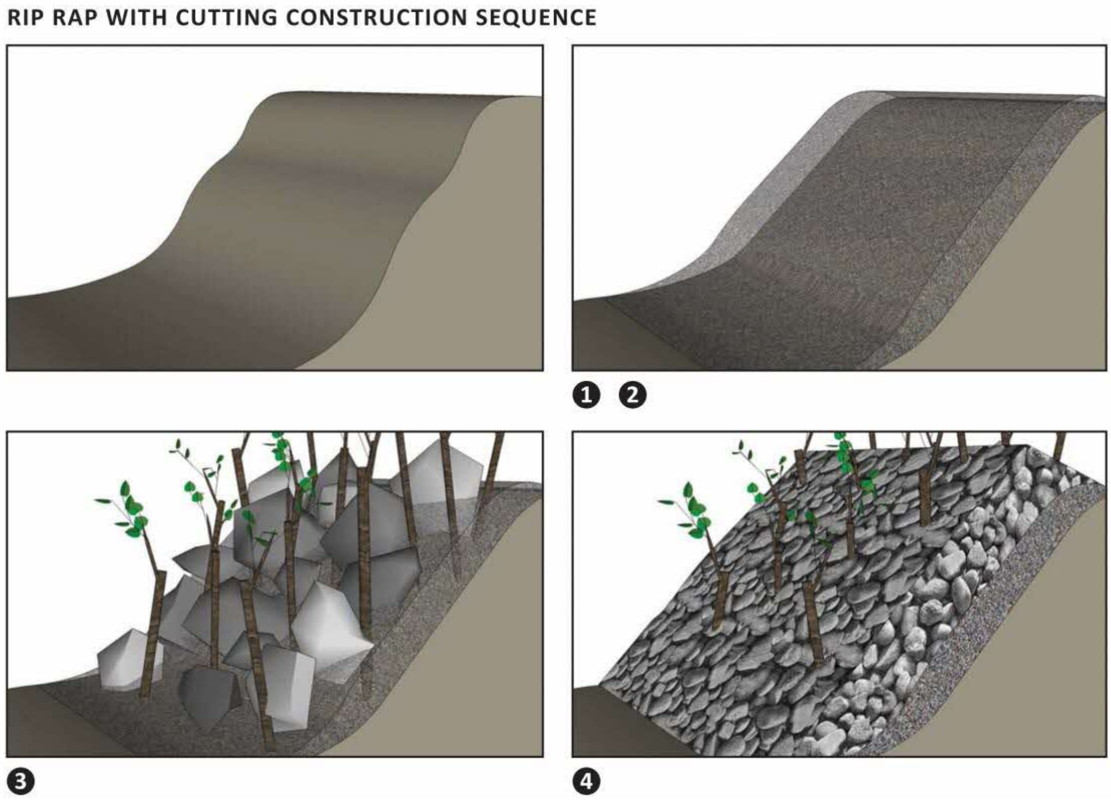


Figure 9.1 Rip rap with cutting construction sequence

9.3.2 Brush Mattress with Live Fascine

Elements / Materials:

Plants

- Live brush of appropriate length (2-5 m)
- Long wooden poles for pressing down and attaching branches to the slope

Fasteners

- Wire
- Long, wooden stakes

Fill

- Suitable fill or excavated soil

Installation of Fascine:

1. Gather and arrange live brush such that they are aligned in the same direction. At every 80-100 cm, tie wires around them to create bundles of approx. 30-40 cm diameter. Ensure that these live fascines are dense and tightly bound and that the ends taper.

2. Dig a trench along the bottom of the brush mattress application. This should be at the mean water level. Ensure that there is a gradual transition between the slope and the trench by tapering off the upper section of the trench. The trench should also be located and angled in such a way that the fascine placed in it will eventually be half covered with water and half with soil.

3. Place a layer of dead brush in the trench, protruding approx. 20-30 cm into the water to prevent scour at the toe of the application.

4. Place the fascine on top of this layer of dead brush and secure it in place by driving long wooden stakes in a staggered pattern. This would prevent it from floating.

Installation of Brush Mattress:

5. Grade the slope to desired angle.
6. Place live brush along the slope, with the bottom end pointing towards the stream and tucked under the fascines in the trench, while the tip is pointing up-slope. These should be arranged at a density of approx. 20-30 stems per linear metre.
7. Drive long wooden stakes in a grid pattern across the entire area of laid live bush, with rows running parallel to the stream. The rows should start 1 m above the live fascine and continue at 1 m intervals between each row.
8. Lay long wooden poles parallel to the stream against these stakes, forming rows of poles for pressing down the live bush.
9. Secure the poles and stakes to the ground by wiring it down. This is done by tying one end of the wire to the bottom of a stake, weaving the wire above and below the live brush, and then tying the other end of the wire to the bottom of another stake. Once the wire is secured, the stake is driven another 2-3 cm into the ground to ensure that it is taut.
10. Once all the poles and stakes are wired down, spread a thin layer of film over the entire mattress, including the fascine. However, ensure that the treatment is not excessively covered. This is to prevent the brush from drying out, but should not stifle it.

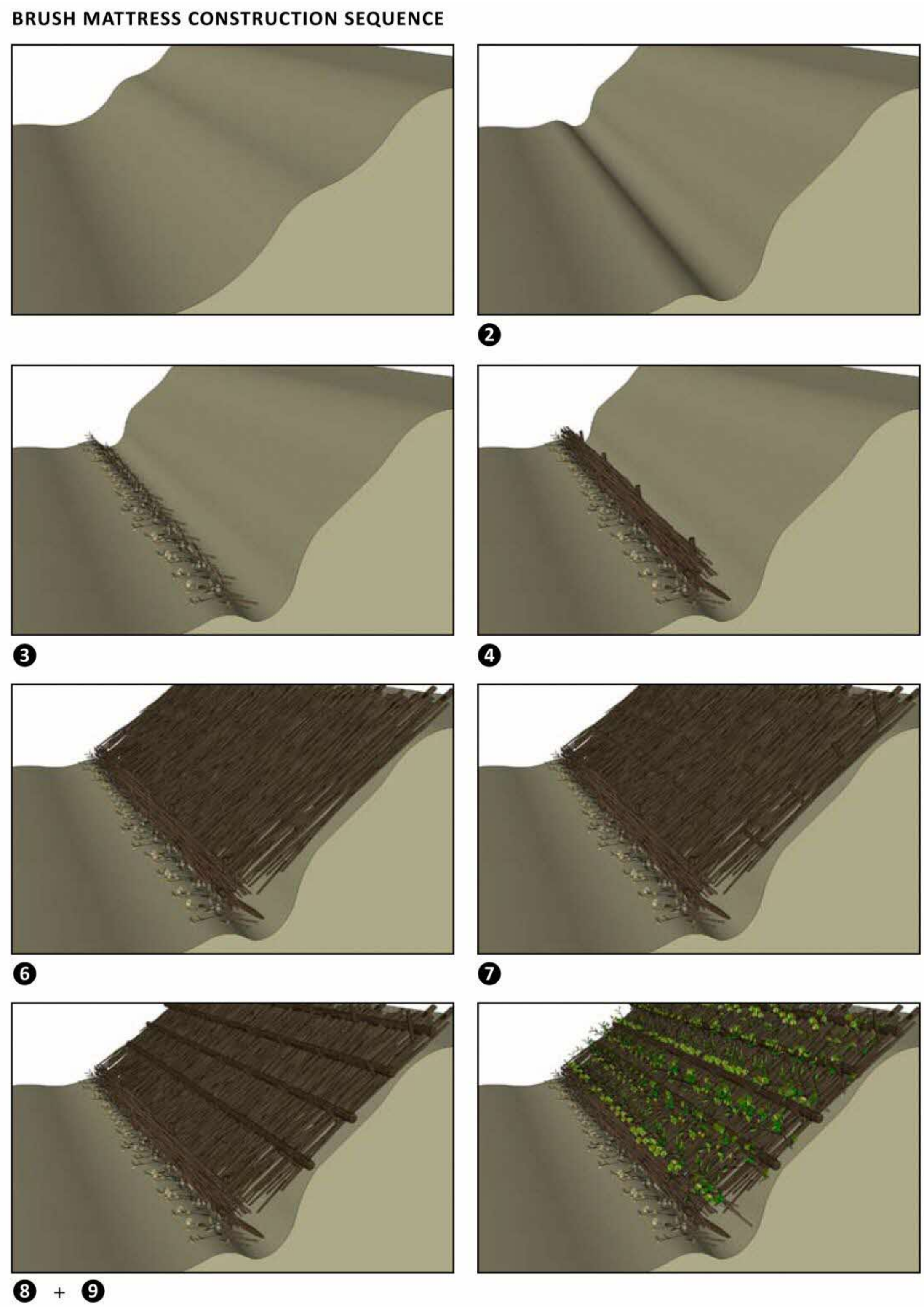


Figure 9.2 Brush mattress construction sequence

9.3.3 Geotextile-Wrapped Soil Lift

Elements / Materials:

Plants

- Live brush of appropriate length (2-5 m)
- Rooted, woody plants
  - Species:

Geotextile

- Coconut fibre mats, 700-900 g/m^2

Fasteners

- Short stakes
  - hard wood species
  - Dimensions: 2-4 cm diameter (round or square), 30-50 cm long
  - One end pointed for easy driving

Fill

- Suitable fill and gravel

Installation:

1. Excavate a length of shelf of approx. width 1 m along the bottom of the stream wall. Ensure that it is sloped slight back towards the slope.
2. Place a layer of branches (sloping back against the stream edge and protruding approx. 20-30 cm out into the water) at the toe of the soil-lift application. This has to be just below mean high water level. This layer is for preventing scours at the stream edge.
3. Secure geotextile along the entire length of this shelf using short stakes driven through the geotextile and into the ground at the edge of this excavation. Ensure that there is a minimum of 20 cm of overlap between adjoining strips of geotextile and that a sufficient length of each geotextile strip overhangs to later wrap over the soil-lift.
4. Place and moderately compact fill materials (soil-gravel-mix), to a height of approx. 50-70 cm, on top of the geotextile.
5. Pull the overhanging geotextile and wrap it over the fill material, securing it at the top by driving short wooden stakes through the back end of the fabric and into the compacted fill, forming a c-profiled soil-lift.
6. Place live brush and rooted plants densely (approx. 20 stems per linear metre) on the soil lift, before covering it with a layer of soil material (approx. 20 cm). Do not include any gravel in this layer.
7. Repeat steps 3-6, building up layers of soil lift, until the desired height is achieved.

GEOTEXTILE-WRAPPED SOIL LIFT CONSTRUCTION SEQUENCE

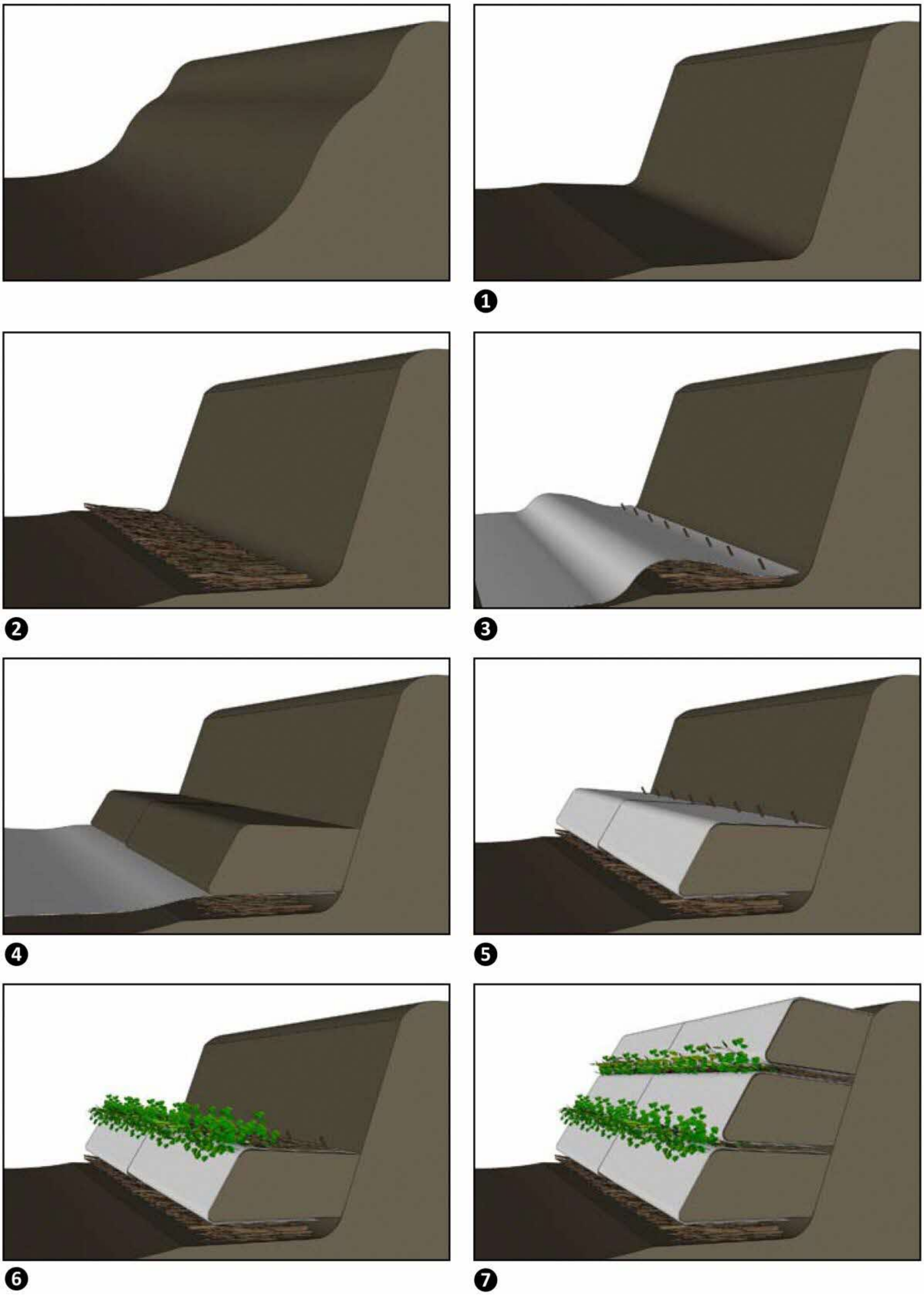


Figure 9.3 Geotextile-wrapped soil lift construction sequence

9.3.4 Fascine with Geotextile

Elements / Materials:

Plants

- Live but non-sprouting brush of appropriate length (2-5 m)

Geotextile

- A variety of coconut fibre and jute mats (400-900 g)

Fasteners

- Wire
- Long and short wooden stakes

Fill

- Suitable fill or excavated soil

Installation of Fascine:

1. Gather and arrange live and dead brush such that they are aligned in the same direction. At every 80-100 cm, tie wires around them to create bundles of approx. 30-40 cm diameter and no less than 4 m long. Ensure that these live fascines are dense and tightly bound, and that the ends taper.
2. Dig a trench along the edge of the stream, at the mean water level. Ensure that there is a gradual transition between the slope and the trench by tapering off the upper section of the trench. The trench should also be located and angled in such a way that the fascine placed in it will eventually be half covered with water and half with soil.
3. Optional: place a layer of dead brush in the trench, protruding approx. 20-30 cm into the water to prevent scour at the toe of the application.
4. Place the fascine on top of this layer of dead brush and secure it in place by driving long wooden stakes in a staggered pattern. This would prevent it from floating.

Installation of Geotextile:

5. Grade the slope to desired angle.
6. Secure the top edge by staking it into a trench (of approx. 15 x 15 cm), after which it is back-filled and compacted. Use 5 small wooden stakes per metre run or according to the manufacturer’s recommendation.
7. Secure the bottom edge either by similarly anchoring it to a trench, or by tucking it under the fascine.
8. Lay erosion control fabrics over the entire application, ensuring a 10-15 cm overlap between adjacent strips of fabric. At 50 cm intervals along the overlapping seams, stake both layers of fabric into the ground using short wooden stakes, or according to the manufacturer’s recommendation.

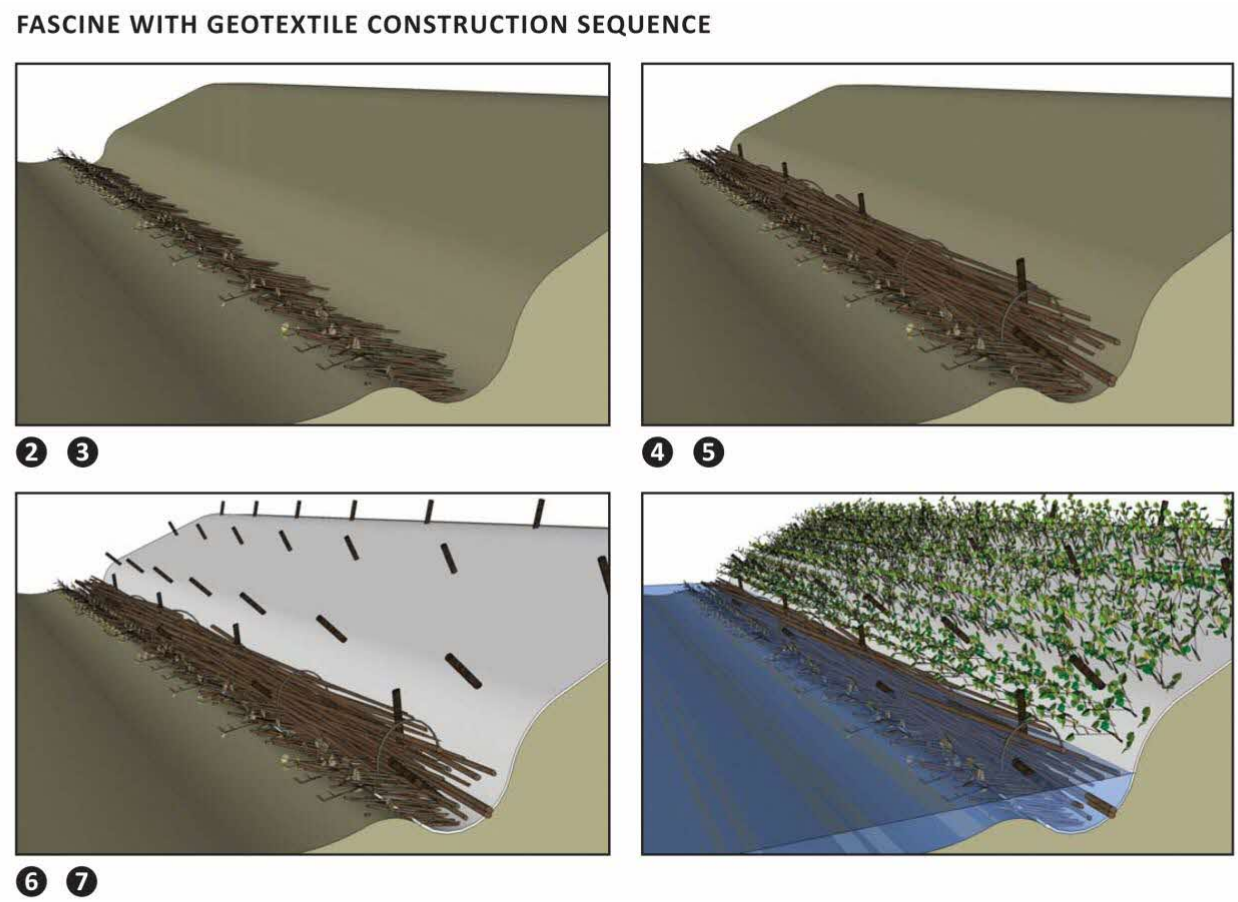


Figure 9.4 Fascine with geotextile construction sequence

9.3.5 Reed Roll

Elements / Materials:

<u>Plants</u>

- Herbaceous plugs

<u>Geotextile</u>

- Coconut Fibre mats (700 g/m^2)

<u>Fasteners</u>

- Short and long wooden stakes

<u>Fill</u>

- Suitable soil or excavated soil

Installation:

1. Dig a trench of approx. 50 x 50 cm along the stream edge.

2. Drive long, wooden stakes along the stream edge of this trench. These will form the edge of the reed roll, and should be driven two-thirds of their lengths vertically into the ground, spaced at 100-120 cm intervals.

3. Lay one end of the geotextile in the trench, with the long side facing the stream. Then fill the trench to half its depth with fill material.

4. Place plant plugs and/or sod in the trench at a spacing of 3-5 plants per linear metre (depending on species size).

5. Fold the geotextile over this line of plants plugs to form a roll of approx. 30-80 cm diameter. Secure it in place using short wooden stakes.

6. Plant plugs and/or sod at remaining area on the slope at approx. 4-5 plants / m^2.

7. Extend the geotextile that is still attached to the reed roll over the slope to cover the entire planted area. Secure it in place using short stakes at approx. 4 stakes /m^2, ensuring that there is a minimum of 20-30 cm overlap between adjacent strips of geotextile. Dig the loose end of the geotextile into the ground at the top of the slope.

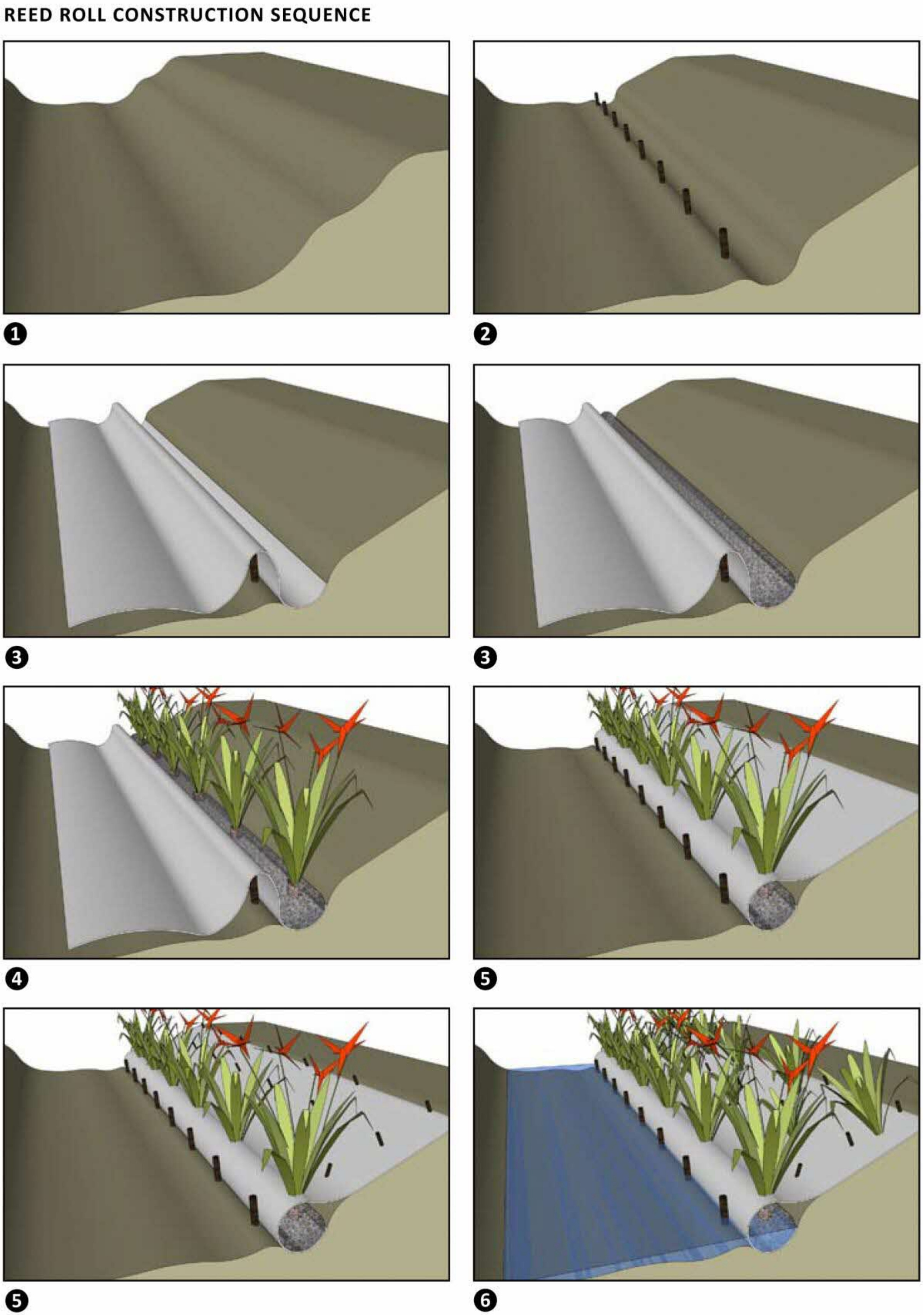


Figure 9.5 Reed roll construction sequence

9.3.6 Stone Wall with Cuttings

Elements / Materials:

Plants

- Cuttings of appropriate length (80-100 cm), and diameter (5-10 cm)
    - ends pointed for easy driving
    - Species:

Stones

- Natural stone or concrete rubble (average diameter 30– 60 cm)
- Filter material under and behind stone wall: sand-gravel mixture (0-200 mm)

Fill

- Suitable fill and gravel

Installation:

1. Spread and compact an even layer of filter material (approx. 20 cm) on graded slope.
2. Place stones on the filter layer, forming a slightly back-inclined wall (towards the slope). Adjust stones by hand to ensure dense interlocking but avoiding cross-joints.
3. Simultaneously place cuttings in the gaps between the stones as they are being laid and adjusted, making sure that they are slightly angled back into the slope and, at the same time, securely locked in between the stones. Approx. 5 cuttings should be driven every m^2 of stone. Cuttings should not protrude more than 10 cm beyond the stone layer.
4. Fill the gaps between the stones with fill material.
5. Backfill the wall with gravel and filter material to facilitate drainage.

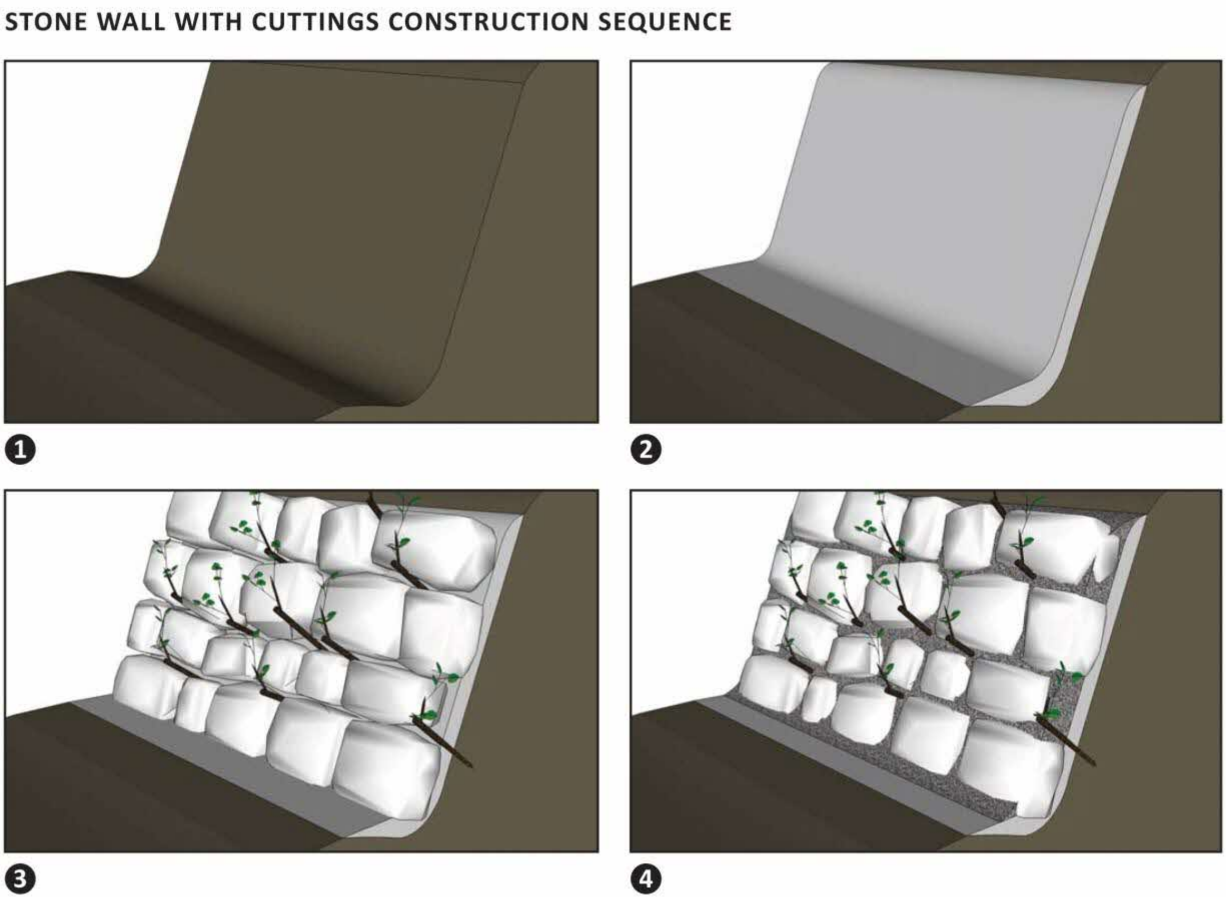


Figure 9.6 Stone wall with cuttings construction sequence

9.3.7 Gabion Walls with Creepers

Elements / Materials:

Gabions

- Cubic wire mesh containers
- Dimensions: 100 cm (W) x 100 cm (L) x 100 cm (H)
- Wire mesh – galvanized steel wire, spot-welded, 4-5 mm diameter, 450N/m² tensile strength, mesh opening 10 x 10 cm
- Subject to Superintending Officer's approval

Plants

- Creeper species of choice (non-invasive, non-woody and herbaceous)

Stones

- Gabion filler: natural stone or concrete rubble (average diameter 15 – 20 cm)
- Filter material under and behind gabion: sand-gravel mixture (0-200 mm)

Filter fabric

- Polypropylene geo-textile, non-woven needle-punched

Installation:

1. Grade the slope of the stream edge with levels of steps / shelves of about 100 cm high and 40-80 cm wide each. These should be slightly sloping backwards, away from the river, and the first step should start below the stream bed level.
2. Evenly spread a 15-20 cm layer of filter material (sand gravel mixture) on the stepped profile. Compact this layer of filter material before laying filter fabric on top, ensuring a 10-15 cm overlap between adjacent lengths of filter fabric.
3. Place each wire mesh basket tightly next to each other on the lowermost level, but maintaining a 5-10 cm gap from the back, away from the graded stream edge. Back-fill the gap behind with filter material.
4. Spread a 20 cm layer of soil material on top of this level of gabion and pack plant material with adequate protection. Construct the next level of gabion on top of planting layer (according to step 3), slightly stepped back and ensuring that the plant material still protrudes out from in between the 2 levels of gabion.
5. Repeat step 4 until the gabion wall reaches the stream bank.

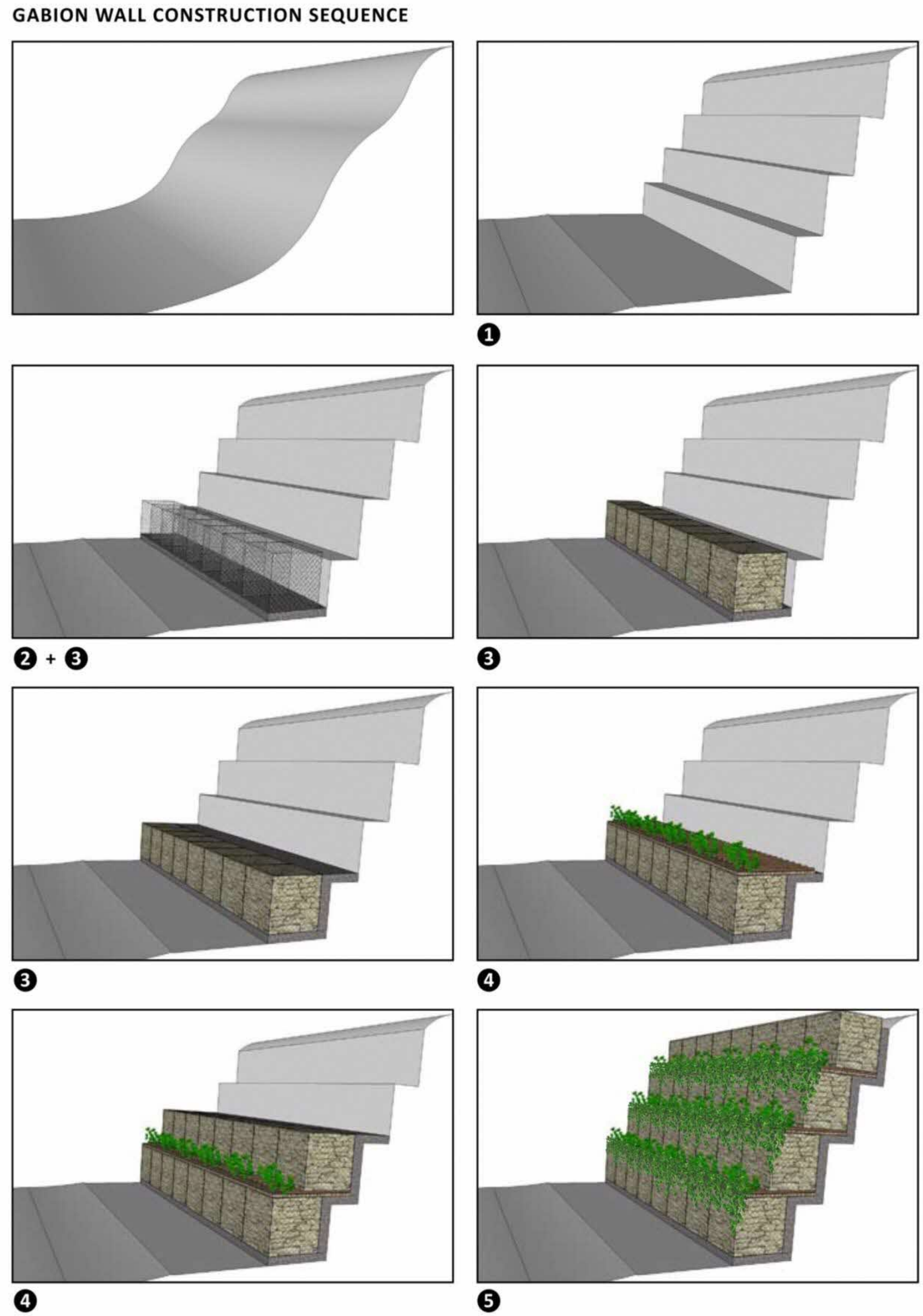


Figure 9.7 Gabion wall construction sequence

9.3.8 Vegetated Reinforced Soil

Elements / Materials:

Reinforcing Cage

- Steel mesh cage
- Dimensions: 60 cm high x 60 cm wide, angled on stream-facing edge to desired profile
- Steel braces at various points for reinforcement

Geotextiles

- Soil-retaining
  - Geo-synthetic fabric
  - Line the inside face of steel mesh to retain fill within reinforcing cage
- Reinforcing filter fabric
  - Separating layer between steel meshes

Plants

- Cuttings of appropriate length (80-100 cm), and diameter (5-10 cm)
  - Ends pointed for easy driving

Fasteners

- Steel securing pins (according to manufacturer's recommendation)

Fill

- Suitable fill

Installation:

1. Grade a step of approx. 60 cm wide at the stream edge, right below the bed level.
2. Lay the filter fabric on this step
3. Install the reinforcing cages on the step using securing pins at approx. 4 pins / m^2.
4. Line the inner face of the steel mesh reinforcing cage with geosynthetic fabric, ensuring that there is an overlap of 10-20 cm between adjacent strips of fabric, and that each strip lap over the top of the mesh.
5. Fix the steel braces in place to support the cage during soil filling and compaction. (refer to manufacturer's instructions for brace installation)
6. Layer fill materials on the slop side of the cage, compacting it at every level, until the soil lift reaches a height of approx. 50 cm.
7. Fold the fabric that is lapping at the top of the cage over this 50 cm soil lift.
8. Repeat steps 2-7 until the desired embankment height is achieved. There should be an overlap of approx. 10 cm between each level of reinforcing cage.
9. Drive cuttings into the reinforced soil lifts.

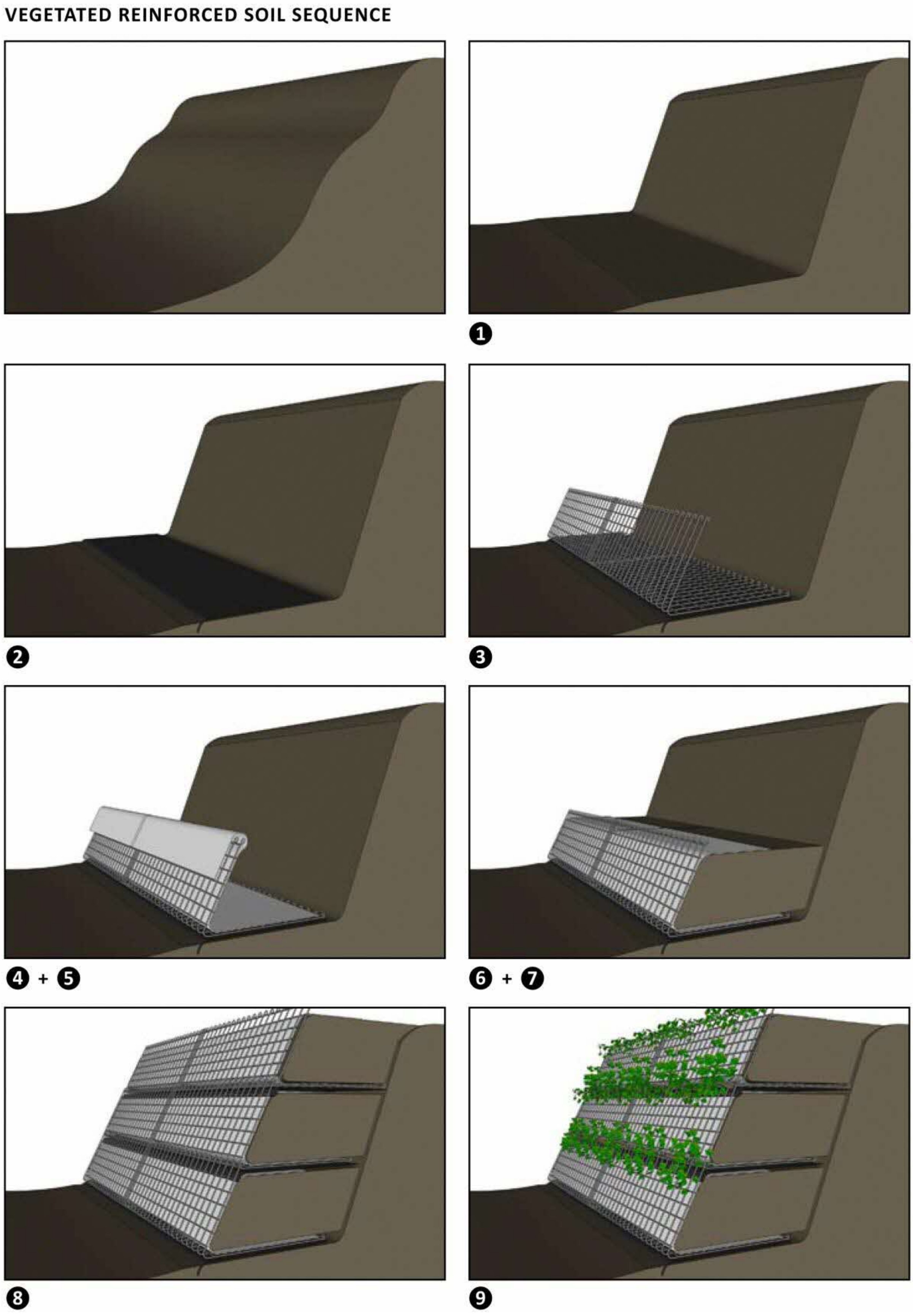


Figure 9.8 Vegetated reinforced soil sequence

9.3.9 Rock Gabion Mat

Elements / Materials:

Gabion Mat

- Cubic wire mesh containers
- Dimensions: 100 cm (W) x 50 cm (L) x 30 cm (H)
- Wire mesh – galvanised steel wire, spot-welded, 4-5 mm diameter, 450N/m² tensile strength, mesh opening 10 x 10 cm
- Subject to Superintending Officer's approval

Stone

- Natural stone or concrete rubble
    - Approx. 150-300 mm diameter

Fasteners

- Galvanised Wire (according to manufacturer's recommendation)
- Steel pins (according to necessity)

Installation:

1. Grade the stream bed to desired profile. Then excavate at extra 30 cm below desired bed level to accommodate gabion mats.
2. Install wire mesh containers side by side and in rows to fill the graded surface, ensuring minimum gap between them.
3. Fill the wire mesh containers with stone and close them securely using galvanised wire or according to manufacturer's recommendation.
4. Secure the top row of gabion mat to the lower ones by weaving galvanised wires between them.
5. Stake the gabion mat to the ground using steel pins for added stability if necessary.

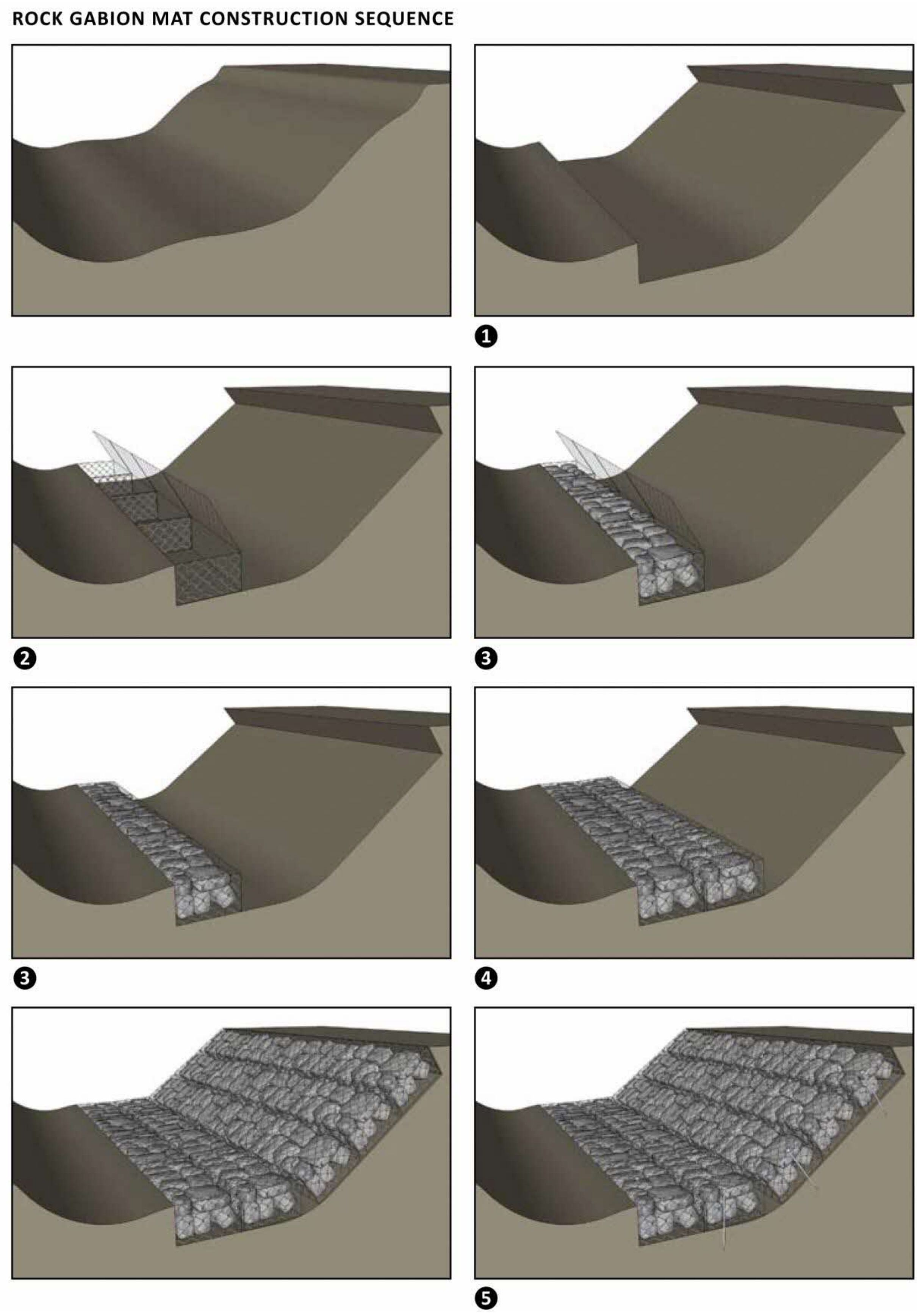


Figure 9.10 Rock garden mat construction sequence

9.3.10 Rock Sill

Elements / Materials:

<u>Plants</u>

- Cuttings of appropriate length (80-100 cm), and diameter (5-10 cm)
  - Ends pointed for easy driving

<u>Stones</u>

- Natural stone (average diameter 50-80 cm)
- Filter material under rock sill:
  - Sand-gravel mixture (0-200 mm)
  - Alternative: concrete C12 / 15
- Gap-filler between stones:
  - Sand-gravel mixture (0-200 mm)

<u>Fill</u>

- Suitable fill

Installation:

1. Lay a layer of approx. 20 cm thick of filter material in an arch shape (arched against the stream flow direction) at a desired location crossing the stream. The width of the arch should be approx. 150 – 200 cm.
2. Place large stones upright on top of this layer of filter material, following the arch shape and width, ensuring that they form a stable, interlocking structure.
3. This arch of stones must protrude out into either bank by at least 50 cm. This protruding part should be completely embedded in the bank.
4. Drive plant cuttings in between the stones around the transition areas between the stream bed and banks.
5. Fill the remaining gaps between the stones with gravel. Fill the exposed stones at banks with fill.

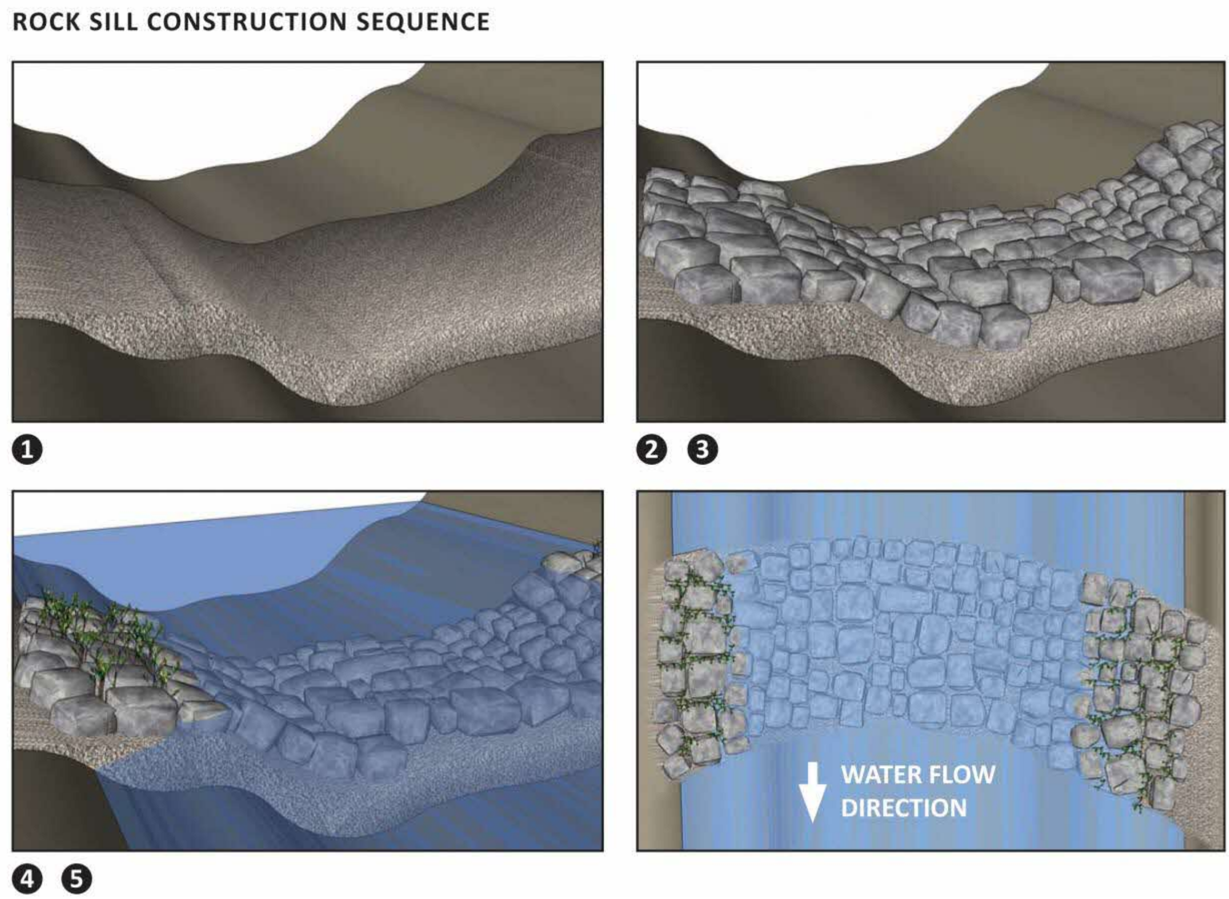


Figure 9.11 Rock sill construction sequence

9. 4 Case Study: Kallang River Bishan-Ang Mo KioPark

Over the years, Singapore has developed a network of water bodies to fulfil the functional needs of catchment, distribution and purification. The longest river in Singapore, the 10-km long Kallang River plays a crucial role in this pervasive network of 17 reservoirs, 32 major rivers, and more than 7,000 kilometres of canals and rivers.

For a stretch of about 3 km, this river passes through Bishan-Ang Mo Kio Park, one of the biggest and most popular regional parks in Singapore. The potential for the river to be better-utilized for recreational activities was not met as it was hidden from view and took the form of a hard concrete canal with railings on one side of the park. As this canal is located downstream of the overflow of two large reservoirs, the water level can drastically change within a short period of time, particularly in Singapore's tropical climate with its extreme storm events.

In 2007, a team of consultants, comprising of Atelier Dreiseitl, CH2M Hill, Geitz & Partner GBR were employed by the PUB, Singapore's National Water Agency and National Parks Board (NParks) and to propose, and eventually implement, a new design for the park that would facilitate the interaction of the public with the river.

As a general strategy, plants act as retention system that slows down the water flow. At the widest point of the park, the course of the stream is meandered / diverted to slow down the flow of the river. It is also at this point that the river is widened and the banks made much gentler to allow the public to walk down the banks and occupy the multi-functional floodplain.

The banks of the river are completely reinforced using various techniques of soil bio-engineering, depending on the slope of the banks, the planting scheme, as well as the type and intensity of recreational activities adjacent to it.

A detailed site survey was conducted, and landscape specialists were engaged to identify existing plants species, give specific advice on transplanting and conservations during the construction phase, as well as recommend appropriate species for new plantings.

Discussions with the soil bioengineering specialist and civil engineer yielded an appropriate method of transitioning between the concrete channels to the naturalized river, as shown in the diagram below.

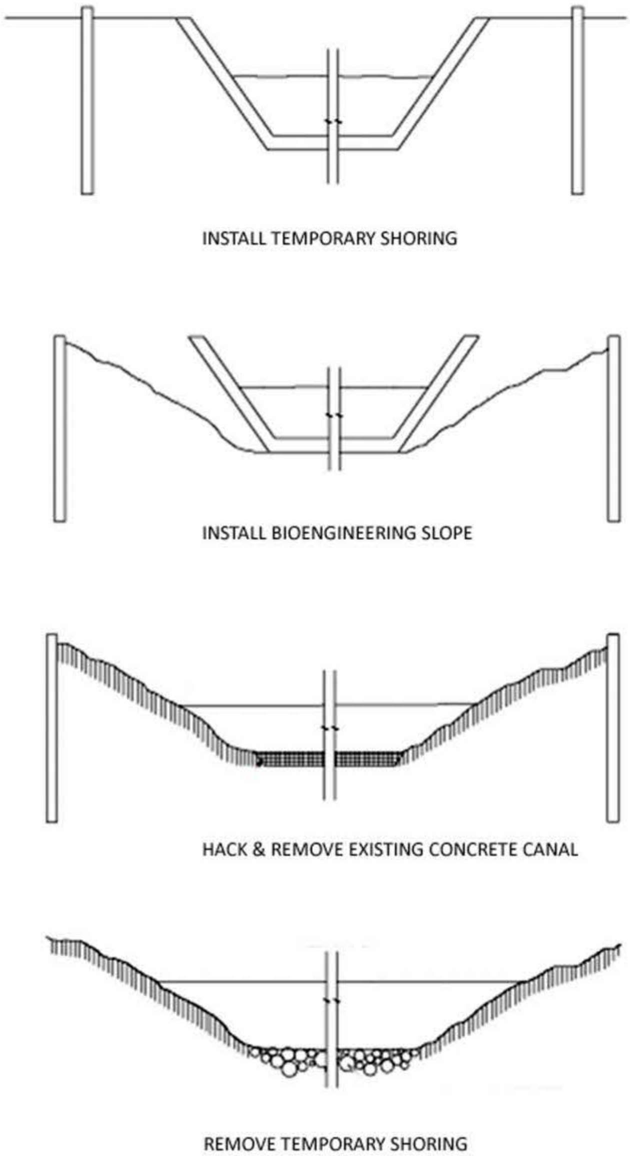


Figure 9.12 Transitioning between the concrete channel to the naturalized river

Modeling & simulation tests

Detailed calculation and modeling were carried out to ensure that the river profiles and hydraulic design of the river would meet the safety standards and the carrying capacity as required by government agencies. Locations of hydraulic stresses are also identified, as input for the application of the appropriate soil bioengineering techniques.

The water flow and water levels corresponding to different n-values were graphed out in order to determine the optimal n-value to be used in the river profiles. Appropriate soil bioengineering techniques and plant species are then selected, taking their roughness coefficients and slope-stabilization capabilities into consideration.

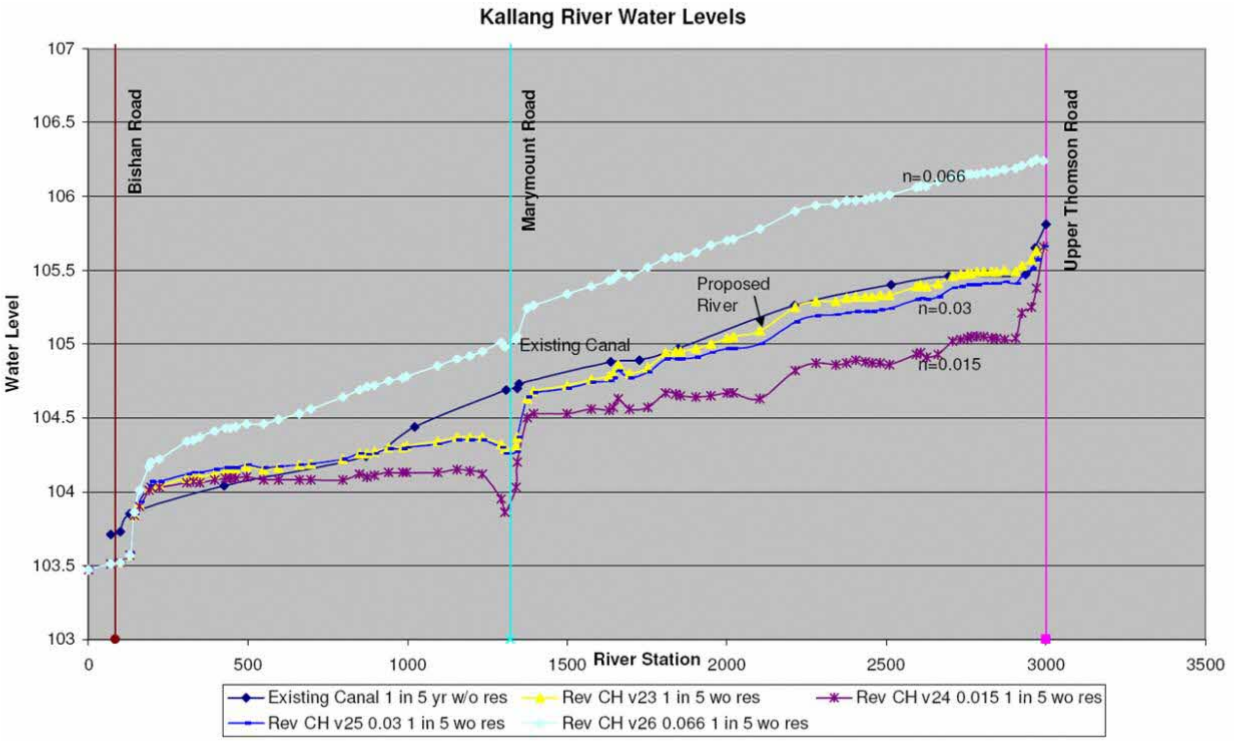


Figure 9.13 Chart showing simulation of river flow and water levels in relation to different n-values for the existing and proposed Kallang River.

Test Reach Implementation

A test reach has been constructed on-site to test out the various soil bioengineering techniques which would eventually be employed in the Kallang River / Bishan Park redevelopment project. The following plan and sections show the various techniques being implemented in the test reach.

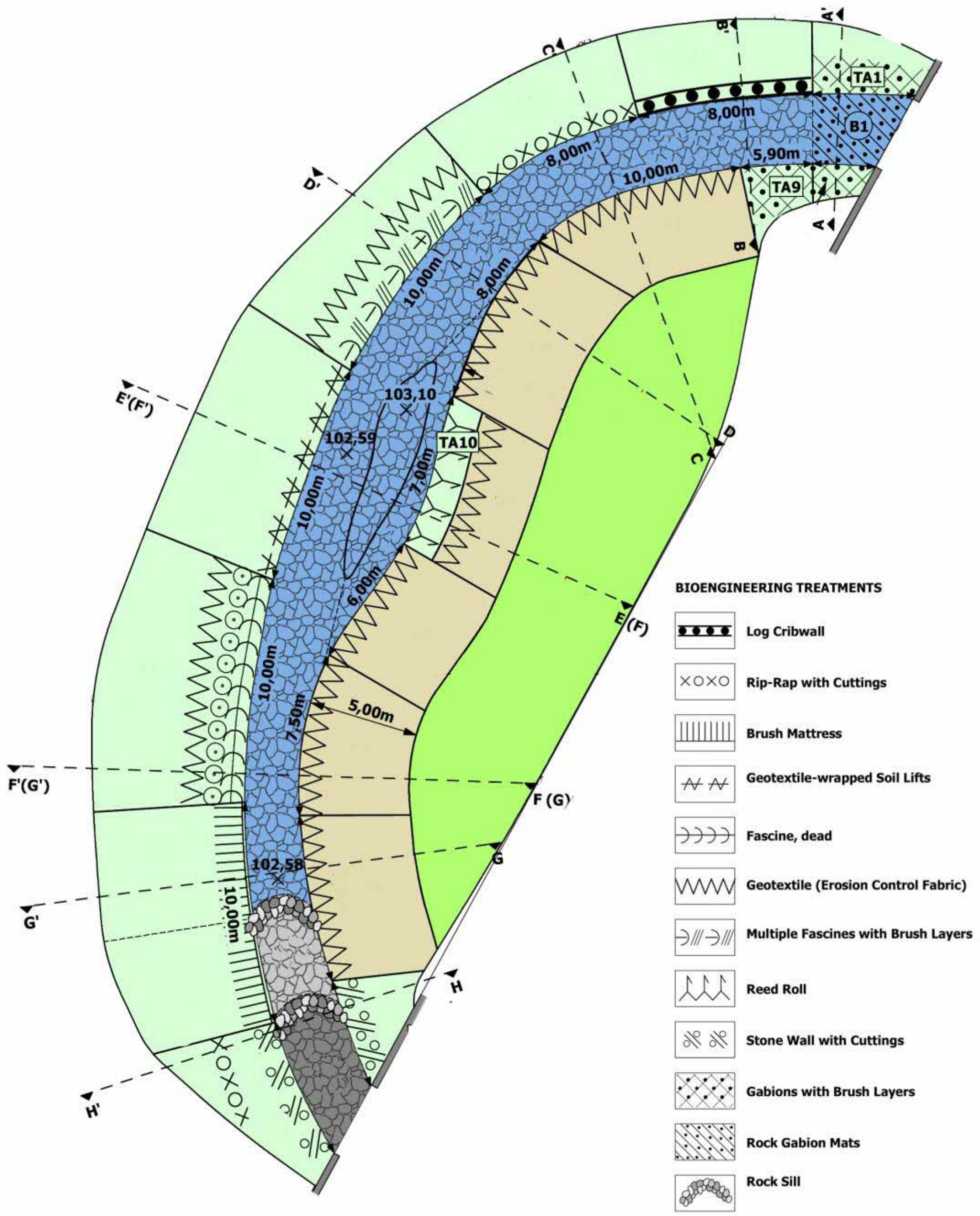


Figure 9.14 Test reach in Kallang River Bishan Park

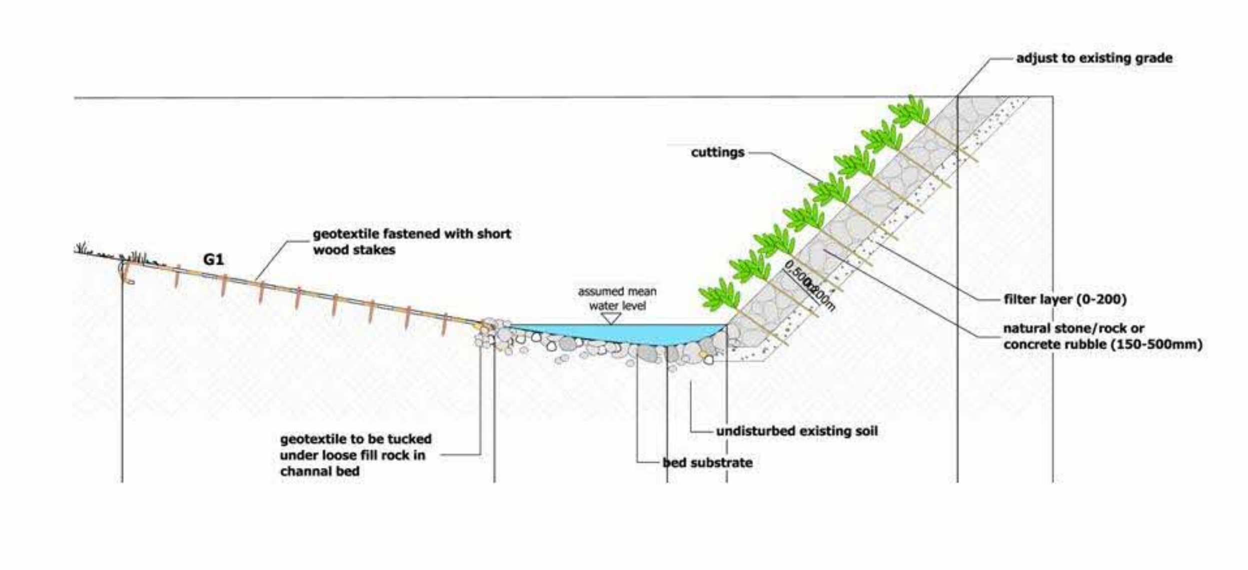


Figure 9.15 Section C-C

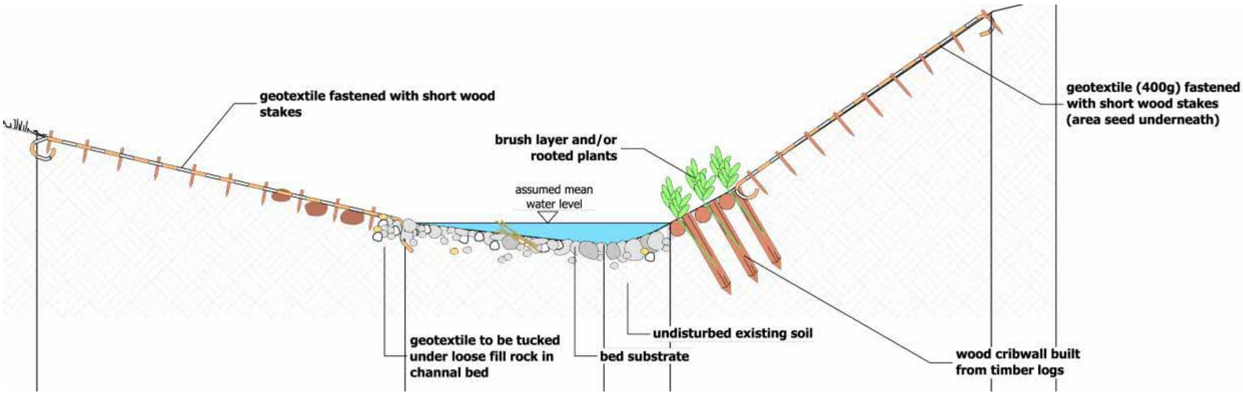


Figure 9.16 Section D-D

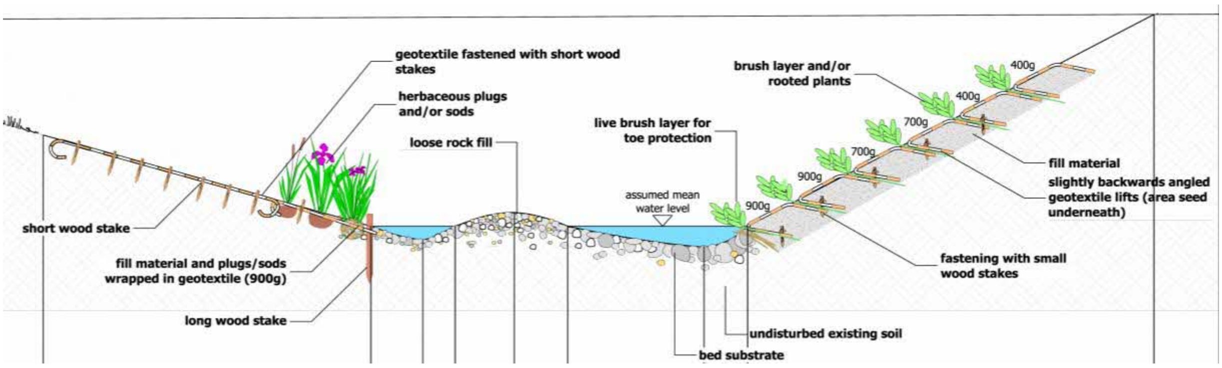


Figure 9.17 Section E-E

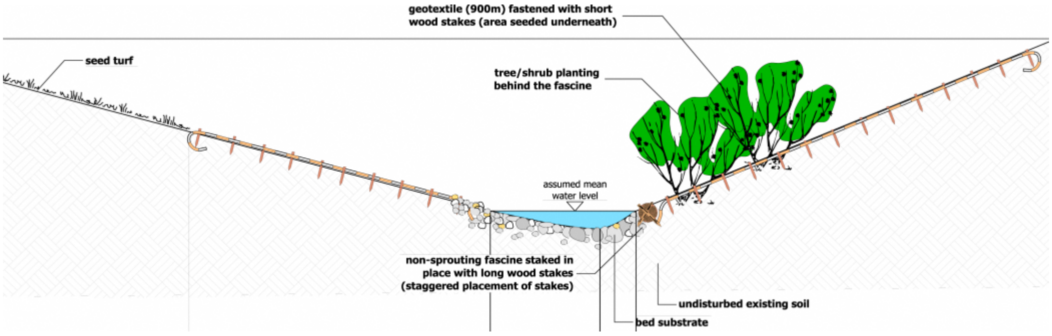


Figure 9.18 Section F-F

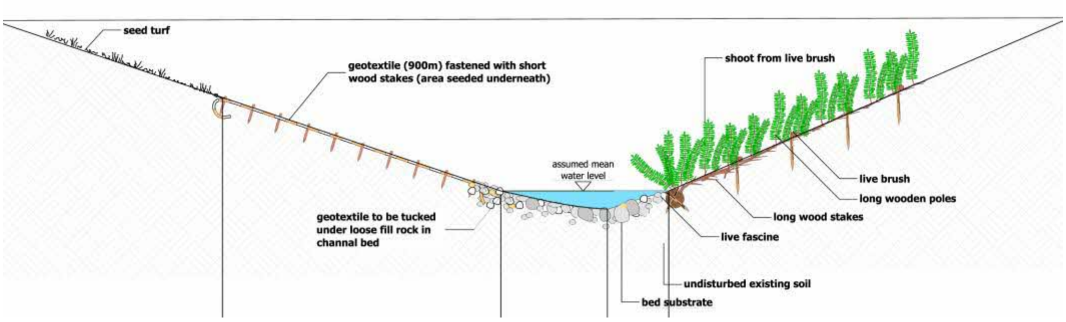


Figure 9.19 Section G-G

Test Reach – Selection of plant material

The plants for the test bed are selected carefully. The following parameter was important for the selection of the plants:

- *Good sprouting characteristics from cuttings based on literature record or personal experience*
- *Water-loving, waterlog-tolerant and river/coastal plants*
- *Availability from NParks*
- *Native plant*

Good sprouting characteristic from cutting is essential as the plants should fulfill the function of slope stabilization using their root system. The use of cutting is a more cost effective for implementation compared to use of whole plant in such a large scale project.

Water-loving, waterlog-resistant and river/coastal plants are more likely to survive and do well in prolong or periodic flooding condition, a condition that is expected of the test bed and the real implementation site. On the dryer side of the test reach, less waterlog-tolerant species can be used.

As most of the species selected are not common commercially available, both in terms of the large quantity and size required, only those with availability from NParks have been chosen.

Species

Dillenia indica 	Hibiscus tiliaceus 
Scaveola taccada 	Dolichandrone spathacea 

Leucaena leucocephala	Eugenia oleina
	
Ardisia elliptica	Macaranga sp
	

Table 9.2 The species which are installed in the different test areas and are selected based on the criteria earlier described.

Test Reach – Monitoring

The objective of the Test Reach is to obtain data during the construction and development phase that indicates to what extent the plants used, in combination with the different bioengineering construction methods, are able to quickly and reliably protect the newly constructed stream banks of the Kallang River from erosion caused by the stormwater flow and surface water runoff.

All relevant data relating to the construction phase and completion including plant materials, soil conditions, weather conditions, etc, will be documented during the monitoring phase.

Figure 9.20 Test bed under construction in Bishan Park 2 (13.02.2009)

Depending on the type of parameter, data will be collected at regular intervals (for plant growth data and environmental conditions) or during specific events (eg. specific storm intensities). Prescribed intervals for data collection are necessary for example, for vegetation growth. Episodic events are sufficient for documenting erosion and sedimentation processes. The data collection needs to be conducted by trained personnel.

When enough data about the different parameters and variables have been collected (over approx. 6 months), conclusions can be drawn from the results, which can then be used for the first construction phase of the Kallang River.

9.5 References

Books

1) Dictionary of Soil Bioengineering (1996)

Society for Soil Bioengineering

ISBN 978-3-7281-3055-6

2007, vdf Hochschulverlag AG an der ETH Zürich

2) Soil Bioengineering: Construction Type Manual

Helgard Zeh

1995 – European Federation for Soil Bioengineering

(Switzerland, Germany, Austria, Italy, Spain)

Surface Flow Wetlands 10

10

Chapter 10 Surface Flow Wetlands

10.1 Introduction

The use of surface flow wetlands (usually referred to as constructed wetlands) for urban stormwater quality improvement is widely adopted in many urban environments, many of which have been successfully incorporated into the urban landscape. Design considerations include the interaction between the wetland hydrology and hydrodynamic behaviour with the various physical, chemical and biological treatment processes. The operating conditions of these systems are stochastic in nature, with intermittent and highly variable hydraulic and pollutant loading.
Constructed wetland systems are shallow extensively vegetated water bodies that use enhanced sedimentation, fine filtration and pollutant uptake processes to remove pollutants from stormwater.

Wetlands generally consist of an inlet zone (sedimentation basin to remove coarse sediments – see Chapter 4 Sedimentation Basins), a macrophyte zone (a shallow heavily vegetated area to remove fine particulates and uptake of soluble pollutants) and a high flow bypass channel (to protect the macrophyte zone). They are designed primarily to remove stormwater pollutants associated with fine to colloidal particulates and dissolved contaminants. Figure 10.1 shows a typical layout of a wetland system. However, given the constraints and shape of land in Singapore, other designs like a linear wetland design (to fit into Drainage Reserves) is also possible.

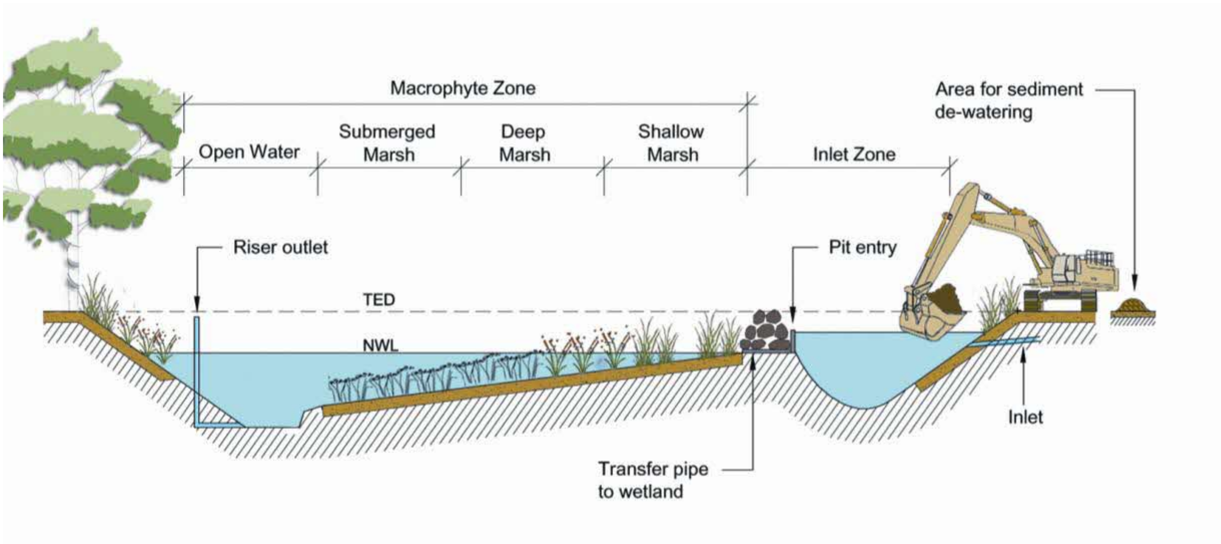


Figure 10.1 Layout of a constructed wetland system

10.2 Design Considerations

The major elements of constructed wetland systems are shown in the figure below. Design considerations include hydrologic and hydrodynamic factors aimed at optimizing the performance of these systems. In addition to these factors, design considerations of stormwater treatment wetland need to also include the botanical structure and layout of the wetland and the hydrologic regime necessary to sustain the botanical structure. Wetland macrophytes support a number very important pollutant removal mechanism, including the removal of fine suspended solids and associated contaminants (e.g. nutrients, metals, organic contaminants and hydrocarbons).
Wetlands provide four key operational functions in a treatment train:

- Promote sedimentation of particles larger than 125µm in the <u>inlet zone</u>.
- Discharge water from the inlet zone into the <u>macrophyte zone</u> for removal of fine particulates and dissolved contaminants through the processes of enhanced sedimentation, filtration, adhesion and biological uptake.
- Ensure that the required detention period is achieved for all flow though the wetland system through the incorporation of a riser outlet system.
- Ensure adequate flood protection of the macrophyte zone from scouring during “above-design” conditions by designing for by-pass operation when inundation in the macrophyte zone reaches the design maximum extended detention depth.

The level of treatment achievable by a wetland system can be maximized by optimising the relationship between detention time, wetland volume and the hydrologic effectiveness. The relationship between detention time and pollutant removal efficiency is largely influenced by the settling velocity of the target particulate.

Figure 10.2 presents an overview of key design elements of a constructed wetland.

10.2.1 Hydrodynamic Design

The performance of constructed wetland in the removal of stormwater pollutants is affected by many factors. The hydrodynamic behaviour of a constructed wetland system is determined by the hydrologic and hydraulic design of the system. Stormwater wetlands are subjected to extended periods of no inflow followed by events of high hydraulic loading and pollutant loads. Flow attenuation can be significant as the detention storage of the wetland fills and drains during these events. Hydrodynamic flow patterns within the wetland can vary at different stages of wetland inundation and thus, the detention period of stormwater inflow for each individual event can be highly variable.

Poor wetland hydrodynamics and lack of appreciation of the stormwater treatment chain are often identified as major contributors to wetland management problems. A summary of desired hydrodynamic characteristics and design elements is presented in Table 10.1.

A controlled inflow wetland can be designed to improve the quality of dry weather flow that is regularly observed in canals and drains in Singapore. A diversion structure is often employed to divert a certain portion of the dry weather flow preferably by gravity from the canal or drain into the inlet zone (functions as a sedimentation basin). After energy dissipation and settling of coarser sediments, the water then enters the macrophyte zone of the constructed wetland for further treatment. The diversion structure often consists of a weir, sump, pipe and control valve (if needed). The rate of flow can be diverted depends on the area available for constructing the wetland and can be calculated using orifice equations based on the water head that can be built up for diversion. Gross pollutant traps (GPT) like gratings or alike should be placed at the upstream of any diversion structure to screen out – litters, stones, dry leaves etc. Regular checking and cleaning shall be performed to avoid the accumulation of sediments and leaves in the GPT, diversion structure and pipes.

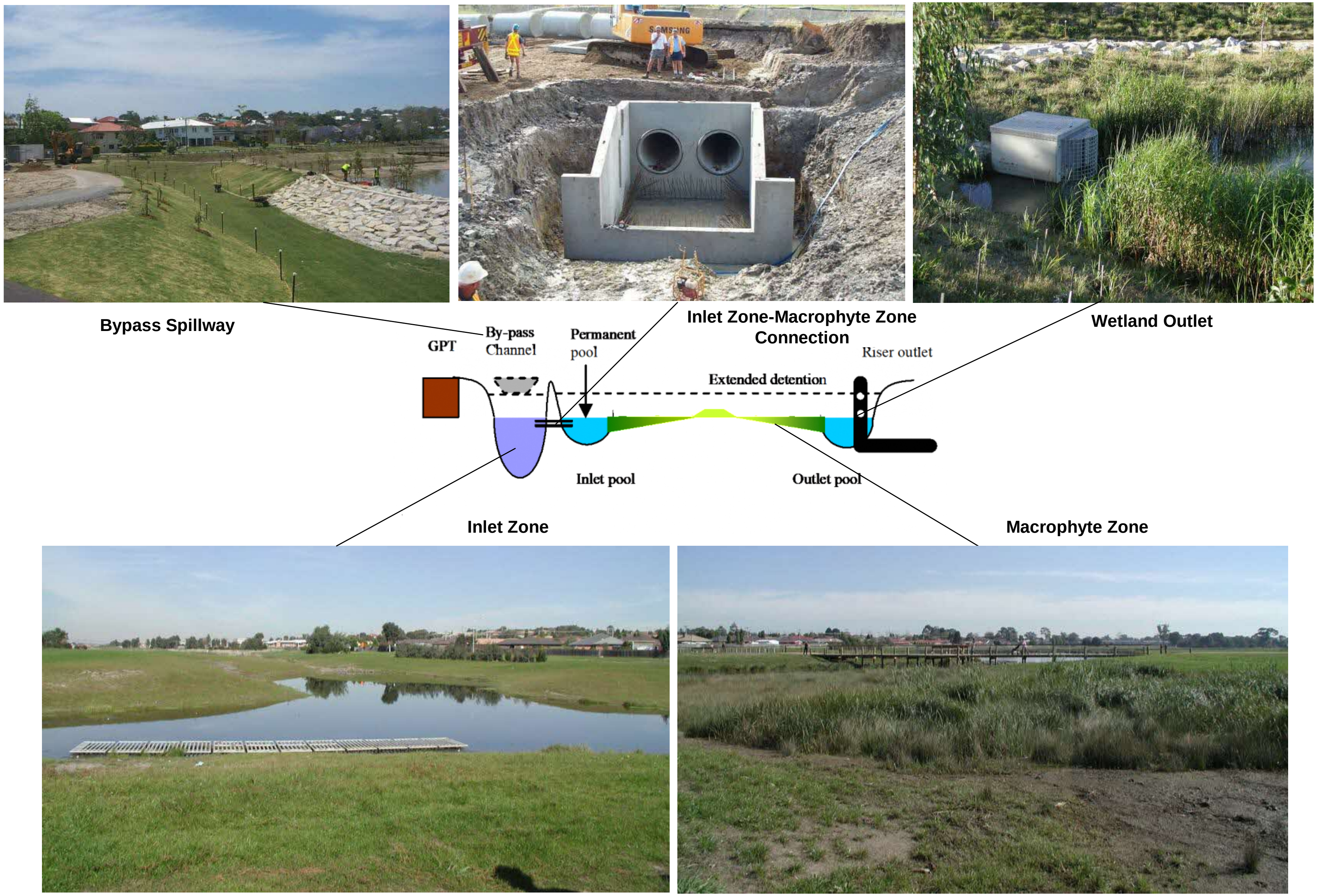


Figure 10.2 Design elements of a constructed wetland

Table 10.1: Desired Wetland Hydrodynamic Characteristics and Design Elements

Hydrodynamic Characteristics	Design Issues	Remarks
Uniform distribution of flow velocity	Wetland shape, inlet and outlet placement and morphological design of wetland to eliminate short-circuit flow paths and "dead zones".	Poor flow pattern within a wetland will lead to zones of stagnant pools which promotes the accumulation of litter, oil and scum as well as potentially supporting mosquito breeding. Short circuit flow paths of high velocities will lead to the wetland being ineffective in water quality improvement.
Inundation depth, wetness gradient, base flow and hydrologic regime	Selection of wetland size and design of outlet control to ensure compatibility with the hydrology and size of the catchment draining into the wetland. Morphological and outlet control design to match botanical layout design and the hydrology of the wetland.	Regular flow throughput in the wetland would promote flushing of the system thus maintaining a dynamic system and avoiding problems associated with stagnant water, e.g. algal blooms, mosquito breeding, oil and scum accumulation etc. Inadequate attention to the inundation depth, wetness gradient of the wetland and the frequency of inundation at various depth ranges would lead to dominance of certain plant species especially weed species over time, which results in a deviation from the intended botanical layout of the wetland. Recent research findings indicate that regular wetting and drying of the substrata of the wetland can prevent releases of phosphorus from the sediment deposited in the wetland.
Uniform vertical velocity profile	Selection of plant species and location of inlet and outlet structures to promote uniform velocity profile	Preliminary research findings have indicated that certain plant species have a tendency to promote stratification of flow conditions within a wetland leading to ineffective water pollution control and increase the potential for algal bloom.
Scour protection	Design of inlet structures and erosion protection of banks	Owing to the highly dynamic nature of stormwater inflow, measures are to be taken to "protect" the wetland from erosion during periods of high inflow rates.

10.2.2 Detention Time and Hydrologic Effectiveness

Detention time is the time taken for an idealized 'parcel' of water entering the wetland to travel through the macrophyte zone assuming 'plug' flow conditions. Simulations using computer models, are often required to optimize, for a given site area, the relationship between wetland detention time[1] and wetland hydrologic effectiveness to ensure treatment performance is maximised. Hydrologic effectiveness is a term used to quantify the mean annual volume of stormwater runoff captured and treated within the wetland and is expressed as a percentage of the mean annual runoff volume generated from the contributing catchment. For well-designed wetlands without any site constraints, the hydrologic effectiveness of constructed wetlands should be greater than 80%.

The relationship between notional detention time and pollutant removal efficiency is largely influenced by the settling velocity of the target particulates, although defining the settling velocity of fine to colloidal particulates is not a straight forward exercise.

Standard equations for settling velocities often do not apply for fine particulates owing to the influence of external factors such as wind and water turbulence. It is therefore recommended that a notional detention time should preferably be 48 hours (and not less than 48 hours) to remove nutrients effectively from urban stormwater.

1 It should be noted that detention time is rarely a constant and the term notional detention time is used throughout this chapter to provide a point of reference in modelling and determining the design criteria for riser outlet structures.

10.2.3 Inlet Zone Design Considerations

The inlet zone of a constructed stormwater wetland is designed as a sedimentation basin (see Chapter 4 Sedimentation Basins) and has two key functional roles, i.e. as a pre-treatment for, and to control and regulate flow into, the downstream macrophyte zone.

The primary role of the inlet zone is to remove coarse to medium sized sediment (i.e. 125µm or larger) prior to flows entering the macrophyte zone. This ensures the vegetation in the macrophyte zone is not smothered by coarse sediment and allows the macrophyte zone to target finer particulates, nutrients and other pollutants. It also meant that maintenance practice are generally focused at the inlet zone for regular desilting (generally from annually to once every 5 years), leaving the downstream macrophyte zone to have a more passive maintenance regime mainly directed at botanical maintenance.

The second role of the inlet zone is the control and regulation of flows entering the macrophyte zone and bypass of flows during ‘above design flow’ conditions. The outlet structures from the inlet zone (i.e. sedimentation basin) are designed such that flows up to the ‘design flow’ (typically the 1 year ARI) enter the macrophyte zone whereas when the macrophyte zone is at its top of extended detention level, catchment inflows are bypassed around the macrophyte zone. In providing this function, the sedimentation basin protects the vegetation in the macrophyte zone against scour during high flows.

Chapter 4 (Sedimentation Basins) presents the range of issues that should be considered when designing an inlet zone.

As a pre-treatment component of a wetland, it should be borne in mind that even when the available space for a constructed wetland system is constrained, the size of the inlet zone (i.e. sedimentation basin) should not be reduced as it is determined to target the medium to coarse particles.

This ensures the larger sediments are effectively trapped and prevented from smothering the macrophyte zone. When the site constrains the size of the constructed wetland, it is the macrophyte zone of the wetland that should be reduced accordingly.

10.2.4 Macrophyte Zone Design Considerations

The layout of the macrophyte zone needs to be configured such that system hydraulic efficiency is optimised and healthy vegetation sustained. Design considerations include:

- The preferred extended detention depth is 0.5 m. Deeper extended detention depths up to a maximum of 0.75 m may be acceptable where the wetland hydrologic effectiveness is greater than 80% and where the botanic design uses plant species tolerant of greater depths of inundation such as *Scirpus grossus* and Typha.
- The bathymetry of the macrophyte zone should be designed to promote a sequence of ephemeral, shallow marsh, marsh and deep marsh zones in addition to small open water zones. The relative proportion of each zone will be dependent on the target pollutant and the wetland hydrologic effectiveness.
- The macrophyte zone is required to retain water permanently and therefore the base must be of suitable material to retain water (e.g. clay). If in-situ soils are unsuitable for water retention, a clay liner (e.g. 300mm thick compacted clay) must be used to ensure there will be permanent water for vegetation and habitat.
- The bathymetry of the macrophyte zone should be designed so that all marsh zones are connected to a deeper open water zone to allow mosquito predators to seek refuge in the deeper open water zones during periods of extended dry weather.
- Particular attention should be given to the placement of the inlet and outlet structures, the length to width ratio of the macrophyte zone and flow control features to promote a high hydraulic efficiency within the macrophyte zone.

- Provision to drain the macrophyte zone for water level management during the plant establishment phase should also be considered.
- The macrophyte zone outlet structure needs to be designed to provide a notional detention time (usually 72 hours in Australia and 48 hours in Singapore, probably due to warmer temperature and faster plant growth) for a wide range of flow depths. The outlet structure should also include measures to exclude debris to prevent clogging.

10.2.5 Wetlands Constructed within Flood Detention Basins

In many urban applications, wetlands can be constructed in the base of flood detention basins, thus reducing the land required for stormwater treatment. In these situations, wetland systems will occasionally become inundated to greater depths than the extended detention depth; however, the inundation duration is usually relatively short (hours) and is unlikely to affect the wetland vegetation provided there is a safe pathway to drain the wetland following flood events which avoids scour of the wetland vegetation and banks.

When designing a wetland within a detention basin, the wetland system should be located at the upstream end of the basin. By pass of the macrophyte zone during large flow events would commence to inundate the basin from the downstream end of the basin away from the wetland. This ensures that flooding of the wetland during the flood detention operation is by 'backwater' inundation across the wetland thus protecting the macrophyte vegetation from scour by high velocity flows.

10.2.6 Landscape Design

Constructed wetlands are often located within accessible open space areas and can become interesting community features. Landscape design aims to ensure that marsh planting fulfils the intended stormwater treatment function as well as integrating with their surrounds. Opportunities to enhance public amenity and safety with viewing areas, pathway links, picnic nodes and other elements should be exploited. Community education through signage and public art can also be explored. It is important that the landscape of constructed wetlands addresses stormwater quality objectives whilst being sensitive to these other important landscape aims.

10.2.7 Vegetation Types

Vegetation planted in the macrophyte zone has an important functional role in treating stormwater flows, as well as adding aesthetic value. Dense planting of the littoral zone will inhibit public access to the macrophyte zone, minimising potential damage to wetland plants and reducing the safety risks posed by water bodies.

Plant species for the wetland area will be selected based on the hydrologic regime, microclimate and soil types of the region, and the life histories, physiological and structural characteristics, natural distribution, and community groups of the wetland plants. The National Parks Board of Singapore should be consulted in determining suitable plantings for constructed wetlands in Singapore. A CUGE publication on "A selection of plants for waterways and waterbodies in the tropics" can be downloaded at https://www.cuge.com.sg/research/A-Selection-of-Plants-for-Waterways-and-Waterbodies-in-the-Tropics.

The planting densities recommended in the list should ensure that 70 - 80 % cover is achieved within two growing seasons. The distribution of the species within the wetland will relate to their structure, function, relationship and compatibility with other species.

10.2.8 Designing to Avoid Mosquitoes

To reduce the risk of high numbers of mosquitoes, there are a number of design features that can be considered. Not all of these will be feasible in any one situation, but they include:

- Providing access for mosquito predators, such as fish, dragons fly nymphs and predatory insects, to all parts of the water body.

- Providing a deep sump of permanent water (for long dry periods or for when water levels are artificially lowered) so that mosquito predators can seek refuge and maintain a presence in the wetland.
- Maintaining a continuous flow through the wetland and/or natural water level fluctuations that disturb the breeding cycle of mosquito species.Where possible, incorporating a steep slope into the water, preferably greater than 30° or 3:1 horizontal to vertical. Note that steep edges may be unacceptable for public safety reasons, and a slope of up to 8:1 horizontal to vertical is generally used.
- Wave action from wind over open water will discourage mosquito egg laying and disrupt the ability of larvae to breathe.
- Providing a bathymetry such that regular wetting and drying is achieved and water draws down evenly so isolated pools are avoided.
- Providing sufficient gross pollutant control at the inlet such that human derived litter does not accumulate and provide breeding habitat.
- Providing ready access for field operators to monitor and treat mosquito larvae.
- Ensuring maintenance procedures do not result in wheel rut and other localised depressions that create isolated pools when water levels fall.
- Ensuring overflow channels don't have depressions that will hold water after a storm event.

Each case has to be considered on its own merits. It may be possible that a well established constructed wetland will have no significant mosquito breeding associated with it; however, changes in climatic and vegetation conditions could change that situation rapidly. Maintaining awareness for mosquito problems and regular monitoring for mosquito activity should be considered as a component of the management of these sites. Effective and environmentally sound control products are available for control of mosquito larvae in these situations.

10.2.9 Designing for Maintenance Access

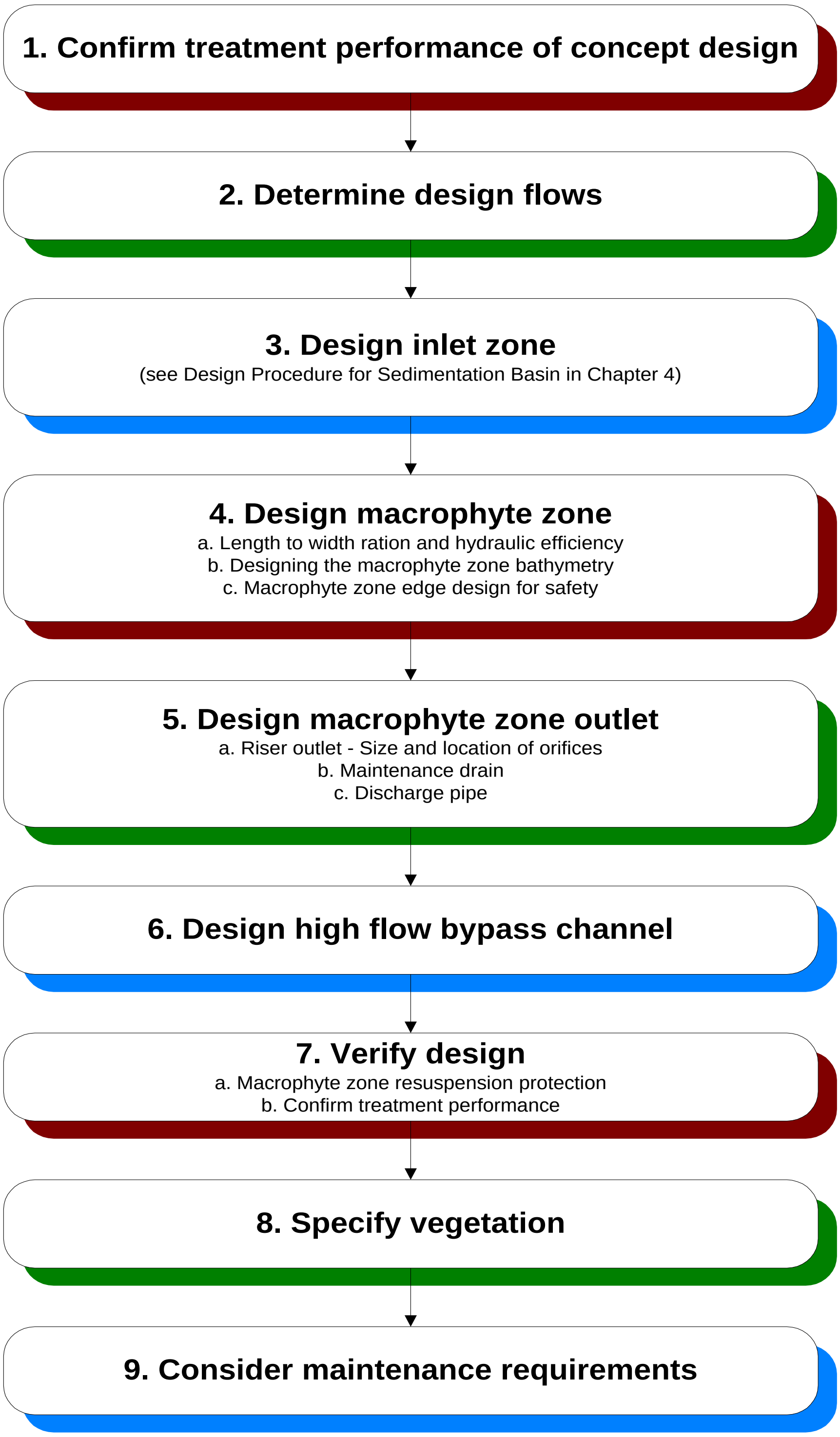
Access to all areas of a constructed wetland is required for maintenance. In particular, inlet zones and gross pollutant traps require a track suitable for heavy machinery for removal of debris and desilting as well as an area for dewatering removed sediments (refer to Chapter 4 Sedimentation Basins). The track should be permanent and have a maximum slope 1:10.

To aid maintenance, it is recommended that the inlet zone is constructed with a hard (i.e. rock) bottom. This is important if maintenance is performed by driving into the basin. It also serves an important role by allowing excavator operators to detect when they have reached the base of the inlet zone during desilting operations.

Macrophyte zones require access to the areas for weeding and replanting as well as regular inspections. Commonly, these access tracks can be incorporated with walking paths around a wetland system. Maintenance access to constructed wetland needs to be considered when determining the layout of a wetland system.

10.3 Design Process

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



10.3.1 Step 1: Confirm treatment size

A conceptual design of a constructed wetland system is typically undertaken prior to detailed design. The performance of the concept design must be checked to ensure that stormwater treatment objectives will be satisfied.

The performance of a concept design is checked using sizing curves. Sizing curves for TSS, TP and TN removal for a range of feasible extended detention depths are given in Figure 10.3 to Figure 10.5.

The curves for Singapore were derived using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC V5), assuming the system receives direct runoff with pre-treatment through an inlet zone of 2m deep and with the macrophyte zone of 0.5 m average permanent pool depth and a notional detention time of 48 hours. The curves provide the detailed designer with a useful visual guide to illustrate the sensitivity of constructed wetland performance to the ratio of macrophyte zone treatment area and contributing equivalent impervious catchment area.

The curves given in Figure 10.3 to Figure 10.5 allow the detailed designer to make a rapid assessment as to whether the concept design falls within the "optimal size range" or if it is potentially under or over-sized, i.e.

- an under-sized system might indicate the wetland is part of a "treatment train" or that another supplementary treatment device may be located somewhere else within the catchment. This should be checked by the detailed designer.
- an over-sized system suggests the concept design of the wetland may have been inadvertently sized the wetland such that it is operating well beyond its point of "diminishing performance" (i.e. where incremental increases in wetland size, and thus cost, result in only a marginal increase in treatment performance). In this instance, the detailed designer should confirm whether or not the wetland size can be reduced or if additional treatment devices may be required.

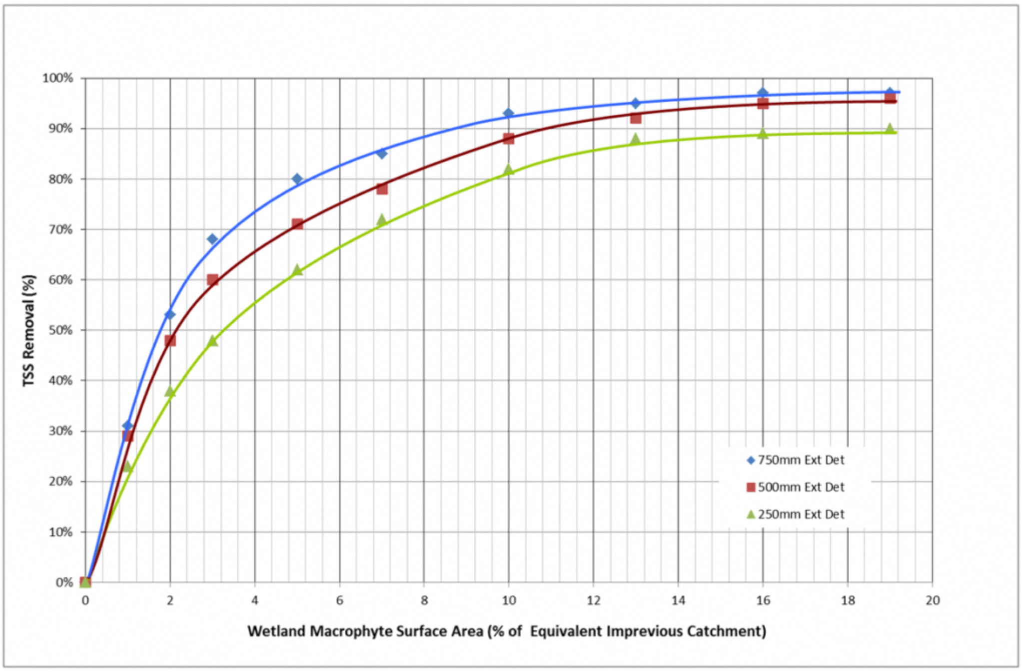


Figure 10.3 Wetland TSS removal performance (Reference: Station 43)

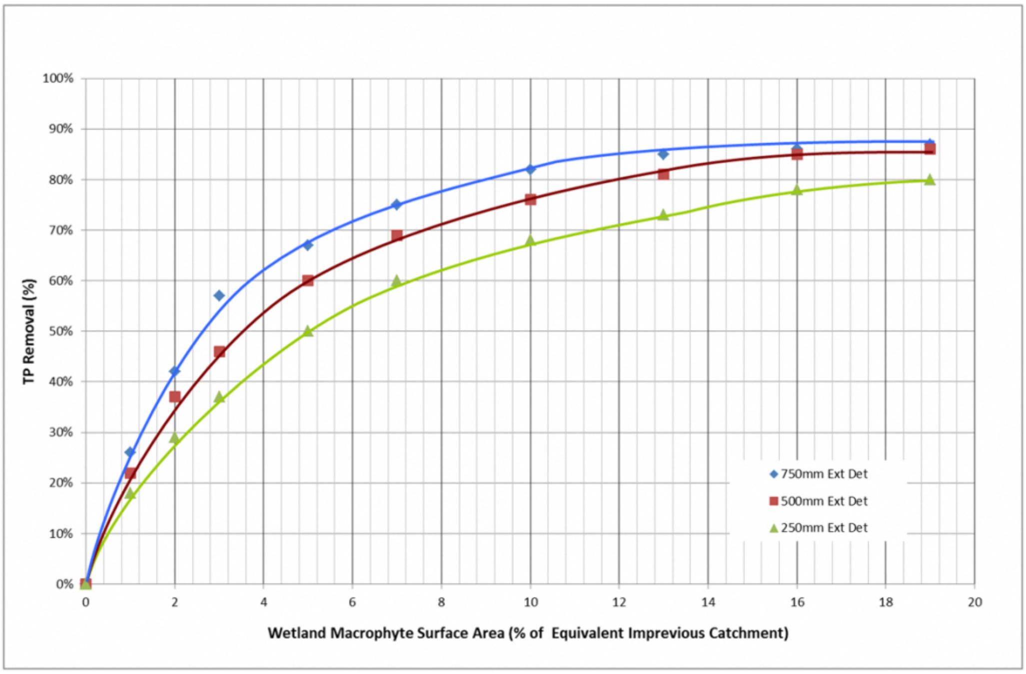


Figure 10.4 Wetland TP removal performance (Reference: Station 43)

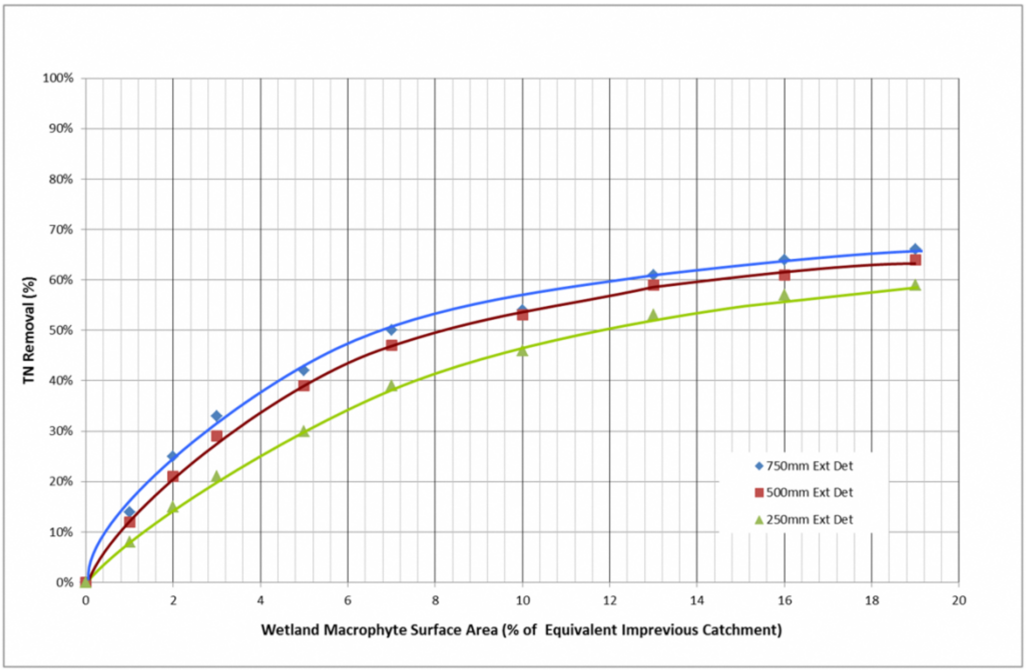


Figure 10.5 Wetland TN removal performance for Singapore

10.3.2 Step 2: Determine Design Flows

10.3.2.1 Design Discharges

The following design flows is required to configure the inlet zone and high flow bypass elements of a constructed wetland:

- Design operation flow (1 year ARI) for sizing the inlet zone (i.e. sedimentation basin) and the 'control' outlet structure (i.e. overflow pit and pipe connection) discharging to macrophyte zone.
- Above design flow for design of the high flow bypass around the macrophyte zone. The discharge capacity for the bypass system may vary depending on the particular situation but will typically correspond to one of the following:
  - Minor design flow – for situations where only the minor drainage system is directed to the inlet zone. Relevant local government guidelines should be referred to for the required design event for the minor design flow.
  - Major flood flow – for situations where both the minor and major drainage system discharge into the inlet zone.

10.3.2.2 Design Flow Estimation

A range of hydrologic methods can be applied to estimate design flows. If the typical catchment areas are relatively small, the Rational Method design procedure is considered to be a suitable method for estimating design flows. However, if the constructed wetland is to form part of a flood detention basin (Section 10.2.5) or if the catchment area to the wetland is large (> 50 ha), then a catchment flood routing modelling approach should be adopted for flood estimation.

10.3.3 Step 3: Design Inlet Zone

As outlined in Section 10.2.3, the inlet zone of a constructed stormwater wetland is designed as a sedimentation basin (refer Chapter 4) and serves two functions:

(1) Pre-treatment of inflow to remove coarse to medium sized sediment; and

(2) Hydrologic control of inflows into the macrophyte zone and bypass of floods during 'above design' operating conditions. The inlet zone consists of the following elements:

- Sedimentation basin 'pool' to capture coarse to medium sediment (125 μm or larger).
- Inlet zone connection to the macrophyte zone (or 'control' structure as defined in Chapter 4) normally consisting of an overflow pit within the inlet zone connected to one or more pipes through the embankment separating the inlet zone and the macrophyte zone.
- High flow bypass weir (or 'spillway' outlet structure) to deliver 'above design' flood flows to the high flow bypass channel.

For more information and design guidance for each of the inlet zone elements listed above, the reader is referred to Chapter 4 Sedimentation Basins. When applying the design procedure outlined in Chapter 4, the following should be used as a guide:

- The inlet zone typically must comprise a deep open water body (> 1.5 m) that operates essentially as a sedimentation basin designed to capture coarse to medium sized sediment (i.e. 125μm or larger).

- It may be necessary for a Gross Pollutant Trap (GPT) to be installed such that litter and large debris can be captured at the interface between the incoming waterway (or pipe) and the open water of the inlet zone. This is particularly necessary if the open water of the inlet zone also serves a landscape feature of high visibility.
- The crest of the overflow pit must be set at the permanent pool level of the inlet zone. It is common practice to set the permanent pool level in the inlet zone to be above the permanent water level of the downstream, receiving macrophyte zone to provide for unimpeded inflow to the macrophyte zone. As the macrophyte zone progressive become inundated over its extended detention depth, the overflow pit in the inlet zone will become submerged where downstream water levels will influence subsequent discharge rates into the wetland, ultimately causing the bypass operation to be activated.
- The overflow pit and connecting pipe between the inlet zone and macrophyte zone should be designed to convey the design operation flow (i.e. 1 year ARI peak discharge). Assuming inlet control operation:-
  - the dimension of the overflow pit (control structure) should be determined to ensure that adequate operating capacity as a weir or a submerged ‘glory hole’ or orifice with a water level corresponding to the crest of the by-pass spillway;
  - the pipe size that connects the inlet zone to the macrophyte zone is determined by assuming the macrophyte zone is at the permanent pool level and with upstream water level at the crest of the overflow pit.
- An energy dissipater is usually required at the end of the pipes to reduce velocities and distribute flows into the macrophyte zone.
- The inlet zone is to have a structural base (e.g. rock) to define the base when desilting and provide support for maintenance plant/ machinery when entering the basin for maintenance.
- The high flow bypass weir (‘spillway’ outlet) is to be set at the same level as the top of extended detention in the macrophyte zone.

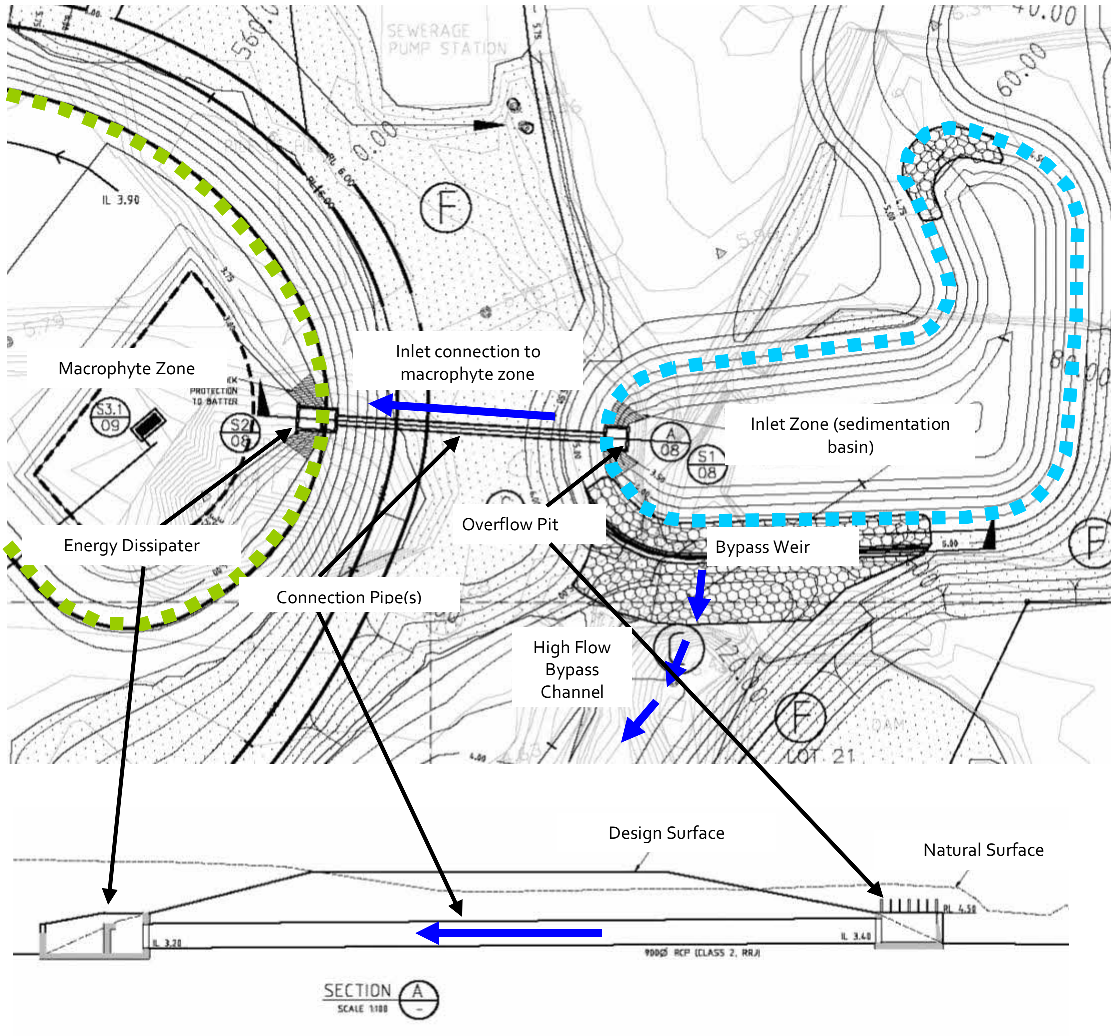


Figure 10.6 Typical inlet zone design for a constructed wetland

10.3.4 Step 4: Designing the Macrophyte Zone

10.3.4.1 Length to Width Ratio and Hydraulic Efficiency

To optimise wetland performance, it is important to avoid short circuit flow paths and poorly mixed regions within the macrophyte zone. One way to minimise this is to adopt a high length to width ratio not less than 5 to 1 for the macrophyte zone. Length to width ratios less than this can lead to poor hydrodynamic conditions and reduced water quality treatment performance.

The expected hydrodynamic characteristics can be defined by the hydraulic efficiency of the wetland, defined by Persson et al. (1999). The hydraulic efficiency is greatly influenced by the length to width ratio of the wetland, the relative position of the inlet and the outlet, and the inclusion and placement of any baffles, islands or flow spreaders. Hydraulic efficiency has a range from 0 to 1, with 1 representing the most efficient configuration for sedimentation. The design of the macrophyte zone of a constructed wetland should aim to have a hydraulic efficiency greater than 0.7. Engineers Australia (2006) recommend that constructed wetland systems should not have a hydraulic efficiency (λ) less than 0.5.

Guidance on estimating hydraulic efficiency is given in Figure 10.7. The shape designed as ‘o’ in diagrams O and P represent islands in the waterbody and the double line in diagram Q represents a weir structure to distribute flows evenly (Persson *et al*. 1999).

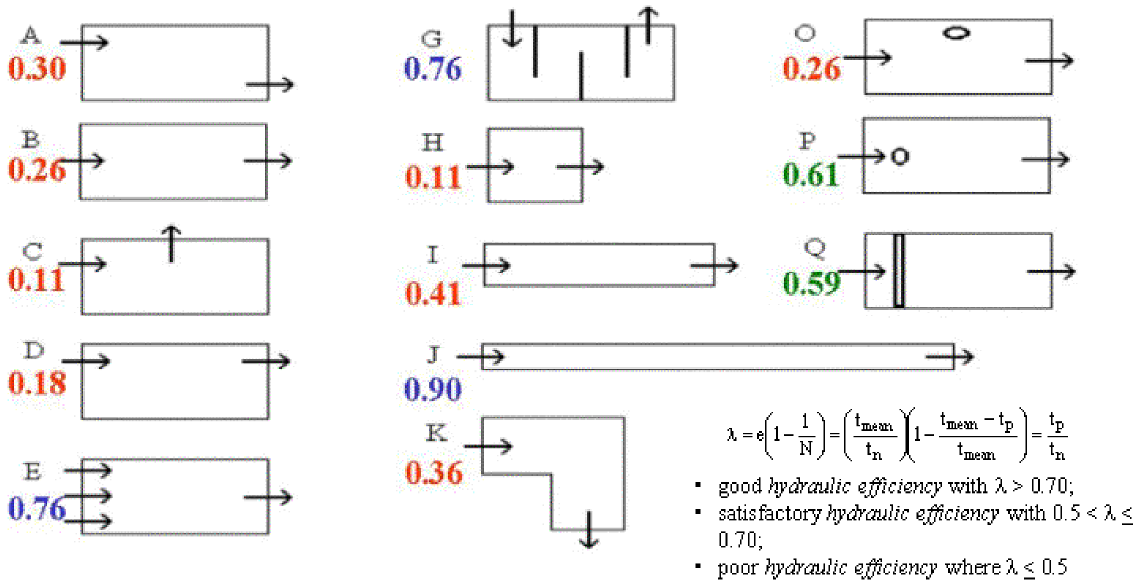


Figure 10.7 Hydraulic Efficiency, λ

10.3.4.2 Designing the Macrophyte Zone Bathymetry

It is good design practice to provide a range of habitat areas within the macrophyte zone to support a variety of plant species, ecological niches and perform a range of treatment processes. The macrophyte zone therefore typically comprises four marsh zones (defined by water depth) and an open water zone. The four marsh zones are ephemeral marsh, shallow marsh, marsh and deep marsh as depicted in Figure 10.8. The bathymetry across the four marsh zones is to vary gradually ranging from 0.2 m above the permanent pool level (i.e. ephemeral marsh) to a maximum of 0.5 m below the permanent pool level (i.e. deep marsh).

The relative proportion of each marsh zone will be dependent on the specific pollutant(s) being targeted by the wetland. For example, a wetland targeting phosphorus removal would typically have a higher proportion of ephemeral marsh zone where the frequent cycles of inundation and draining promotes the 'locking' of phosphorus onto the soil particles within the macrophyte zone substrate. Conversely, if nitrogen is the target pollutant, the macrophyte zone would typically have a higher proportion of marsh and deep marsh. The marsh and deep marsh zones facilitate nitrogen cycling within the aerobic and anaerobic substrate conditions as well as biological processing of soluble nitrogen from the water column by algal epiphytes and biofilms attached to the submerged part of the macrophytes in these zones.

The depth of the open water zones should be not less than 1m below the permanent pool level to avoid colonisation by emergent macrophytes and typically not more than 1.5m depth. Colonisation for submerged macrophytes should be discouraged as their decomposition raises the BOD and turbidity of the water..

To ensure optimal hydraulic efficiency of a wetland for a given shape and aspect ratio, wetland zones are arranged in bands running across (i.e. perpendicular to) the flow path (see Figure 10.9). The appropriate bathymetry, coupled with uniform plant establishment, ensures the macrophyte zone cross section has uniform hydraulic conveyance, thus reducing the risk of short circuiting.

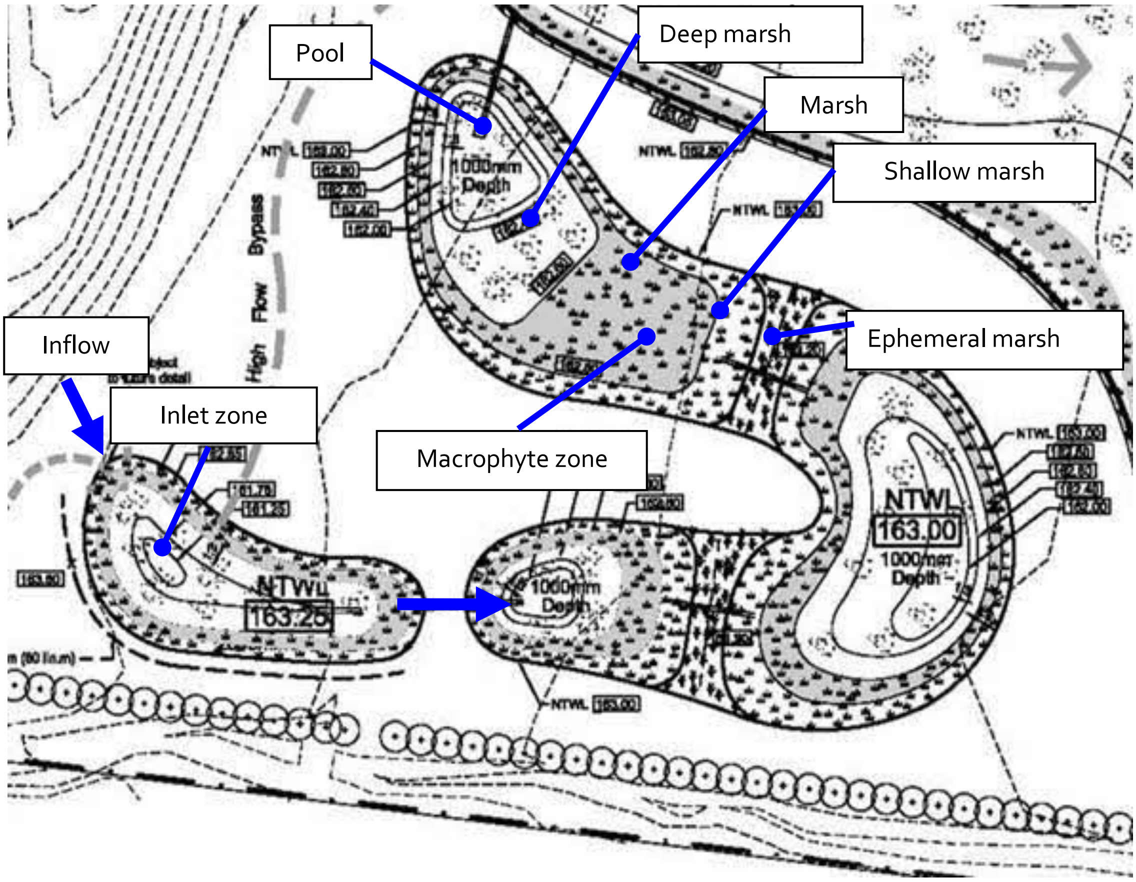


Figure 10.8 Example Bathymetry of a Constructed Wetland System (GBLA 2004)

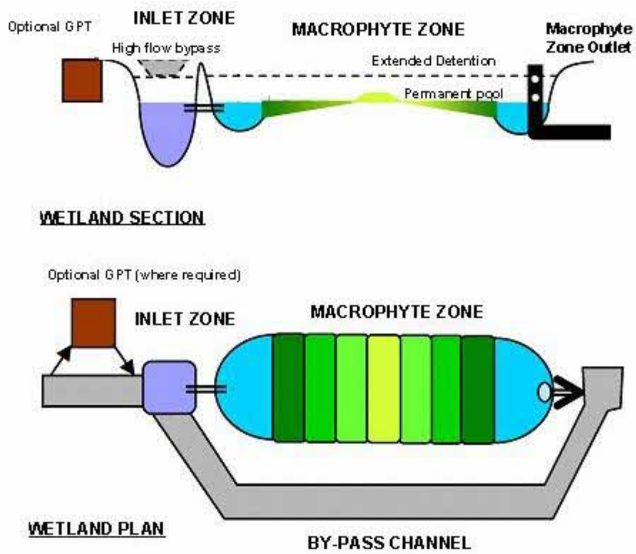


Figure 10.9 Schematic layout of a constructed wetland (see also Figure 10.2)

10.3.4.3 Macrophyte Zone Edge Design for Safety

The batter slopes on approaches and immediately under the permanent water level have to be configured with consideration of public safety (refer to Figure 10.10).

It is recommended that a gentle slope to the water edge and extending below the water line be adopted before the batter slope steepens into deeper areas. An alternative to the adoption of a flat batter slope is to provide a 3 m "safety bench" that is less than 0.2 m deep below the permanent pool level be built around the wetland.

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

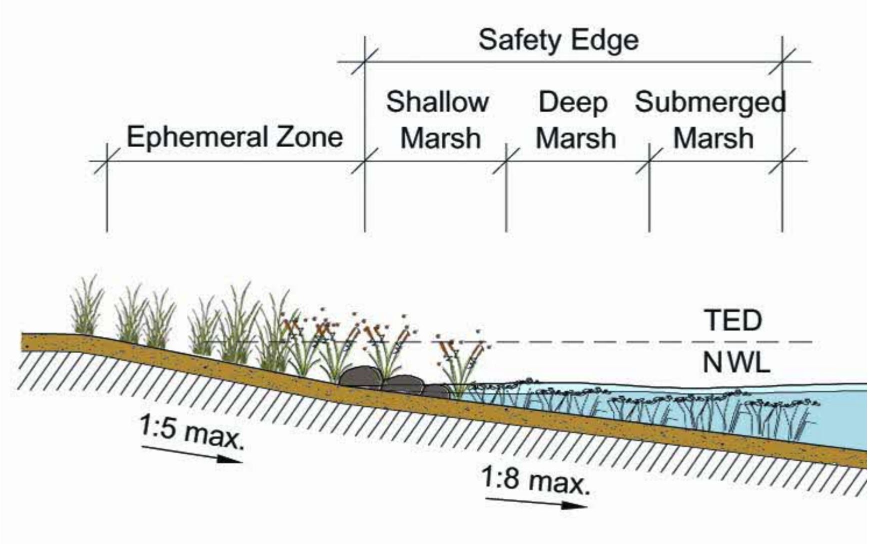


Figure 10.10 Example of edge design to a constructed wetland system

10.3.4.4 Macrophyte Zone Soil Testing

Constructed wetlands are permanent water bodies and therefore the soils in the base must be capable of retaining water. Geotechnical investigations of the suitability of the in-situ soils are required to establish the water holding capacity of the soils. Where the infiltration rates are too high for permanent water retention, tilling and compaction of in-situ soils may be sufficient to create a suitable base for the wetland. Where in-situ soils are unsuitable for water retention, a compacted clay liner may be required (e.g. 300 mm thick). Specialist geotechnical testing and advice must be sought.

10.3.5 Step 5: Design Macrophyte Zone Outlet

A macrophyte zone outlet has two purposes: (1) hydrologic control of the water level and flows in the macrophyte zone to achieve the design detention time; and (2) to allow the wetland permanent pool to be drained for maintenance.

10.3.5.1 Riser Outlet – Size and Location of Orifices

The riser is designed to provide a uniform notional detention time over the full range of the extended detention depth[2]. The target maximum discharge may be computed as the ratio of the volume of the extended detention to the notional detention time, i.e.

$$Q_{\text{max riser}} = \frac{\text{extended detention storage volume (m}^3\text{)}}{\text{notional detention time (s)}}$$ **Equation 10.1**

The placement of outlet orifices and determining their appropriate diameters is designed iteratively by varying outlet diameters and levels, using the orifice equation (Equation 10.2) 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.

$$A_o = \frac{Q_{\max\ riser}}{C_d\sqrt{2 \cdot g \cdot h}}$$ **Equation 10.2**

[2] It should be noted that detention time is never a constant and the term notional detention time is used to provide a point of reference in modelling and determining the design criteria for riser outlet structures.

As the outlet orifices can be expected to be small, it is important that the orifices are prevented from clogging by debris. Some form of debris guard is recommended as illustrated in the images below.

Figure 10.11 Example devices to prevent clogging of the riser

An alternative to using debris guard is to install the riser within a pit. The pit is connected to the permanent pool of the macrophyte zone via a submerged pipe culvert. The connection should be adequately sized such that there is minimal water level difference between the water within the pit and the water level in the macrophyte zone. With the water entering into the outlet pit being drawn from below the permanent pool level, floating debris are prevented from entering the outlet pit while heavier debris would normally settle onto the bottom of the permanent pool.

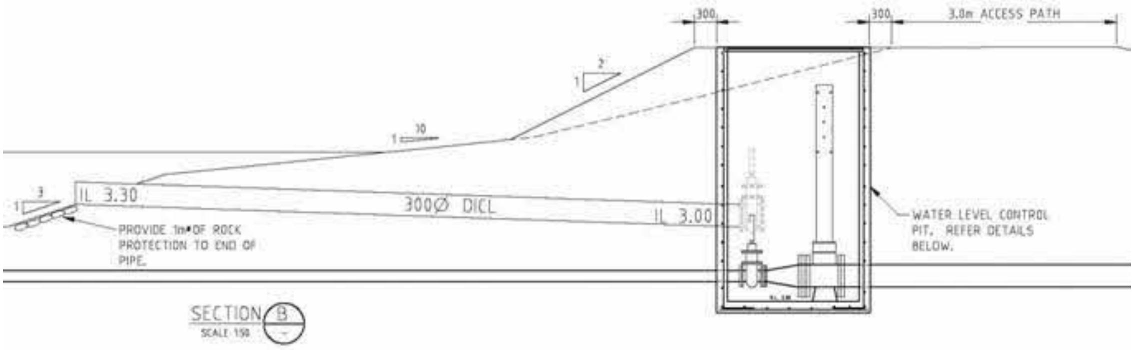


Figure 10.12 Typical macrophyte zone outlet arrangement

10.3.5.2 Maintenance Drain

To allow access for maintenance, the wetland should have appropriate allowance for draining. A maintenance drainage pipe should be provided that connects the low points in the macrophyte zone bathymetry to the macrophyte zone outlet. A valve is provided on the maintenance drainage pipe (typically located in the outlet pit), which can be operated manually. The maintenance drainage pipe should be sized to draw down the permanent pool within 12 hours (i.e. overnight). If a weir plate is used as a riser outlet, provision should be made to remove the weir plate and allow drainage for maintenance.

10.3.5.3 Discharge Pipe

The discharge pipe of the wetland conveys the outflow of the macrophyte zone to the receiving waters (or existing drainage infrastructure). The conveyance capacity of the discharge pipe is to be sized to match the higher of the two discharges, i.e. maximum discharge from the riser or the maximum discharge from the maintenance drain.

10.3.6 Step 6: Design High Flow Bypass Channel

To protect the integrity of the macrophyte zone of the wetland, it is necessary to consider the desired above-design operation of the wetland system. This is generally provided for with a high flow route that by-passes the macrophyte zone during flow conditions that may lead to scour and damage to the wetland vegetation. As outlined in Section 10.3.3, a function of the inlet zone is to provide hydrologic control of inflow into the macrophyte zone. A by-pass weir is to be included in the design of the inlet zone, together with a by-pass floodway (channel) to direct high flows around the macrophyte zone.

Ideally, the bypass weir level should be set at the top of the extended detention level in the macrophyte zone. This would ensure that a significant proportion of catchment inflow will bypass the macrophyte zone once it has reached its maximum operating extended detention level. The width of the spillway is to be sized to safely pass the maximum discharge conveyed into the inlet zone (as defined in Section 10.3.3) with the maximum water level above the crest of the weir to be defined by the top of embankment level (plus a suitable freeboard provision).

Figure 10.13 Examples of high flow bypass systems

10.3.7 Step 7: Verify Design

10.3.7.1 Macrophyte Zone Re-suspension Protection

The principle pathway for biological uptake of soluble nutrients in wetlands is through biofilms (epiphytes) attached to the surface of the macrophyte vegetation. The biofilms, being mostly algae and bacteria, are susceptible to wash out under high flow conditions. Further, wetland surveys indicate that up to 90% of the total nutrients are stored in the sediments, therefore, the key to effective retention of pollutants is managing high velocity flows that could potentially resuspend and remobilise these stored pollutants.

A velocity check is to be conducted for design conditions, when the wetland water level is at the top of the extended detention level and the riser is operating at design capacity, to ensure velocities are less than 0.05 m/s through all zones of the wetland. The following condition must be met:

$$\frac{Q_{\text{max riser}}}{A_{\text{section}}} < 0.05\text{m/s}$$ **Equation 10.3**

Where

$Q_{\text{max riser}}$ = target maximum discharge (defined in Equation 10.1) (m^3/s)

A_{section} = wetland cross sectional area at narrowest point*, measured from top of extended detention (m^2)

Minimum wetland cross-section is used when undertaking this velocity check

10.3.7.2 Confirm Treatment Performance

If the basic wetland parameters established by the conceptual design phase have changed during the course of undertaking detailed design (e.g. macrophyte zone area, extended detention depth, etc.) then the designer should verify that the current design meets the required water quality improvement performance. This can be done by simulating the current design using MUSIC.

10.3.8 Step 8: Specify Vegetation

Vegetation planted in the macrophyte zone (i.e. marsh and pool areas) is designed to treat stormwater flows, as well as add aesthetic value. Dense planting of the littoral berm zone will inhibit public access to the macrophyte zone, minimising potential damage to the plants and the safety risks posed by water bodies. Terrestrial planting may also be recommended to screen areas and provide an access barrier to uncontrolled areas of the stormwater treatment system.

Plant species for the wetland area will be selected based on the water regime, microclimate and soil types of the region, and the life histories, physiological and structural characteristics, natural distribution, and community groups of the wetland plants. The reader is referred the National parks Board of Singapore for a list of suggested plant species suitable for constructed wetland systems in Singapore and recommended planting densities. The distribution of the species within the wetland will relate to their structure, function, relationship and compatibility with other species. Planting densities should ensure that 70-80% cover is achieved after two growing seasons (2 years) will be recommended.

10.3.9 Step 9: Consider Maintenance Requirements

A specific maintenance plan and schedule should be developed for the wetland, either as part of a maintenance plan for the whole treatment train, or for each individual asset. Guidance on maintenance plans is provided in Section 10.6. The maintenance plan should consider how maintenance is to be performed on the wetland, for example, where and how the wetland can be accessed and where litter is likely to collect.

10.3.10 Design Calculation Summary

Following is a design calculation summary sheet for the key design elements.

	Constructed Wetland	**CALCULATION SUMMARY**		
	CALCULATION TASK	OUTCOME		CHECK
	Catchment characteristics			
	- Land Uses			
	Residential		Ha	
	Commercial		Ha	
	Roads /pavements		Ha	
	- Fraction Impervious			
	Residential			
	Commercial			
	Roads / pavements			
	Weighted average			
	- Site Slope		m	
	Conceptual Design			
	Macropyte Area		m^2	
	Aspect Ratio (macrophyte zone)		(W)	
			(L)	
	Permanent pool level of macrophyte zone		m AHD	
	Extended detention depth (0.25-0.5m)		m	
	Notional detention time or hydraulic retention time (HRT)		hrs	
	Identify design criteria			
	Design ARI Flow for Inlet Zone		year	
	Target Sediment Size for Inlet Zone		mm	
	Design ARI Flow for Bypass Spillway		year	
	Extended Detention Volume		m^3	
1	**Confirm treatment size of conceptual design**			
	TSS load reduction		%	
	TP load reduction		%	
	TN load reduction		%	
2	**Estimate design flow rates**			
	Time of concentration			
	Estimate from flow path length and velocities		minutes	
	Identify rainfall intensities			
	1 year ARI		mm/hr	
	100 year ARI		mm/hr	
	Design runoff coefficient			
	Catchment use			
	C			
	Peak design flows			
	Minor Storm (selected design storm ARI and flow)	ARI	m3/s	
	Major Storm (selected design storm ARI and flow)	ARI	m3/s	
3	**Inlet zone**			
	(refer to sedimentation basin calculation checksheet)			

	Is a GPT required?		
	Suitable GPT selected and maintenance considered?		☐
	Inlet zone size		
	Target Sediment Size for Inlet Zone	µm	
	Capture efficiency	%	
	Inlet zone area	m^2	
	$V_s > V_{s:5yr}$		☐
	Inlet zone connection to macrophyte zone		
	Overflow pit crest level	m AHD	
	Overflow pit dimension	L x W	
	Discharge capacity of overflow pit	m^3/s	
	Provision of debris trap		☐
	Connection pipe dimension	mm diam	
	Connection pipe invert level	m AHD	☐
	High flow by-pass weir		
	Weir Length	m	
	High flow by-pass weir crest level (top of extended detention)	m AHD	☐
4	**Macrophyte Zone Layout**		
	Area of Macrophyte Zone	m^2	
	Aspect Ratio	L:W	
	Hydraulic Efficiency		☐
5	**Macrophyte Zone Outlet** **Riser outlet**		
	Target maximum discharge (Q_{max})	m^3/s	
	Uniform Detention Time Relationship for Riser		☐
	Maintenance drainage rate (drain over 12hrs)	m^3/s	
	Diameter of maintenance drain pipe	mm	
	Diameter of maintenance drain valve	mm	☐
	Discharge Pipe		
	Diameter of discharge pipe	mm	☐
6	**High flow bypass channel**		
	Discharge Capacity of Bypass Weir	m^3/s	
	Longitudinal slope	%	
	Base width	m	
	Batter slopes	H:V	☐
7	**Verification checks**		
	Macrophyte zone re-suspension protection		☐

10.4 Checking tools

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

Checklists are provided for:

- Design assessments
- Construction (during and post)
- Maintenance and inspections

10.4.1 Design assessment checklist

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

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

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

WETLAND DESIGN ASSESSMENT CHECKLIST		
Wetland Location:		
Hydraulics:	**Design operational flow (m^3/s):**	**Above design flow (m^3/s):**
Area:	**Catchment Area (ha):**	**Wetland Area (ha):**

TREATMENT	**Y**	**N**
MUSIC modelling performed?		
INLET ZONE	**Y**	**N**
Discharge pipe/structure to inlet zone sufficient for maximum design flow?		
Scour protection provided at inlet for inflow velocities?		
Configuration of inlet zone (aspect, depth and flows) allows settling of particles >125µm?		
Bypass weir incorporated into inlet zone?		
Bypass weir length sufficient to convey 'above design flow' ?		
Bypass weir crest at macrophyte zone top of extended detention depth?		
Bypass channel has sufficient capacity to convey 'above design flow'?		
Bypass channel has sufficient scour protection for design velocities?		
Inlet zone connection to macrophyte zone overflow pit and connection pipe sized to convey the design operation flow?		
Inlet zone connection to macrophyte zone allows energy dissipation?		
Structure from inlet zone to macrophyte zone enables isolation of the macrophyte zone for maintenance?		
Inlet zone permanent pool level above macrophyte permanent pool level?		
Maintenance access allowed for into base of inlet zone?		
Public safety design considerations included in inlet zone design?		
Where required, gross pollutant protection measures provided on inlet structures (both inflows and to macrophyte zone)		
MACROPHYTE ZONE	**Y**	**N**
Extended detention depth >0.25m and <0.5m?		
Vegetation bands perpendicular to flow path?		
Appropriate range of macrophyte vegetation (ephemeral, shallow, marsh, deep marsh)?		
Sequencing of vegetation bands provides continuous gradient to open water zones?		
Vegetation appropriate to selected band?		
Aspect ratio provides hydraulic efficiency =>0.5?		
Velocities from inlet zone <0.05 m/s or scouring protection provided?		
Public safety design considerations included in macrophyte zone (i.e. batter slopes less than 5(H):1(V)?		
Maintenance access provided into areas of the macrophyte zone (especially open water zones)?		
Safety audit of publicly accessible areas undertaken?		
Freeboard provided above extended detention depth to define embankments?		
OUTLET STRUCTURES	**Y**	**N**
Riser outlet provided in macrophyte zone?		
Notional detention time of 48-72 hours?		
Orifice configuration allows for a linear storage-discharge relationship for full range of the extended detention depth?		
Maintenance drain provided?		
Discharge pipe has sufficient capacity to convey maximum of either the maintenance drain flows or riser pipe flows with scour protection?		
Protection against clogging of orifice provided on outlet structure?		
COMMENTS		

10.5 Construction advice

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

10.5.1.1 Protection from existing flows

It is important to have protection from upstream flows during construction of a wetland system. A mechanism to divert flows around a construction site, protect from litter and debris is required. This can be achieved by constructing a high flow bypass channel initially and then diverting all inflows along the channel until the wetland system is complete.

During building construction, it is recommended that the inlet zone form a sedimentation basin reducing the load of coarse sediment discharging to the macrophyte zone (Leinster, 2006). The disconnection between the inlet and macrophyte zone should remain in place to ensure the majority of flows from the catchment continue to bypass the macrophyte zone thus allowing the wetland plants to reach full maturity without the risk of being smothered with coarse sediment. At the completion of all building activity the inlet zone is de-silted, the disconnection between the inlet zone and macrohpyte zone is removed and the constructed wetland allowed to operate in accordance with the design.

10.5.1.2 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.

10.5.1.3 Erosion control

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

10.5.1.4 Inlet erosion checks

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

10.5.1.5 Tolerances

It is importance to stress that particular attention be placed on ensuring that construction tolerances of key wetlands features (e.g base, longitudinal and batters) are kept to a minimum. The relative levels of the control structure are particularly important in achieving the required hydraulic performance. It is also important to ensure that as water levels reduce (e.g. for maintenance) that areas drain back into designated pools with distributed shallow pools across the wetland to be avoided. Generally plus or minus 5 mm is acceptable.

The bathymetry of the macrophyte zone must be free from localised depressions and low points resulting from earthworks. This is particularly important to achieve a well distributed flow path and to prevent isolated pools from forming (potentially creating mosquito habitat) when the wetland drains. Generally an earthworks tolerance of plus or minus 25 mm is considered acceptable.

10.5.1.6 Transitions

It is important to pay attention to the detail of earthworks to ensure smooth transitions between benches and batter slopes. This will allow for strong edge vegetation to establish and avoid local ponding (that can enhance mosquito breeding habitat).

10.5.1.7 Inlet zone access

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

10.5.1.8 Inlet zone base

To aid maintenance it is recommended to construct the inlet zone with a hard (i.e rock) bottom. This is important if maintenance of the wetland requires driving into the basin. It also serves an important role for determining the levels that excavation should extend to (i.e. how deep to dig) for either systems cleaned from the banks or directly accessed.

10.5.1.9 Dewatering collected 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.

10.5.1.10 Timing for planting

Timing of vegetation planting is dependent on a suitable time of year (and potential irrigation requirements) as well as timing in relation to the phases of development. Temporary sediment controls should always be used prior to planting as lead times from earthworks to planting are often long.

10.5.1.11 Vegetation establishment

During the establishment phase water levels should be controlled carefully to prevent seedlings from being desiccated or drowned. This is best achieved with the use of maintenance drains. Once plants are established water levels can be raised to operational levels.

10.5.1.12 Bird protection

Protection from bird feeding on newly planted vegetation (e.g. nets) should be considered in consultant with the National Parks Board of Singapore.

10.5.2 Construction Inspection Checklist

The following checklist presents the key items to be reviewed when inspecting the constructed wetland system during and at the completion of construction. The checklist is to be used by Construction Site Supervisors and local authority Compliance Inspectors to ensure all the elements of the constructed wetland have been constructed in accordance with the design. If an item is ticked as unsatisfactory appropriate actions must be specified and delivered to rectify the construction issue before final inspection sign-off is given.

WETLAND CONSTRUCTION INSPECTION CHECKLIST			
Site:		**Date:**	
		Time:	
Constructed by:		**Weather:**	
		Contact During Visit:	

Items inspected	**Checked**		**Adequate**		**Items inspected**	**Checked**		**Adequate**	
	Y	**N**	**Y**	**N**		**Y**	**N**	**Y**	**N**
DURING CONSTRUCTION									
Preliminary Works					**Structural components cont**				
1. Erosion and sediment control plan adopted					22. Ensure spillway is level				
2. Limit public access					23. Provision of maintenance drain(s)				
3. Location same as plans					24. Collar installed on pipes				
4. Site protection from existing flows					25. Low flow channel is adequate				
5. All required permits in place					26. Protection of riser from debris				
Earthworks					27. Bypass channel stabilised				
6. Integrity of banks					28. Erosion protection at macrophyte outlet				
7. Batter slopes as plans					**Vegetation**				
8. Impermeable (eg. clay) base installed					29. Vegetation appropriate to zone (depth)				
9. Maintenance access to whole wetland					30. Weed removal prior to planting				
10. Compaction process as designed					31.Provision for water level control				
11. Placement of adequate topsoil					32. Vegetation layout and densities as designed				
12. Levels as designed for base, benches, banks and spillway (including freeboard)					33. Provision for bird protection				
13. Check for groundwater intrusion					34. By-pass channel vegetated				
14. Stabilisation with sterile grass					**Erosion and Sediment Control**				
Structural components					35. Disconnect inlet zone from macrophyte zone (flows via high flow bypass)				
15. Location and levels of outlet as designed					36. Inlet zone to be used as sediment basin during construction				
16. Safety protection provided					37. Stabilisation immediately following earthworks and planting of terrestrial landscape around basin				
17. Pipe joints and connections as designed					38. Silt fences and traffic control in place				
18. Concrete and reinforcement as designed					39. Inlet zone desilted prior to wetland online				
19. Inlets appropriately installed					40. Inlet zone disconnection removed				
20. Inlet energy dissipation installed									
21. No seepage through banks									

FINAL INSPECTION									
1. Confirm levels of inlets and outlets					8. Public safety adequate				
2. Confirm structural element sizes					9. Check for uneven settling of banks				
3. Check batter slopes					10. Evidence of stagnant water, short circuiting or vegetation scouring				
4. Vegetation planting as designed					11. Evidence of litter or excessive debris				
5. Erosion protection measures working					12. Provision of removed sediment drainage area				
6. Pre-treatment installed and operational					13. Evidence of debris in high flow bypass				
7. Maintenance access provided					14. Macrophyte outlet free of debris				

COMMENTS ON INSPECTION

ACTIONS REQUIRED
Inspection officer signature:

10.6 Maintenance requirements

Wetlands treat runoff by filtering it through vegetation, the action of microorganisms in the substrate and providing extended detention to allow sedimentation to occur. In addition, they have a flow management role that needs to be maintained to ensure adequate flood protection for local properties and protection of the wetland ecosystem.

Maintaining healthy and vibrant vegetation and adequate flow conditions in a wetland are the key maintenance considerations. Weeding, planting and debris removal are the dominant tasks. In addition the wetland needs to be protected from high loads of sediment and debris and the inlet zone needs to be maintained in the same way as sedimentation basins (see Chapter 4).

The most intensive period of maintenance is during the plant establishment period when weed removal and replanting may be required. Manual weed removal and terrestrial weed removal can be achioeved by flooding the macrophyte zone.

Other components of the system that will require careful consideration are the inlet points. Inlets can be prone to scour and build up of litter. Occasional litter removal and potential replanting may be required as part of maintaining an inlet zone.

Maintenance is primarily concerned with:

- Maintenance of flow to and through the system
- Maintaining vegetation and remove plant litter
- 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
- Checking plant health and presence of pests (eggs, larvae and adults) and disease
- Re-establishment of plants that die

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

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

10.6.1 Operation & maintenance inspection form

The form below should be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time. Inspections should occur weekly to monthly. Please refer to maintenance checklist for details. More detailed site specific maintenance schedules should be developed for major wetland systems and include a brief overview of the operation of the system and key aspects to be checked during each inspection. An example is presented as part of the worked example in Section 10.7.

WETLAND MAINTENANCE CHECKLIST				
Inspection Frequency:	Weekly to monthly	**Date of Visit:**		
Location:				
Description together with photographic record of wetland components:				
Site Visit by:				
INSPECTION ITEMS	**FREQUENCY**	**Y**	**N**	**ACTION REQUIRED (DETAILS)**
Sediment accumulation at inflow points?	monthly			
Litter (non-plant) within inlet or macrophyte zones?	Weekly			
Plant litter within inlet or macrophyte zones?	Weekly			
Sediment within inlet zone requires removal (record depth, remove if >50%)?	Monthly			
Evidence of dumping (building waste, oils etc)?	Weekly			
Outlet structure free of debris?	Weekly			
Maintenance drain operational (check)?	Weekly			
Overflow structure integrity satisfactory?	Monthly			
Terrestrial vegetation condition satisfactory (density, weeds, health, evidence of chlorosis, disease or pest)?	Monthly			
Aquatic vegetation condition satisfactory (density, weeds, health, evidence of chlorosis, disease or pest)?	Monthly			
Removal of diseased, pest infested or dead plants and replanting required?	Monthly			
Settling or erosion of bunds/batters present?	Monthly			
Evidence of isolated shallow ponding?	Monthly			
Damage/vandalism to structures present?	Monthly			
Resetting of system required?	Monthly			
COMMENTS		Monthly		

10.7 Worked example

10.7.1 Worked example introduction

As part of a residential development in Singapore, stormwater runoff is to be conveyed to a constructed wetland for water quality treatment. An illustration of the site and proposed layout of the wetland is shown in the figure below. This worked example describes the design process for each component of the constructed wetland: inlet zone (including the bypass weir), macrophyte zone, macrophyte zone outlet and high flow bypass channel.

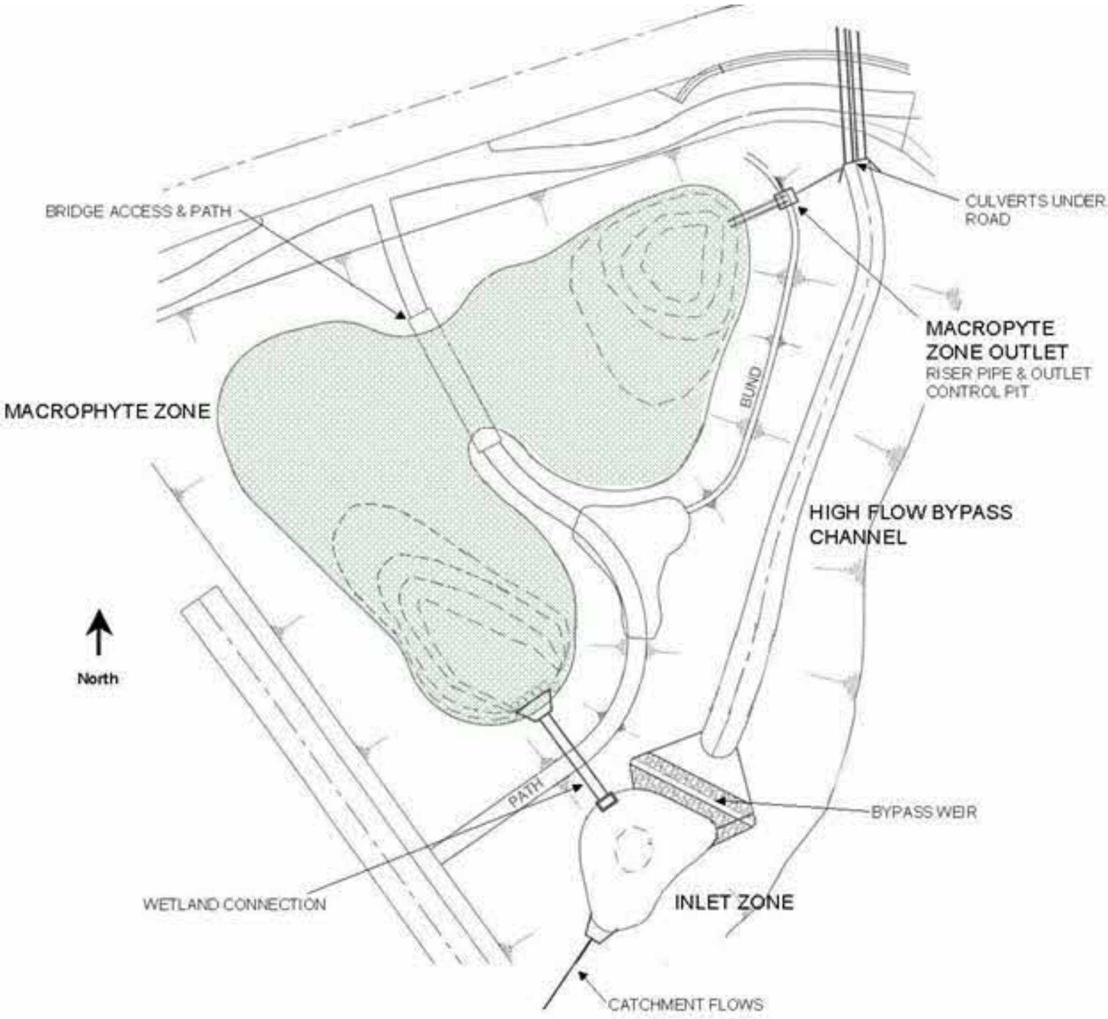


Figure 10.14 Layout of Proposed Wetland System

<u>Catchment and site description</u>

The contributing catchment area of the proposed wetland is 10ha. The catchment is densely developed with residential and industrial developments.

The site has a moderate fall of 2.5 m from south to north and is constrained by roads to the west and north and by steeper grades to the east. Soils through the site have been classified as clay. Stormwater runoff is collected and conveyed to the wetland inlet zone via conventional piped drainage with minor storm (i.e. the 5 year ARI event) flows discharged to the wetland inlet zone via a 975 mm diameter pipe and major storm (100yr ARI) entering via overland flow.

<u>Design Objectives</u>

The design objectives for the wetland system are to:

- Promote sedimentation of particles larger than 125μm within the inlet zone.
- Optimise the relationship between detention time, wetland volume and the hydrologic effectiveness of the system to maximise treatment given the wetland volume site constraints.
- Ensure that the required detention period is achieved for all flow though the wetland system through the incorporation of a riser outlet system.
- Provide for by-pass operation when the inundation of the macrophyte zone reaches the design maximum extended detention depth.

Concept Design Criteria

The conceptual design of the constructed wetland (as shown in Figure 10.14) established the following key design elements to ensure effective operation:

- Wetland macrophyte zone extended detention depth of 0.5m, permanent pool level of 11.5m and an area of 15,000 m^2 (equivalent to 15% of the catchment area)
- Inlet zone permanent pool level of 11.7m, which is 0.2m above the permanent pool level of the macrophyte zone
- Bypass weir ('spillway' outlet) level of 12.0m set at the top of extended detention in the wetland macrophyte zone and 0.3 m above the inlet zone permanent pool level
- High flow bypass channel longitudinal grade of 1.5%.

Landscape Requirements

In addition, a landscape design will be required and they include:

- Macrophyte zone vegetation (including edge vegetation)
- Terrestrial vegetation.

10.7.2 Calculation Steps

The design of a constructed wetland system has been divided into the following 7 calculations steps:

Step 1 Confirm treatment size
Step 2 Estimate design flows
Step 3 Design inlet zone
Step 4 Macrophyte zone
Step 5 Macrophyte zone outlet
Step 6 Design High Flow Bypass Channel
Step 7 Verification Checks

Details for each calculation step are provided below. A design calculation summary has been completed for the worked example and is given at the conclusion of the calculation steps.

Step 1 Confirm treatment size

As a basic check of the adequacy of the size of the wetland, reference is made to the performance curves presented in Figure 10.3 to Figure 10.5. A macrophyte area of 1.5ha (equivalent to 15% of the catchment area) provides a pollutant load reduction of 78%, 72% and 50% reduction of TSS, TP and TN respectively from mean annual loads typically generated from an urban catchment.

Step 2 Estimate design flows

The site has a contributing catchment of 10ha which is drained via conventional pipe drainage. Both the minor storm (5yr ARI) and the major storm (100yr ARI) flows enter the inlet zone of the wetland. Therefore, the 'above design flow' is set to correspond to the 100year ARI peak flow. The 'design operation flow', which is required to size the inlet zone and the inlet zone connection to the macrophyte zone, is set to correspond to the 1year ARI peak flow.

Design flows are established using the Rational Method, as given in the Code of Practice on Surface Water Drainage (PUB, 2000). The rational method is given by

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

Where

C	=	Runoff coefficient
I	=	Rainfall intensity (mm/hr)
A_c	=	Catchment area (ha)

a. time of concentration

The time of concentration of the catchment was determined to be 10min.

b. Runoff coefficients

The Code of Practice on Surface Water Drainage describes runoff coefficients based on the degree and type of development within the catchment. The catchment for the worked example is densely developed with residential and industrial developments. The corresponding runoff coefficient is 0.8.

c. Rainfall Intensities

The rainfall intensity at a time of concentration of 10minutes for the 1yr ARI and 100yr ARI event are determined from the IDF curve for Singapore contained in Appendix 2 of the Code of Practice on Surface Water Drainage (2000), i.e.

I_1 = 106 mm/hr $\qquad$ I_{100} = 271 mm/hr

d. Design flows

Applying the Rational Method for the above parameters and a catchment area of 10ha gives the following design flows:

Q_1 = 2.3 m^3/s $\qquad$ Q_{100} = 6.0 m^3/s

Step 3 Design inlet zone

The design of the inlet zone is undertaken in accordance with the design procedures outlined in Chapter 4 (Sedimentation Basin). A summary of the key inlet zone elements is provided below.

<u>Inlet Zone (Sedimentation Basin) Size</u>

a. Sediment Basin Area

An initial estimate of the inlet zone area can be established using the curves given in Figure 4.3. For a peak 1 year ARI flow of 2.3 m^3/s, a basin area of 520 m^2 is required to capture 90% of the 125μm particles for flows up to the 'design operation flow' (1 year ARI = 2.3 m^3/s). This area represents approximately 0.25% of the site area.

A more detailed design procedure for a sedimentation basin is contained in Chapter 4 of this document.

b. Clean-out frequency

The inlet zone (sedimentation basin) should have adequate storage to ensure desilting is not more frequent than once a year. Desilting is required when sediment storage reaches half the volume of the permanent pool volume of the basin.

The design depth of the permanent pool is adopted to be 1.5 m. Adopting a batter slope of 1:3 below the water line, the sedimentation volume available for storage is 544 m^3. Thus, clean out of the sedimentation basin is required when sediment deposition volume exceeds 272 m^3.

Assuming a sediment loading rate of 3.0 m^3/ha/yr and a capture efficiency of 90%, the cleanout frequency I computed to be:-

Cleanout frequency (years) = $\frac{272}{3 \times 10 \times 0.9} = 10$ years OK

Inlet Zone Connection to Macrophyte Zone

The hydraulic structure connecting the inlet zone to the macrophyte zone consists of an overflow pit (in the inlet zone) and a connection pipe with the capacity to convey the 'design operation flow' (1yr ARI = 2.3 m^3/s). The conceptual design defined the following design elements:

- Inlet zone permanent pool level (overflow pit crest level) = 11.7m which is 0.2m above the permanent pool level of the macrophyte zone
- Bypass weir ('spillway' outlet) crest level = 12m which is the top of extended detention for the wetland and 0.3 m above the inlet zone permanent pool level.

It is common practice to allow for 0.3m of freeboard above the afflux level when setting the top of embankment elevation.

Overflow Pit

Two possible flow conditions need to be checked for overflow conditions: weir flow conditions (with extended detention of 0.3 m) and orifice flow conditions.

a. Weir Flow Conditions

The required perimeter of the outlet pit to pass the 1yr ARI flow (2.3 m^3/s) with an afflux of 0.3 m can be calculated using the following equation assuming 50% blockage:

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

$$P = \frac{2.3}{0.5 \times 1.7 \times 0.3^{3/2}}$$

$$P = 16.5m$$

The equivalent area, assuming the pit is square, is 18m^2.

b. Orifice Flow Conditions

The required area of the outlet pit can be calculated as follows:

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

$$A_o = \frac{2.3}{0.5 \times 0.6 \times \sqrt{2 \times 9.81 \times 0.3}}$$

$$A_o = 3.2m^2$$

In this case the weir flow condition is limiting. Considering the overflow pit is to convey the 'design operation flow' (1yr ARI) or slightly greater, a minimum pit of 1.8m x 1.8m will be required (area 3.2 m^2). The top of the pit is to be fitted with a letter box grate. This will ensure large debris does not enter the 'control' structure while avoiding the likely of blockage of the grate by smaller debris.

c. Connection Pipe(s)

As the connection pipe (i.e. between the inlet zone and the macrophyte zone) is to be submerged, the size can be determined by first estimating the required velocity in the connection pipe using the following:

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

Where

h	=	head level driving flow through the pipe (defined as the 'spillway' outlet level minus the normal water level in the downstream treatment system)
	=	12m – 11.5m = 0.5m
g	=	gravity (9.81 m/s^2)

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

Back calculating gives a velocity of 2.2m/s in the connection pipe. The pipe size required to carry the 1yr ARI design flow of 2.3 m^3/s is hence 1.1m^2. It is recommended that multiple pipes be used to connect the inlet and the macrophyte zones. Three pipes of 750 mm diameter will be required.

The obvert of the pipes is to be set below the permanent water level in the wetland macrophyte zone (11.5 m) meaning the invert is set at 10.80 m. The dimension of the over pit to accommodate the pipe connection is thus 3 m by 1.5 m.

In summary, the control outlet structure will be an overflow pit, 3m by 1.5m with the crest level at RL 11.5m and a raised grated cover set at RL 11.6m. The outlet/connection pipe to the wetland will be three 750mm in diameter pipes with their inverts set at RL 10.8m.

High Flow Bypass Weir

All flows in excess of the 'design operation flow' and up to the 'above design flow' are to bypass the wetland macrophyte zone. This is facilitated by a high flow bypass weir ('spillway' outlet) designed to convey the 'above design flow' (100yr ARI) with the weir crest level 0.3 m above the permanent pool of the inlet pond (RL 12.0m).

Assuming a maximum afflux of 0.3 m, the weir length is calculated using the weir flow equation

$$L = \frac{Q_{100yr}}{C_w \cdot h^{3/2}}$$

$$L = \frac{6}{1.7 \times 0.3^{\frac{3}{2}}}$$

$$L = 22m$$

To ensure no flows breach the embankment separating the inlet zone and the macrophyte zone the embankment crest level is to be set at 12.6 m AHD (i.e. 0.3 m freeboard on top of the maximum afflux level over the high flow bypass weir).

Summary of inlet zone dimensions

The dimensions for the sedimentation basin are summarised below.

Open water area	=	520m^2
Basin width	=	45m
Basin length	=	12m
Depth of permanent pool	=	1.5m
Overflow pit	=	3.0m x 1.5m with grate set at RL 11.7 m
Pipe connection (to wetland)	=	3 x 750mm RCPs at RL 10.8m
High flow bypass weir	=	22m length at RL 12.0m

Step 4 Macrophyte zone

Length to width ratio and hydraulic efficiency

The hydraulic efficiency describes the effectiveness of the basin to retain sediment. A reasonable estimate of the hydraulic efficiency can be made based on the length to width ratio of the basin, the relative position of entry and exit points and any flow diverting systems (e.g. baffles).

The concept design (Figure 10.14) describes the macrophyte zone as L-shaped and an area of 15,000m^2, and a width to length ratio of 1:6. “Case K” (λ= 0.37) shown in (Figure 10.7) has a L-shaped layout but with an equivalent aspect ratio of 1:3. “Case G” (λ= 0.76) has an equivalent aspect ratio of 1:8 with a sinuous shape. With a length to width ratio of 1:6, and an L-shaped configuration, a hydraulic efficiency of 0.6 is considered reasonable.

Designing the macrophyte zone bathymetry

The macrophyte zone of the wetland is divided into four marsh zones and an open water zone as described below:

- The bathymetry across the four marsh zones is to vary gradually over the length of the macrophyte zone, ranging from 0.2 m above the permanent pool level (ephemeral zone) to 0.5 m below the permanent pool level (Table 10.2). The ephemeral marsh zone is to be located adjacent to the pathway and bridge crossing mid way along the wetland.
- The permanent pools upstream and downstream of the ephemeral zone are to be connected via the maintenance drain to ensure the upstream permanent pool can drain down to RL11.5 m following a rainfall event.
- The depth of the open water zone in the vicinity of the outlet structure is to be 1 m below the permanent pool level.
- The marsh zones are arranged in bands of equal depth running across the flow path to optimise hydraulic efficiency and reduce the risk of short-circuiting.

Table 10.2: Indicative Break of Marsh Zones

Zone	Depth Range (m)	Proportion of Macrophyte Zone Surface Area (m)
Open Water (Pool)	>1.0 below permanent pool	10%
Transition	0.5 – 1.0 below permanent pool	10%
Deep Marsh	0.35 – 0.5 below permanent pool	20%
Marsh	0.2 – 0.35 below permanent pool	20%
Shallow Marsh	0.0 – 0.2 below permanent pool	20%
Ephemeral Marsh	0.2 – 0.0 above permanent pool	20%

Macrophyte Zone Edge Design for Safety

The batter slopes on approaches and immediately under the permanent water level have to been configured with consideration of public safety:

- Generally, batter slopes of 1(V):8(H) from the top of the extended detention depth to 0.3 m beneath the water line has been adopted.
- The general grade through the wetland below the waterline is 1(V):8(H) or flatter.
- The batters directly adjacent and within the open water zones of the macrophyte are limited to 1(V):8(H).

Step 5 Macrophyte zone outlet

Riser Outlet – Size and Location of Orifices

The target maximum discharge may be computed as the ratio of the volume of the extended detention to the notional detention time, i.e.

$$Q_{max\ riser} = \frac{\text{extended detention storage volume } (m^3)}{\text{notional detention time (s)}}$$

The extended detention storage volume is approximated as 7,500m^3 (given a surface area of 15,000m^2 and an extended detention depth of 0.5m). The wetland will be designed for a notional detention time of 72hrs. Hence the maximum discharge from the riser is 0.03m^3/s.

The placement of outlet orifices and determining their appropriate diameters is designed iteratively by varying outlet diameters and levels, using the orifice equation (Equation 10.2) applied over discrete depths along the length of a riser up to the maximum detention depth. The outlet diameters and positioning are varied to ensure a 72hr nominal detention time at each outlet position.

The final iteration is presented in the excel spreadsheet below. The resulting orifice configuration is described schematically in Figure 10.15. Note that the riser pipe has no role in managing the flows greater than 0.03m^3/s ($Q_{max,\ riser}$). For flows above the extended detention depth the high flow bypass is activated. A notional upstand riser pipe diameter of 225 mm is selected.

Orifice Position (invert level)			0	0.125	0.25	0.375	m (above 11.5m)	
Orifice diameter			40	40	30	30	(mm)	
Number of orifices			6	4	5	3	(-)	
Area/orifice			1.3E-03 ($1.3 \cdot 10^{-3}$)	1.3E-03 ($1.3 \cdot 10^{-3}$)	7.1E-04 ($7.1 \cdot 10^{-4}$)	7.1E-04 ($7.1 \cdot 10^{-4}$)	(m^2)	
Position No.	Water depth (m)	Volume (m^3)	Flow at given extended detention depth (L/s)				Total Flow (L/s)	t_{det} (hrs)
1	0	0	0				0	
2	0.125	1875	7.08				7.1	74
3	0.25	3750	10.02	4.72			14.7	71
4	0.375	5625	12.27	6.68	3.32		22.3	70
5	0.5	7500	14.17	8.18	4.70	1.99	29.0	72

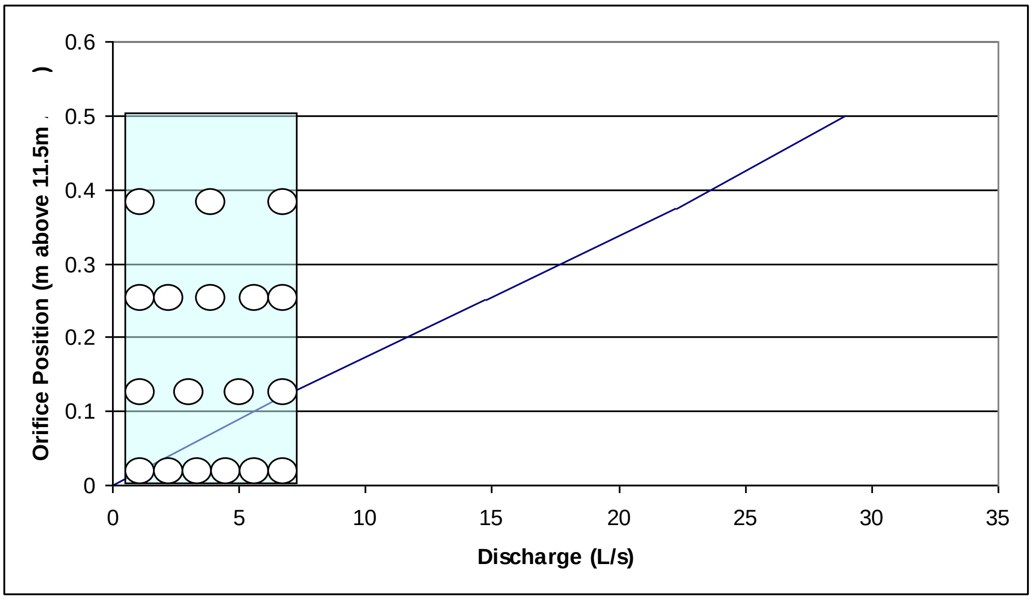


Figure 10.15 Riser configuration

As the required orifices are small, it is necessary to include measures to prevent blocking of the orifices. The riser is to be installed within an outlet pit with a pipe connection to the permanent pool of the macrophyte zone. The connection is via a 225mm diameter pipe. The pit is accessed via the locked screen on top of the pit.

Maintenance Drains

To allow access for maintenance, the wetland is to be drained via a maintenance drain (i.e. pipe) that connects the low points in the macrophyte bathymetry. The drain must be sized to draw down the permanent pool of the macrophyte zone in 12 hours with allowance for manual operation (i.e. inclusion of valve). The permanent pool is assumed to have a mean depth of 0.25m.

The mean flow rate to draw down the macrophyte zone over a notional 12 hour period based on a permanent pool volume of 3,750m^2 is calculated as follows

$$Q = \frac{3750}{12 \times 3600} = 87 L/s$$

The size of the maintenance drain can be established using the Manning's equation assuming the drain/ pipe is flowing full and at 0.5 % grade:

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

Where

Q = The mean flow rate required for maintenance (0.087m^3/s)

A = Cross sectional area of pipe (m^2)

R = Hydraulic radius (m), equivalent to the cross sectional area divided by the wetted perimeter

S = Slope of drainage pipe (0.5%)

n = Roughness coefficient (0.015m for finished concrete)

The pipe diameter is calculated to be 320mm.

The size of the valve can be established using the orifice equation, assuming the orifice operates under inlet control:

$$A_o = \frac{Q}{C_d \sqrt{2 \cdot g \cdot h}}$$

Q = Flow (m^3/s)

C_d = Discharge Coefficient (0.6)

A_o = Valve area (0.06m^2)

g = 9.81 m/s^2

h = Hydraulic head (m)

The valve area is calculated as 0.06m^2, given the discharge coefficient is 0.6 and h is one third the hydraulic head. The equivalent diameter is 270mm.

Discharge Pipe

The discharge pipe of the wetland conveys the outflow of the macrophyte zone to the receiving waters (or existing drainage infrastructure). Under normal operating conditions, this pipe will need to have sufficient capacity to convey the larger of the discharges from the riser (30L/s) or the maintenance drain (87L/s). Considering the maintenance drain flow is the larger of the two flows the discharge pipe size is set to the size of the maintenance drain (320mm pipe at 0.5% as calculated above).

Summary of Macrophyte Zone Outlet

Riser outlet = 225mm diameter pipe with following orifice detail:

Level	Orifices	Orifice Diameter
RL11.5 m	6	40 mm
RL11.625 m	4	40 mm
RL11.75 m	5	30 mm
RL11.875 m	3	30 mm

Maintenance drain = 320mm diameter pipe at 0.5% grade

Maintenance control = 2000 x 4000 mm with letter box grate set at RL11.7 m

Discharge pipe = 270mm diameter valve

Step 6 Design High Flow Bypass Channel

The bypass channel accepts 'above design flow' (100yr ARI = 6.0m^3/s) from the inlet zone (via the bypass weir) and conveys this flow around the macrophyte zone of the wetland. The configuration of the bypass channel can be designed using Manning's Equation:

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

Where

Q	=	'above design flow' (100yr ARI = 6.0m^3/s)
A	=	Cross sectional area (m^2)
R	=	Hydraulic radius (m), equivalent to the cross sectional area divided by the wetted perimeter
S	=	Slope of drainage pipe (1.5%)
n	=	Roughness coefficient (0.035m for earth with gravel and weeds)

A turf finish is to be adopted for the bypass channel. A Manning's n of 0.035 (for earth with gravel and weeds) stipulated in the Singapore Code of Practice on Surface Water Drainage is considered appropriate for flow depths more than double the height of the grass.

Assuming there is a 0.3m drop from the bypass weir crest to the upstream invert of the bypass channel and 5(H):1(V) batters, the base width of the bypass channel can be established by setting the maximum flow depth in the bypass channel at 0.3m. This ensures flow in the channel does not backwater (i.e. submerge) the bypass weir.

For a base width of 19m, the flow through the channel is calculated as 9.0m^3/s, which is greater than the 100yr ARI flow. Hence, the bypass channel is adequately sized.

Step 7 Verification Checks

Macrophyte Zone Re-suspension Protection

A velocity check is to be conducted for when the wetland is at the top of the extended detention level and the riser is operating at design capacity. This check is to ensure velocities through the macrophyte zone ($V_{\text{macrophyte zone}}$) are less than 0.05 m/s to avoid potential scour of biofilms from the wetland plants (macrophytes) and re-suspension of the sediments (Equation 10.3).

The flow rate through the riser was calculated as 30L/s. The cross sectional area refers to the narrowest section of the wetland measured from the top of the extended detention. The narrowest point in the wetland is the ephemeral marsh (refer to Figure 10.14). If the minimum depth at the top of the ephemeral marsh is 0.3m, and the length of the wetland is 50m (given a cross sectional area of 15,000m^2 and a width to length ratio of 1:6), the cross sectional area is 15m^2. The velocity is calculated to be:

$$\frac{0.03}{15} = 0.002 \langle 0.05 m/s$$

Confirm Treatment Performance

The key functional elements of the constructed wetland developed as part of the conceptual design (i.e. area, extended detention depth) were not adjusted as part of the detailed design. Therefore, the performance check undertaken in Step 1 still applies.

Step 8 Vegetation Specification

The vegetation specification for the various zones within the wetland will be advised once the list of recommended plantings has been established by National Parks Board of Singapore.

10.7.3 Design Calculation Summary

The sheet below summarises the results of the design calculations.

Constructed Wetand — **CALCULATION SUMMARY**

CALCULATION TASK		OUTCOME		CHECK
Catchment characteristics				
- Land Uses				
	Residential	10	Ha	
	Commercial	0	Ha	
	Roads /pavements	0	Ha	
- Fraction Impervious				
	Residential	50%		
	Commercial			
	Roads / pavements			
	Weighted average	0.5		
- Site Slope		2.5	m	
Conceptual Design				
	Macropyte Area	15000	m^2	
	Aspect Ratio (macrophyte zone)	1	(W)	
		6	(L)	
	Permanent pool level of macrophyte zone	1	m AHD	
	Extended detention depth (0.25-0.5m)	0.5	m	
	Notional detention time	72	hrs	
Identify design criteria				
	Design ARI Flow for Inlet Zone	1yr	year	
	Target Sediment Size for Inlet Zone	100yr	mm	
	Design ARI Flow for Bypass Spillway	100yr	year	
	Extended Detention Volume	7500	m^3	
1. Confirm treatment size of conceptual design				
	TSS load reduction	HOLD	%	
	TP load reduction	HOLD	%	
	TN load reduction	HOLD	%	
2. Estimate design flow rates				
Time of concentration				
	estimate from flow path length and velocities	10	minutes	
Identify rainfall intensities				
	station used for IFD data	Singapore		
	1 year ARI	106	mm/hr	
	100 year ARI	271	mm/hr	
Design runoff coefficient				
	Catchment use	Residential area-densely built up		
	C	0.80		
Peak design flows				
	Q_1	2.3	m^3/s	
	Q_{100}	6.0	m^3/s	
3. Inlet zone				
	(refer to sedimentation basin calculation checksheet)			
Is a GPT required?				
	Suitable GPT selected and maintenance considered?			
Inlet zone size				
	Target Sediment Size for Inlet Zone	125	μm	
	Capture efficiency	90%	%	
	Inlet zone area	599	m^2	
	$V_s > V_{5yr}$	yes		
Inlet zone connection to macrophyte zone				
	Overflow pit crest level	11.7	m AHD	
	Overflow pit dimension	5000 x 4000	L x W	
	Provision of debris trap	yes		
	Connection pipe dimension	750	mm diam	
	Connection pipe invert level	10.8	m AHD	
High flow by-pass weir				
	Weir Length	21.6	m	
	High flow by-pass weir crest level (top of extended detention)	12	m AHD	
4. Macrophyte Zone Layout				
	Area of Macrophyte Zone	15000	m^2	
	Aspect Ratio	1:6	L:W	
	Hydraulic Efficiency	0.6		
5. Macrophyte Zone Outlet				
Riser outlet				
	Target maximum discharge (Q_{max})	0.03	m^3/s	
	Uniform Detention Time Relationship for Riser	yes		
Maintenance Drain				
	Maintenance drainage rate (drain over 12hrs)	0.09	m^3/s	
	Diameter of maintenance drain pipe	318	mm	
	Diameter of maintenance drain valve	268	mm	
Discharge Pipe				
	Diameter of discharge pipe	318	mm	
6. High flow bypass channel				
	Discharge Capacity of Bypass Weir	9.0	m^3/s	
	Longitudinal slope	1.5	%	
	Base width	19	m	
	Batter slopes	5:1	H:V	
7. Verification checks				
	Macrophyte zone re-suspension protection	yes		

10.8 References

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Infiltration Systems 11

11

Chapter 11 Infiltration Systems

11.1 Introduction

Stormwater infiltration systems capture stormwater runoff and encourage infiltration into surrounding in-situ soils and underlying groundwater. This has the benefit of reducing stormwater runoff peak flows and volumes, reducing downstream flooding, managing the hydrologic regime entering downstream aquatic ecosystems and improving groundwater recharge.

The purpose of infiltration systems in a stormwater management strategy is as a conveyance measure (to capture and infiltrate flows), NOT as a stormwater treatment system. Appropriate pre-treatment of stormwater entering infiltration systems is required to avoid clogging and to protect groundwater quality.

Infiltration systems generally consist of a 'detention volume' and an 'infiltration area' (or infiltration surface):

- The 'detention volume' can be located above or below ground and is designed to detain a certain volume of runoff and make it available for infiltration. When the 'detention volume' is exceeded, the system is designed to overflow to the downstream drainage systems and the receiving environment.
- The 'infiltration area' is the surface or interface between the detention volume and the in-situ soils through which the collected water is infiltrated.

The application of infiltration systems is best suited to moderately to highly permeable in-situ soils (i.e. sandy loam to sandy soils); however, infiltration systems can still be applied in locations with less permeable soils by providing larger detention volumes and infiltration areas.

There are four basic types of infiltration systems:

Leaky Well

A leaky well is typically used in small scale residential applications and consists of a vertical perforated pipe (concrete or PVC) and an open base (Figure 11.1). Pretreated stormwater enters via an inlet pipe at the top of the well and when the detention volume is full, an overflow pipe delivers excess waters to the downstream drainage system. The perforations in the open pipe and the base are covered with a geotextile (non-woven) and the pipe is surrounded by a ring of clean gravel (5 - 10 mm particle size diameter).

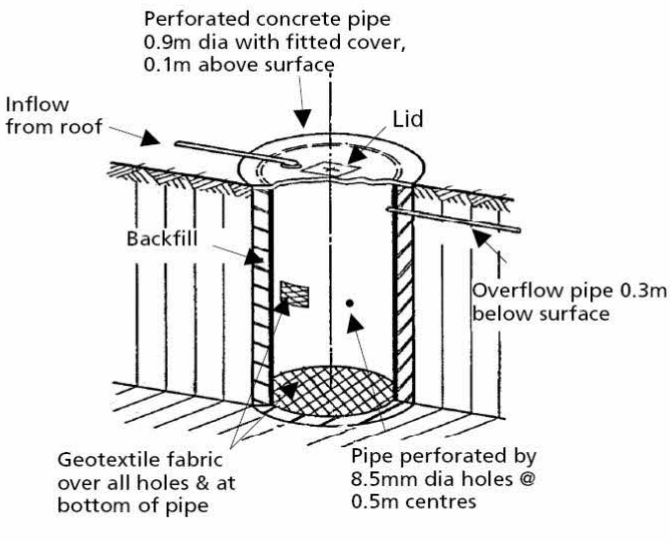


Figure 11.1 'Leaky Well' Infiltration System (Engineers Australia 2006)

Infiltration Trench

Infiltration trenches consist of a trench, typically 0.5 - 1.5 m deep, filled with gravel or modular plastic cells lined with geotextile (non-woven) and placed under 300 mm of backfill (topsoil or sandy loam). Pretreated runoff enters the trench either directly or via an inlet control pit, with excess waters delivered downstream via an overflow pipe. If the trench contains gravel fill then a perforated distribution pipe is incorporated into the system to ensure effective distribution of stormwater into the detention volume. A typical configuration of an infiltration trench is shown in Figure 11.2**.**

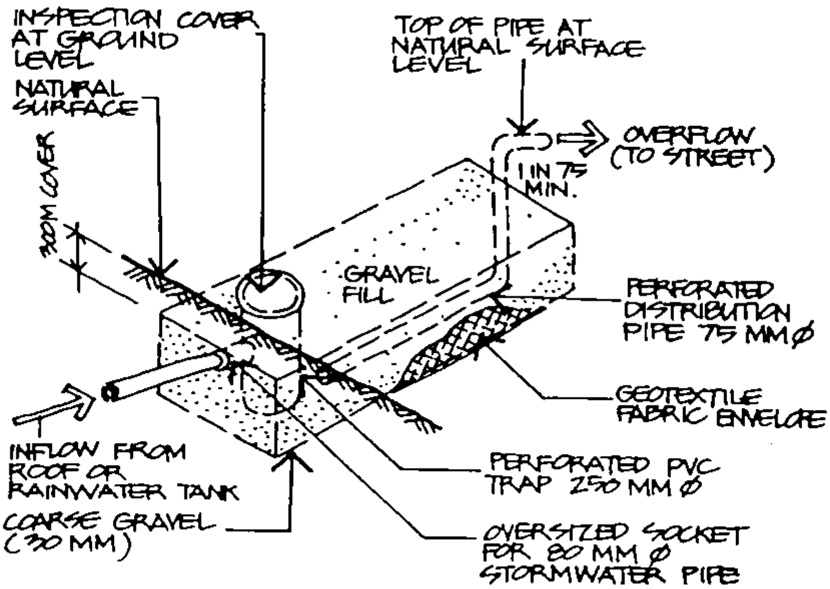


Figure 11.2 Infiltration Trench (Engineers Australia 2006)

Infiltration 'Soak-away'

Soak-aways are similar to trenches in operation but have a larger plan area, being typically rectangular, and of shallower depth (Figure 11.3). Infiltration soak-aways can be applied across a range of scales from residential allotments through to open space or parklands.

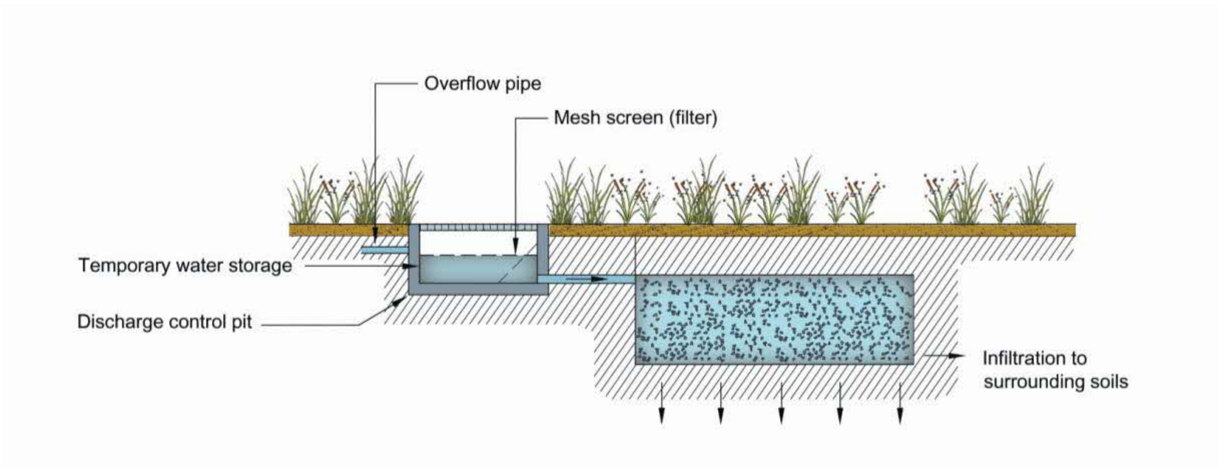


Figure 11.3 Operation of a Gravel Filled 'Trench' or Soak-away' Type Infiltration System

<u>Infiltration Basin</u>

Infiltration basins are typically used in larger scale applications where space is not a constraint (e.g. parklands). They consist of natural or constructed depressions designed to capture and store stormwater runoff on the surface (i.e. the detention volume is located above ground) prior to infiltration into the in-situ soils (Figure 11.4).

A typical section through an infiltration basin is provided in Figure 11.5. Infiltration basins are best suited to sand or sandy-clay in-situ soils and can be planted out with a range of vegetation to blend into the local landscape. Pre-treatment of stormwater entering infiltration basins is required with the level of pre-treatment varying depending on in-situ soil type and basin vegetation. Further guidance in this regard is provided in Section11.2.4.

Figure 11.4 Infiltration Basin

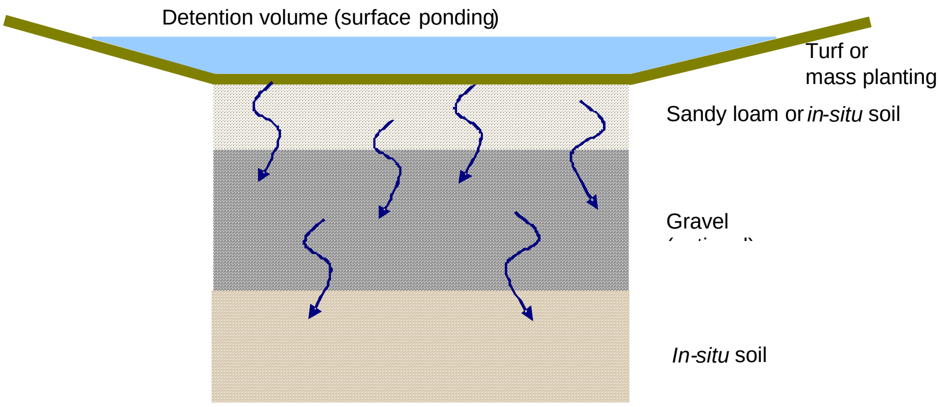


Figure 11.5 Infiltration Basin Typical Section

11.2 Design Considerations

11.2.1. Design Objectives

Infiltration systems can be designed to achieve a range of objectives including:

- Minimising the volume of stormwater runoff from a development
- Preserving pre-development hydrology
- Capturing and infiltrating flows up to a particular design flow
- Enhancing groundwater recharge or preserving pre-development groundwater recharge.

The design objective will vary from one location to another and will depend on site characteristics, development form and the requirements of the receiving ecosystems. It is essential that these objectives are established as part of the conceptual design process and approved by the Public Utilities Board prior to commencing the engineering design.

11.2.2. Selecting the Type of Infiltration System

Selection of the type of infiltration system for a particular application must occur as part of the conceptual design process (i.e. Site Based Stormwater Management Plan) by assessing the site conditions against the relative merits of the four basic types of infiltration systems described in Section 0. There is a range of resources available to assist with this selection process, including *Australian Runoff Quality* (Engineers Australia 2006), *Water Sensitive Urban Design: Basic Procedures for 'Source Control' of Stormwater* (Argue 2004) and *Water Sensitive Urban Design: Technical Guidelines for Western Sydney* (UPRCT 2004).

In general, selection of the type of infiltration system is determined by the size of the contributing catchment. Table 11.1 provides guidance on selection by listing the type of infiltration systems against typical scales of application.

Table 11.1 Infiltration Types and Associated Application Scales

Infiltration Type	Allotment Scale (< 0.1 ha)*	Medium Scale (0.1 - 10 ha)*	Large Scale (> 10 ha)*
Leaky Wells	✓		
Infiltration Trenches	✓	✓	
Infiltration Soak-aways		✓	
Infiltration Basins		✓	✓

* Catchment area directing flow to the infiltration system

11.2.3. Design (Sizing) Methods

Establishing the size of an infiltration system requires consideration of the volume and frequency of runoff discharged into the infiltration system, the available 'detention volume' and the infiltration rate (product of 'infiltration area' and hydraulic conductivity of in-situ soils). The approach for establishing these design elements depends on the design objectives as outlined in Section 11.2.1. For the purposes of these guidelines, the infiltration system design objectives can be addressed by two design methods: the hydrologic effectiveness method and the design storm method. These methods are summarised in **Table 11.2** and discussed in the following sections.

Table 11.2 Design (Sizing) Methods to Deliver Infiltration System Design Objectives

Infiltration Design objective	*Hydrologic Effectiveness Method	*Design Storm Method
Minimise the volume of stormwater runoff from a development	✓	
Preserve pre-development hydrology	✓	
Capture and infiltrate flows up to a particular design flow		✓
Enhance groundwater recharge or preserve pre-development groundwater recharge	✓	

11.2.3.1. Hydrologic Effectiveness Method

Where the design objective is the infiltration of a specific proportion of the mean annual runoff, the hydraulic effectiveness approach can be adopted for sizing infiltration systems. For a given catchment area and meteorological conditions, the hydrologic effectiveness of an infiltration system is determined by the combined effect of the quality and quantity of runoff, the 'detention volume', in-situ soil hydraulic conductivity and 'infiltration area'.

The hydrologic effectiveness of an infiltration system requires long term continuous simulation which can be undertaken using the *Model for Urban Stormwater Improvement Conceptualisation* (MUSIC) (CRCCH 2005). However, in most situations, where a number of the design considerations can be fixed (i.e. frequency of runoff, depth of detention storage, saturated hydraulic conductivity); hydrologic effectiveness curves can be generated and used as the design tool for establishing the infiltration system size.

The hydrologic effectiveness curves derived for infiltration systems (with defined parameters) located in Singapore are presented in Section 11.3.6.1 and represent Step 6 in the design steps required for infiltration measures.

11.2.3.2. Design Storm Method

Where the design objective for a particular infiltration system is peak discharge attenuation or the capture and infiltration of a particular design storm event (e.g. 3-month ARI event), then the design storm approach can be adopted for sizing the infiltration system.

This method involves defining the required 'detention volume' by comparing the volume of inflow and outflow for a particular design storm, and then calculating the 'infiltration area' to ensure the system empties within a specified period of time. This approach requires further development for application in Singapore and therefore, unless otherwise approved by Public Utilities Board, the Hydrologic Effectiveness Method must be used.

11.2.4. Pre-treatment of Stormwater

Pre-treatment of stormwater entering an infiltration system is primarily required to minimise the potential for clogging of the infiltration media and to protect groundwater quality. In line with these requirements, there are two levels of stormwater pre-treatment required:

Level 1 Pre-treatment - To prevent blockage of the infiltration system media, stormwater should be treated to remove coarse and medium sized sediments and litter. Level 1 pre-treatment applies to all four types of infiltration system.

Level 2 Pre-treatment - To protect groundwater quality, pre-treatment is required to remove fine particulates and dissolved pollutants, such as nutrients and metals. This second level of treatment is the most stringent as any stormwater infiltrated must be of equal, or preferably superior, quality to that of the receiving groundwater to ensure the groundwater quality and values are protected. To determine an appropriate level of pre-treatment, assessment in consultation with PUB of the groundwater aquifer quality, possible uses and suitability of treated stormwater for recharge is required.

Level 2 pre-treatment applies to leaky wells, infiltration trenches and infiltration soak-aways. It also applies to most infiltration basin applications. However, level 2 pre-treatment is not required if the infiltration system can be designed to function as a bioretention system i.e. where basins are located on sandy loam to clay soils of low hydraulic conductivity (<180 mm/hr) and the depth to groundwater is greater than 1.0 m. The system can be planted out with rush and reed species and pollutant removal takes place prior to waters entering the underlying groundwater. A summary of pre-treatment requirements for each of the infiltration system types is presented in Table 11.3.

Table 11.3 Pre-treatment Requirements for Each Type of Infiltration System

Infiltration Type	Level 1 Pre-treatment	Level 2 Pre-treatment
Leaky Well	✓	✓
Infiltration Trench	✓	✓
Infiltration Soak-away	✓	✓
Infiltration Basin		
- Sandy clay to clay soils (K_{sat} < 180 mm/hr) + dense ground cover	✓	
- Sandy clay to clay soils (K_{sat} < 180 mm/hr) + turf ground cover	✓	✓
- Sandy soils (K_{sat} > 180 mm/hr)	✓	✓

Note K_{sat} = saturated hydraulic conductivity (mm/hr) of in-situ soil (see Section 11.2.6.1)

11.2.5. Site Terrain

Infiltration into steep terrain can result in stormwater re-emerging onto the surface at some point downslope. The likelihood of this pathway for infiltrated water is dependent on the soil structure. Duplex soils and shallow soil over rock create situations where re-emergence of infiltrated water to the surface is most likely to occur. These soil conditions do not necessarily preclude infiltrating stormwater, unless leaching of soil salt is associated with this process. The provision for managing this pathway will need to be taken into consideration at the design stage to ensure hazards or nuisance to downstream sites are avoided.

Additionally, the introduction of infiltration systems on steep terrain can increase the risk of slope instability. Installation of infiltration systems on slopes greater than 10 % will not be approved by the Public Utilities Board unless a detailed engineering assessment has been undertaken.

11.2.6. In-Situ Soils

11.2.6.1. Hydraulic Conductivity

Hydraulic conductivity of the in-situ soil is the rate at which water passes through a soil medium. It influences both the suitability of infiltration systems and the size of the infiltration area. Therefore, it is essential that field measurement of hydraulic conductivity be undertaken to confirm assumptions of soil hydraulic conductivity adopted during the concept design stage. The determination of hydraulic conductivity must be undertaken in accordance with procedures which provides an estimate of saturated hydraulic conductivity (K_{sat})(i.e. the hydraulic conductivity of a soil when it is fully saturated with water). The typical ranges of saturated hydraulic conductivities for homogeneous soils are provided in **Table 11.4**.

Table 11.4 Typical Soil Types and Associated Saturate Hydraulic Conductivity (Engineers Australia 2006)

Soil Type	Saturated Hydraulic Conductivity	
	m/s	mm/hr
Coarse Sand	>1 x 10-4	>360
Sand	5 x 10-5 to 1 x 10-4	180 – 360
Sandy Loam	1 x 10-5 to 5 x 10-5	36 to 180
Sandy Clay	1 x 10-6 to 1 x 10-5	3.6 to 36
Medium clay	1 x 10-7 to 1 x 10-6	0.36 – 3.6
Heavy Clay	1 x 10-7	0.0 to 0.36

When assessing the appropriateness of infiltration systems and the type of in-situ soils, the following issues must be considered:

Soils with a saturated hydraulic conductivity of 3.6 mm/hr to 360 mm/hr are preferred for infiltration application.

Infiltration systems are neither appropriate nor functional where the in-situ soils are very heavy clays (i.e. < 0.36 mm/hr).

Soils with a low hydraulic conductivity (0.36 - 3.6 mm/hr) do not necessarily preclude the use of infiltration systems even though the required infiltration/ storage area may become prohibitively large. However, soils with lower hydraulic conductivities will be more susceptible to clogging and will therefore require enhanced pre-treatment to remove sediment.

11.2.6.2. Soil Salinity

Infiltration systems must be avoided in areas with poor soil conditions, in particular sodic/ saline and dispersive soils, and shallow saline groundwater. If the 'Site and Soil Evaluation' (refer to Section 11.3.1) identifies poor soil conditions, then the Public Utilities Board will not approve the use of infiltration systems.

11.2.6.3. Impermeable Subsoil, Rock and Shale

Infiltration systems must not be placed in locations where soils are underlain by rock or a soil layer with little or no permeability (i.e. K_{sat} < 0.36 mm/hr). In locations where fractured or weathered rocks prevail, the use of infiltration systems may be approved by the Public Utilities Board provided detailed engineering testing has been carried out to ensure the rock will accept infiltration.

11.2.7. Groundwater

11.2.7.1. Groundwater Quality

As outlined in Section 11.2.4, the suitability of infiltrating stormwater and the necessary pre-treatment requires assessment of the groundwater quality. The principle legislation governing the management of groundwater quality is that there should be no deterioration in groundwater quality. This means the stormwater being infiltrated must be of equal or preferably superior quality to that of the receiving groundwater in order to ensure the groundwater quality and values are protected. To determine an appropriate level of pre-treatment for stormwater, assessment of the groundwater aquifer quality, possible uses and suitability for recharge is required and must be approved by the public Utilities Board.

11.2.7.2. Groundwater Table

A second groundwater related design consideration is to ensure that the base of an infiltration system is always above the groundwater table. It is generally recommended that the base of the infiltration system be a minimum of 1.0 m above the seasonal high water table.

If a shallow groundwater table is likely to be encountered, investigation of the seasonal variation of groundwater levels is essential. This should include an assessment of potential groundwater mounding (i.e. localised raising of the water table in the immediate vicinity of the infiltration system) that in shallow groundwater areas could cause problems with nearby structures.

11.2.8. Building Setbacks (Clearances)

Infiltration systems should not be placed near building footings to avoid the influence of continually wet sub-surface or greatly varying soil moisture content on the structural integrity. *Australian Runoff Quality* (Engineers Australia 2006) recommends minimum distances from structures and property boundaries (to protect possible future buildings in neighbouring properties) for different soil types. These values are shown in Table 11.5.

Table 11.5 Minimum Setback Distances (adapted from Engineers Australia 2006)

Soil Type	Saturated Hydraulic Conductivity (mm/hr)	Minimum distance from structures and property boundaries
Sands	>180	1.0 m
Sandy Loam	36 to 180	2.0 m
Sandy Clay	3.6 to 36	4.0 m
Medium to Heavy Clay	0.0 to 3.6	5.0 m

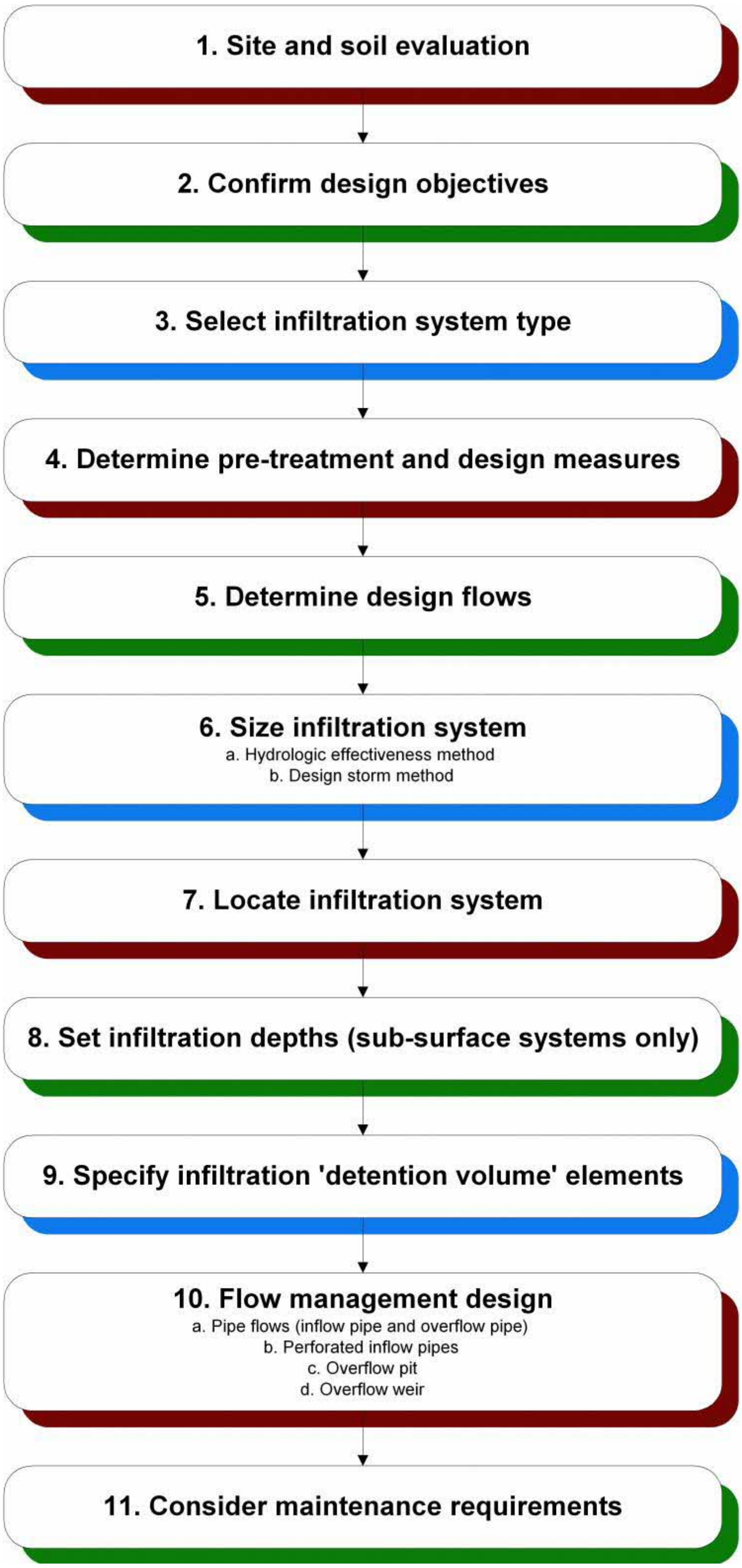
11.2.9. Flow Management

The following issues should be considered when designing the hydraulic control structures within infiltration systems:

- For large scale systems (i.e. infiltration basins), the surface of the 'infiltration area' must be flat to ensure uniform distribution of flow and to prevent hydraulic overloading on a small portion of the 'infiltration area'.
- For gravel filled infiltration systems, flow should be delivered to the 'detention volume' via a perforated pipe(s) network that is located and sized to convey the design flow into the infiltration systems and allow distribution of flows across the entire infiltration area.
- Where possible, 'above design' flows will bypass the infiltration systems. This can be achieved in a number of ways. For smaller applications, an overflow pipe or pit, which is connected to the downstream drainage system, can be used. For larger applications, a discharge control pit can be located upstream of the infiltration system. This will function much like the inlet zone of a constructed wetland to regulate flows (i.e. 1 year ARI) into the infiltration systems and bypass above design flows (i.e. > 1 year ARI).

11.3 Design Process

The following sections detail the design steps required for infiltration measures. Key design steps are as follows:-



11.3.1. Step 1: Site and Soil Evaluation

As outlined in Section 11.2.6, there are a range of site and soil conditions which influence infiltration system design. To define the site's capability to infiltrate stormwater, a 'Site and Soil Evaluation' must be undertaken. The evaluation should provide the following:

- Soil type
- Hydraulic conductivity
- Presence of soil salinity (where applicable)
- Presence of rock shale
- Slope of terrain (%)
- Groundwater details (depth, quality and uses).

11.3.2. Step 2: Confirm Design Objectives

This step involves confirming the design objectives, defined as part of the conceptual design, to ensure the correct infiltration system design method is selected (refer to Table 11.2).

11.3.3. Step 3: Select Infiltration System Type

This step involves selecting the type of infiltration system by assessing the site conditions against the relative merits of the four infiltration systems described in Section 0. In general, the scale of application dictates selection of the infiltration system. Table 11.1 provides guidance in this regard.

For further guidance in selecting infiltration systems, designer should refer to *Australian Runoff Quality* (Engineers Australia 2006), *Water Sensitive Urban Design: Basic Procedures for 'Source Control' of Stormwater* (Argue 2004) and the *Water Sensitive Urban Design: Technical Guidelines for Western Sydney* (UPRCT 2004).

11.3.4. Step 4: Pre-treatment Design

As outlined in Section 11.2.4 and Table 11.3, both Level 1 Pre-treatment (minimising risk of clogging) and Level 2 Pre-treatment (groundwater protection) are required for all infiltration systems except for specific infiltration basin applications. To determine Level 2 requirements, an assessment of the groundwater must be undertaken to define existing water quality, potential uses (current and future) and suitability for recharge.

Pre-treatment measures for roof runoff include the provision of leaf and roof litter guards along the roof gutter and rainwater tanks. Pre-treatment for urban runoff includes sediment basins, vegetated swales, bioretention systems or constructed wetlands as outlined in the other chapters of this guideline.

11.3.5. Step 5: Determine Design Flows

11.3.5.1. Design Flows

To configure the inflow system and high flow bypass elements of the infiltration system the following design flows generally apply:

- Design operation flow' for sizing the inlet to the infiltration system. This will typically correspond to one of the following:
    - 1 year ARI or less – for situations where a discharge control pit is used to regulate flows into the infiltration system and bypass larger flows
    - 2 - 10 year ARI (minor design flow, typically the 5 year ARI event in Singapore) – for situations where the minor drainage system is directed to the infiltration system.

- 'Above design flow' for design of the high flow bypass around the infiltration system. The discharge capacity for the bypass system may vary depending on the particular situation but will typically correspond to one of the following:
  - 2 - 10 year ARI (minor design flow) – for situations where only the minor drainage system is directed to the infiltration system.
  - 50 - 100 year ARI (major design flow) – for situations where both the minor and major drainage system discharge to the infiltration system.

11.3.5.2. Design Flow Estimation

A range of hydrologic methods can be applied to estimate design flows. If typical catchment areas are relatively small, the Rational Method design procedure is considered suitable. However, if the infiltration system is to form part of a detention basin or if the catchment area to the system is large (> 50 ha) then a full flood routing computation method should be used to estimate design flows.

11.3.6. Step 6: Size Infiltration System

As outlined in Section 11.2.3, there are two design methods available for establishing the size of the detention volume and infiltration area of infiltration systems: the hydrologic effectiveness method and the design storm method. Unless otherwise approved by the Public Utilities Board, the hydrologic effectiveness method must be used when designing infiltration systems.

11.3.6.1. Hydrologic Effectiveness Method

Figure 11.6 below shows the relationship between the hydrologic effectiveness, infiltration area and detention storage for a range of soil hydraulic conductivities, detention storage depths and detention storage volumes (adjusted for media porosity) for the reference station 43 in Singapore. The curves were derived using the *Model for Urban Stormwater Improvement Conceptualisation*.

The curves in Figure 11.6 are generally applicable to infiltration measure applications within residential, industrial and commercial land uses. If the configuration of the infiltration measure concept design is significantly different to that described below then the curves may not provide an accurate indication of performance and the detailed designer should use MUSIC to size the infiltration system.

The curves were derived conservatively assuming that the systems have the following characteristics:

- varying in-situ soil hydraulic conductivity
- 'detention volume' area was confined to the space allocated for the 'infiltration area'
- 'detention volume' effective depth of 1.0 m. Note that this is equivalent to an actual depth 1 m when the media porosity is 1.0 (i.e. an open detention volume with no fill media) or 3.0 m and porosity of 0.33 (gravel filled detention volume)

These curves can be used to establish the size of both the 'detention volume' and 'infiltration area' of the infiltration systems to achieve a particular hydrologic effectiveness. The designer is required to select the relevant hydrologic effectiveness curve by establishing the likely configuration and form of the infiltration system, namely whether it will be an open void detention volume (porosity = 1.0) or gravel filled (porosity = 0.35).

If linear interpolation between the curves is used to estimate the infiltration area required for systems with hydraulic conductivities between those shown on the charts, it should be noted that the relationship between the curves is not linear. As a result, these interpolations do not provide an exact representation of the size of infiltration area as a %

of catchment area. Designers must be careful not to under size infiltration areas through this process.

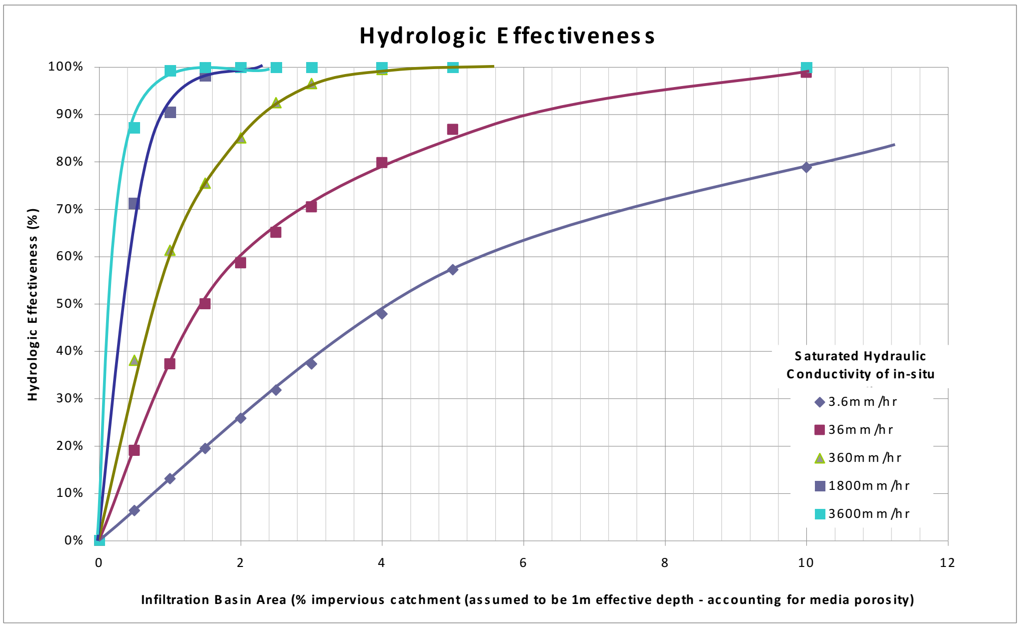


Figure 11.6 Hydrologic Effectiveness of Detention Storages for Infiltration Systems [Reference Station 43]

11.3.7. Step 7: Locate Infiltration System

This step involves locating the infiltration system in accordance with the requirement set out in Section 11.2.8 and Table 11.5 to minimise the risk of damage to structures from wetting and drying of soils (i.e. swelling and shrinking of soils and slope stability).

11.3.8. Step 8: Set Infiltration Depths (sub-surface systems only)

For sub-surface infiltration systems, selection of the optimum depth requires consideration of the seasonal high water table and the appropriate cover of soil to the surface.

- Seasonal groundwater table - As outlined in Section 11.2.7.2, it is generally recommended that the base of the infiltration system be a minimum of 1 m above the seasonal high water table.
- Cover (i.e. depth of soil above top of infiltration system) – Minimum cover of 0.3 m. For systems created using modular plastic cell storage units, an engineering assessment is required.

11.3.9. Step 9: Specify Infiltration 'Detention Volume' Elements

The following design and specification requirements must be documented as part of the design process for 'leaky wells', infiltration trenches and 'soak-aways'.

11.3.9.1. Gravel

Where the infiltration 'detention volume' is created through the use of a gravel-filled trench then the gravel must be a uniform size of between 25 - 100 mm diameter and must be clean (free of fines).

11.3.9.2. Modular Plastic Cells

Where the infiltration detention volume is created through the use of modular plastic cells, the design must be accompanied by an engineering assessment of the plastic cells and their appropriateness considering the loading above the infiltration system. A minimum 150 mm thick layer of coarse sand or fine gravel must underlie the base of the plastic cells.

11.3.9.3. Geofabric

Geofabric must be installed along the side walls and through the base of the infiltration detention volume to prevent the migration of in-situ soils into the system. For this reason, in infiltration systems the use of non-woven geofabric with a minimum perforation or mesh of 0.25 mm is most appropriate.

11.3.9.4. Inspection Wells

It is good design practice to install inspection wells at numerous locations in an infiltration system. This allows water levels to be monitored during and after storm events and for infiltration rates to be confirmed over time.

11.3.10. Step 10: Flow Management Design

The design of the hydraulic control for infiltration systems varies for the different types of systems. For smaller applications, all pretreated flows will enter directly into the infiltration system and an overflow pipe or pit will be used to convey excess flow to the downstream drainage system. For larger applications, a discharge control pit will be located upstream of the infiltration system. This control pit functions to regulate flows for the design return interval (3-month ARI) into the infiltration systems and acts to bypass flows above the design criteria (> 3-month ARI) in a manner similar to the inlet zone of a constructed wetland.

Table **11.6** summarises the typical hydraulic control requirements for the different types of infiltration system.

Table 11.6 Typical Hydraulic Control Requirements for Infiltration Systems

Infiltration Type	Inflow		Overflow/ Bypass	
	Direct inflow	Discharge control pit	Overflow pipe/ pit	Discharge control pit
Leaky Wells	✓		✓	
Infiltration Trenches	✓		✓	
Infiltration Soak-aways		✓		✓
Infiltration Basins	✓	✓	✓	✓

Note: For gravel filled infiltration systems, flow should be delivered to the 'detention volume' via a perforated pipe network.

The hydraulic control measures described in

Table 11.6 are designed using the following techniques.

11.3.10.1. Pipe Flows (Inflow Pipe and Overflow Pipe)

Pipe flows are to be calculated in accordance the Singapore Code of Practice which use standard pipe equations that account for energy losses associated with inlet and outlet conditions and friction losses within the pipe. For most applications, the pipe or culvert will operate under outlet control with the inlet and outlet of the pipe/ culvert being fully submerged. With relatively short pipe connections, friction losses are typically small and can be computed using Manning's equation.

11.3.10.2. Perforated Inflow Pipes

Two design checks are required to ensure that the perforated inflow pipes within the gravel of the filled infiltration systems have sufficient capacity to convey the 'design operation flow' (Section 11.3.5) and distribute this flow into the infiltration system,:

- Ensure the pipe itself has capacity to convey the 'design operation flow'
- Ensure the perforations are adequate to pass the 'design operation flow'.

It is recommended that the maximum spacing of the perforated pipes is 3 m (centres) and that the minimum grade is 0.5 % from the inflow point. The inflow pipes should be extended to the surface of the infiltration system to allow inspection and maintenance when required. The base of the infiltration system must remain flat.

Perforated Pipe Conveyance

To confirm the capacity of the perforated pipes to convey the 'design operation flow', Manning's equation can be used (which assumes pipe full flow but not under pressure). When completing this calculation it should be noted that installing multiple perforated pipes in parallel is a means of increasing the capacity of the perforated pipe system.

Perforated Pipe Slot Conveyance

The capacity of the slots in the perforated pipe needs to be greater than the maximum infiltration rate to ensure the slots does not become the hydraulic 'control' in the infiltration system (i.e. to ensure the in-situ soils and 'detention volume' set the hydraulic behaviour rather than the slots in the perforated pipe). To do this, orifice flow can be assumed to occur through the slots and the sharp edged orifice equation used to calculate the flow through the slots for the full length of perforated pipe. Firstly, the number and size of perforations needs to be determined (typically from manufacturer's specifications) and used to estimate the flow rate out of the pipes, with the driving head being the difference between the overflow level and the invert of the perforated pipe. It is conservative, but reasonable, to use a blockage factor to account for partial blockage of the perforations. A 50 % blockage should be used.

$$Q_{perf} = B \cdot C_d \cdot A \cdot \sqrt{2 \cdot g \cdot h} \qquad \text{Equation 11-1}$$

Where

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

B = blockage factor (0.5)

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

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

g = gravity (9.79 m/s^2)

h = head above the centroid of the perforated pipe (m)

If the capacity of the perforated pipe system is unable to convey the 'design operation flow' then additional perforated pipes will be required.

11.3.10.3. Overflow Pit

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

While the smaller of the two would normally suffice, the larger of the two pit configurations should be adopted to provide a level of conservatism. Furthermore, the size of the pit should also be selected to ensure that it would adequately accommodate the stormwater pipe draining from it.

For free overfall conditions (weir equation):

$$Q_{weir} = B \cdot C_w \cdot L \cdot h^{3/2} \qquad \text{Equation 11-2}$$

Where

Q_{weir} = flow into pit (weir) under free overfall conditions (m^3/s)

B = blockage factor (= 0.5)

C_w = weir coefficient (= 1.66)

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

h = flow depth above the weir (pit) (m)

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

For drowned outlet conditions (orifice equation):

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

Equation 11-3

Where *B*, *g* and *h* have the same meaning as above

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

C_d = discharge coefficient for sharp edge orifice (adopt 0.6)

A = area of orifice (perforations in inlet grate) (m^2)

When designing grated field inlet pits, reference is to the requirements of the Public Utilities Board.

11.3.10.4. Overflow Weir

In applications where infiltration systems require a discharge control pit, a 'spillway' outlet weir will form part of the high flow bypass system to convey the 'above design flow'. The 'spillway' outlet weir level will be set at the top of the 'detention storage' to ensure catchment flows bypass the infiltration system once the 'detention volume' is full. The length of the 'spillway' outlet weir is to be sized to safely pass the maximum flow discharged to the discharge control pit (as defined the 'above design flow' in Section 11.3.5).

The required length of the 'spillway' outlet weir can be computed using the weir flow equation (Equation 11.2) and the 'above design flow' (Section 11.3.5).

11.3.11. Step 11: Consider Maintenance Requirements

Consider how maintenance is to be performed on the infiltration system (e.g. how and where is access available, where sediment likely to collect etc.). A specific maintenance plan and schedule should be developed for the infiltration system, either as part of a maintenance plan for the whole treatment train, or for each individual asset. Guidance on maintenance plans is provided in Section 11.4.1.

11.3.12. Design Calculation Summary

Following is a design calculation summary sheet for the key design elements of an infiltration system to aid the design process.

INFILTRATION SYSTEMS DESIGN CALCULATION SUMMARY				
		CALCULATION SUMMARY		
	Calculation Task	Outcome		Check
	Catchment Characteristics			
	Catchment area		ha	
	Catchment landuse (i.e residential, commercial etc.)			
	Storm event entering infiltration system (minor or major)		year ARI	
1	**Site and soil evaluation**			
	Site and Soil Evaluation' undertaken in accordance with AS1547-2000 Clause 4.1.3			
	Soil type			
	Hydraulic conductivity (K_{sat})		mm/hr	
	Presence of soil salinity			
	Presence of rock/shale			
	Infiltration site terrain (% slope)			
	Groundwater level		m HD m below	
	Groundwater quality			
	Groundwater uses			
2	**Confirm design objectives**			
	Confirm design objective as defined by conceptual design			
3	**Select infiltration system type**			
	Leaky Well			
	Infiltration Trench			
	Infiltration 'Soak-away'			
	Infiltration Basin			
4	**Pre-treatment design**			
	Level 1 Pre-treatment (avoid clogging)			
	Level 2 Pre-treatment (groundwater quality protection)			
5	**Determine design flows**			
	'Design operation flow' (i.e. 1 year ARI)		year ARI	
	'Above design flow' (i.e. 2 - 100 year ARI)		year ARI	
	Time of concentration			
			minutes	
	Identify rainfall intensities			
	'Design operation flow' - $I_{1\ year\ ARI}$		mm/hr	
	'Above design flow'- $I_{2\ –100\ year\ ARI}$		mm/hr	
	Design runoff coefficient			
	'Design operation flow' - $C_{1\ year\ ARI}$			
	'Above design flow'- $C_{2\ –100\ year\ ARI}$			
	Peak design flows			
	'Design operation flow' - 1 year ARI		m^3/s	
	'Above design flow' (2-100 year ARI)		m^3/s	
6	**Size infiltration system**			
	Hydrologic effectiveness approach			
	Hydrologic effectiveness objective		%	
	Depth		m	
	Porosity (void = 1.0, gravel filled = 0.35)			

INFILTRATION SYSTEMS DESIGN CALCULATION SUMMARY

	Calculation Task		CALCULATION SUMMARY Outcome		Check
		'Infiltration Area'		m^2	
		'Detention Volume'		m^3	
7	**Locate infiltration system**				
		Minimum distance from boundary (**Table 11.5**)		m	
		Width		m	
		Length		m	
8	**Set infiltration depths (sub-surface systems only)**				
		Ground surface level		m HD	
		Groundwater level		m HD m below	
		Infiltration system depth		m	
		Top of infiltration system		m HD	
		Base of infiltration system		m HD	
		Cover		m	
		Depth to water table		m	
9	**Specify infiltration 'detention volume' elements**				
		Gravel size		mm diam.	
		Modular plastic cells			
		Geofabric			
10	**Flow management design**				
	Inflow/Overflow structures				
		Direct inflow			
		Overflow pit/pipe			
		Discharge control pit			
	Discharge pipe				
		Pipe capacity		m^3/s	
		Pipe size		mm diam.	
	Inflow pipe				
		Pipe capacity		m^3/s	
		Pipe size		mm diam.	
	Overflow pipe				
		Pipe capacity		m^3/s	
		Pipe size		mm diam.	
	Overflow pit				
		Pit capacity		m^3/s	
		Pit size		mm x mm	
	Perforated inflow pipes				
		No. of pipes			
		Pipe size		mm	
	Discharge control pit				
		Pit size		mm x mm	
		Weir length		m	

11.4 Construction and Establishment

It is important to note in the context of a development site and associated construction/building works, delivering infiltration measures can be a challenging task.

- Construction activities which can generate large sediment loads in runoff which can clog infiltration measures
- Construction traffic and other works can result in damage to the infiltration measures.

A careful construction and establishment approach is needed to ensure that the system is delivered in accordance with its design intent. A staged construction and establishment methodology for infiltration measures is provided in Leinster (2006).

11.4.1. Maintenance Requirements

Maintenance for infiltration measures aims at ensuring the system does not clog with sediments and that an appropriate infiltration rate is maintained. The most important consideration during maintenance is to ensure the pre-treatment elements are operating as designed to prevent sediments from blocking the infiltration measure and to prevent groundwater contamination.

To ensure the system is operating as designed, the infiltration zone should be inspected every 1 - 6 months (or after each major rainfall event) depending on the size and complexity of the system. Typical maintenance of infiltration systems will involve:

- Routine inspection to identify any surface ponding after the design infiltration period, which would indicate clogging/ blockage of the underlying aggregate or the base of the trench.
- Routine inspection of inlet points to identify any areas of scour, litter build up, sediment accumulation or blockages.
- Removal of accumulated sediment and clearing of blockages to inlets.
- Tilling of the infiltration surface, or removing the surface layer, if there is evidence of clogging.
- Maintaining the surface vegetation (if present).

11.5 Checking Tools

This section provides a number of checking aids for designers and assessment officers. In addition, the following checking tools are provided:

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

11.5.1. Design Assessment Checklist

The Design Assessment Checklist presents the key design features that are to be reviewed when assessing the design of an infiltration system. These considerations include configuration, safety, maintenance and operational issues that need to be addressed during the design phase. If an item receives an 'N' when reviewing the design, referral is to be made back to the design procedure to determine the impact of the omission or error.

In addition to the checklist, a proposed design should have all necessary permits for its installation. Assessment officers will require that all relevant permits are in place prior to accepting a design.

11.5.2. Construction Checklist

The Construction Checklist presents the key items to be reviewed when inspecting the infiltration measure during and at the completion of construction. The checklist is to be used by Construction Site Supervisors and Compliance Inspectors to ensure all the elements of the infiltration measure have been constructed in accordance with the design. If an item receives an 'N' in satisfactory criteria then appropriate actions must be specified and delivered to rectify the construction issue before final inspection sign-off is given.

11.5.3. Maintenance and Inspection Checklist

In addition to checking and maintaining the function of pre-treatment elements, the Operation and Maintenance Form can be used during routine maintenance inspections of the infiltration measure and kept as a record on the asset condition and quantity of removed pollutants over time. Inspections should occur every 1 - 6 months depending on the size and complexity of the system. More detailed site specific maintenance schedules should be developed for major infiltration systems and include a brief overview of the operation of the system and key aspects to be checked during each inspection.

11.5.4. Asset Transfer Checklist

Land ownership and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the asset transfer checklist. The Asset Transfer Checklist provides a template for facilitating asset transfer following the maintenance period.

Infiltration Measure Design Assessment Checklist

Asset I.D.			
Infiltration Measure Location:			
Hydraulics:	Design operational flow (m^3/s):	Above design flow (m^3/s):	
Area:	Catchment Area (ha):	Infiltration Area (m^2):	Detention Volume (m^3):

SITE AND SOIL EVALUATION	**Y**	**N**
Site and Soil Evaluation undertaken		
Soil types appropriate for infiltration (K_{sat} > 0.36mm/hr, no salinity problems, no rock/shale)?		
PRE-TREATMENT		
Groundwater conditions assessed and objectives established?		
Level 1 Pre-Treatment provided?		
Level 2 Pre-Treatment provided?		
INFILTRATION SYSTEM	**Y**	**N**
Design objective established?		
Has the appropriate design approach been adopted?		
Infiltration system setbacks appropriate?		
Base of infiltration system >1m above seasonal high groundwater table?		
Has appropriate cover (soil depth above infiltration system) been provided?		
If placed on >10% terrain (ground slope), has engineering assessment been undertaken?		
FLOW MANAGEMENT	**Y**	**N**
Overall flow conveyance system sufficient for design flood event?		
Are the inflow systems designed to convey design flows?		
Bypass/ overflow sufficient for conveyance of design flood event?		

COMMENTS

Infiltration Measures Construction Inspection Checklist

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

Items inspected	**Checked**		**Satisfactory**		**Items inspected**	**Checked**		**Satisfactory**	
	Y	**N**	**Y**	**N**		**Y**	**N**	**Y**	**N**
DURING CONSTRUCTION									
A. FUNCTIONAL INSTALLATION					**Structural components**				
Preliminary Works					10. Location and levels of infiltration system and overflow points as designed				
1. Erosion and sediment control plan adopted					11. Pipe joints and connections as designed				
2. Traffic control measures					12. Concrete and reinforcement as designed				
3. Location same as plans					13. Inlets appropriately installed				
4. Site protection from existing flows					14.Provision of geofabric to sides and base				
Earthworks					15. Correct fill media/modular system used				
5. Excavation as designed					**B. SEDIMENT & EROSION CONTROL (if required)**				
6. Side slopes are stable					16. Stabilisation immediately following earthworks				
Pre-treatment					17. Silt fences and traffic control in place				
7. Maintenance access provided					18. Temporary protection layers in place				
8. Invert levels as designed					**C. OPERATIONAL ESTABLISHMENT**				
9. Ability to freely drain					19. Temporary protection layers and associated silt removed				

FINAL INSPECTION									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of surface				
2. Traffic control in place					7. No surface clogging				
3. Confirm structural element sizes					8. Maintenance access provided				
4. Gravel as specified					9. Construction generated sediment and debris removed				
5. Confirm pre-treatment is working									

COMMENTS ON INSPECTION

ACTIONS REQUIRED
1.
2.
3.
4.
5.
Inspection officer signature:

Infiltration Measures Maintenance Checklist			
Asset I.D.			
Inspection Frequency:	1 to 6 monthly	**Date of Visit:**	
Location:			
Description:			
Site Visit by:			

INSPECTION ITEMS	Y	N	ACTION REQUIRED (DETAILS)
Sediment accumulation in pre-treatment zone?			
Erosion at inlet or other key structures?			
Evidence of dumping (eg building waste)?			
Evidence of extended ponding times (eg. algal growth)?			
Evidence of silt and clogging within 'detention volume'?			
Clogging of flow management systems (sediment or debris)?			
Damage/vandalism to structures present?			
Drainage system inspected?			
Resetting of system required?			
COMMENTS			

Infiltration Measure Asset Transfer Checklist		
Asset Description:		
Asset ID:		
Asset Location:		
Construction by:		
'On-maintenance' Period:		
TREATMENT	**Y**	**N**
System appears to be working as designed visually?		
No obvious signs of under-performance?		
MAINTENANCE	**Y**	**N**
Maintenance plans and indicative maintenance costs provided for each asset?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
ASSET INSPECTED FOR DEFECTS AND/OR MAINTENANCE ISSUES AT TIME OF ASSET TRANFSFER	**Y**	**N**
Sediment accumulation at inflow points?		
Litter present?		
Erosion at inlet or other key structures?		
Traffic damage present?		
Evidence of dumping (e.g. building waste)?		
Evidence of ponding?		
Surface clogging visible?		
Damage/vandalism to structures present?		
COMMENTS		
ASSET INFORMATION	**Y**	**N**
Design Assessment Checklist provided?		
As constructed plans provided?		
Copies of all required permits (both construction and operational) submitted?		
Proprietary information provided (if applicable)?		
Digital files (eg drawings, survey, models) provided?		
Asset listed on asset register or database?		

11.6 Infiltration measure worked example

11.6.1. Worked example introduction

An infiltration system is to be installed to treat stormwater runoff from a residential allotment. Pre-treatment of stormwater prior to discharge into the ground via infiltration is essential to ensure sustainable operation of the infiltration system and protection of groundwater (as discussed in Australian Runoff Quality, Engineers Australia, 2006). Suspended solids and sediment are the key water quality constituents requiring pre-treatment prior to infiltration. Roof runoff is directed into a rainwater tank for storage and to be used as an alternative source of water. Overflow from the rainwater tank can be discharged directly into the gravel trench for infiltration into the surrounding sandy soil without further "pre-treatment". Stormwater runoff from paved areas will be directed to a pre-treatment vegetated swale and then into a gravel trench for temporary storage and infiltration. An illustration of the proposed allotment stormwater management scheme is shown in Figure 11.7.

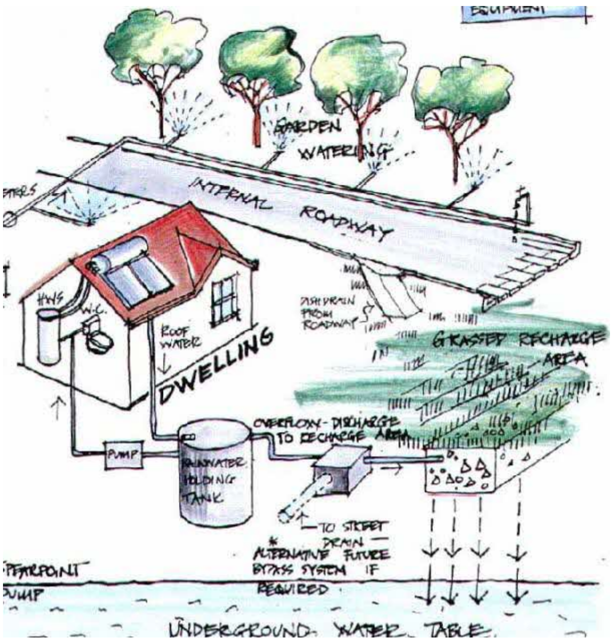


Figure 11.7 Illustration of Allotment Stormwater Management Scheme

[source: Urban Water Resource Centre, University of South Australia; http://www.unisa.edu.au/uwrc/ham.htm]

The allotment in question in this worked example is 10000 m^2 in area on a rectangular site with an overall impervious surface area of 5000 m^2. The site layout is shown in Figure 11.8.

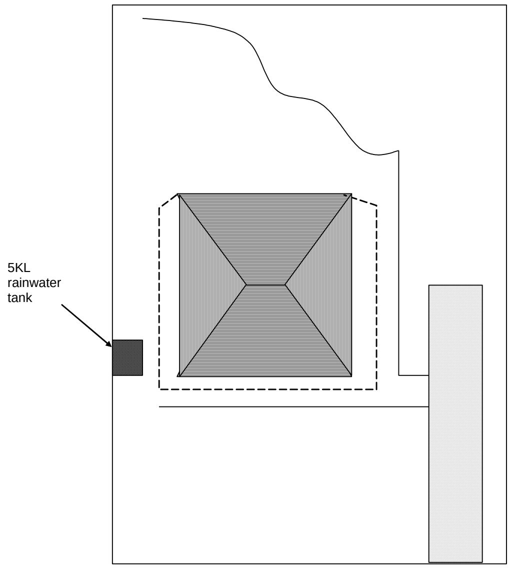


Figure 11.8 Site Layout

Of the impervious surfaces, roof areas make up a total of 2100 m^2, while on-ground impervious surfaces make up the remaining 2900 m^2. There is no formal stormwater drainage system, with stormwater runoff discharging into a small table drain in the front of the property. The design objective of the infiltration system is retention of stormwater runoff from the allotment with a hydrologic effectiveness of 95%. Stormwater flows in excess of the detention capacity of the infiltration system are directed towards the road table drain at the front of the property.

Roof runoff is directed to a 5 kL rainwater tank. Although rainwater tanks can provide significant peak discharge reduction owing to their available storage capacity, in this worked example an assumption is made that the 5kL tank will be full. The design criteria for the infiltration system are to:

- Provide pre-treatment of stormwater runoff.
- Determine an appropriate size of infiltration system.
- Ensure that the inlet configuration to the infiltration system includes provision for by-pass of stormwater when the infiltration system is operating at its full capacity.

This worked example focuses on the design of the infiltration system and associated hydraulic structures.

11.6.2. Step 1: Site and Soil Evaluation

The site characteristics are summarised as follows:

- Catchment area 2100 m^2 (roof)
- 2900 m^2 (ground level paved)
- 5000 m^2 (pervious)
- 10000 m^2 (Total)
- Landuse/surface type – pervious area is either grassed or landscaped with garden beds.
- Overland flow slope – Lot is 25m wide, 40m deep, slope = 3%
- To define the site's suitability for infiltration of stormwater Boreholes were drilled at 2 locations within the site and the results are as follows:
- Soil type = sandy loam

- Hydraulic conductivity = 80 mm/hr
- Presence of soil salinity = no problems discovered
- Presence of rock or shale = no rock or shale discovered
- Slope/ terrain (%) = 2 – 4 %, ground level 10 m Height Datum (HD) in infiltration location
- Groundwater details (depth, quality and uses) = water table 5 m below surface (5 m HD), moderate water quality with local bores used for irrigation.
- Field tests found the soil to be suitable for infiltration.

11.6.3. Step 2: Confirm Design Objectives

The design objectives are summarised as follows:

- Size infiltration trench to retain 95% of the mean annual runoff volume from the site.
- Design the inlet and outlet structures to convey the peak 3-month ARI flow from the critical (flow rate) storm event. Ensure the inlet configuration includes provision for stormwater bypass when the infiltration system is full.
- Pre-treat stormwater runoff.
- Design appropriate ground cover and terrestrial vegetation over the infiltration trench.

11.6.4. Step 3: Select Infiltration System Type

Based on the site attributes, the scale of the infiltration application (i.e. 1.0 ha) and Table 11.1, an infiltration ‘soak-away’ system is selected.

11.6.5. Step 4: Pre-treatment Design

An infiltration ‘soak-away’ has been selected for the site, reference to Section 11.2.4 and Table 11.3 indicates that Level 1 pre-treatment is required. Roof runoff is directed to a rainwater tank. Although the tank may often be full, it nevertheless serves a useful function as a sedimentation basin. A conservative approach to calculating the infiltration capacity was taken by assuming that the 5kL tank will be full at the commencement of the design event. This configuration is considered sufficient to provide the required sediment pre-treatment for roof runoff.

Stormwater runoff from paved areas is directed to a combination of grass buffer areas and a vegetated swale area which is slightly depressed to provide for trapping of suspended solids conveyed by stormwater. Stormwater flows from the swale area into a grated sump pit and then into the infiltration system.

Pre-treatment for sediment removal is therefore provided by the following:

- Connection of roof runoff into a rainwater tank;
- Paved area runoff is conveyed to a combination of grassed buffer areas and a vegetated swale.

11.6.6. Step 5: Determine Design Flows

As described in Section 11.3.5.1, the ‘design operation flow’ is required to size the inlet to the infiltration system. In this case, flows into the infiltration system are to be regulated through a discharge control pit, which will deliver flows up to the 3-month ARI into the infiltration system. Flows greater than the 3-month ARI, or when the infiltration system is full, will bypass the infiltration system by overtopping the overflow weir in the discharge control pit. Therefore:

- ‘design operation flow’ = 3-month ARI

Design flows and Runoff Coefficients were estimated using the Rational Method as described in the Code of Practice on Surface Water Drainage (Public Utilities Board 2006).

Catchment area = 10000 m^2

t_c ~ 6 min

C = 0.65

Rainfall Intensities t_c = 6 mins

$I_{3\ month}$ = 60.6 mm/hr

I_{100} = 275 mm/hr

Rational Method Q = CIA/360 [A = 1.0 Ha]

$Q_{3\ month}$ = 0.109 m^3/s

Q_{100} = 0.497 m^3/s

11.6.7. Step 6: Size Infiltration System

Estimating the required storage volume of the infiltration system is through computer simulation or reference to the design curves in Figure 11.6.

With a sandy loam in situ soil, a saturated hydraulic conductivity of 360 mm/hr is adopted and Figure 11.6 shows that the required storage area (assuming an effective depth of 1m) is 3% of the contributing impervious area. Thus the storage volume required is 0.65 x 10000 x 0.03 = 195 m^3.

11.6.8. Step 7: Locate Infiltration System

As the general fall of the site is to the front of the property, it is proposed that the infiltration system be sited near the front of the property with paved area runoff directed to grassed buffers and a feature vegetated landscaped area adjacent to the infiltration system. Given the sandy soil profile of the site, the minimum distance of the infiltration system from structures and property boundary is 1 m.

Overflow from the infiltration system will be directed to the table drain of the street in front of the property.

11.6.9. Step 8: Set Infiltration Depths

The depth of the infiltration systems must be set to ensure the base is a minimum of 1.0 m above the seasonal high water table and there is a minimum of 0.3 m cover. Considering the water table sits 5 m below surface, a maximum infiltration depth of 3.7 m applies with 0.5m of soil covering the soak-away.

Infiltration depth = 1.0 m

Depth to water table = 3.5 m

The available site area is approximately 48 m^2 and therefore, the effective depth to achieve storage of 195m^3 is 4.1 m. A gravel-filled trench will be used and will have a porosity of 0.35. The resulting actual depth of the infiltration tank will need to be 11.7m.

The proposed layout of the infiltration system is shown in Figure 11.9.

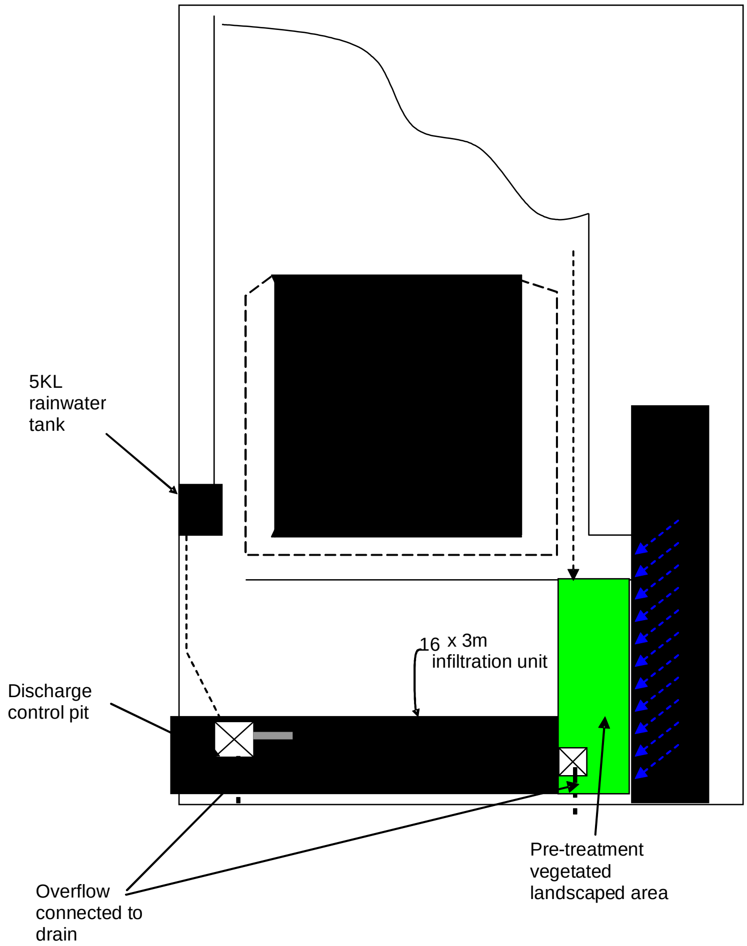


Figure 11.9 Layout of Stormwater Infiltration System

11.6.10. Step 9: Specify Infiltration 'Detention Volume' Elements

The following design specification applies to the infiltration 'soak-away':

- Gravel - clean (fines free) stone/ gravel with a uniform size of 5 mm diameter.
- Geofabric - Geofabric must to be installed along the side walls and through the base of the infiltration detention volume to prevent the migration of in-situ soils into the system. Geofabric must be non-woven type with a minimum perforation or mesh size of 0.25 mm.

11.6.11. Step 10: Hydraulic Control Design

Flow into the infiltration 'soak-away' will be regulated through a discharge control pit with overflow or bypass flows being directed into the piped drainage system located in the road reserve. As depicted in Figure 11.10. The discharge control pit consists of the following:

- Inflow pipe - connection between the pit and the infiltration basin sized to convey 'design operation flow' (3-month ARI)
- Perforated inflow pipes - to distribute ' design operation flow' (3-month ARI) into the gravel filled 'detention volume'
- overflow weir – flows above the 3-month ARI to bypass the infiltration system and to be directed to the street table drain.

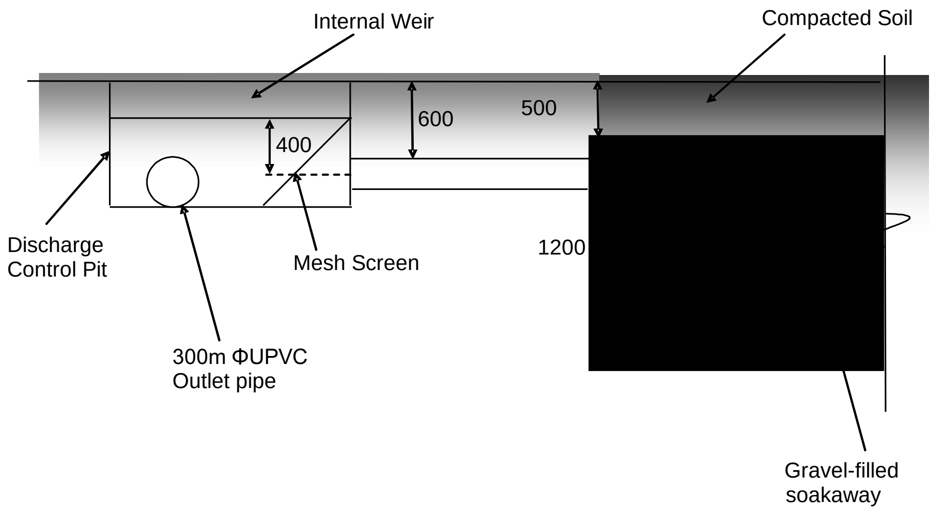


Figure 11.10 Pit Inlet design – Connection to Infiltration System

Peak 3-month design flow = 0.109 m^3/s (calculated previously) and assuming pervious area not contributing any runoff. There will be approximately 0.046 m^3/s discharging from the rainwater tank overflow and 0.063 m^3/s from other paved areas.

There are two inlets to the infiltration system, i.e. one from the rainwater tank and the second from the driveway (see Figure 11.9). These inlets are to be designed to discharge flows up to 0.063 m^3/s each into the infiltration trench with overflows directed to the table drain on the street in front of the property.

Pipe connections from the inlet pits to the infiltration system and street table drain are computed using the orifice flow

$$A_o = \frac{Q}{C_d \sqrt{2gh}}$$

C_d	=	Discharge Coefficient for sharp edge orifice (0.6)
h	=	Depth of water above the centroid of the orifice (m)
A_o	=	Orifice area (m^2)

For pipe connections to the infiltration system, adopt h = 0.15 m; Q = 0.063m^3/s

This gives an orifice area (A_o) of 0.062 m^2, equivalent to a 280 mm diameter pipe ➔adopt 300 mm diameter uPVC pipe.

11.6.12. Perforated Inflow Pipes

To ensure appropriate distribution of flows into the gravel filled ‘detention volume’, three 300 mm diameter perforated pipes laid in parallel (0.75 m apart) are to accept flows from the 300 mm diameter RCP.

Two design checks are required:

- Ensure the pipe has capacity to convey the ‘design operation flow’ (0.109 m^3/s).
- Ensure the perforations are adequate to pass the ‘design operation flow’.

Perforated Pipe Conveyance

Manning's equation is applied to estimate the flow rate in the perforated pipes and confirm the capacity of the pipes is sufficient to convey the 'design operation flow' (0.109 m^3/s). The three 300 mm diameter perforated pipes are to be laid in parallel at a grade of 0.5 %.

$$Q = \frac{A.R^{2/3}.S^{1/2}}{n} \qquad \text{Manning's equation}$$

Applying the Manning's equation using Manning's n = 0.015 finds:

Q (flow per pipe) = 0.320 m^3/s

Q_{Total} = 0.960 m^3/s (for three pipes) > 0.109 m^3/s, and hence OK.

Perforated Pipe Slot Conveyance

To ensure the perforated pipe slots are not a hydraulic choke in the system, the flow capacity of perforated pipe slots is estimated and compared with the 'design operation flow' (0.109 m^3/s). To estimate the flow rate, an orifice equation (equation 11-5) is applied as follows:

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

Where:

Head *(h)* = 0.4 m

Blockage *(B)* = 0.5 (50 % blocked)

Area *(A)* = 3150 mm^2/m clear perforations, hence blocked area
= 1575 mm^2/m

Slot Width = 1.5 mm

Slot Length = 7.5 mm

Pipe diameter = 300 mm

Coefficient *(C_d)* = 0.61 (assume slot width acts as a sharp edged orifice).

Number of unblocked slots per metre = (1575)/(1.5x7.5) = 140

(Note: blockage factor (*B*) already accounted for in 'Area' calculation above)

Inlet capacity /m of pipe = $[0.61 \times (0.0015 \times 0.0075) \times \sqrt{2 \times 9.81 \times 0.4}] \times 140$

= 0.0027 m^3/s

Inlet capacity/m x total length (3 lengths of 16 m)

= 0.0027 x (3 x 16) = 0.129 m^3/s > 0.109, hence OK.

Perforated pipes = 3 x 300 mm diameter perforated pipe laid in parallel, 0.75 m apart and at 0.5 % grade.

11.6.13. Bypass Design

An overflow weir (internal weir) located within the discharge control pit (Figure 11.11) separates the inflow pipe to the infiltration system from the overflow pipe that conveys excess flows to the street table drain. The overflow internal weirs in discharge control pits are to be sized to convey the peak 3-month ARI flow and the overflow weir is designed to provide at least 150m freeboard i.e.

(For conservative design, choose the larger flow for design of discharge control pit, 0.063 m^3/s)

Q_{design} = 2 x 0.063 m^3/s (two inlet pits) = 0.126m^3/s

The weir flow equation (equation 11-6) is used to determine the required weir length:

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

So, using the

Q_{design} flow = 0.126 m^3/s

B = 1.0 (no blockage for internal weir)

C_w = 1.66

h = 0.3 m

We can solve for L, giving a weir length (*L)* = 0.46 m.

To size the pipe connection to the street table drain, use the orifice equation and solve for A

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

Adopting:

h =0.40 m;

Q = 0.126 m^3/s

B = 1

g = 9.81

C_d = 0.6 (assumes sharp edge orifice)

This gives an orifice area (A) of 0.075 m^2, equivalent to a 310 mm diameter pipe ➔ adopt 450 mm diameter uPVC pipe.

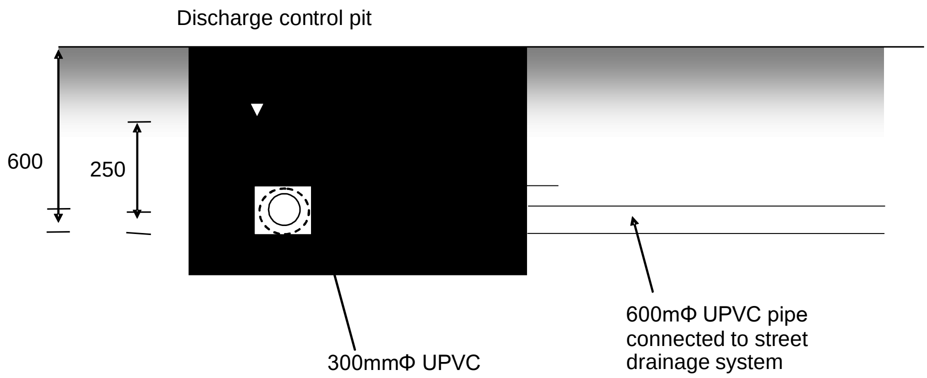


Figure 11.11 Weir used for infiltration system bypass

11.6.14. Design Calculation Summary

	INFILTRATION SYSTEMS DESIGN CALCULATION SUMMARY			
		CALCULATION SUMMARY		
	Calculation Task	**Outcome**		**Check**
	Catchment Characteristics			
	Catchment area	0.1	ha	✓
	Catchment landuse (i.e residential, commercial etc.)	Residential		✓
	Storm event entering infiltration system (minor or major)	3-month	year ARI	✓
1	**Site and soil evaluation**			
	Site and Soil Evaluation' undertaken in accordance			
	Soil type	Sandy-		✓
	Hydraulic conductivity (K_{sat})	360	mm/hr	✓
	Presence of soil salinity	No		✓
	Presence of rock/shale	No		✓
	Infiltration site terrain (% slope)	3		✓
	Groundwater level	RL 5	m	✓
		5	m below	✓
	Groundwater quality	Moderate		✓
	Groundwater uses	Irrigation		✓
2	**Confirm design objectives**			
	Confirm design objective as defined by conceptual design	95% HE		✓
3	**Select infiltration system type**			
	Leaky Well			
	Infiltration Trench			✓
	Infiltration 'Soak-away'			
	Infiltration Basin			
4	**Pre-treatment design**			
	Level 1 Pre-treatment (avoid clogging)			✓
	Level 2 Pre-treatment (groundwater quality protection)			
5	**Determine design flows**			
	'Design operation flow' (< or =1 year ARI)	3-month	year ARI	✓
	'Above design flow' (2 - 100 year ARI)	100	year ARI	✓
	Time of concentration			
		6	minutes	✓
	Identify rainfall intensities			
	'Design operation flow' - $I_{1\ year\ ARI}$	60.6	mm/hr	✓
	'Above design flow'- $I_{2-100\ year\ ARI}$	275	mm/hr	✓
	Design runoff coefficient			
	'Design operation flow' - $C_{1\ year\ ARI}$	0.65		✓
	'Above design flow'- $C_{2-100\ year\ ARI}$	0.65		✓
	Peak design flows			
	'Design operation flow' – 3-month ARI	0.109	m^3/s	✓
	'Above design flow' (100 year ARI)	0.497	m^3/s	✓
6	**Size infiltration system**			
	Hydrologic effectiveness approach			
	Hydrologic effectiveness objective	95	%	✓
	Depth	-	m	
	Porosity (void = 1.0, gravel filled = 0.35)	0.35		
	'Infiltration Area'	48	m^2	✓
	'Detention Volume'	19.5	m^3	✓

INFILTRATION SYSTEMS DESIGN CALCULATION SUMMARY

		CALCULATION SUMMARY		
	Calculation Task	**Outcome**		**Check**
7	**Locate infiltration system**			
	Minimum distance from boundary (**Table 11.5**)	1.2	m	✓
	Width	3	m	✓
	Length	16	m	✓
8	**Set infiltration depths (sub-surface systems only)**			
	Ground surface level	RL10	m	✓
	Groundwater level	RL 5	m	✓
		5	m below surface	✓
	Infiltration system depth	1.2	m	✓
	Top of infiltration system	RL 4.5	m	✓
	Base of infiltration system	RL 3.3	m	✓
	Cover	0.5	m	✓
	Depth to water table	1.7	m	✓
9	**Specify infiltration 'detention volume' elements**			
	Gravel size	5	mm diam.	✓
	Modular plastic cells			
	Geofabric	✓		✓
10	**Flow management design**			
	Inflow/Overflow structures			
	Direct inflow			
	Overflow pit/pipe			✓
	Discharge control pit			
	Discharge pipe			
	Pipe capacity	0.126	m^3/s	✓
	Pipe size	450	mm diam.	✓
	Inflow pipe			
	Pipe capacity	0.063	m^3/s	✓
	Pipe size	300	mm diam.	
	Overflow pipe			
	Pipe capacity	0.126	m^3/s	✓
	Pipe size	450	mm diam.	✓
	Overflow pit			
	Pit capacity	-	m^3/s	N/A
	Pit size	-	mm x mm	N/A
	Perforated inflow pipes			
	No. of pipes	3		✓
	Pipe size	300	mm	✓
	Discharge control pit			
	Pit size		mm x mm	N/A
	Weir length	1	m	N/A

11.7 References

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