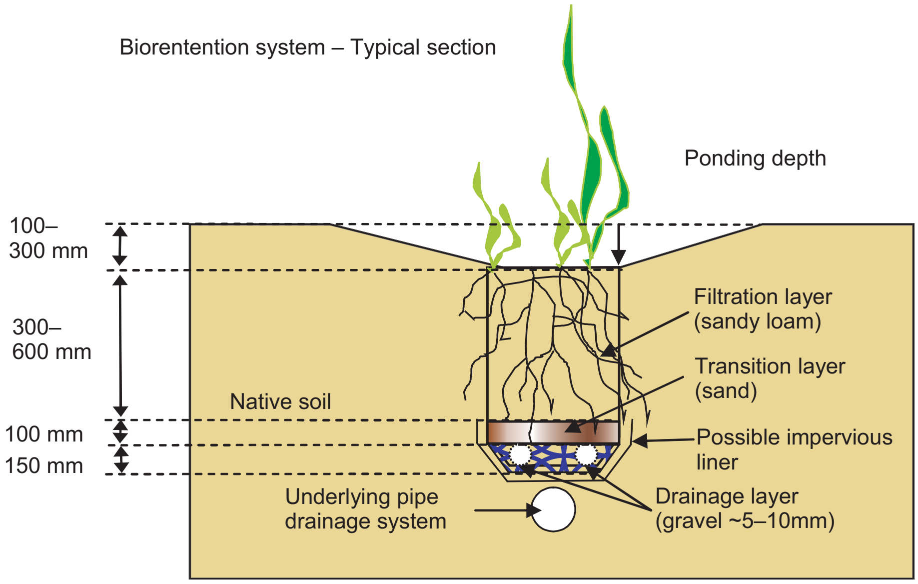


Figure 6.2 Section of a typical bioretention basin.

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

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

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

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

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

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

Key design issues to be considered are:

1. verifying size and configuration for treatment
2. determining design capacity and treatment flows
3. specifying details of the filtration media
4. checking above-ground design:
   - velocities
   - design of **inlet zone** and overflow pits
   - above design flow operation
5. checking below-ground design:

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

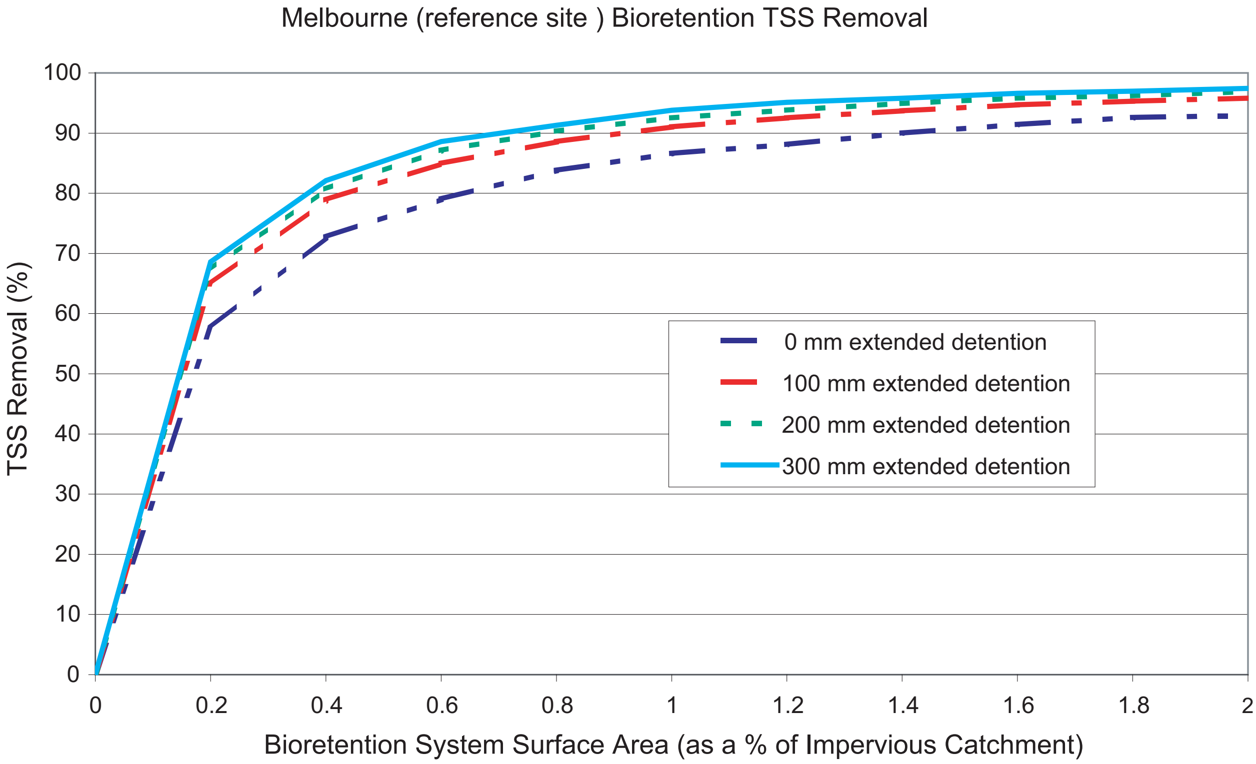


Figure 6.3 Performance of a bioretention system in removing Total Soluble Solids (TSS) in Melbourne.

6. recommending plant species and planting densities
7. providing maintenance.

6.2 Verifying size for treatment

The curves below (Figures 6.3–6.5) show the pollutant removal performance expected for bioretention basins with varying depths of ponding. The curves are based on the performance of the system in Melbourne and were derived using the Model for Urban Stormwater Improvement Conceptualisation (**MUSIC**) (Cooperative Research Centre for Catchment Hydrology 2003). To estimate an equivalent performance at other locations in Victoria, the hydrologic design region relationships should be used to convert the treatment area into an equivalent treatment area in Melbourne (reference site) (see Chapter 2). In preference to using the curves, local data should be used to model the specific treatment performance of the system.

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

- hydraulic conductivity of 180 mm/hr
- filtration media depth of 600 mm
- particle size of 0.45 mm.

These curves can be used to check the expected performance of the bioretention system for removal of Total Soluble Solids (TSS), Total Phosphorus (TP) and Total Nitrogen (TN).

6.3 Design procedure: bioretention basins

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

6.3.1 Estimating design flows

Three design flows are required for bioretention basins:

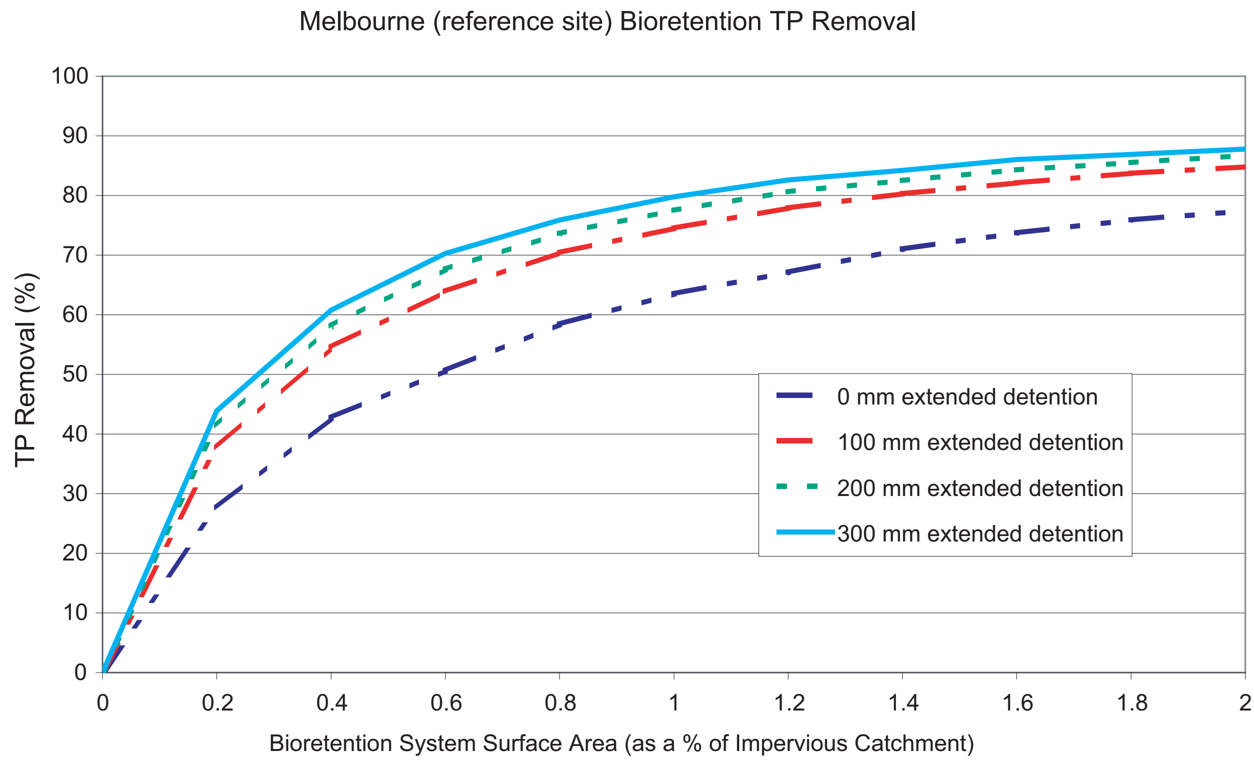


Figure 6.4 Performance of a bioretention system in removing Total Phosphorus (TP) in Melbourne.

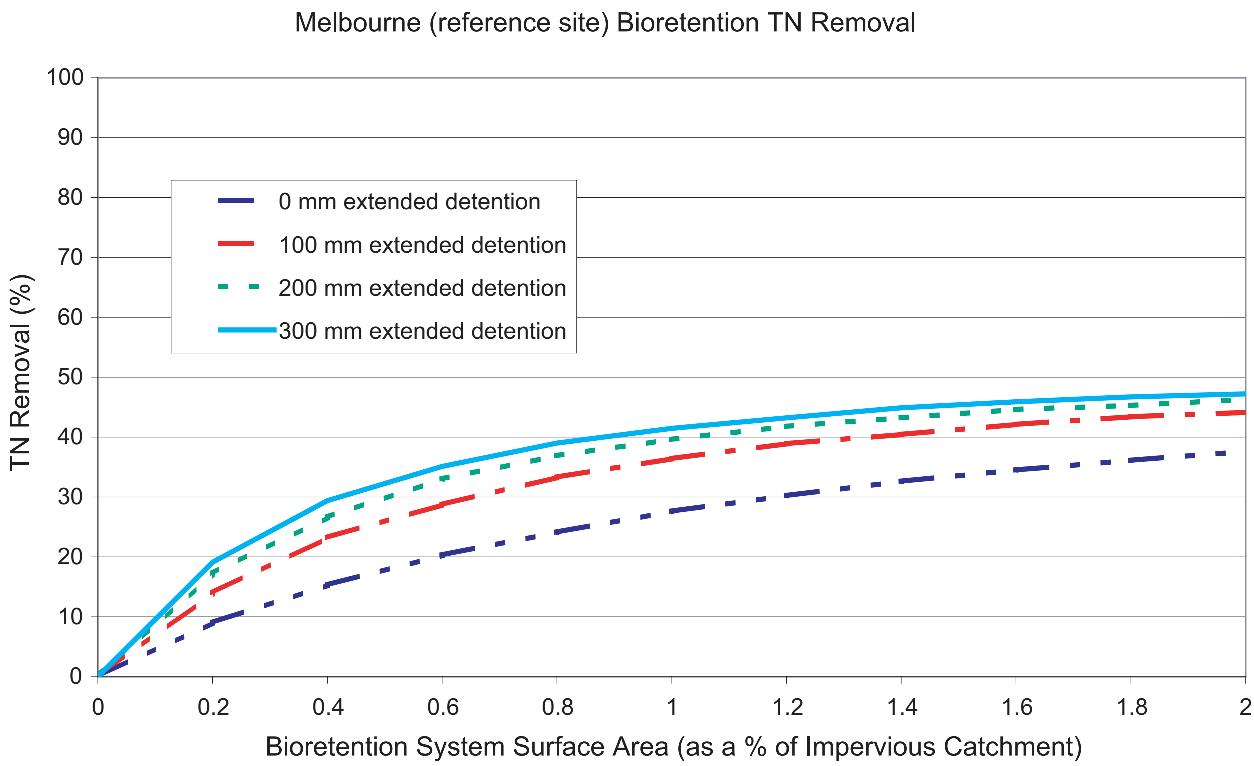


Figure 6.5 Performance of a bioretention system in removing Total Nitrogen (TN) removal in Melbourne.

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

6.3.1.1 Minor and major flood estimation

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

6.3.1.2 Maximum infiltration rate

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

A maximum infiltration rate (Q_{max}) can be estimated by applying Darcy's equation (Equation 6.1):

$$Q_{max} = k \times L \times W_{base} \frac{h_{max} + d}{d} \qquad \text{(Equation 6.1)}$$

where k is the hydraulic conductivity of the soil filter (m/s)
W is the average width of the ponded cross section above the sand filter (m)
L is the length of the bioretention zone (m)
h_{max} is the depth of pondage above the sand filter (m).

6.3.2 Inlet details

Two checks of inlet details are required for bioretention basins: checking the width of flow in the gutter at the inlet (so traffic is not affected); and checking velocities to ensure scour does not occur at the entry for both minor and major storms.

6.3.2.1 Flow widths at entry

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

6.3.2.2 Kerb opening width at entry

To determine the width of the inlet slot in the kerb into the bioretention basin, Manning's equation can be used with the kerb, gutter and road profile to estimate flow depths at the entry point. Once the flow depths for the minor storm (e.g. five-year ARI) is estimated, this can be used to calculate the required width of opening in the kerb by applying a broad-crested **weir** equation (Equation 6.2). This ensures free-draining flows into the bioretention basin. The opening width is estimated by applying the flow depth in the gutter (as H) and solving for L (opening width).

$$Q = C \times L \times H^{3/2} \text{ with } C = 1.7 \qquad \text{(Equation 6.2)}$$

where C = weir coefficient

6.3.2.3 Inlet scour protection

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

6.3.3 Vegetation scour velocity check

Scour velocities over the vegetation are checked through the bioretention basin by assuming the system flows at a depth equal to the ponding depth across the full with of the system. Then by dividing the design flow rate by the cross-sectional area, flow velocity can be estimated. 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 for discharges should be kept below:

- 0.5 m/s for five-year ARI
- 1.0 m/s for 100-year ARI.

6.3.4 Size of slotted collection pipes

The slotted collection pipes at the base of bioretention filter media collect treated water for conveyance downstream. They should be sized so that the filtration media are freely drained and the collection system does not become a 'choke' in the system.

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

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

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

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

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

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

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

6.3.4.1 Perforations inflow check

To estimate the capacity of flows through the perforations, orifice flow conditions are assumed and a sharp-edged orifice equation (Equation 6.3) can be used. First, the number and size of perforations needs to be determined (typically from manufacturer's specifications) and used to estimate the flow rate into the pipes using a head of the filtration media depth plus the ponding depth. Second, it is conservative but reasonable to use a blockage factor (B) to account for partial blockage of the perforations by the drainage layer media. A factor of two is considered adequate.

$$Q_{\text{perforations}} = C \times A\sqrt{2gh} / B \qquad \text{(Equation 6.3)}$$

where $Q_{\text{perforations}}$ = flow through the perforation
g = acceleration due to gravity (9.81 m/s^2)
A = total area of the orifice (m)
h = maximum depth of water above the pipe (m)
C = orifice coefficient

6.3.4. Perforated pipe capacity

The Colebrook-White equation (Equation 6.4) can be applied to estimate the flow rate in the perforated pipe. Manning's equation could be used as an alternative. The capacity of this pipe needs to exceed the maximum infiltration rate.

$$Q = [-2(2gDS_f)^{0.5}\log_{10}(k/(3.7D) + 2.51\nu/D(2gDS_f)^{0.5})] \times A \qquad \text{(Equation 6.4)}$$

6.3.4.3 Drainage layer hydraulic conductivity

The drainage layer is specified with the other soil media used in bioretention systems; however, it should be considered when selecting the perforated pipe system, in particular the slot sizes. Coarser material (e.g. fine gravel) should be used if the slot sizes are large enough for sand to be washed into the slots. If fine gravels are used, then a transition layer is recommended to prevent the filtration media from washing into the perforated pipes. The addition of a transition layer increases the overall depth of the bioretention system and may be an important consideration for some sites (therefore pipes with smaller perforations may be preferable).

6.3.4.4 Impervious liner requirement

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

Where bioretention basins are installed near to significant structures care should be taken to minimise any leakage from the bioretention system. The surrounding soils should be tested and the expected hydraulic conductivity estimated (see Chapter 11 of Engineers Australia 2003).

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

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

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

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

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

6.3.5 High-flow route and bypass design

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

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

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

To size a grated overflow pit, two checks should be made to estimate either drowned or free-flowing conditions. A broad-crested weir equation (Equation 6.5) can be used to determine the length of weir required (assuming free-flowing conditions) (L) and an orifice equation (Equation 6.6) used to estimate the area between opening required (assumed drowned outlet conditions). The larger of the two pit configurations should be adopted. In addition, a blockage factor (B) is to be used that assumes the orifice is 50% blocked.

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

$$Q_{\text{minor}} = B \times C \times L \times H^{3/2} \qquad \text{(Equation 6.5)}$$

with B = blockage factor (0.5), C = 1.7 and H = available head above the weir crest

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

For drowned outlet conditions (orifice equation) (Equation 6.6):

$$Q_{\text{minor}} = B \times C \times A\sqrt{2gh} \qquad \text{(Equation 6.6)}$$

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

6.3.6 Soil media specification

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

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

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

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

6.3.6.1 Filter media specifications

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

The material shall meet the geotechnical requirements set out below:

- **Material** – Sandy loam or equivalent material (ie similar hydraulic conductivity, 50–200 mm/hr) free of rubbish and deleterious material.
- **Particle size** – Soils with infiltration rates in the appropriate range typically vary from sandy loams to loamy sands. Soils with the following composition are likely to have an infiltration rate in the appropriate range: clay 5%–15 %, silt < 30 %, sand 50%–70 %, assuming the following particle sizes ranges (clay < 0.002 mm, silt 0.002 mm–0.05 mm, sand 0.05 mm–2.0 mm).

  Soils with most particles in this range would be suitable. Variation in large particle size is flexible (i.e. an approved material does not have to be screened). Substratum materials should avoid the lower particle size ranges unless tests can demonstrate an adequate hydraulic conductivity (1–5 × $10^{[-]5}$ m/s).

- **Organic content** – between 5% and 10%, measured in accordance with AS1289 4.1.1.
- **pH** – is variable, but preferably neutral, nominal pH 6.0 to pH 7.5 range. Optimum pH for denitrification, which is a target process in this system, is pH 7–8. Siliceous materials may have lower pH values.

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

6.3.6.2 Transition layer specifications

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

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

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

The transition layer is recommended to be a minimum of 100 mm thick. Hydraulic conductivities are shown for a range of media sizes (based on d_{50} sizes) that can be applied in either the transition or drainage layers (Table 6.1).

Table 6.1 Hydraulic conductivity for a range of media particle sizes (d_{50})
Engineers Australia (2003)

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

6.3.6.3 Drainage layer specifications

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

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

6.3.7 Vegetation specification

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

6.3.8 Design calculation summary

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

6.4 Checking tools

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

Checklists are provided for:

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

6.4.1 Design assessment checklist

The *Bioretention Basin Design Assessment Checklist* 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, the design procedure should be assessed to determine the effect of the omission or error.

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

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

6.4.2 Construction advice

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

Building phase damage

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

Traffic and deliveries

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

Inlet erosion checks

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

Sediment build-up on roads

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

Timing for planting

Timing of planting vegetation depends on a suitable time of year (and potential irrigation requirements) as well as timing in relation to the phases of development. For example,

Bioretention Basin Design Assessment Checklist					
Bioretention location:					
Hydraulics	Minor flood: (m^3/s)		Major flood: (m^3/s)		
Area	Catchment area (ha):		Bioretention area (ha)		
Treatment				**Y**	**N**
Treatment performance verified from curves?					
Inlet zone/hydraulics				**Y**	**N**
Station selected for IFD appropriate for location?					
Overall flow conveyance system sufficient for design flood event?					
Maximum upstream flood conveyance width does not impact on traffic amenity?					
Velocities at inlet and within bioretention system will not cause scour?					
Bypass sufficient for conveyance of design flood event?					
Bypass has set down of at least 100 mm below kerb invert?					
Collection system				**Y**	**N**
Slotted pipe capacity > infiltration capacity of filter media?					
Maximum spacing of collection pipes <1.5 m?					
Transition layer/geofabric barrier provided to prevent clogging of drainage layer?					
Basin				**Y**	**N**
Maximum ponding depth will not impact on public safety?					
Selected filter media hydraulic conductivity > 10x hydraulic conductivity of surrounding soil?					
Maintenance access provided to base of bioretention (where reach to any part of a basin >6 m)?					
Protection from gross pollutants provided (for larger systems)?					
Vegetation				**Y**	**N**
Plant species selected can tolerate periodic inundation?					
Plant species selected integrate with surrounding landscape design?					
Detailed soil specification included in design?					

temporary planting during construction for sediment control (e.g. with turf) can then be removed and the area planted out with long-term vegetation.

Planting strategy

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

Perforated pipes

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

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 build-up and water level fluctuations when required and allow easy access for maintenance. The vertical component of the pipe should not be perforated otherwise short circuiting can occur.

Clean filter media

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

6.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Bioretention basins

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

SITE: ______________________

CONSTRUCTED BY: ______________________

DURING CONSTRUCTION

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

FINAL INSPECTION

1. Confirm levels of inlets and outlets					6. Check for uneven settling of soil				
2. Traffic control in place					7. Inlet erosion protection working				
3. Confirm structural element sizes					8. Maintenance access provided				
4. Check batter slopes					9. Construction generated sediment removed				
5. Vegetation as designed									

COMMENTS ON INSPECTION

ACTIONS REQUIRED

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

6.4.4 Asset handover checklist

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

6.5 Maintenance requirements

Bioretention basins treat runoff by filtering it through vegetation and then passing the runoff vertically through a filtration media which filters the runoff. Besides vegetative filtration, treatment relies upon infiltration of runoff into an underdrain. Vegetation is key 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 plant establishment (first two years) when weed removal and replanting may be required. It is also when large loads of sediments could affectplant growth particularly in developing catchments with poor building controls.

Maintenance is primarily concerned with:

- 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

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

the available ponding volume. Should excessive sediment build-up, it will affepact 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. Inspection and removal of debris should be done regularly, but debris should be removed whenever it is observed on the site.

6.5.1 Operation and maintenance inspection form

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

6.6 Bioretention basin worked example

6.6.1 Worked example introduction

A series of bioretention basins, designed as street traffic parking 'out-stands' is to be retrofitted into a local street to treat road runoff. The local street is in inner Melbourne. A proposed layout of the bioretention system is shown in Figure 6.6 and an image of a similar system to that proposed is shown in Figure 6.7.

The contributing catchment areas to each of the individual bioretention basins consist of 300 m^2 of road and footpath pavement and 600 m^2 of adjoining properties. Runoff from adjoining properties (about 60% impervious) is discharged into the road gutter and, together with road runoff, is conveyed along a conventional roadside gutter to the bioretention cell.

The aim of the design is to facilitate effective treatment of stormwater runoff while maintaining a five-year ARI level of flood protection for the local street. Analysis during the concept design of the system has found that a bioretention basin area of 6 m^2 with an extended detention depth of 200 mm, and consisting of a sandy loam soil filtration medium, would treat

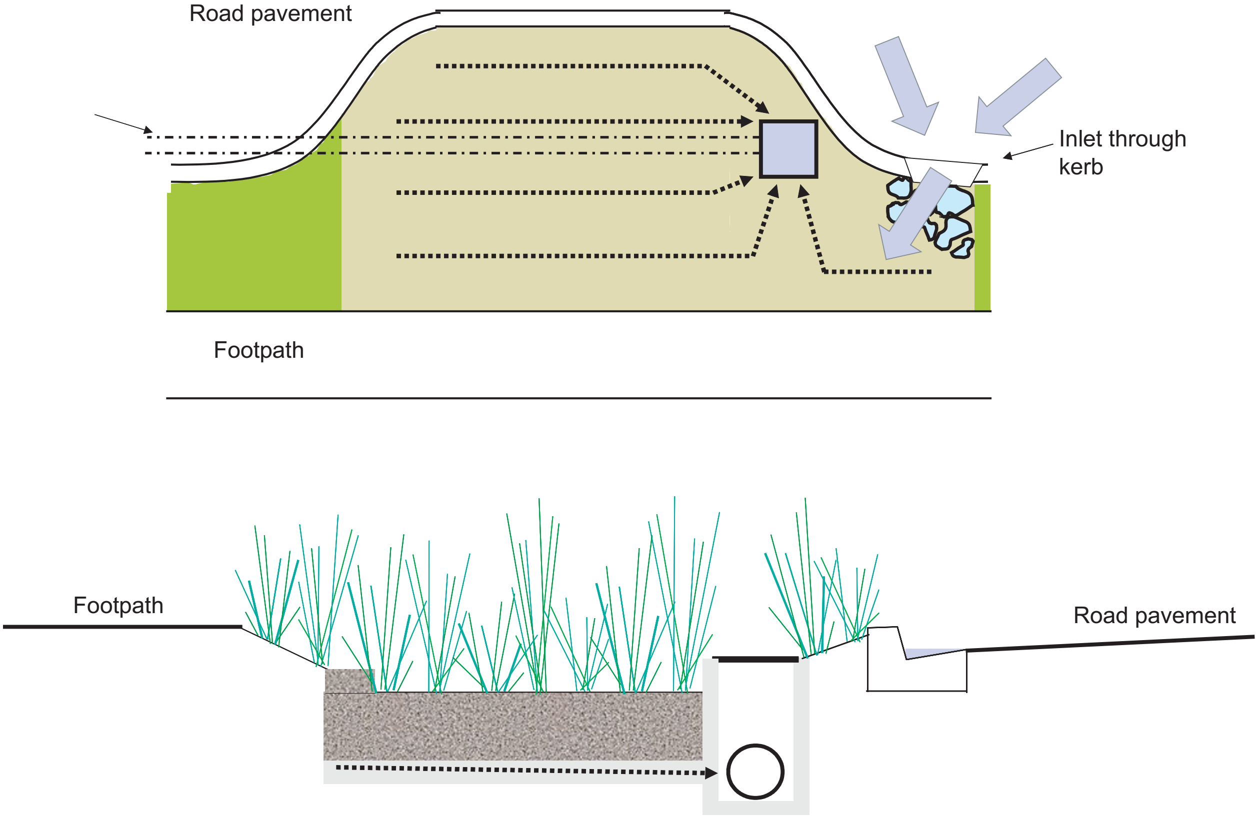


Figure 6.6 General layout and cross section of proposed bioretention system in inner Melbourne.

Figure 7 Retrofitted bioretention system in a street.

stormwater runoff adequately to best practice objectives. The actual size of the cell may, however, be increased to suit other streetscape objectives. The maximum width (measured perpendicular to the alignment of the road) of the bioretention basin is to be 2 m. Analyses to detail the operation of the bioretention basin are shown below and demonstrate the design procedures. The analyses include:

- road and gutter details to convey water into the basin
- detailing inlet conditions to provide for erosion protection
- configuring and designing a system for above-design operation that will provide the required five-year ARI flood protection for the local street
- sizing of below-ground drainage system
- specification of the soil filtration medium
- landscape layout and details of vegetation.

6.6.1.1 Design objectives

The design objectives of the bioretention basin are to:

- maximise reductions of TSS, TP and TN, respectively, while maintaining a five-year ARI level of flood protection for the local street.

6.6.1.2 Constraints and concept design criteria

Analyses during a concept design determined the following criteria:

- bioretention basin area of 6 m^2 (minimum) is required to achieve the water quality objectives
- maximum width of the bioretention basin is to be 2m.
- extended detention depth is 200 mm.
- filter media shall be a sandy loam.

6.6.1.3 Site characteristics

The site characteristics for the bioretention basin are:

- urban, paved carpark and footpaths, lots land use.
- typical overland flow slope of 1%.
- clay soil assumed
- catchment area: carpark, 300 m^2; lots, 600 m^2
- fraction impervious is: carpark, 0.90; lots, 0.60.

6.6.2 Confirm size for treatment

Interpretation of Figures 6.3 to 6.5 with the input parameters below is used to estimate the reduction performance of the bioretention basin for the three pollutants.

- Melbourne location
- 200 mm extended detention
- treatment area to impervious area ratio: 6 m^2/ $[(0.9 \times 300) + (0.6 \times 600)]m^2 = 0.95\%$

From the graphs, the expected pollutant reductions are 92%, 75% and 38% for TSS, TP and TN respectively, and exceed the design requirements of 80%, 45% and 45%.

6.6.3 Estimating design flows

6.6.3.1 Major and minor design flows

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

Time of concentration (t_c)

- Lot, flow path length, is 15 m

  adopt Horton's n (roughness coefficient) = 0.030 (grassed surface)

  slope (S) = 1%

$$\text{Friend's equation } t = \frac{107 \times nL^{0.333}}{S^{0.2}} \qquad \text{(Equation 6.7)}$$

$$t = (107 \times 0.03 \times 15^{\,0.333})/1^{0.2} = 7.9 \text{ min}$$

- Gutter flow: adopt flow path length of 50 m to bioretention.
  Velocity = 1 m/s
  Flow time = 50 m / 1 m/s = 50 s
  Adopt t_c = 7.9 + 0.8 = 8.7 min, say 8 min.

Design rainfall intensities

Adopt the values from from IFD (Intensity–Frequency Duration) table for Melbourne (Table 6.2).

Table 6.2 Design rainfall intensities

	100 yr	5 yr	1 yr
Intensity (mm/hr)	150	72	39.3

Design runoff coefficient

To calculate the design runoff coefficient, apply the method outlined in ARR (Institution of Engineers 2001, Book VIII, 5.1.5.5 iii)

$C^1_{10} = 0.1 + 0.0133({}^{10}I_1 - 25)$, where C^1_{10} is the pervious year runoff coefficient
$C_{10} = 0.9f + C^1_{10}(1-f)$, where f is the fraction impervious.

$${}^{10}I_1 = 28.6 \text{ mm/hr}$$

$$C^1_{10} = 0.15$$

$$f = 0.06 \times 0.6 + 0.03 \times [0.90/(0.06 + 0.03)] = 0.70.$$

$$C_{10} = 0.67$$

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

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

$$C_{3\text{ Month}} = C_1 = 0.8 \times C_{10} = 0.54.$$

Peak design flows

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

$$Q = CIA/360$$

$$Q_5 = 0.012 \text{ m}^3/\text{s}$$

$$Q_{100} = 0.030 \text{ m}^3/\text{s}$$

6.6.3.2 Maximum infiltration rate

The maximum infiltration rate reaching the perforated pipe at the base of the soil media is estimated by using the hydraulic conductivity of the media (k) and head above the pipes (h_{max}) and applying Darcy's equation (Equation 6.1):

Saturation permeability = 180 mm/hr
Flow capacity of the infiltration media (assume no blockage)
Assume $\Upsilon = 0$ (no blockage – maximise infiltration)

Maximum infiltration rate = $(0.18 \times 6)/3600 \times (0.2 + 0.6/0.6) = 0.0004 \text{ m}^3/\text{s}$.

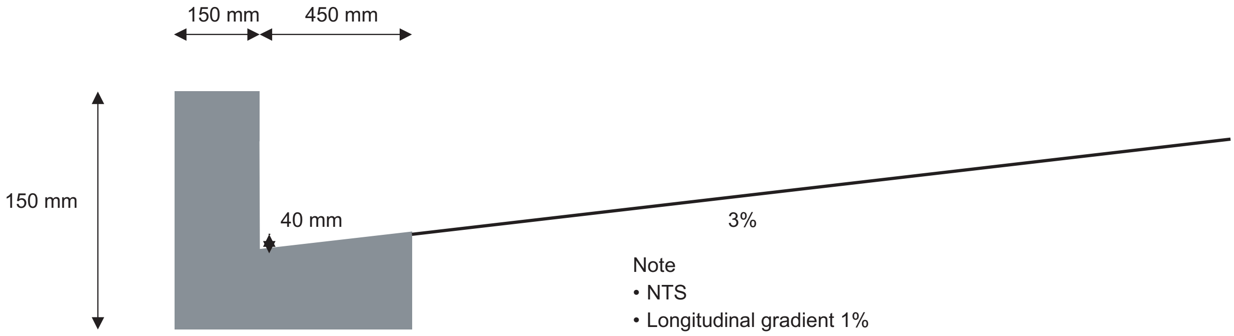


Figure 6.8 Gutter details

6.6.3.3 Inlet details

Flow width at entry

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

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

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

$$Q_{5\text{-year}} = 0.012 \text{ m}^3/\text{s, depth of flow} = 0\ 55 \text{ mm}$$

$$\text{width of flow} = 900 \text{ mm (within gutter)}$$

$$\text{velocity} = 0.6 \text{ m/s (within gutter).}$$

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

$$Q_{100} = 0.030 \text{ m}^3/\text{s, depth of flow} = 70 \text{ mm}$$

width of flow = 1.45 m (within gutter)
velocity = 0.8 m/s (within gutter).

Kerb opening at entry

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

$$Q_5 = 0.012 \text{ m}^3/\text{s.}$$

Assume broad-crested weir flow conditions (Equation 6.5) through the slot

$$Q = C.L.H^{3/2}$$

Adopt $C = 1.7$.
Flow depth (Q_5) = 55 mm, adopt $H = 0.055$ m
Therefore,

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

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

Inlet scour protection

Rock beaching is to be provided at the inlet to manage flow velocities from the kerb and into the bioretention system. This detail is shown on Figure 6.6.

6.6.4 Vegetation scour velocity check

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

Width of bioretention = 2 m
Extended detention depth = 0.2 m
Area = 2 m x 0.2 m= 0.4 m^2

Q_5 average velocity = 0.012 m^3/s / 0.4 m^2 = 0.03 m/s, which is < 0.5 m/s – therefore OK.

Q_{100} average velocity = 0.03 m^3/s / 0.4 m^2 = 0.08 m/s, which is < 1.0 m/s – therefore OK.

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

6.6.5 Sizing of perforated collection pipes

6.6.5.1 Perforations inflow check

Estimate the inlet capacity of subsurface drainage system (perforated pipe) to ensure it is not a choke in the system. To build in conservatism, it is assumed that 50% of the holes are blocked. A standard perforated pipe was selected that is widely available. To estimate the flow rate, an orifice equation (Equation 6.3) is applied using the following parameters:

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

Assume subsurface drains with half of all pipes blocked:
Clear opening = 2100 mm^2/m, hence blocked openings are 1050 mm^2/m
Slot width is 1.5 mm
Slot length, 7.5 mm
No. of rows, 6
Diameter = 100 mm,
Number of slots per metre = (1050)/(1.5 × 7.5) = 93.3
Assume orifice flow conditions – $Q = CA\sqrt{2gh}$
C = 0.61 (assume slot width acts as a sharp-edged orifice, see Equation 6.3).
Inlet capacity per metre of pipe =

$$[0.61 \times (0.0015 \times 0.0075) \times \sqrt{2 \times 9.88 \times 0.85}] \times 93.3 = 0.0025 \text{ m}^3/\text{s}$$

Inlet capacity per metre × total length = 0.0025 × (6/2) = 0.008 m^3/s, which is > 0.004 (maximum infiltration rate), hence OK.

6.6.5.2 Perforated pipe capacity

The Colebrook-White equation (Equation 6.4) is applied to estimate the flow rate in the perforated pipe. Manning's equation could be used as an alternative. A slope of 0.5% is assumed and a 100 mm perforated pipe (as above) was used. Should the capacity not be sufficient, either a second pipe could be used or a steeper slope. The capacity of this pipe needs to exceed the maximum infiltration rate.

Estimate applying the Colebrook-White equation (see Equation 6.4):

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

Adopt D = 0.15 m
S_f = 0.005 m/m
g = 9.81 m^2/s
k = 0.007 m

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

$Q_{cap} = 0.004\ m^3/s$ (for one pipe), which is > $0.004\ m^3/s$, and is hence OK.

Adopt 1 × ϕ (diameter) 100 mm perforated pipe for the underdrainage system.

6.6.5.3 Drainage layer hydraulic conductivity

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

6.6.5.4 Impervious liner requirement

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

6.6.6 High-flow route and bypass design

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

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

First, check using a broad-crested weir equation (Equation 6.2):

$Q_{minor} = B \times C \times L \times H^{3/2}$ with $B = 0.5$, $C = 1.7$ and $H = 0.1$ and solving for L

Gives $L = 0.44$ m of weir length required (equivalent to 115 × 115 mm pit).

Second, check for drowned conditions:

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

$$0.12 = 0.5 \times 0.6 \times A \times \sqrt{2g} \times 0.1 \text{ gives } A = 0.029\ \mathrm{m}^2 \text{ (equivalent to 170 x 170 pit)}$$

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

6.6.7 Soil media specification

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

6.6.7.1 Filter media specifications

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

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

6.6.7.2 Transition layer specifications

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

percentage passing 1.4 mm, 100%; 1.0 mm, 80%; 0.7 mm, 44%; 0.5 mm, 8.4%.

6.6.7.3 Drainage layer specifications

The drainage layer is to be 5 mm screenings.

6.6.8 Vegetation specification

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

6.6.9 Calculation summary

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

Bioretention basins	CALCULATION SUMMARY		
CALCULATION TASK	**OUTCOME**		**CHECK**
1 Identify design criteria			✓
Conveyance flow standard (ARI)	5	year	
Area of bioretention	6	m^2	
Maximum ponding depth	200	mm	
Filter media type	180	mm/hr	
2 Catchment characteristics			✓
Car park area	300	m^2	
Allotment area	600	m^2	
Slope	1	%	
Fraction impervious			✓
Car park	0.9		
Allotments	0.6		
3 Estimate design flow rates			
Time of concentration			
Estimate from flow path length and velocities	8	minutes	✓
Identify rainfall intensities			
Station used for IFD data:	Melbourne		
100-year ARI	150	mm/hr	
5-year ARI	72	mm/hr	
Peak design flows			
Q_5	0.012	m^3/s	
Q_{100}	0.030	m^3/s	
Q_{infil}	0.0003	m^3/s	✓
4 Slotted collection pipe capacity			
Pipe diameter	100	mm	
Number of pipes	1		
Pipe capacity	0.004	m^3/s	
Capacity of perforations	0.015	m^3/s	
Soil media infiltration capacity	0.004	m^3/s	
CHECK PIPE CAPACITY > SOIL CAPACITY	YES		✓
5 Check flow widths in upstream gutter			
Q_5 flow width	0.9	m	
CHECK ADEQUATE LANES TRAFFICABLE	YES		✓
6 Kerb opening width			
Width of break in kerb for inflows	0.6	m	✓
7 Velocities over vegetation			
Velocity for 5-year flow (<0.5 m/s)	0.03	m/s	
Velocity for 100-year flow (<1.0 m/s)	0.08	m/s	✓
8 Overflow system			
System to convey minor floods	grated pit 600 x 600		✓
9 Surrounding soil check			
Soil hydraulic conductivity	0.36	mm/hr	
Filter media	180	mm/hr	
MORE THAN 10 TIMES HIGHER THAN SOILS?	YES (no liner)		✓
10 Filter media specification			
Filtration media	sandy loam		
Transition layer	coarse sand		
Drainage layer	fine gravel		✓
11 Plant selection			
	Carex appressa		✓

6.6.10 Construction drawings

Figure 6.9 shows the construction drawing for the worked example.

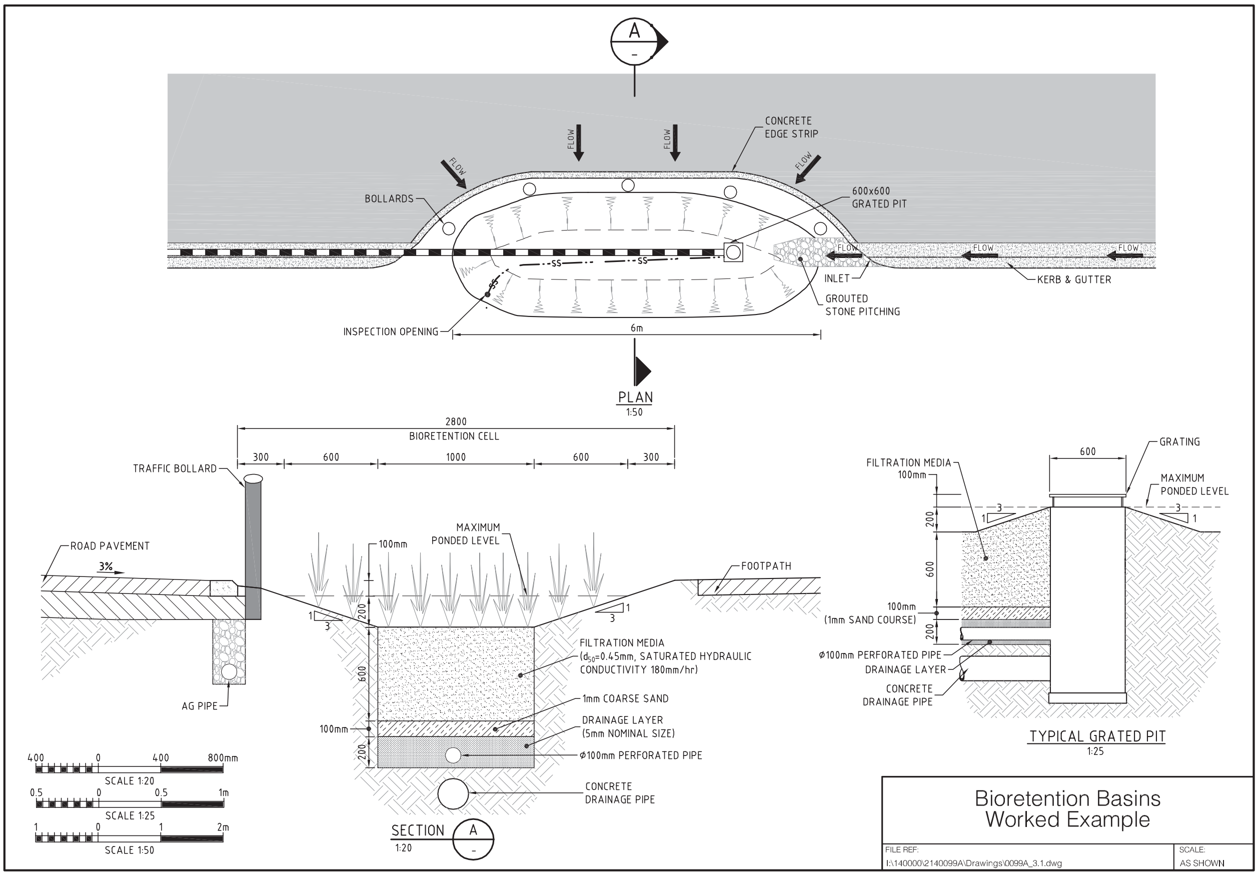


Figure 6.9 Construction drawing and a section view of the bioretention basin.

6.7 References

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

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

Institution of Engineers, Australia (1997). *Australian Rainfall and Runoff – A Guide to Flood Estimation*, Revised edn, Pilgram, D.H. (Ed.), Institution of Engineers, Australia, Barton, ACT.

Standards Australia (1997). AS 1289.4.1.1: Methods of testing soils for engineering purposes – Soil chemical tests – Determination of the organic matter content of a soil – Normal method, Standards Australia, Sydney.

Chapter 7 **Sand filters**

Sand filters for detention and filtration of stormwater runoff

7.1 Introduction

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

Other filter media, such as peat, mulch or gravel have also been used in filtration systems, however, only sand filters are discussed in this chapter.

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

Sand is the filtration media and its hydraulic conductivity ranges from 1×10^{-4} m/s (360 mm/hr) to 1×10^{-3} m/s (3600 mm/hr).

A sand filter system typically consists of three chambers (Figures 7.1 and 7.2).

Water firstly enters a **sedimentation** chamber where gross pollutants and coarse to medium-sized sediment are retained. **Stormwater** enters this chamber either via a conventional side entry pit or through an underground pipe network. The sedimentation chamber can be designed to have either permanent water between events or to drain between storm events with weep holes. There are advantages and disadvantages with each approach.

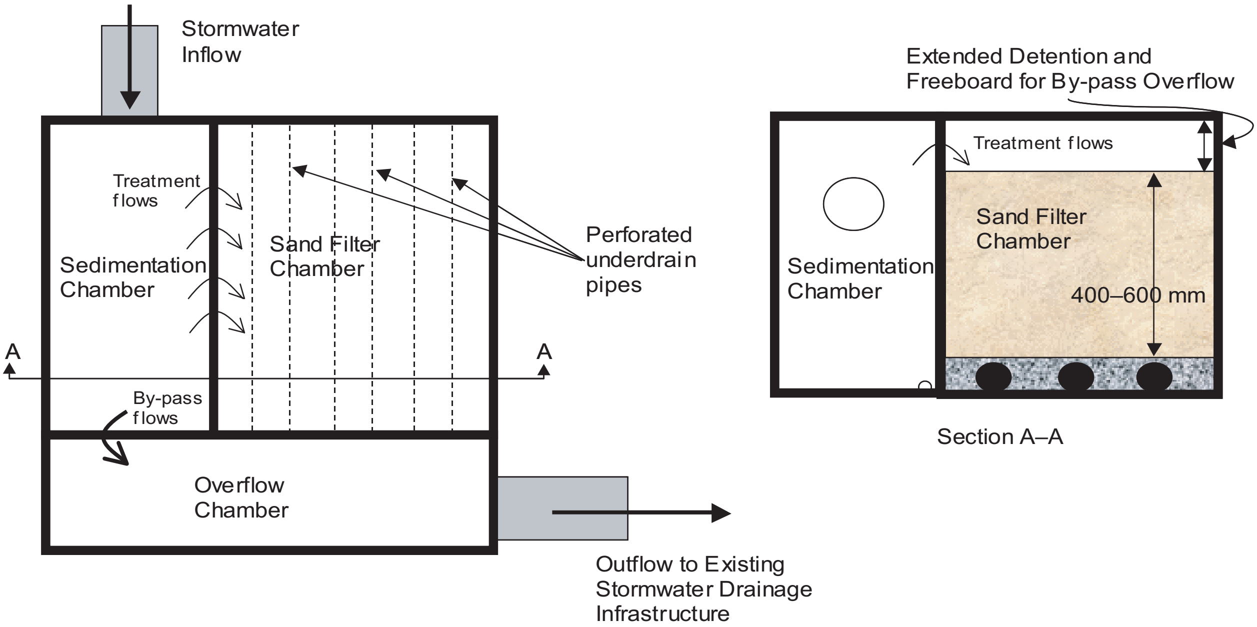


Figure 7.1 Proposed layout of a sand filter.

Having a permanent water body reduces the likelihood of resuspension of sediments at the start of the following rainfall event as inflows do not fall and scour collected sediments. The potential for mosquito breeding is minimised because of the likelihood of sufficient surface oil on incoming flows to discourage mosquitoes. However, this system requires the removal of wet material from the sedimentation chamber during maintenance.

Allowing the sedimentation chamber to drain between events (by installation of weep holes) reduces the likelihood of pollutant transformation during the interevent period. The high organic loads and stagnant water can lead to anaerobic conditions that can also lead to release of soluble pollutants (e.g. phosphorus). Release of these bioavailable pollutants can cause water

Figure 7.2 Underground sand filter for a car park in Auckland, New Zealand.

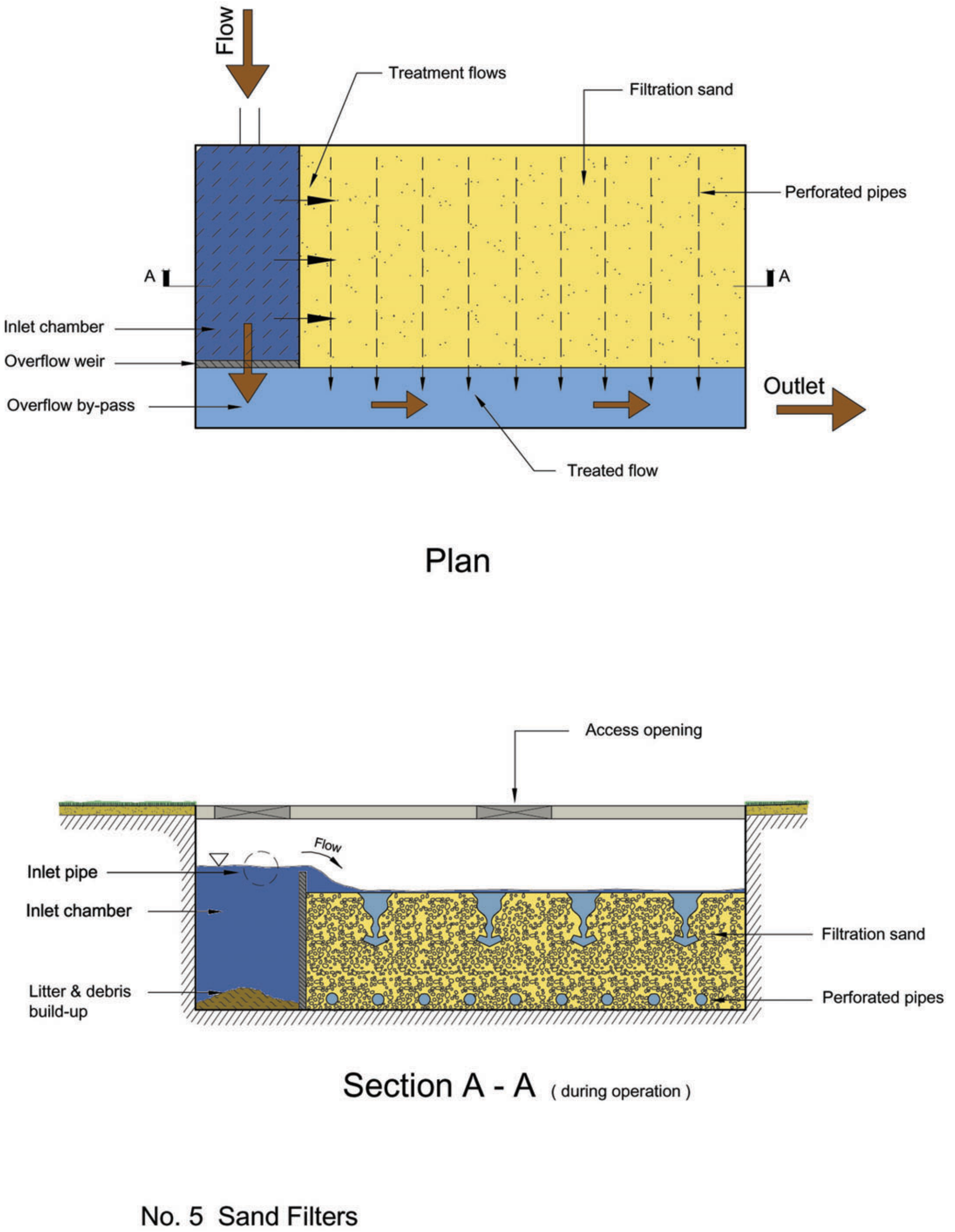


Figure 7.3 A sand filter during operation.

quality problems downstream (e.g. excessive algal growth). The challenge with this system, however, is to design weep holes such that they can continue to drain as material (litter, organic material and sediment) accumulates and the holes do not block.

These factors need to be considered when designing the sedimentation chamber.

Stormwater overflows the sedimentation chamber into a sand filter chamber via a **weir**. Water percolates through the sand filtration media (typically 400 mm–600 mm depth) and filtered water is collected by perforated underdrain pipes in a similar manner as in bioretention systems. A notional extended detention depth is provided within this chamber above which water will flow into an overflow chamber (usually via the sedimentation chamber). Owing to the high saturated hydraulic conductivity of sand as a filtration media, analyses have found that only a small (about 200 mm) extended detention depth is required.

Figure 7.2 shows a sand filter in Auckland and Figure 7.3 illustrates how a sand filter may be configured and operates during storm events.

Key functions of a sand filter include the following:

- capture of gross pollutants
- sedimentation of particles larger than 125 μm within a sedimentation chamber for flows up to a one-year ARI (unattenuated) peak discharge
- filtration of stormwater following sedimentation pretreatment through a sand filtration layer.

7.2 Verifying size for treatment

Figures 7.4, 7.5 and 7.6 show expected performance of sand filters for retention of Total Soluble Solids (TSS), Total Phosphorus (TP) and Total Nitrogen (TN), respectively, for Melbourne conditions. These curves were derived using **Model for Urban Stormwater Improvement Conceptualisation (MUSIC)** (Cooperative Research Centre for Catchment Hydrology 2003) an assumed sand filter depth of 600 mm.

These performance curves can be used to verify the selected size of a proposed sand filter. The regional hydrological design equations for bioretention systems can be used for sand filters.

7.3 Design procedure: sand filters

The following sections describe the design steps required for sand filters.

7.3.1 Estimating design flows

Three design flows are required for sand filters:

- sedimentation chamber design flow – this would normally correspond to the one-year ARI peak discharge as standard practice for sedimentation basins
- sand filter design flow – this is the product of the maximum infiltration rate and the surface area of the sand filter, used to determine the minimum discharge capacity of the underdrains to allow the filter media to freely drain
- overflow chamber design flow – this would normally correspond to the minor drainage system (typically five-year ARI) to size the weir connecting the sand filter to the overflow chamber. This allows minor floods to be safely conveyed and not increase any flooding risk compared to conventional stormwater systems.

7.3.1.1 Minor and major flood estimation

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

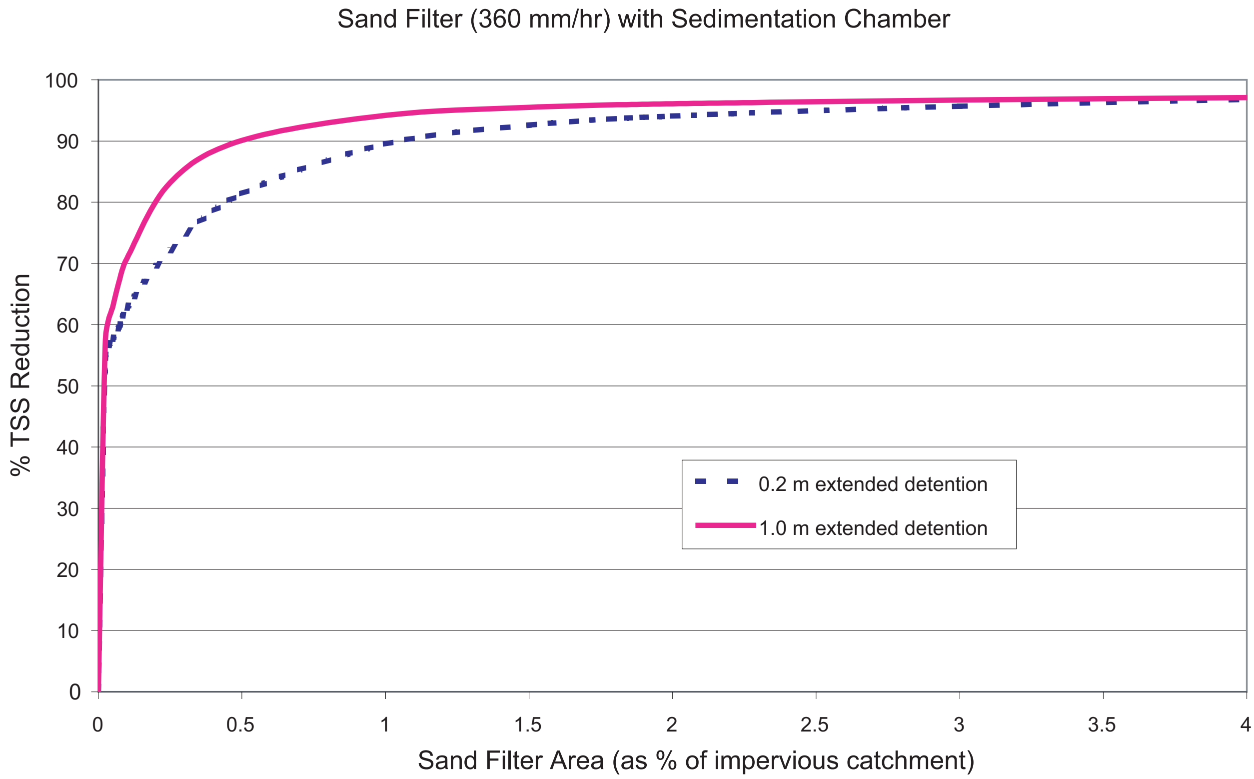
7.3.1.2 Maximum infiltration rate

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

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

$$Q_{max} = k \times A \times \frac{h_{max} + d}{d} \qquad \text{(Equation 7.1)}$$

where k is the hydraulic conductivity of the soil filter (m/s)
A is the surface area of the sand filter (m^2)
h_{max} is the depth of pondage above the sand filter (m)
d is the depth of the filter media (m).



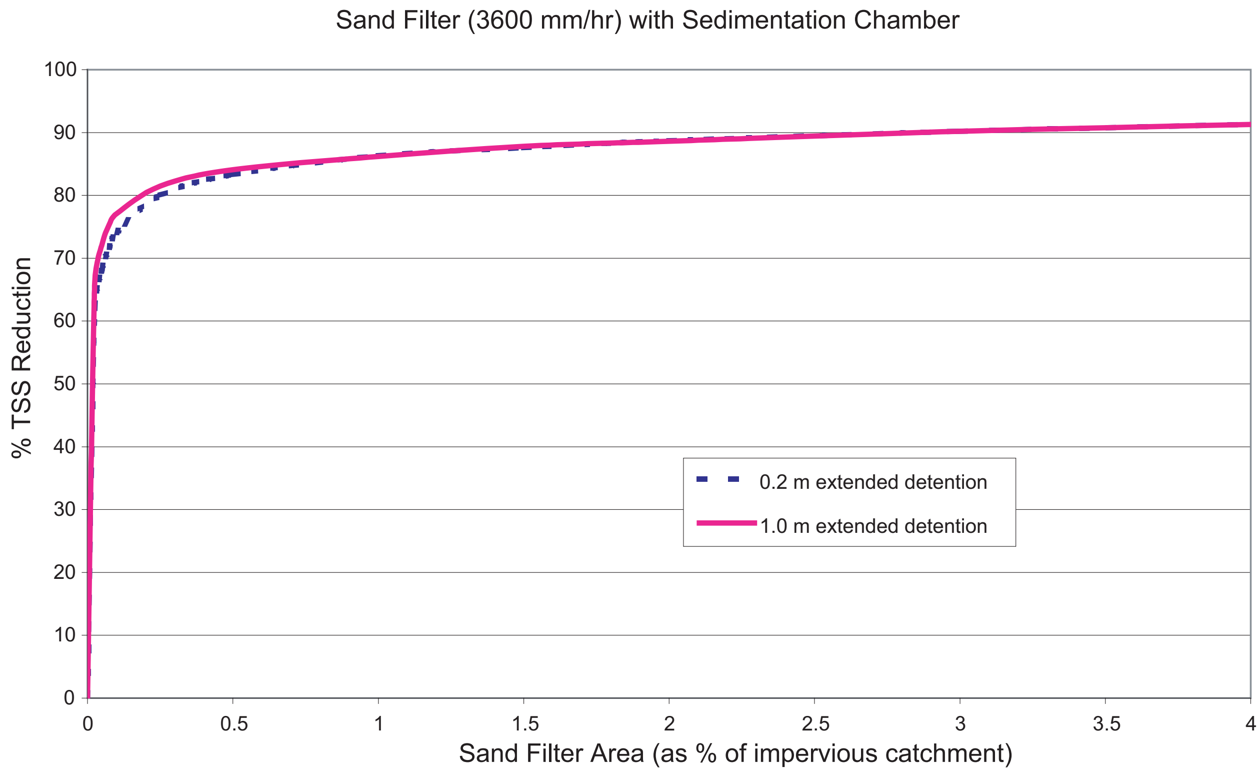
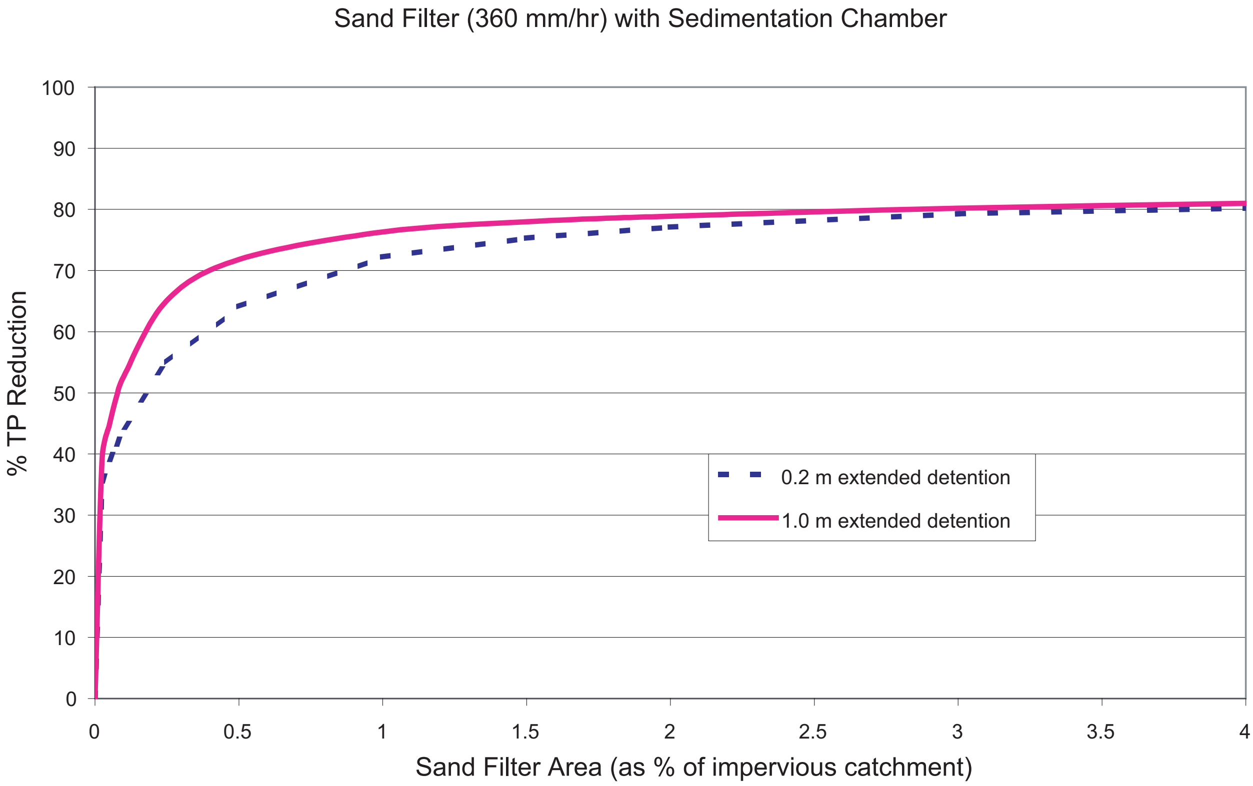


Figure 7.4 Performance of a sand filter in Melbourne in removing total suspended solids in Melbourne.

7.3.2 Hydraulic structure details

7.3.2.1 Sedimentation chamber

Inlet into the sand filter is via the sedimentation chamber. The dimension of this chamber should be sized to retain sediment larger than 125 μm for the design flow and to have adequate capacity to retain settled sediment such that the cleanout frequency is once a year or longer. A



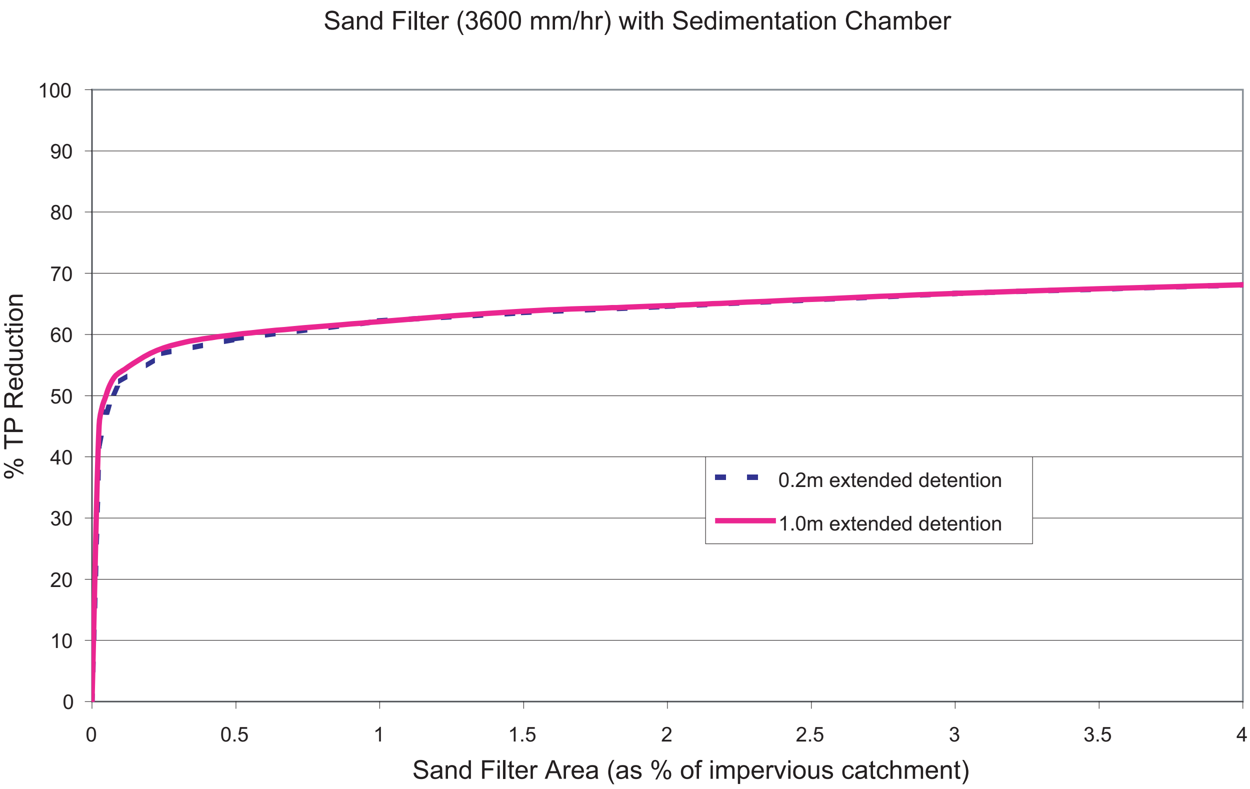
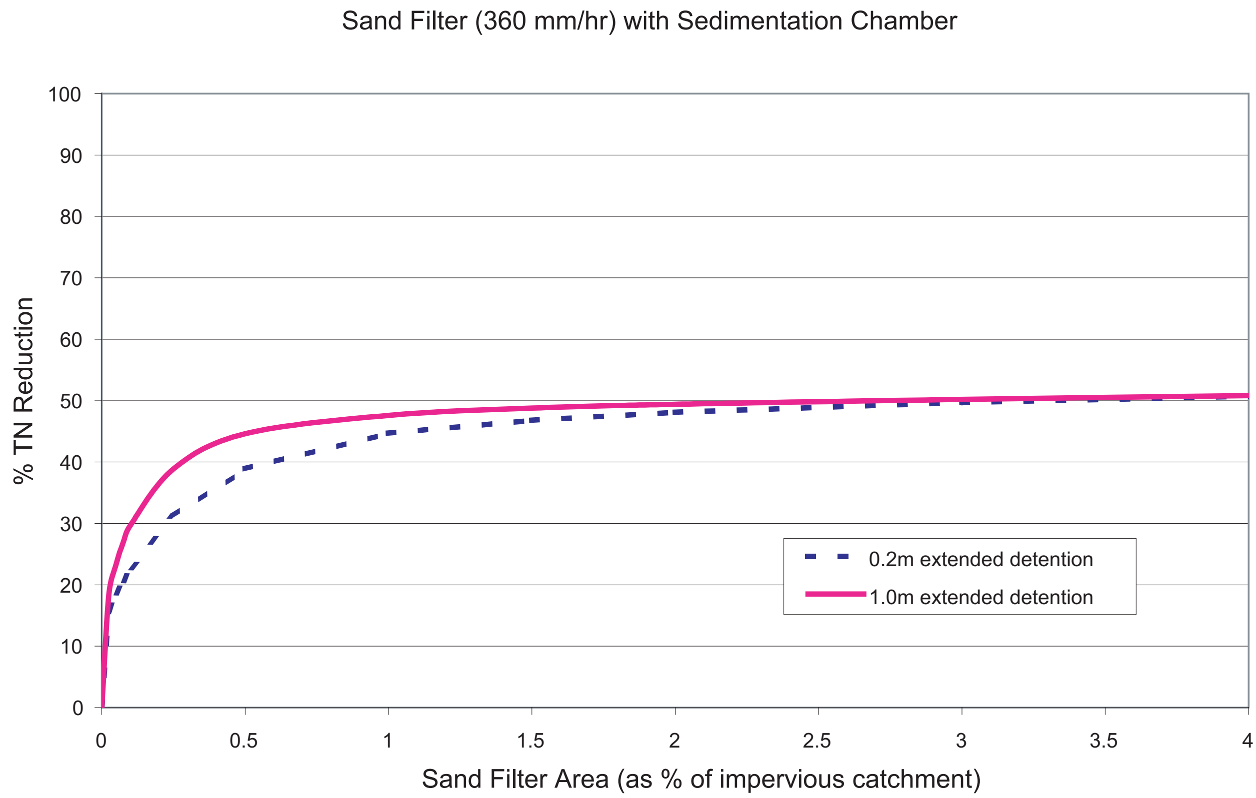


Figure 7.5 Performance of a sand filter in Melbourne in removing TP.

target sediment capture efficient of 70% is recommended. This is lower than the recommendation for sedimentation basins that do not form part of a sand filter (see Chapter 4). With a sand filter, lower capture efficiencies can be supported because of the maintenance regime of the filter media (inspections and either scraping or removal of the surface of the sand filter twice per year) and particle size range in the sand filter being of a similar order of magnitude as the target sediment size of 125 μm. Inspections should also be carried out after



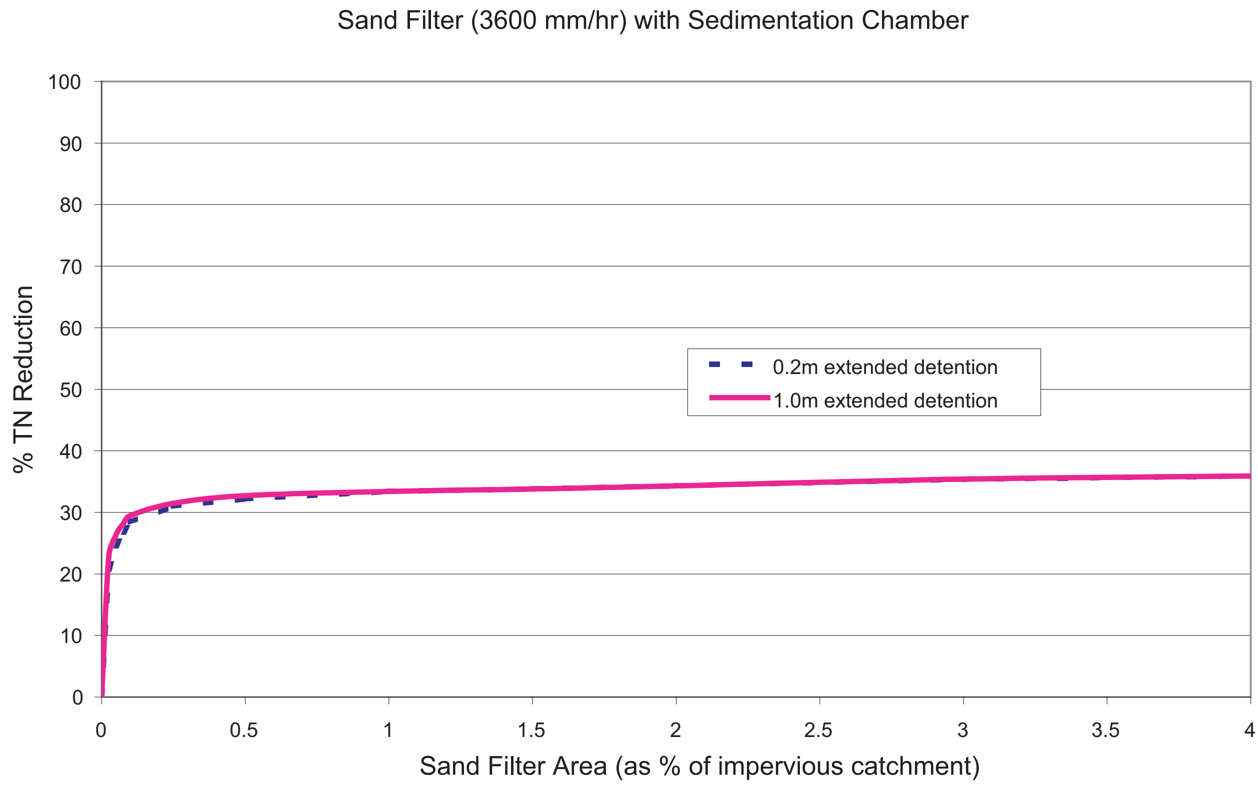


Figure 7.6 Performance of a sand filter in Melbourne in removing total nitrogen (TN).

significant rainfall events soon after the device has been constructed to ensure the sediment and litter loads can be controlled in the sedimentation chamber.

Inspections of the sedimentation chamber would be performed every six months (same as the sand filter); however, sediment clean out may only be required once per year. This will vary from site to site and records of inspections should be kept from each inspection (see Section 7.5.1).

One or more weep holes are also provided when a sedimentation chamber is designed to drain following storm events. Stormwater in the sedimentation chamber is discharged (via surcharge) into the sand filter chamber via a weir during storms. This weir will have a minimum discharge capacity that is equal to the sand filter design flow.

Deposited sediments of the target sediment size or larger should not be resuspended during the passage of the design peak discharge for the overflow chamber. A maximum flow velocity of 0.2 m/s is recommended. Sizing the sedimentation chamber is discussed in Chapter 4.

7.3.2.2 Sand filter chamber

The filter media in the sand filter chamber consist of two layers (i.e. a drainage layer consisting of gravel size material to encase the perforated underdrains and the sand filtration layer). The surface of the sand filter should be set at the crest of the weir connecting the sedimentation chamber to the sand filter chamber. This would minimise any scouring of the sand surface as water is conveyed into the sand filter chamber.

Filter media specifications

A range of particle size ranges can be used for sand filters depending on the likely size of generated sediments. Material with particle size distributions described below has been reported as effective for stormwater treatment, based on Stormwater Management Devices (Auckland Regional Council 2003):

Percentage passing	9.5 mm	100%
	6.3 mm	95%–100%
	3.17 mm	80%–100%
	1.5 mm	50%–85%
	0.8 mm	25%–60%
	0.5 mm	10%–30%
	0.25 mm	2%–10%

Alternatively finer material can be used (e.g. Unimin 16/30 FG sand, details below); however, this requires more attention to maintenance to ensure the material maintains its hydraulic conductivity and does not become blocked. Inspections should be carried out every three months during the initial year of operation as well as after major storms to check for surface clogging.

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

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

Drainage layer specifications

The drainage layer specification can be either coarse sand or fine gravel, such as a 5 mm or 10 mm screenings. Specification of the drainage layer should take into consideration the perforated pipe system, in particular the slot sizes. Fine gravel should be used if the slot sizes are large enough for the sand to be washed into the slots.

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

Impervious liner requirements

Sand filters are considered as conveyance filtration devices rather than infiltration systems. Stormwater is treated via filtration through a specified soil media with the filtrate collected via a subsurface drainage system to be either discharged as treated surface flow or collected for reuse. The amount of water lost to surrounding soils depends largely on local soils and the hydraulic conductivity of the filtration media in the sand filter.

Where sand filters are installed near to significant structures care should be taken to minimise any leakage from the sand filter. The surrounding soils should be tested, including those on the typical hydraulic conductivity.

Should surrounding soils be very sensitive to any seepage from sand filters (e.g. sodic soils, shallow groundwater or close proximity to significant structures), it is necessary to ascertain if the saturated hydraulic conductivity of the surrounding soils is less than one order of magnitude of the filtration media. If this is the case, an impervious liner can be used to contain all water within the sand filter. The liner could be a flexible membrane or a concrete casing. A leakage test should be done immediately after construction to ensure that leakage from the filter does not occur.

7.3.2.3 Overflow chamber

The overflow chamber conveys excess flow to downstream drainage infrastructure and the overflow weir should be sized to ensure that it has sufficient capacity to convey the design discharge from the sedimentation chamber. The overflow weir should be located in the sedimentation chamber.

When water levels in the sedimentation and sand filter chambers exceed the extended detention depth, water will overflow into the overflow chamber and be conveyed into the downstream drainage system. Water levels in the overflow chamber should ideally be lower than the crest of the overflow weir although some level of weir submergence is not expected to severely reduce the discharge capacity of the overflow weir. Water levels should remain below ground when operating at the design discharge for the minor stormwater drainage system.

A broad-crested weir equation can be used to determine the length of the overflow weir:

$$Q_{\text{weir}} = C_{\text{w}} \times L \times H^{1.5} \qquad \text{(Equation 7.2)}$$

where Q_{weir} = flow over the weir
C_{w} is the weir coefficient (~1.7)
L is the length of the weir (m)
H is the **afflux** (m)

7.3.3 Size of slotted collection pipes

Either flexible perforated pipes (e.g. AG pipe) or slotted PVC pipes can be used for the collection pipes; however, care needs to be taken to ensure the slots in the pipes are not so large that sediment would freely flow into the pipes from the drainage layer. The slotted or perforated collection pipes at the base of the sand filter collect treated water for conveyance to downstream drainage infrastructure. They should be sized so that the filtration media are freely drained and the collection system does not become a 'choke' in the system. There are, however, circumstances where it may be desirable to restrict the discharge capacity of the collection system so as to promote a longer detention period within the sand media. One such circumstance is when depth constraints may require a shallower filtration depth and a larger surface area, leading to a higher than desired maximum infiltration rate.

Treated water that has passed through the filtration media is directed into slotted pipes via a 'drainage layer' (typically fine gravel or coarse sand, 2 mm–10 mm diameter). The purpose of the drainage layer is to efficiently convey treated flows into the collection pipes while preventing any of the filtration media from being washed downstream.

It is considered reasonable for the maximum spacing of the slotted or perforated collection pipes to be 1.5 m (centre to centre) so that the distance water needs to travel through the drainage layer does not hinder drainage of the filtration media.

Installing parallel pipes is a means to increase the capacity of the collection pipe system. A 100 mm diameter is considered to be a maximum size for the collection pipes.

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

- the perforations (slots) are adequate to pass the maximum infiltration rate (or the maximum required outflow)
- the pipe itself has sufficient capacity
- the drainage layer has sufficient hydraulic conductivity and will not be washed into the perforated pipes.

7.3.3.1 Perforations inflow check

To estimate the capacity of flows through the perforations ($Q_{perforations}$), orifice flow conditions are assumed and a sharp-edged orifice equation can be used (Equation 7.3). First, the number and size of perforations needs to be determined (typically from manufacturer's specifications) and used to estimate the flow rate into the pipes using a head of the filtration media depth plus the ponding depth. Second, it is conservative but reasonable to use a blockage factor (B) (e.g. 50%–75% blocked) to account for partial blockage of the perforations by the drainage layer media.

$$Q_{perforations} = B \times C \times A_{perforation} \sqrt{2gh} \qquad \text{(Equation 7.3)}$$

where $Q_{perforations}$ is the capacity of flows through the perforations
B is the blockage factor (0.5–0.75)
C is the orifice coefficient (about 0.6)
A is the area of the perforation
h is depth of water over the collection pipe.

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

Prevention of clogging of the perforations is essential and a drainage layer consisting of gravel encasing the slotted pipe is recommended.

7.3.3.2 Perforated pipe capacity

The discharge capacity of the collection pipe (Q_{pipe}) can be calculated simply using an orifice flow equation similar to Equation 7.3:

$$Q_{\text{pipe}} = C \times A_{\text{pipe}} \sqrt{2gh}$$

The capacity of this pipe needs to exceed the maximum infiltration rate.

7.3.4 Design principles to facilitate maintenance

There are several key decisions during the design process that have significant effect on the ability to perform maintenance on a sand filter. As sand filters do not support vegetation, maintenance is paramount to performance, especially in maintaining the porosity of the surface of the sand filtration media.

Easy access is the most important maintenance consideration during design. This includes both access to the site (e.g. traffic management) as well as access to the sedimentation and sand filter chambers (as well as less frequent access to the overflow chamber). Regular inspections are also required, particularly following construction and should be conducted following the first several significant rainfall events. This reinforces the requirement for easy access to the site.

Access into the sand filter chamber is particularly important because of the requirement to remove the fine sediments from the surface layer of the sand filter (top 25 mm–50 mm) from the entire surface area when accumulated fine sediment forms a 'crust'. This may require multiple entry points to the chamber depending in the scale of the filter. If maintenance crews cannot access part of the sand filter chamber, it will quickly become blocked and be unable to improve water quality.

The sedimentation chamber is required to be drained for maintenance purposes (regardless of whether it is designed to drain between storm events). A drainage valve that can drain this chamber needs to be designed into systems that have no weep holes. Having freely drained material significantly reduces the removal and disposal costs from the sedimentation chamber.

The perforated collection pipes at the base of the sand filter are also important maintenance considerations. Provision should be made for flushing (and downstream capture of flushed material) of any sediment build-up that occurs in the pipes. This can be achieved with solid pipe returns to the surface for inspection openings (at the upstream end of the pipes) and a temporary filter sock or equivalent placed over the outlet pipe in the overflow chamber to capture flushed sediment.

7.3.5 Design calculation summary

A *Sand Filters Calculation Checksheet* is included for the key design elements to aid the design process.

Sand Filters	CALCULATION CHECKLIST	
CALCULATION TASK	OUTCOME	CHECK
1 Identify design criteria		
Conveyance flow standard (ARI)	year	
Treatment flow rate (ARI)	year	
Pretreatment objective	μm	
Sand filter area	m^2	
Sand filter depth	m	
Maximum ponding depth	mm	☐
2 Catchment characteristics		
Area	m^2	
Slope	%	
Fraction impervious		☐
3 Estimate design flow rates		
Time of concentration		
Estimate from flow path length and velocities	minutes	☐
Identify rainfall intensities		
Station used for IFD data		
100-year ARI	mm/hr	
1-year ARI	mm/hr	☐
Design runoff coefficient		
C_{10}		
C_{100}		☐
Peak design flows		
Q_1	m^3/s	
Q_{100}	m^3/s	☐
4 Sedimentation chamber		
Required surface area	m^2	
Length:width ratio		
Length x width	m	
Permanent pool depth	m	
Extended detention depth	m	
CHECK SCOUR VELOCITY (depends on particle size)	<0.2 m/s	☐
5 Sand filter chamber		
Inlet weir length	m	
Particle sizes	mm	
Filter saturated hydraulic conductivity	mm/hr	
Extended detention depth	m	
Overflow weir capacity	m^3/s	
CHECK OVERFLOW CAPACITY		
6 Slotted collection pipe capacity		
Pipe diameter	mm	
Number of pipes		
Pipe capacity	m^3/s	
Capacity of perforations	m^3/s	
Soil media infiltration capacity	m^3/s	
CHECK PIPE CAPACITY > SOIL CAPACITY		☐
7 Sand filter properties		
Particle size	% Passing	
	%	
	%	
	%	
	%	
	%	
	%	
	%	☐

7.4 Checking tools

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

Checklists are provided for:

- design assessments
- construction (during and post)

- operation and maintenance inspections
- asset transfer (following defects period).

7.4.1 Design assessment checklist

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

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

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

Land and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the *Asset Handover Checklist* (see Section 7.4.4).

7.4.2 Construction advice

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

Building phase damage

It is important to protect filtration media during the building phase as uncontrolled building site runoff is likely to cause excessive sedimentation, introduce debris and litter and could cause clogging of the sand media. Upstream measures should be employed to control the quality of building site runoff. If a sand filter is not protected during the building phase, it is likely to require replacement of the sand filter media. An additional system of installing a geotextile fabric over the surface of the sand filter during the building phase can also protect the sand filter media below. Accumulated sediment and the geotextile fabric can then be removed after most of the upstream building activity has finished.

Traffic and deliveries

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

Sediment basin drainage

When a sediment chamber is designed to drain between storms (so that pollutants are stored in a drained state) weep holes can be used that are protected from blockage. Blockage can be avoided by constructing a protective sleeve e.g. 5 mm screen (to protect the holes from debris blockage) around small holes at the base of the bypass weir. Sediment basin drainage can also be achieved with a vertical slotted PolyVinyl Chloride (PVC) pipe, with protection from impact and an inspection opening at the surface to check for sediment accumulation. The weep holes should be sized so that they only pass small flows (e.g. 10–15 mm diameter).

Perforated pipes

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

Sand Filter Design Assessment Checklist					
Sand filter location:					
Hydraulics	Minor flood: (m^3/s)		Major flood: (m^3/s)		
Area	Catchment area (ha):		Sand filter area (m^2)		
Treatment				**Y**	**N**
Treatment performance verified from curves?					
Inlet zone/hydraulics				**Y**	**N**
Station selected for IFD appropriate for location?					
Sediment chamber dimensions sufficient to retain 125 µm particles?					
Drainage facilities for sediment chamber provided?					
Overall flow conveyance system sufficient for design flood event?					
Velocities at inlet and within sand filter will not cause scour?					
Bypass sufficient for conveyance of design flood event?					
Collection system				**Y**	**N**
Slotted pipe capacity > infiltration capacity of filter media (where appropriate) ?					
Maximum spacing of collection pipes <1.5 m?					
Drainage layer >150 mm?					
Transition layer provided to prevent clogging of drainage layer?					
Filter Basin				**Y**	**N**
Maximum ponding depth will not impact on public safety?					
Selected filter media hydraulic conductivity > 10x hydraulic conductivity of surrounding soil?					
Maintenance access provided to base of filter media (where reach to any part of a basin >6 m)?					
Protection from gross pollutants provided (for larger systems)?					
Sand media specification included in design?					

Inspection openings in perforated pipes

It is good design practice to have inspection openings at the end of the perforated pipes. The pipes should be brought to the surface (with solid pipes) and have a sealed capping. This allows inspection of sediment build-up when required and easy access for maintenance, such as flushing out of accumulated sediments. Sediment controls downstream should be used when flushing out sediments from the pipes to prevent sediments reaching downstream waterways.

Clean filter media

Ensure drainage media is washed prior to placement to remove fines and prevent premature clogging of the system.

7.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Sand filters

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

SITE: ______________________

CONSTRUCTED BY: ______________________

DURING CONSTRUCTION

Items inspected	Checked Y	Checked N	Satisfactory	Unsatisfactory		Checked Y	Checked N	Satisfactory	Unsatisfactory
Preliminary works	Y	N			**Structural components**	Y	N		
1. Erosion and sediment control plan adopted					14. Location and levels of pits as designed				
2. Traffic control measures					15. Safety protection provided				
3. Location same as plans					16. Pipe joints and connections as designed				
4. Site protection from existing flows					17. Concrete and reinforcement as designed				
Earthworks					18. Inlets appropriately installed				
5. Level bed					19. Pipe joints and connections as designed				
6. Side slopes are stable					20. Concrete and reinforcement as designed				
7. Provision of liner					21. Inlets appropriately installed				
8. Perforated pipe installed as designed					**Filtration system**				
9. Drainage layer media as designed					22. Provision of liner				
10. Sand media specifications checked					23. Adequate maintenance access				
Sedimentation chamber					24. Inlet and outlet as designed				
11. Adequate maintenance access									
12. Invert level correct									
13. Ability to freely drain (weep holes)									

FINAL INSPECTION

1. Confirm levels of inlets and outlets					6. Check for uneven settling of sand				
2. Traffic control in place					7. No surface clogging				
3. Confirm structural element sizes					8. Maintenance access provided				
4. Sand filter media as specified					9. Construction generated sediment and debris removed				
5. Sedimentation chamber freely drains									

COMMENTS ON INSPECTION

ACTIONS REQUIRED

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

7.4.4 Asset handover checklist

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

7.5 Maintenance requirements

Maintenance of sand filters is primarily concerned with:

- regular inspections (every three to six months) to check the sedimentation chamber and the sand media surface
- flows to and through the sand filter
- removal of accumulated sediments and litter and debris from the sedimentation chamber
- ensuring the weep holes and overflow weirs are not blocked with debris.

Maintaining the flow through a sand filter involves regular inspection and removal of the top layer of accumulated sediment. Inspections should be conducted after the first few significant rainfall events following installation and then at least every six months following. The inspections will help to determine the long-term cleaning frequency for the sedimentation chamber and the surface of the sand media.

Removing fine sediment from the surface of the sand media can typically be performed with a flat-bottomed shovel or vacuum machinery. Tilling below this surface layer can also maintain infiltration rates. Access is required to the complete surface area of the sand filter and this needs to be considered during design.

Sediment accumulation in the sedimentation chamber also needs to be monitored. Depending on the catchment activities (e.g. building phase) the deposition of sediment can overwhelm the sedimentation chamber and reduce flow capacities.

Similar to other types of practices, debris removal is an ongoing maintenance function. Debris, if not removed, can block inlets or outlets, and can be unsightly Inspection and removal

Sand Filter Maintenance Checklist			
Inspection frequency: 6 monthly	Date of visit:		
Location:			
Description:			
Site visit by:			
Inspection items	**Y**	**N**	**Action required (details)**
Litter within filter?			
Scour present within sediment chamber or filter?			
Traffic damage present?			
Evidence of dumping (e.g. building waste)?			
Clogging of drainage weep holes or outlet?			
Evidence of ponding?			
Damage/vandalism to structures present?			
Surface clogging visible?			
Drainage system inspected?			
Removal of fine sediment required?			
Comments:			

of debris should be done regularly, but debris should be removed whenever it is observed on the site.

7.5.1 Operation and maintenance inspection form

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

7.6 Sand filter worked example

7.6.1 Worked example introduction

A sand filter system is proposed to treat stormwater runoff from a courtyard/plaza area in Melbourne. The site is nested among several tall buildings and is to be fully paved as a multi-purpose courtyard. Stormwater runoff from the surrounding building is to be directed to bioretention planter boxes while runoff from this 5000 m^2 courtyard will be directed into an underground sand filter as determined by surface levels. Provision for overflow into the underground drainage infrastructure ensures that the site is not subjected to flood ponding for storm events up to the 100-year average recurrence interval. The existing stormwater drainage infrastructure has the capacity to accommodate the 100-year ARI peak discharge from this relatively small catchment.

Key functions of a sand filter include the following:

- promote the capture of gross pollutants
- promote sedimentation of particles larger than 125 μm within the **inlet zone** for flows up to a one-year ARI (unattenuated) peak discharge
- promote filtration of stormwater following sedimentation pretreatment through a sand layer
- provide for bypass operation by configuring and designing the bypass chamber.

The concept design suggests that the required area of the sand filter chamber is 40 m^2 and the depth of the sand filter is 600 mm. Outflows from the sand filter are conveyed into a stormwater pipe for discharge into existing stormwater infrastructure (legal point of discharge) via a third chamber, an overflow chamber. Flows in excess of a 200 mm extended detention depth would overflow and discharge directly into the underground stormwater pipe and bypass the sand filter.

7.6.1.1 Design objectives

The design objectives of a sand filter include:

- three chambers: a sedimentation (and **gross pollutant trapping**, **GPT**) chamber, a sand filter chamber and an overflow chamber
- capture of particles larger than 125 μm for flows up to the peak one-year ARI design flow with a capture efficiency of 80% – the chamber outlet will need to be configured to direct flows up to the one-year ARI into the sand filter, wheras flows in excess of the one-year ARI will bypass to the overflow chamber
- filtration of the peak one-year ARI flow – perforated subsoil drainage pipes are to be provided at the base of the sand filter and will need to be sized to ensure the flow can enter the pipes (check inlet capacity) and to ensure they have adequate flow capacity
- an overflow chamber designed to capture and convey flows in excess of the one-year ARI peak flow and up to the 100-year ARI peak discharge
- a sedimentation chamber to retain sediment and gross pollutants in a dry state and to have sufficient storage capacity to limit sediment clean-out frequency to once per year
- inlet/outlet pipes to be sized to convey the 100-year ARI peak discharge.

7.6.1.2 Site characteristics

The site characteristics are:

- catchment area of 5000 m^2 (100 m × 50 m)
- Paved courtyard land use/surface type
- a 1.0% overland flow slope
- soil is clay
- fraction impervious is 0.90.

7.6.2 Verifying size for treatment

The nominated area of the sand filter is 40 m^2.

A sand filter area of approximately 1% of the impervious area with a hydraulic conductivity of 360 mm/hr will be necessary to attain best practice objectives.

With a fraction impervious of 0.80, the impervious area of the courtyard is 4000 m^2 and the required sand filter area is 40.0 m^2 → OK

Sand filter area provided is adequate.

7.6.3 Estimating design flows

Length of the longest flow path is assumed to consist of an overland flow path ($\frac{1}{2}$ width of the courtyard is 25 m) and gutter flow ($\frac{1}{2}$ perimeter length of the courtyard is 150 m).

The travel time of the overland flow path can be estimated using the overland kinematic wave equation (Equation 7.4) presented in Australian Rainfall and Runoff (Institution of Engineers 2001, Book VIII), i.e.

$$t = \frac{6.94(L \times n^*)^{0.6}}{I^{0.4} \times S^{0.3}} \qquad \text{(Equation 7.4)}$$

where: t is the is the overland travel time (minutes); L is the overland flow path length (m); n^* is the surface roughness (concrete or asphalt ~0.013); I is the design rainfall intensity (mm/hr); and S is the slope.

An iterative application of Equation 7.4 will be required since the travel time will define the time of concentration of the catchment which in turn defines the appropriate design rainfall intensity.

Assuming a time of concentration (t_c) of six minutes: $I^{1}_{6\text{min}}$ = 44 mm/hr; $I^{100}_{6\text{min}}$ = 170 mm/hr.

From Equation 7.4, the overland flow travel times for the one-year and 100-year storms are calculated to be 3 min and 2 min, respectively. Gutter flow travel time, estimated from an assumed flow velocity of 1 m/s, is 2 min–3 min, giving an estimate total travel time of between 4 min and 6 min → OK to adopt a 6 min time of concentration.

Design rainfall intensities are I_1 = 44 mm/hr; I_{100} = 170 mm/hr.

Design runoff coefficients are computed using the method outlined in *Australian Rainfall and Runoff* (Institution of Engineers 2001, Book VIII).

$^{10}I_1$ = 28.6 mm/hr; f = 0.80

$C^1_{10} = 0.1 + 0.0133\ (^{10}I_1 - 25) = 0.15$, where C^1_{10} is the pervious runoff coefficient

$C_{10} = 0.9f + C^1_{10}\ (1\ [-]\ f)$, where f is the fraction impervious, 0.8.

C_{10} = 0.75.

From Table 1.6 in Institution of Engineers 2001 Book VII;

$C_1 = 0.8 \times C_{10} = 0.60$

$C_{100} = 1.2 \times C_{10} = 0.90$.

Peak design flows (Q) are calculated using the Rational Method as follows:

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

where C is the runoff coefficient; I is the design rainfall intensity (mm/hr); A is the catchment area (ha).

Q_1 = 0.037 m^3/s.

Q_{100} = 0.21 m^3/s.

7.6.3.1 Maximum infiltration rate

The maximum infiltration rate (Q_{max}) through the sand filter is computed using Equation 7.1 (Darcy's equation):

$$Q_{max} = k \times A \times \frac{h_{max} + d}{d} = 0.05\ \text{m}^3/\text{s}$$

where k is the hydraulic conductivity of sand = 1×10^{-4} m/s (Engineers Australia 2003, Ch. 9); A is the surface area of the sand filter, 40 m^2; h_{max} is the depth of pondage above the sand filter = 0.2 m; d is the depth of the sand filter = 0.6 m.

Design flows Q_1 = 0.004 m^3/s; Q_{100} = 0.21 m^3/s;
Maximum infiltration rate = 0.005 m^3/s.

7.6.4 Hydraulic structures

7.6.4.1 Sizing of sedimentation basin

The sedimentation chamber is to be sized to remove the 125 μm particles for the peak one-year flow.

Pollutant removal is estimated using Equation 4.3 (see Chapter 4):

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

A notional aspect ratio of 1 (W) to 2 (L) is adopted. From Figure 4.3, the hydraulic efficiency (λ) is 0.3. The turbulence factor (***n***) is computed from Equation 4.2 to be 1.4.

Hydraulic efficiency (λ) = 0.3
Turbulence factor (**n**) = 1.4.

The proposed extended detention depth of the basin is 50 mm (0.05 m) (see Section 7.6.1) and a notional **permanent pool** depth of 0.95 m (equal to the depth of the sand filter) has been adopted:

d_p = 0.95 m

d^* = 0.95 m

d_e = 0.05 m

V_s = 0.011 m/s for 125 μm particles

Q = design flow rate = 0.37 m^3/s.

The required sedimentation basin area to achieve target sediment (125 μm) capture efficiency of 70% is 7 m^2. With a W to L ratio of 1:2, the notional dimensions of the basin are 2 m × 3.5 m. This size is validated against the curves presented in Figure 4.2 (see Chapter 4).

The available sediment storage is 7 × 0.95 = 6.7 m^3. Clean-out is to be scheduled when the storage is half full. Using a sediment discharge rate of 1.6 m^3/ha per year, the clean-out frequency is estimated to be:

$$\text{Frequency of basin desilting} = \frac{0.5 \times 6.7}{0.7 \times 1.6 \times 0.5} \text{ years} > 1 \text{ year} \rightarrow \text{OK}$$

During the 100-year ARI storm, peak discharge through the sedimentation chamber will be 0.21 m^3/s with flow depth of 0.95 m. It is necessary to check that flow velocity does not resuspend deposited sediment of 125 μm or larger (≤ 0.2 m/s).

The mean velocity in the chamber is calculated as follows:

$$V_{100} = 0.21/(2 \times 0.95) = 0.11 \text{ m/s} \rightarrow \text{OK}$$

The length of the sedimentation chamber is 3.5 m. Provide 0.2 m high slots of total length of 2 m connecting it to the sand filter chamber. The connection discharge capacity ($Q_{connection}$) should be greater than the 100-year ARI peak flow (0.21 m^3/s) and can be calculated using the broad-crested weir equation (Equation 7.2) as follows:

$$Q_{connection} = C_w \, L \, H^{1.5}$$

where: C_w is the weir coefficient (assume = 1.4 for a broad-crested weir); H is the afflux = 0.2 m (2 m) weir; L is the length of the weir.

The discharge capacity calculated from the above equation is 0.25 m^3/s >> 100-year ARI discharge of 0.21 m^3/s.

Sedimentation chamber = 7 m^2
Width = 2 m; Length = 3.5 m
Total weir length of connection to sand filter chamber = 2 m
Depth of chamber from weir connection to sand filter = 0.6 m
Depth of Extended Detention (d_e) = 0.05 m.

7.6.4.2 Sand filter chamber

Dimensions

With the length of sedimentation chamber being 3.5 m, the dimension of the sand filter chamber is determined to be 3.5 m × 11.5 m, giving an area of 40.25 m^2.

Sand filter chamber dimension: 3.5 m × 11.5 m.

Media specifications

Sand filter layer to consist of sand material with a typical particle size distribution (based on a Unimin 16/30 FG sand grading) is provided:

Percentage passing	1.4 mm	100%
	1.0 mm	80%
	3.17 mm	44%
	1.5 mm	8.4%

The drainage layer is to consist of fine gravel, of 5 mm screenings.

No impervious liner is necessary as *in situ* soil is clay.

The filter layer is to be 600 mm deep and consist of sand with approximately 50% finer than 1 mm diameter. The drainage layer to be 200 mm deep and consist of 5 mm gravel.

7.6.4.3 Overflow chamber

A weir set at 0.95 m from the base of the sedimentation chamber (or 0.2 m above the surface of the sand filter) of 2 m length needs to convey flows up to the 100-year ARI peak discharging from the sand filter chamber into the overflow chamber.

To calculate the afflux resulting from conveying the 100-year ARI peak discharge through a 2 m length weir, perform the following:

$$H = \left(\frac{Q_{wer}}{C_w \times L}\right)^{0.667} = 0.16 \text{ m, say } 0.2 \text{ m} \qquad \text{(Equation 7.5)}$$

where: Q_{weir} is the design discharge = 0.21 m^3/s
C_w is the weir coefficient (~1.7)
L is the length of the weir (m)
H is the afflux (m).

With an afflux of 0.2 m, the discharge capacity of the overflow weir is 0.30 m^3/s > 100-year ARI peak flow of 0.23 m^3/s.

Crest of overflow weir = 0.2 m above surface of sand filter
Length of overflow weir = 2 m
100-year ARI afflux = 0.2 m
Roof of facility to be at least 0.4 m above sand filter surface.

7.6.5 Size of slotted collection pipes

7.6.5.1 Perforations inflow check

The following are the characteristics of the selected slotted pipe:

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

For a pipe length of 3.5 m, the total number of slots = 2100/(1.5 × 7.5) = 186.

Discharge capacity of each slot can be calculated using the orifice flow equation (Equation 7.3), i.e.

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

where: h is the head above the slotted pipe, calculated to be 0.80 m; C is the orifice coefficient (about 0.6). The inflow capacity of the slotted pipe is thus
$2.67 \times 10^{-5} \times 186 = 5 \times 10^{-3}$ m^3/s per metre of length.

If a blockage factor of 0.5 is adopted, this gives the inlet capacity of each slotted pipe to be 2.5×10^{-3} m^3/s per metre of length.

Maximum infiltration rate is 0.005 m^3/s. Therefore, the minimum length of slotted pipe ($L_{\text{slotted pipe}}$) required is:

$$L_{\text{slotted pipe}} = 0.005/2.5 \times 10^{-3} = 2 \text{ m}$$

The minimum recommended pipe spacing is 1.5 m (refer Section 7.3.3),
therefore, six slotted pipes (3.5 m length) at 1.5 m spacing are required.

7.6.5.2 Slotted pipe capacity

The diameter of the slotted pipe is 100 mm. The discharge capacity of the collection pipe is calculated using an orifice flow equation (Equation 7.3):

$$Q_{\text{pipe}} = C \times A_{\text{pipe}}\sqrt{2gh} = 0.019 \text{ m}^3/\text{s} \qquad \text{(Equation 7.6)}$$

Total discharge capacity (six pipes) = 0.11 m^3/s > maximum infiltration rate of 0.005 m^3/s → OK

Combined slotted pipe discharge capacity = 0.11 m^3/s
and this exceeds the maximum infiltration rate.

7.6.6 Design calculation summary

The completed *Sand Filters Calculation Checksheet* shows the results of the design calculations.

Sand Filters	CALCULATION SUMMARY		
CALCULATION TASK	OUTCOME		CHECK
1 Identify design criteria			
Conveyance flow standard (ARI)	100	year	
Treatment flow rate (ARI)	1	year	
Pretreatment objective	125	µm	
Sand filter area	10	m^2	
Sand filter depth	0.6	m	
Maximum ponding depth	200	mm	✓
2 Catchment characteristics			
Area	5000	m^2	
Slope	1	%	
Fraction impervious	0.9		✓
3 Estimate design flow rates			
Time of concentration			
Estimate from flow path length and velocities	6	minutes	✓
Identify rainfall intensities			
Station used for IFD data	Melbourne		
100-year ARI	170	mm/hr	
1-year ARI	44	mm/hr	✓
Design runoff coefficient			
C_{10}	0.82		
C_{100}	0.99		✓
Peak design flows			
Q_1	0.04	m^3/s	
Q_{100}	0.24	m^3/s	✓
4 Sedimentation chamber			
Required surface area	7	m^2	
Length:width ratio	1(W):2(L)		
Length x width	2 x 3.5	m	
Permanent pool depth	0.95	m	
Extended detention depth	0.05		
CHECK SCOUR VELOCITY (depends on particle size)	0.11	<0.2 m/s	✓
5 Sand filter chamber			
Inlet weir length	2	m	
Particle sizes	0.7	mm	
Filter saturated hydraulic conductivity	360.0	mm/hr	
Extended detention depth	0.2	m^3/s	
Overflow weir capacity	0.3	m^3/s	
CHECK OVERFLOW CAPACITY	YES		✓
6 Slotted collection pipe capacity			
Pipe diameter	100	mm	
Number of pipes	6		
Pipe capacity	0.11	m^3/s	
Capacity of perforations	0.05	m^3/s	
Soil media infiltration capacity	0.005	m^3/s	
CHECK PIPE CAPACITY > SOIL CAPACITY	YES		✓
7 Sand filter properties			
Percent Passing 1.40	100	%	
Unimin 16/30 FG 1.18	96	%	
1.00	80	%	
0.85	63	%	
0.71	44	%	
0.60	24	%	
0.50	8	%	
0.425	1	%	✓

7.6.7 Construction drawings

The following page shows the construction drawing for the worked example.

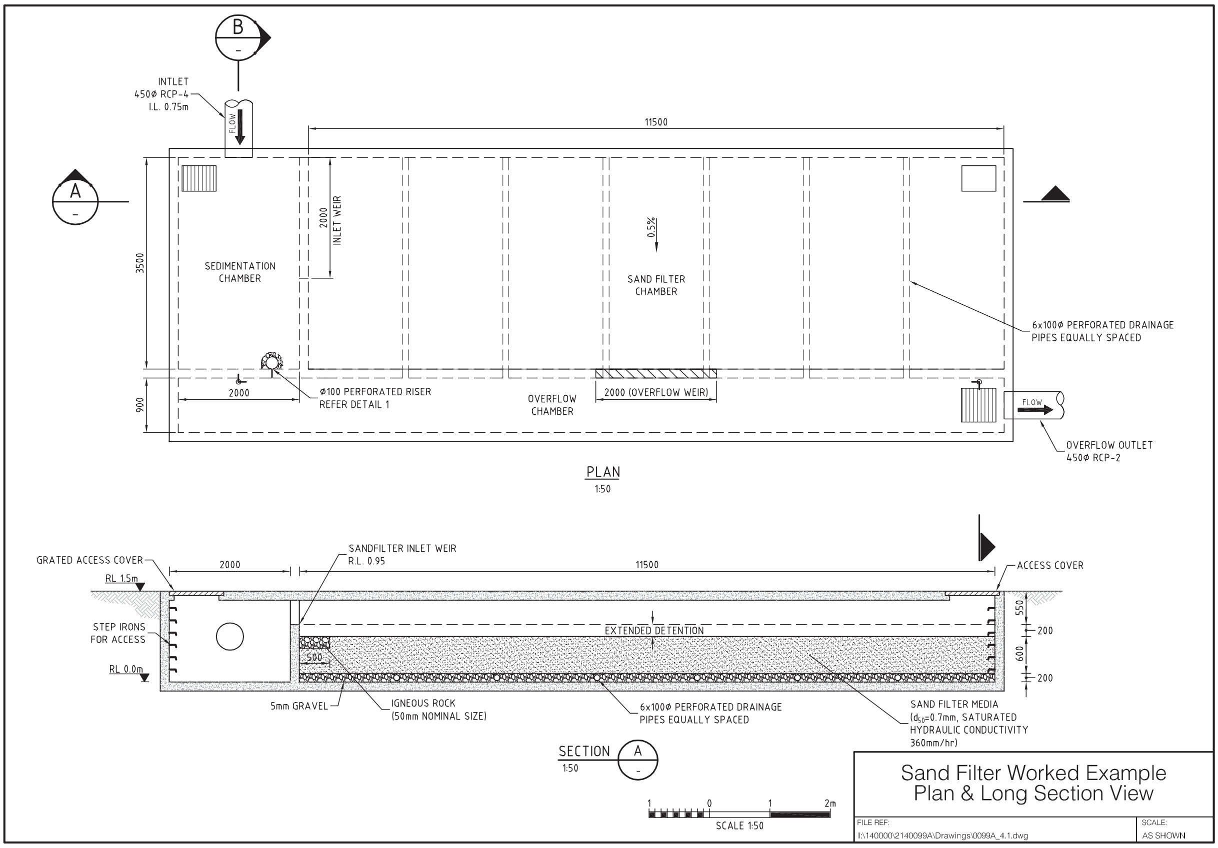


Figure 7.8 Plan and long section view

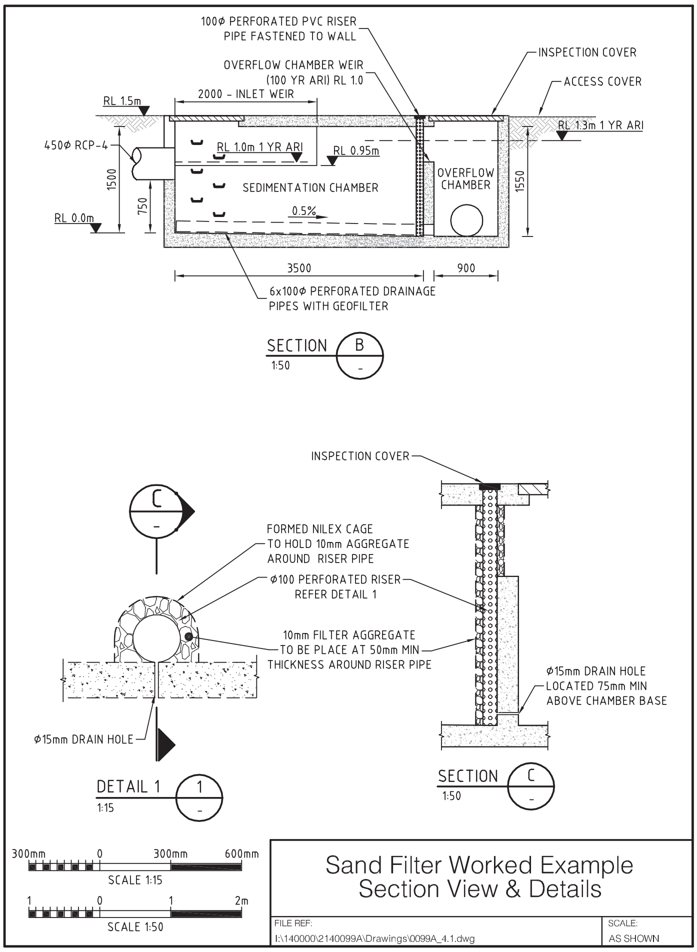


Figure 7.9 Section view and details

7.7 References

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

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

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

Institution of Engineers Australia (1997). *Australian Rainfall and Runoff – A Guide to Flood Estimation*, Pilgram, D.H. (Ed.), Institution of Engineers, Australia, Barton, ACT.

Chapter 8 **Swales and buffer strips**

Swale systems can be attractive elements in urban developments

8.1 Introduction

Vegetated **swales** are used to convey **stormwater** in lieu of pipes and to provide for removal of coarse and medium sediment and are commonly combined with **buffer** strips. The system uses overland flow and mild slopes to slowly convey water downstream. Swales also provide a disconnection of impervious areas from hydraulically efficient pipe drainage systems. This results in slower travel times, thus reducing the impact of increased **catchment** imperviousness on peak flow rates.

Figure 8.1 shows illustrations of a vegetated swales with different versions of driveway crossings, including at-grade crossings (with mild side slopes) and with elevated crossings.

The interaction between flow and vegetation along swales facilitates pollutant settlement and retention. Swale vegetation acts to spread and slow velocities, which in turn aids sediment deposition. Swales alone can rarely provide sufficient treatment to meet objectives for all pollutants, but can provide an important pretreatment function for other **Water Sensitive Urban Design (WSUD)** measures. They are particularly good at coarse sediment removal and can be incorporated in street designs to enhance the aesthetics of an area.

Buffer strips (or buffers) are areas of vegetation through which runoff passes while travelling to a **discharge** point. They reduce sediment loads by passing a shallow depth of flow though vegetation and rely upon well-distributed shallow flows across them. Interaction with the vegetation tends to slow velocities and coarse sediments are retained. Buffers can be used as edges to swales, particularly where flows are distributed along the banks of the swale.

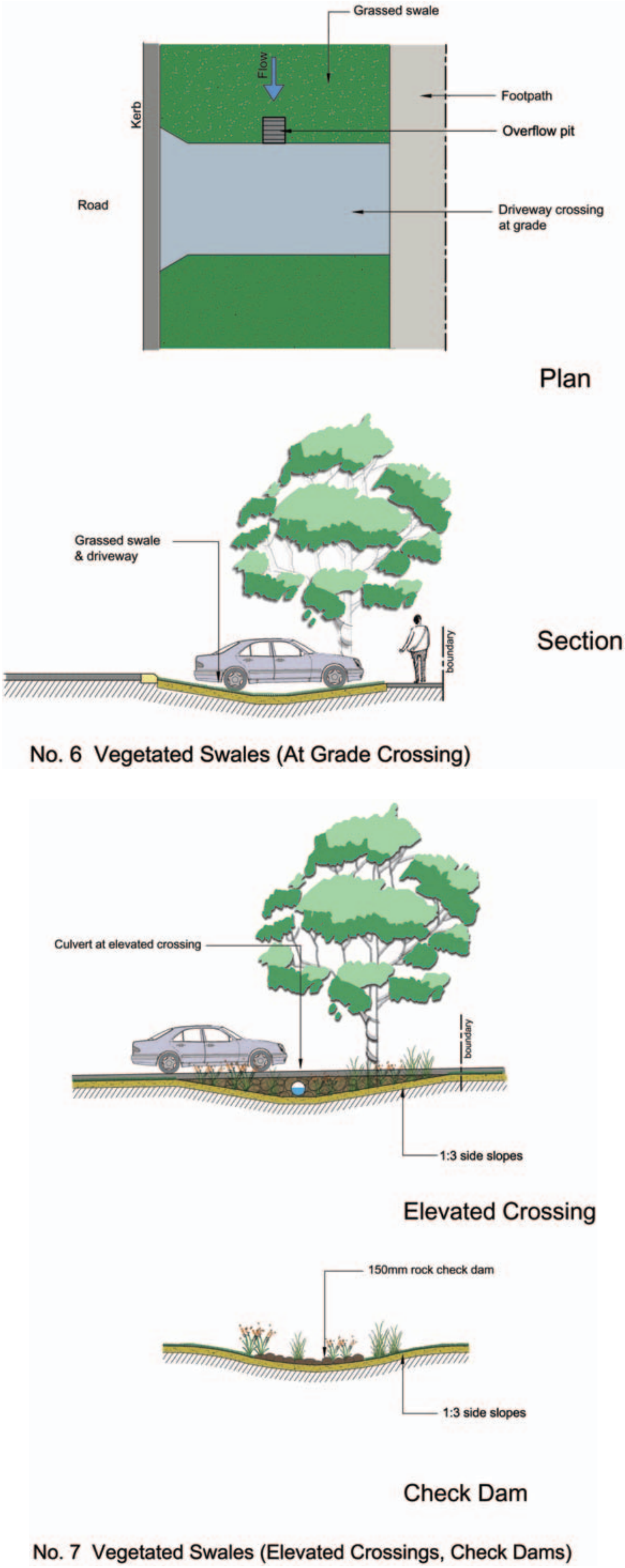


Figure 8.1 Swales with at-grade driveway crossing (plan view and section), elevated crossing and a check dam for flow spreading.

Figure 8.2 Swale systems: (L to R) heavily vegetated; use of check dams; grass swale with elevated crossings.

To convey flood flows along swales, in excess of a treatment design flow (typically the peak three-month ARI (Average Recurrence Interval) flow), pits draining to underground pipes can be used. Water surcharges from the swale down the pit. This is particularly useful in areas that have narrow verges, where a swale can only accommodate flows associated with the minor drainage system (e.g. five-year ARI for a certain length.

The longitudinal slope of a swale is the most important consideration in their design. They generally operate best with between 1% and 4% slopes. Slopes milder than this can tend to become waterlogged and have stagnant ponding because of the difficulty in constructing swales with small tolerances. However, shallow underdrains or a thin sand layer can alleviate this problem by providing a drainage path for small depressions along a swale. For slopes steeper than 4%, **check banks** (small porous rock walls) along swales can help to distribute flows evenly across the swales as well as reduce velocities.

Swales can use a variety of vegetation types including turf, sedges and tussock grasses. Vegetation is required to cover the whole width of the swale, be capable of withstanding design flows and be of sufficient density to provide good filtration. For best performance, the vegetation height should be above the treatment flow water level.

Grassed swales are commonly used and can appear as a typical road verge; however, the short vegetation offers sediment retention only to shallow flows. In addition, the grass is required to be mown and well maintained for the swale to operate effectively. Denser vegetated swales can offer improved sediment retention by slowing flows more and providing filtration for deeper flows. Conversely, vegetated swales have higher hydraulic roughness and therefore require a larger area to convey flows compared to grass swales. These swales can become features of a landscape, will require minimal maintenance once established, and be hardy enough to withstand large flows.

Another key consideration when designing swales is road or driveway crossings. Crossings can provide an opportunity for **check dams** (to distribute flows) or to provide temporary ponding above a bioretention system (refer to Section 8.3.5.2). A limitation with 'elevated' crossings can be their expense compared to at-grade crossings (particularly in dense urban

Figure 8.3 Elevated and at-grade driveway crossings across swales.

developments), safety concerns with traffic movement adjacent to the inlet and outlet and the potential for blockage of relatively small culvert systems.

Crossings can also be constructed at grade and act like a ford during high flows; however, this reduces maximum swale **batter slopes** to about 1 in 9 (with a flat base) to allow for traffic movement. These systems can be cheaper to construct than elevated crossings but require more space. They are well suited to low density developments.

Swales can also be constructed as centre medians in divided roads and in this case would also enhance the aesthetics of the street. This also avoids issues associated with crossings.

Traffic and deliveries needs to be kept off swales. Traffic (should swales be used for parking) can tend to ruin the vegetation and provide ruts that cause preferential flow paths that do not offer filtration. Traffic control can be achieved by selecting swale vegetation that discourages the movement of traffic or by providing physical barriers to traffic movement. For example, barrier kerbs with breaks in them (to allow distributed water entry, albeit with reduced uniformity of flows compared with flush kerbs) or bollards along flush kerbs can be used to prevent vehicle movement onto swales.

With flood flows being conveyed along a swale surface, it is important to ensure velocities are kept low to avoid scouring of collected pollutants and vegetation. These devices can be installed at various scales, for example, in local streets or on large highways.

The design process for swales involves firstly designing the system for conveyance and secondly ensuring the system has features that maximise its treatment performance.

Key design issues to be considered are:

1. verifying treatment performance and relationship to other measures in a treatment train
2. determining design flows
3. sizing the swale with site constraints
4. checking above-ground design:
   - velocities
   - slopes
   - design of **inlet zone** and overflow pits
   - above-design flow operation
5. making allowances to preclude traffic on swales
6. recommending plant species and planting densities
7. providing maintenance.

8.2 Verifying size for treatment

The curves below (Figures 8.4–8.9) show the pollutant removal performance expected for swales with varying slopes (1%, 3% and 5%) and vegetation height (0.05–0.5 m). Swales in isolation provide limited treatment for fine pollutants, but can perform pretreatment for other measures.

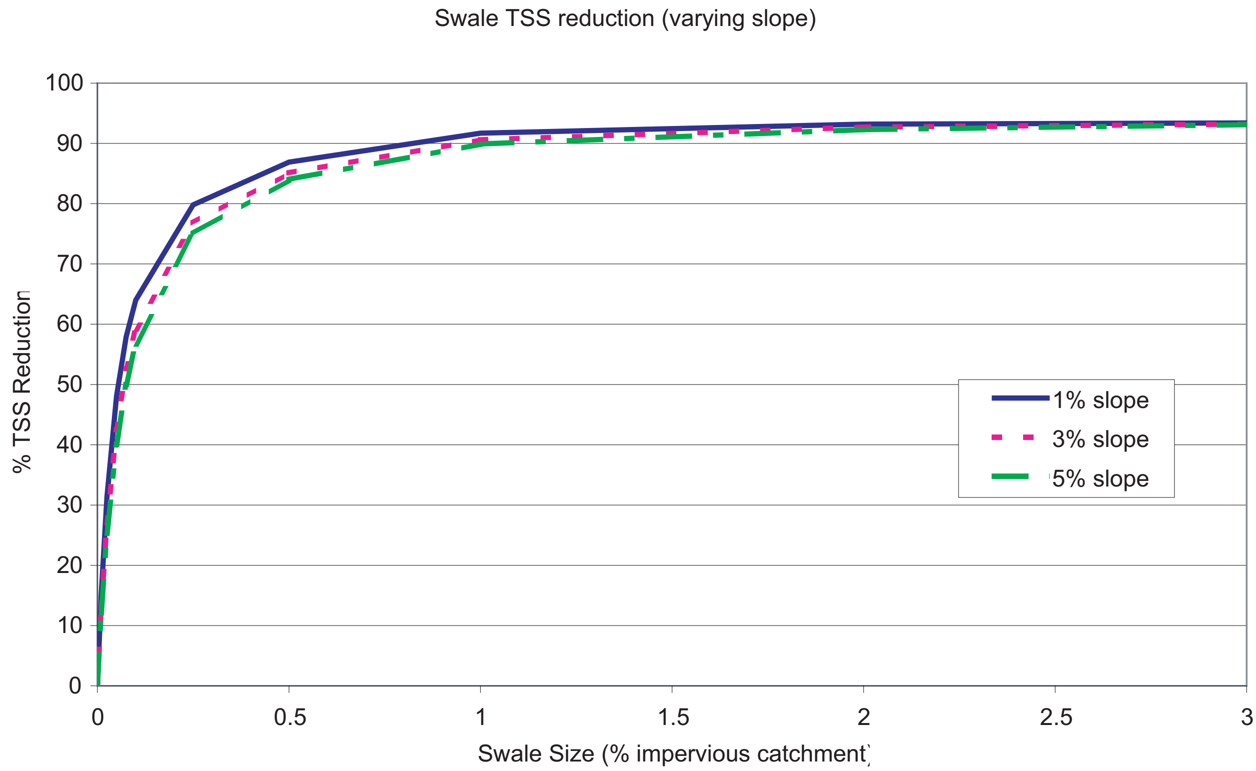


Figure 8.4 Performance of a swale in removing Total Soluble Solids (TSS) in Melbourne with varying channel slopes (vegetation height = 0.25 m).

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

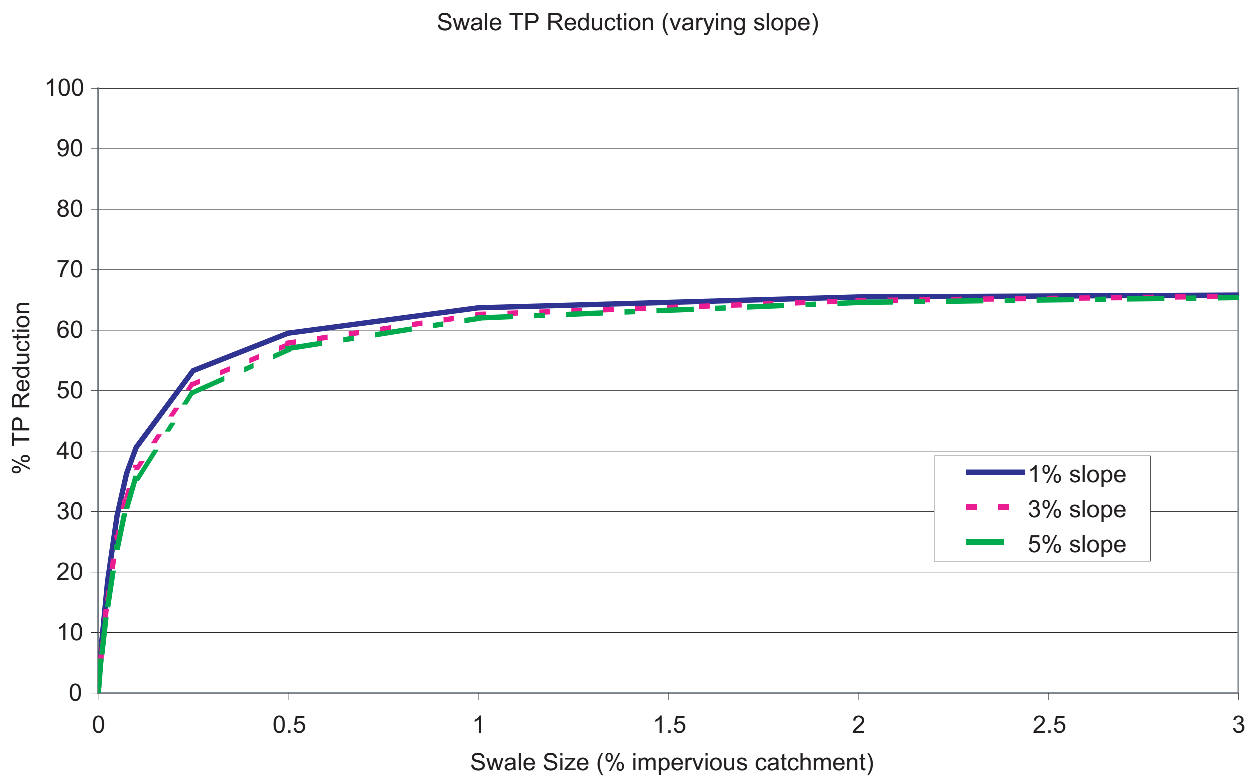


Figure 8.5 Performance of a swale in removing Total Phosphorus (TP) in Melbourne with varying channel slopes (vegetation height = 0.25 m).

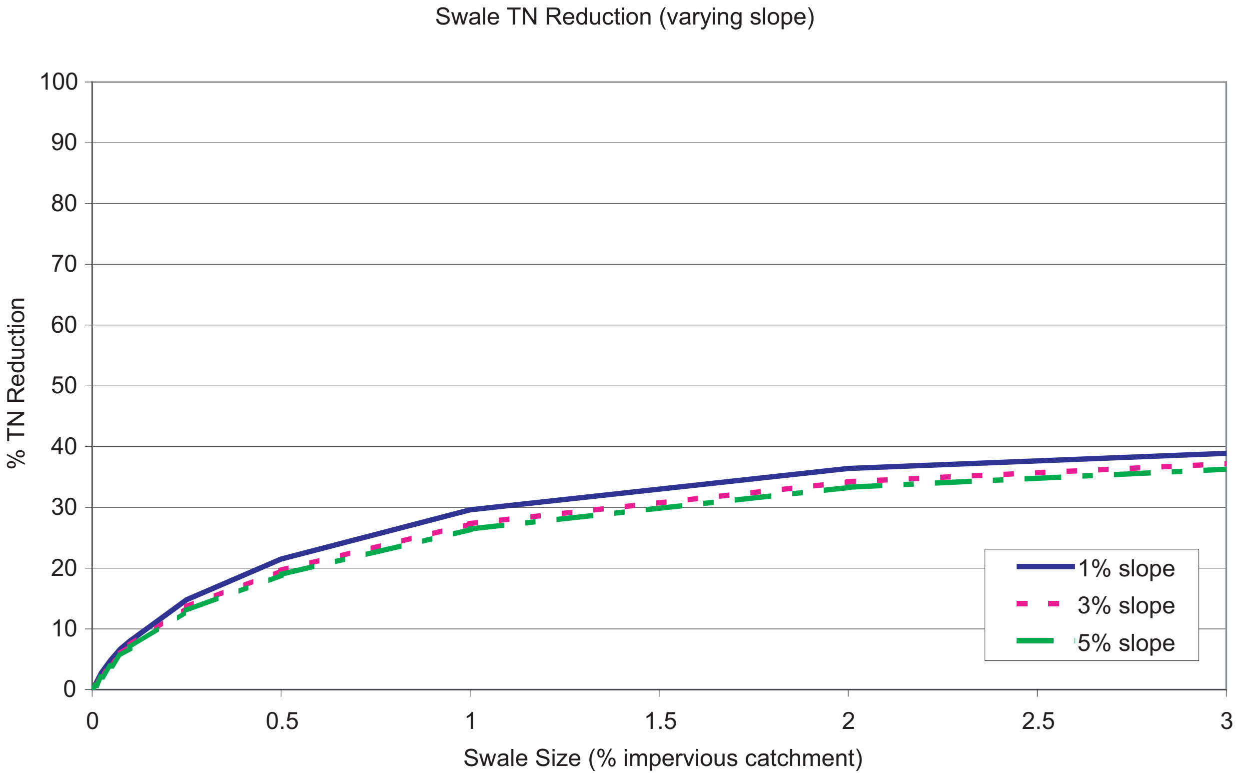


Figure 8.6 Performance of a swale in removing Total Nitrogen (TN) in Melbourne with varying channel slopes (vegetation height = 0.25 m).

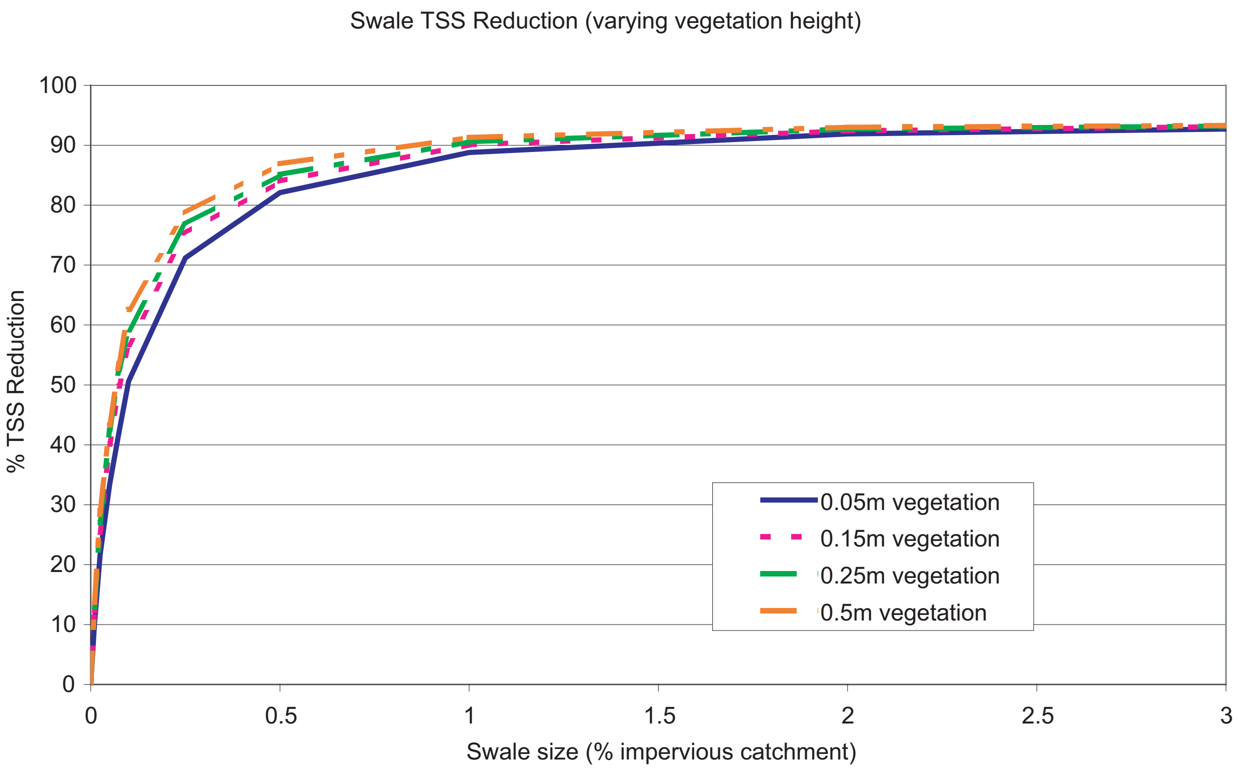


Figure 8.7 Performance of a swale in removing Total Soluble Solids (TSS) in Melbourne with varying vegetation height (channel slope = 3%).

site, see Chapter 2). In preference to using the curves, local data should be used to model the specific treatment performance of the system.

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

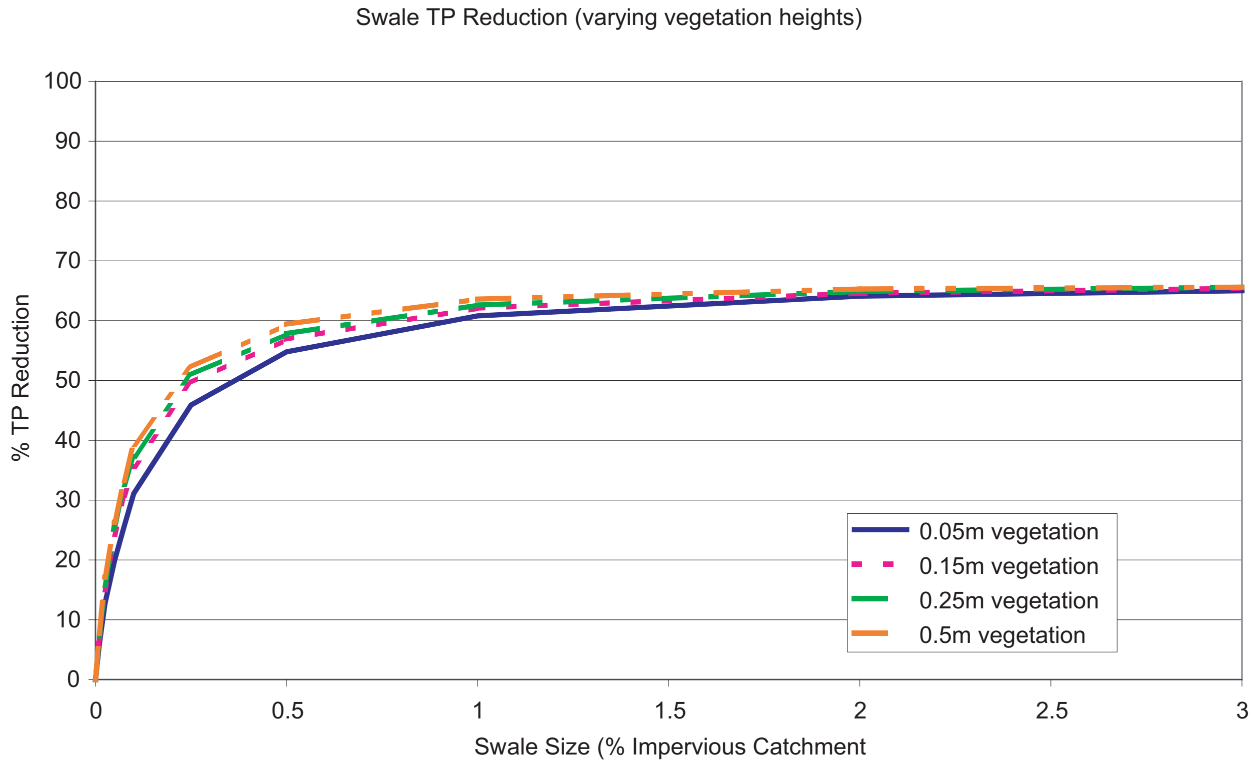


Figure 8.8 Performance of a swale in removing Total Phosphorus (TP) in Melbourne with varying vegetation height (channel slope = 3%).

Figure 8.9 Performance of a swale in removing Total Nitrogen (TN) in Melbourne with varying vegetation height (channel slope = 3%).

- base width of 2 m
- top width of 6 m
- 1 in 6 side slopes
- no infiltration through the base of the swale.

These curves can be used to check the expected performance of swales for removal of Total Soluble Solids (TSS), Total Phosphorus (TP) and Total Nitrogen (TN) with similar cross sections to the dimensions assumed above. If dimensions of a swale vary significantly from the values above, more detailed modelling of performance should be conducted. The *swale size* is represented as the *top width of the swale times its length* divided by the contributing *impervious catchment*.

8.3 Design procedure: swales

The following sections describe the design steps required for swale systems.

8.3.1 Estimating design flows

Two design flows are required for swale systems:

- minor flood rates (typically five-year ARI) to size the overflows to allow minor floods to be safely conveyed and not increase any flooding risk compared to conventional stormwater systems
- major flood rates (typically 100-year ARI) to check that flow velocities are not too large in the swale, that could potentially scour pollutants or damage vegetation.

8.3.1.1 Minor and major flood estimation

A range of hydrologic methods can be applied to estimate design flows. With typical catchment areas being relatively small, the **Rational Method** Design Procedure (Institution of Engineers 1987) is considered to be a suitable method for estimating design flows.

8.3.2 Dimensioning a swale

Constraints relating to a swale alignment and size need to be identified before a swale size can be checked against its flow capacity requirements. Iterations between these factors and an urban concept design may be necessary. Many of these factors should be considered during concept design; nevertheless, these should also be checked during detail design. Factors to be considered are:

- allowable width, given urban layout
- how flows are delivered into a swale (e.g. cover requirements for pipes or kerb details)
- longitudinal slope
- maximum side slopes and base width
- provision of crossings (elevated or at-grade).

Depending on which of the above factors are fixed, other variables can be 'gamed' to derive an acceptable swale configuration.

Once design flows are established, either a swale is sized to convey a particular flood frequency or the maximum length of swale is determined for a particular flood frequency. The calculation steps are identical in either approach. The following sections outline some considerations in relation to dimensioning a swale.

8.3.2.1 Side slopes and maximum width of a swale

A maximum width of swale is usually determined from an urban layout, particularly in redevelopment scenarios. This maximum width needs to be identified early in the design process as it informs the remainder of the swale design.

Alternatively, calculations can be made to estimate a required swale width to accommodate a particular flow (e.g. conveyance as the minor drainage system) to inform an urban design. Other considerations that may influence a swale width are how water is delivered to it and the maximum batter slopes (which can be affected by crossing types).

Selection of an appropriate side slope depends heavily on local council regulations and will be related to traffic access and the provision of crossings (if required). The provision of driveway crossings can significantly affect the required width of the swale. The slope of at-grade crossings (therefore, the swale) are governed by the trafficability of the change in slope across the base of the swale. Typically 1 in 9 side slopes with a small flat base will provide sufficient transitions to allow for suitable traffic movement for at-grade crossings.

Where narrower swales are required, elevated crossings can be used (with side slopes typically of between 1 in 3 and 1 in 6) and these will require provision for drainage under the crossings with a culvert or similar.

Crossings can provide good locations for overflow points in a swale. However, the distance between crossings will determine the feasibility of having overflow points at each one.

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

8.3.2.2 Maximum length of a swale

In many urban situations, the length of a swale is determined by the maximum allowable width and side slopes (therefore, depth). A swale of a set dimension (and vegetation type) will be capable of conveying flows up to a specific rate, after which flows will overtop the banks. This point is considered the maximum length of a swale. Overflow pits can be used in these situations where flows surcharge into underground pits and underground pipe networks for conveyance. A swale thus can be adjacent to a long length of road; however, it will not convey flows from an entire upstream catchment.

Manning's equation is used to size the swale, given the site conditions. This calculation is sensitive to the selection of **Manning's *n*** and this should vary according to flow depth (as it decreases significantly once flow depths exceed vegetation height). Consideration of the landscape and maintenance of the vegetation will need to be made before selecting a vegetation type.

8.3.3 Swale capacity – selection of Manning's *n*

To calculate the flow capacity of a swale, Manning's equation can be used. This allows the flow rate (*Q*) and levels to be determined for variations in dimensions, vegetation type and slopes.

$$\text{Manning's } Q = (A \times R^{2/3} \times S_o^{1/2}) \qquad \text{(Equation 8.1)}$$

Where A = cross-sectional area
R = hydraulic radius
S_o = channel slope
n = roughness factor.

Manning's n is a critical variable in the Manning's equation that relates to roughness of the channel. It varies with flow depth, channel dimensions and the vegetation type. For constructed swale systems, the values are recommended to be between 0.15 and 0.4 for flow depths shallower than the vegetation height (preferable for treatment) and can be significantly lower (e.g. 0.03) for flows with greater depth than the vegetation (however, it can vary greatly with channel slope and cross-section configuration) (see Cooperative Research Centre for Catchment Hydrology 2003, Appendix E).

It is considered reasonable for Manning's n to have a maximum at the vegetation height and then sharply reduce as depths increase (e.g. Figure 8.10). It is reasonable to expect the shape of the Manning's n relationship with flow depth to be consistent with other swale configurations, with the vegetation height at the boundary between 'Low flows' and 'Intermediate flows' (Figure 8.10) on the top axis of the diagram. The bottom axis of the plot has been modified from Barling and Moore (1993).

8.3.4 Inlet details

Inlets for swale systems can be from distributed runoff (e.g. from flush kerbs along a road) or from point outlets such as pipes. Combinations of these two entrance pathways can also be used.

8.3.4.1 Distributed flows (buffers)

An advantage of flows entering a swale system in a distributed manner (i.e. entering perpendicular to the direction of the swale) is that flow depths are shallow which maximises contact with vegetation. This area is often called a buffer. The requirement of the area is to ensure there is dense vegetation growth, flow depths are kept shallow (below the vegetation height) and erosion is avoided. This provides good pretreatment prior to flows being conveyed

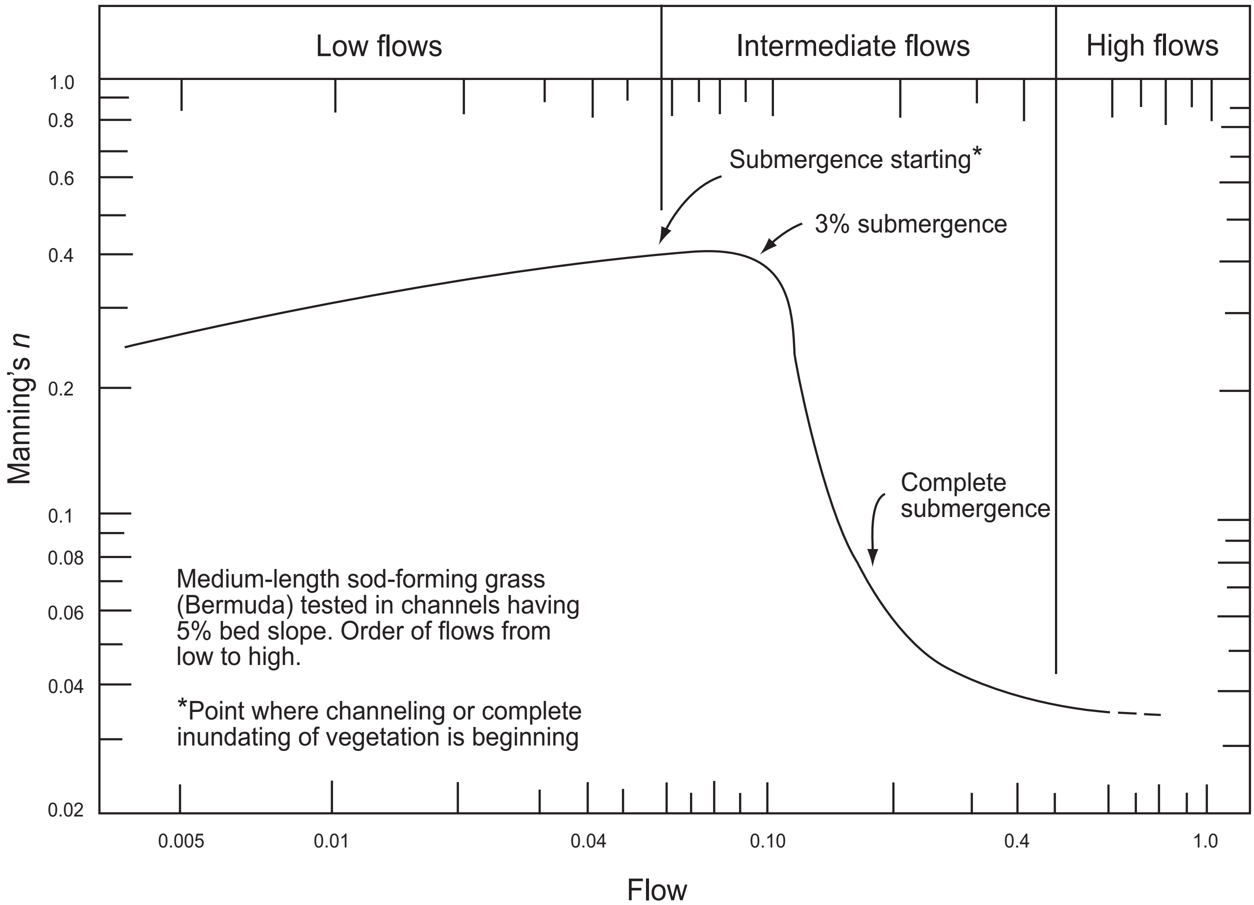


Figure 8.10 The effect of flow depth on hydraulic roughness (after Barling and Moore 1993).

down a swale. Creating distributed flows can be achieved either by having a flush kerb (Figure 8.11) or by using kerbs with regular breaks in them to allow for even flows across the buffer surface (Figure 8.12).

For distributed flows, it is important to provide an area for coarse sediments to accumulate (i.e. off the road surface). Sediment will accumulate on a street surface where the vegetation is the same level as the road (Figure 8.11). To avoid this accumulation, a tapered flush kerb can be used that sets the top of the vegetation between 40 mm and –50 mm lower than the road surface (Figure 8.11, diagram), which requires the top of the ground surface (before turf is placed) to be between 80 mm and –100 mm below the road surface. This allows sediments to accumulate off any trafficable surface.

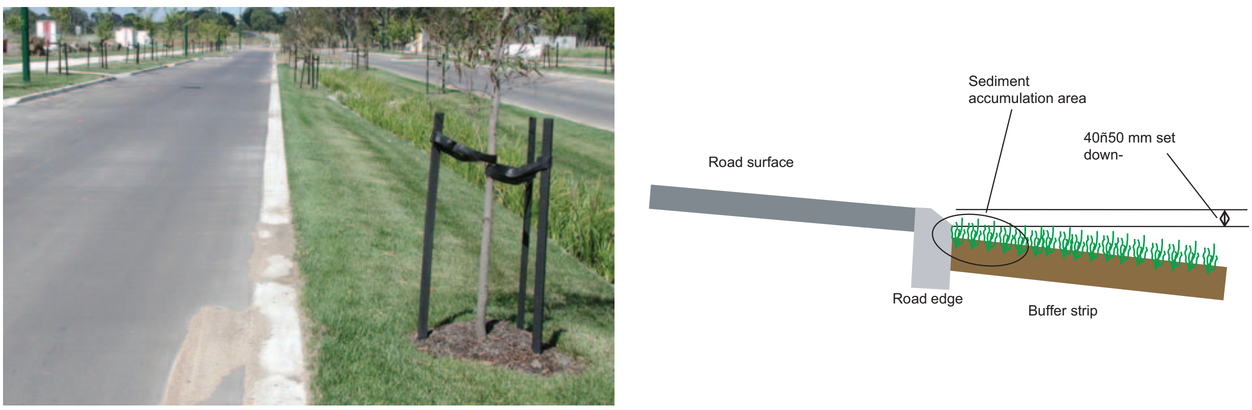


Figure 8.11 A flush kerb without setdown that shows accumulation of sediment on the street surface, and edge detail showing a recommended amount of setdown.

Figure 8.12 Different arrangements of kerbs with breaks to distribute inflows.

8.3.4.2 Direct entry points

Direct entry of flows can be from either overland flow or from a pipe system. For all point entrances into swales, it is important to consider energy dissipation at the inlet point to minimise any erosion potential. This can usually be achieved with rock beaching and dense vegetation.

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

In situations where geometry does not permit the pipe to reach the surface, a 'surcharge' pit can be used to bring flows to the surface. Surcharge pits should be designed so that they are as shallow as possible and have pervious bases to avoid long-term ponding in the pits (this may require underdrains to ensure drainage, 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.

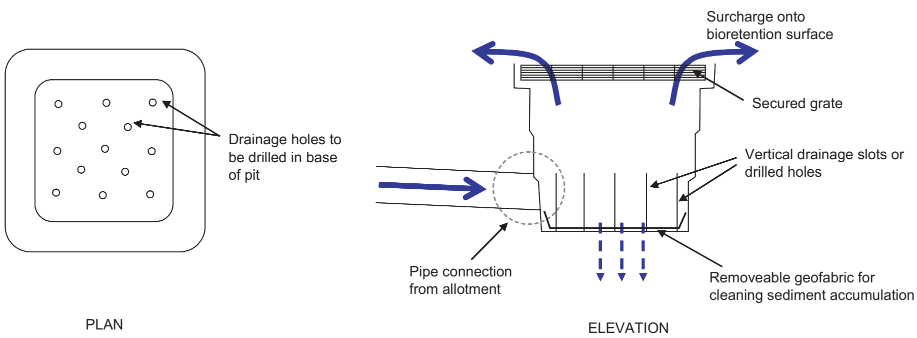


Figure 8.13 A surcharge pit for discharging allotment runoff into a swale.

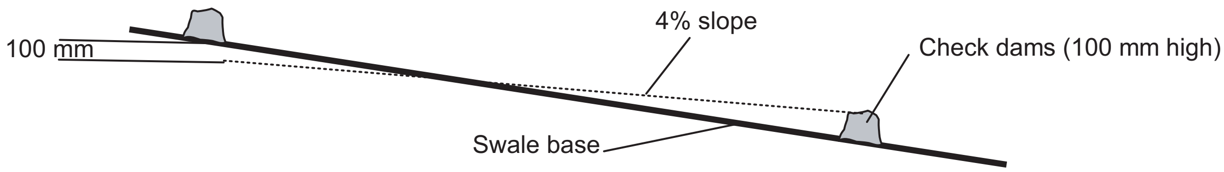


Figure 8.14 Location of check dams in swales.

These systems are most frequently used when allotment runoff is required to cross a road into a swale on the opposite side. Several allotments can generally be combined prior to crossing the road to minimise the number of road crossings. Figure 8.13 shows an example of a surcharge pit discharging into a swale.

8.3.5 Vegetation scour velocity check

Scour velocities over the vegetation along the swale are checked by applying Manning's equation. Selection of Manning's n needs to be appropriate to suit the vegetation height (see Section 8.3.3).

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

- less than 0.5 m/s for minor storm (e.g. five-year ARI) discharges
- less than 1.0 m/s for major storm (e.g.100-year ARI) discharges.

8.3.5.1 Velocity check – safety

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

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

8.3.5.2 Check dams

For steep swales (> 4%), check dams can be used to help distribute flows across a swale to avoid preferential flow paths and maximise contact with vegetation. Check dams are typically low level (e.g. 100 mm) rock weirs or driveway crossings that are constructed across the base of a swale. A rule of thumb for locating check dams is for the crest of a downstream check dam to be at 4% grade from 100 mm below the toe of an upstream check dam (Figure 8.14).

8.3.6 High-flow route and overflow design

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

The locations of overflow pits is variable, but it is desirable to locate them just upstream of crossings to reduce flows across the crossing.

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

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

For free overfall conditions (weir equation):

$$Q_{minor} = B \times C \times L \times H^{3/2} \qquad \text{(Equation 8.2)}$$

with B = blockage factor (0.5), C = 1.7 and H = available head above the weir crest

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

For drowned outlet conditions (orifice equation):

$$Q_{minor} = B \times C \times A\sqrt{2gh}$$

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

8.3.7 Vegetation specification

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

8.3.8 Design calculation summary

Swales — **CALCULATION CHECKLIST**

CALCULATION TASK	OUTCOME		CHECK
1 Identify design criteria			
Conveyance flow standard (ARI)		year	
Vegetation height		mm	☐
2 Catchment characteristics			
		m²	
		m²	
Slope		%	
Fraction impervious			
f_{imp}			☐
3 Estimate design flow rates			
Time of concentration			
Estimate from flow path length and velocities		minutes	☐
Identify rainfall intensities			
Station used for IFD data:			
Major flood – 100-year ARI		mm/hr	
Minor flood – 5-year ARI		mm/hr	
Peak design flows			
Q_{minor}		m³/s	
Q_{100}		m³/s	☐
4 Swale design			
Manning's *n* below vegetation height			
Manning's *n* at capacity			☐
5 Inlet details			
Adequate erosion and scour protection?			
Flush kerb setdown?		mm	☐
6 Velocities over vegetation			
Velocity for 5-year flow (<0.5 m/s)		m/s	
Velocity for 100-year flow (<1.0 m/s)		m/s	
Safety: Vel x Depth (<0.4)		m²/s	☐
7 Overflow system			
Spacing of overflow pits			
Pit type			☐
8 Plant selection			☐

8.4 Checking tools

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

Checklists are provided for:

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

8.4.1 Design assessment checklist

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

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

In addition to the*Checklist*, a proposed design should have all necessary permits for its installations. The referral agency should ensure that all relevant permits are in place.

Land and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the *Asset Handover Checklist* (see Section 8.4.4).

8.4.2 Construction advice

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

Building phase damage

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

Traffic and deliveries

Ensure traffic and deliveries do not access swales during construction. Traffic can compact the filter media and cause preferential flow paths, deliveries can smother vegetation. Washdown wastes (e.g. concrete) can disturb vegetation and cause uneven slopes along a swale. Swales should be fenced off during building phase and controls implemented to avoid washdown wastes.

Inlet erosion checks

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

Sediment build-up on roads

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

Timing for planting

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

Swale Design Assessment Checklist					
Swale location:					
Hydraulics	Minor flood: (m³/s)		Major flood: (m³/s)		
Area	Catchment area (ha):				
Treatment				**Y**	**N**
Treatment performance verified from curves?					
Inlet zone/hydraulics				**Y**	**N**
Station selected for IFD appropriate for location?					
Longitundinal 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 amenity?					
Overflow pits provided where flow capacity exceeded?					
Inlet flows appropriately distributed?					
Energy dissipation provided at inlet?					
Velocities within swale cells will not cause scour?					
Set down of at least 50 mm below kerb invert incorporated?					
Cells				**Y**	**N**
Maximum ponding depth and velocity will not impact on public safety (V x D <0.4)?					
Maintenance access provided to invert of conveyance channel?					
Protection from gross pollutants provided (for larger systems)?					
Vegetation				**Y**	**N**
Plant species selected can tolerate periodic inundation and design velocites?					
Plant species selected integrate with surrounding landscape design?					

8.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Wetlands

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

SITE: ______________________

CONSTRUCTED BY: ______________________

DURING CONSTRUCTION									
Items inspected	Checked		Satisfactory	Unsatisfactory		Checked		Satisfactory	Unsatisfactory
Preliminary works	Y	N				Y	N		
1. Erosion and sediment control plan adopted					18. Concrete and reinforcement as designed				
					19. Inlets appropriately installed				
2. Limit public access					20. Inlet energy dissipation installed				
3. Location same as plans					21. No seepage through banks				
4. Site protection from existing flows					22. Ensure spillway is level				
5. All required permits in place					23. Provision of maintenance drain(s)				
Earthworks					24. Collar installed on pipes				
6. Integrity of banks					25. Low flow channel rocks are adequate				
7. Batter slopes as plans					26. Protection of riser from debris				
8. Impermeable (eg. clay) base installed					27. Bypass channel stabilised				
9. Maintenance access to whole wetland					28. Erosion protection at macrophyte outlet				
10. Compaction process as designed					**Vegetation**				
11. Placement of adequate topsoil					29. Vegetation appropriate to zone (depth)				
12. Levels as designed for base, benches, banks and spillway (including freeboard)					30. Weed removal prior to planting				
13. Check for groundwater intrusion					31.Provision for water level control during establishment				
14. Stabilisation with sterile grass					32. Vegetation layout and densities as designed				
Structural components									
15. Location and levels of outlet as designed					33. Provision for bird protection				
16. Safety protection provided					34. By-pass channel vegetated				
17. Pipe joints and connections as designed									
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					9. Check for uneven settling of banks				
2. Confirm structural element sizes					10. Evidence of stagnant water, short circuiting or vegetation scouring				
3. Check batter slopes									
4. Vegetation planting as designed					11. Evidence of litter or excessive debris				
5. Erosion protection measures working					12. Provision of removed sediment drainage area				
6. Pre-treatment installed and operational									
7. Maintenance access provided					13. Evidence of debris in high flow bypass				
8. Public safety adequate					14. Macrophyte outlet free of debris				

COMMENTS ON INSPECTION

ACTIONS REQUIRED

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

8.4.4 Asset handover checklist

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

8.5 Maintenance requirements

Swale systems treat runoff by filtering it through vegetation and then passing the runoff downstream. Treatment relies upon contact with vegetation and, therefore, maintaining vegetation growth is the main maintenance objective. In addition, they are used for flood conveyance and need to be maintained to ensure adequate flood protection for local properties.

The potential for rilling and erosion along a swale needs to be carefully monitored, particularly during establishment stages of the system.

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

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

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

Maintenance is primarily concerned with:

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

Vegetation maintenance will include:

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

Sediment accumulation at the inlet points needs to be monitored. Depending on the catchment activities (e.g. building phase), the deposition of sediment can tend to smother plants and reduce the ponding volume available. Should excessive sediment build-up, it will affect 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. Inspection and removal of debris should be done regularly, but debris should be removed whenever it is observed on a site.

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

8.5.1 Operation and maintenance inspection form

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

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

8.6 Swale worked example

8.6.1 Worked example introduction

As part of a development in Ballarat, runoff from allotments and a street surface is to be collected and conveyed in a vegetated swale system to downstream treatments, the intention being for a turf swale system. An additional exercise in this worked example is to investigate the consequences on flow capacity of using a vegetated (e.g. sedges) swale (vegetation height equal to 300 mm).

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

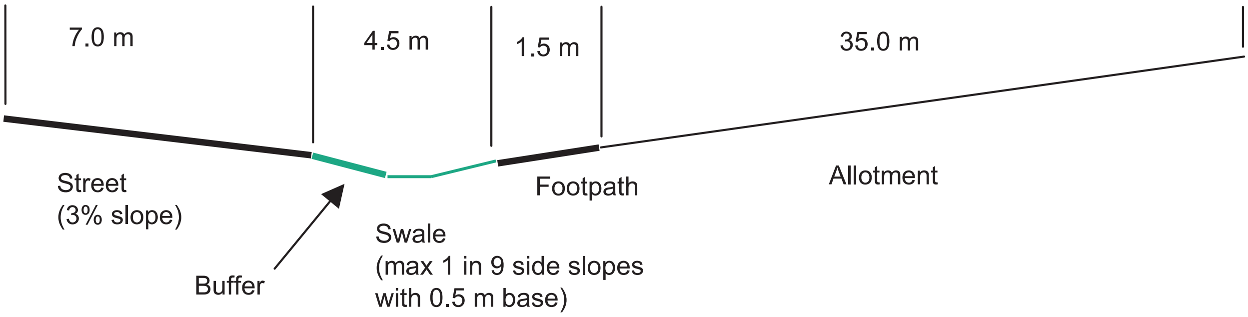


Figure 8.15 Cross section of proposed buffer/swale system.

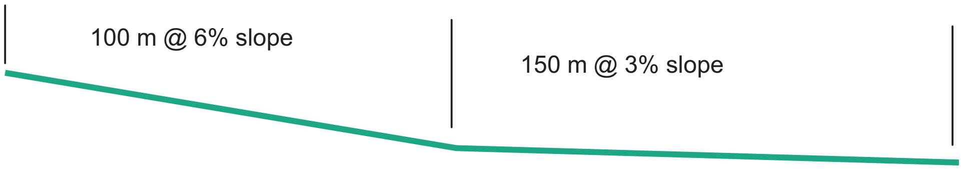


Figure 8.16 Long section of proposed buffer/swale system.

The swale is to convey minor flood events, including all flows up to a five-year ARI storm. However, the width of the swale is fixed (at 4.5 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. Access to the allotments will be via an at-grade crossover with a maximum slope of 1 in 9 (11%).

The contributing catchment area includes 35 m deep (10 m wide) allotments on one side, a 7 m wide road pavement surface and a 1.5 m footpath, and 4.5 m swale and services easement (Figure 8.15). The area is 250 m long with the top 100 m having a 6% slope and the bottom 150 m having a 3% slope (Figure 8.16).

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

The 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 given side slope limits
- be able to convey five-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:

Figure 8.17 Similar buffer swale system for conveying runoff.

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

Additional design elements will be required, including:

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

8.6.1.1 Design objectives

The design objectives of the swale are to:

- convey at least all flows up to the peak five-year ARI storm event
- promote sedimentation of coarse particles within the buffer by providing an even flow distribution
- prevent traffic damage to the buffer swale system
- control flow velocities to prevent erosion
- allow for suitable driveway gradients (max 1:9) to be provided at crossovers into properties.

8.6.1.2 Site characteristics

The site characteristics for the swale are as follows.

Catchment area	Lots	8 750 m^2
	Roads and concrete footpath	2 125 m^2
	Swale and services easement	1 125 m^2
	Total	12 000 m^2.
Landuse/surface type	Residential lots, roads/concrete footpaths, swale and service easement.	
Overland flow slope:	Total main flow path length = 250 m	
	Upper section = 100 m at a 6% slope	
	Lower section = 150 m at a 3% slope.	
Soil type:	Clay	
Fraction impervious:	Lots $f = 0.65$.	
	Roads/footpath $f = 1.00$	
	Swale/service easement $f = 0.10$.	

8.6.1.3 Confirm size for treatment

Interpretation of Figures 8.4 to 8.9 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. Note that the treatment areas need to be adjusted to the equivalent areas at the reference site (Melbourne) using the hydrologic design region **adjustment factors** to interpret Figure 8.4 to 8.9.

- Ballarat location (Western Plains hydrologic design region)
- Average slope of 5% along swale
- Vegetation height of 50 mm.

To interpret the graphs the area of swale base to the impervious catchment needs to be estimated. Then this percentage needs to be adjusted back to the equivalent area for the Melbourne region (the reference site). This value can then be used to interpret the performance graphs in Figure 8.4 to 8.9.

Area of swale base / impervious catchment area:

$$0.5 \times 250/[(0.65 \times 8750) + (1.0 \times 2125) + (0.1 \times 1125)] = 1.6\%.$$

Adopting the Western Plains hydrologic design region adjustment factor equation for swales (see Chapter 2, Table 2.1):

Adjustment factor = 0.539 (MAR) + 0.622
= 0.539 (0.70) + 0.622 = 0.999.

Therefore, the area required in Melbourne $\times$ 0.999 = area required in Ballarat.

To apply the performance curves the area = 1.6%/0.999 = 1.6%.

From the figures using an equivalent area in the reference site, it is estimated that pollutant reductions are 90%, 63% and 28% for TSS, TP and TN, respectively.

8.6.2 Estimating design flows

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

8.6.2.1 Major and minor design flows

Time of concentration (t_c)

Approach:

The time of concentration is estimated assuming overland flow across the allotments and along the swale. From procedures in *Australian Rainfall and Runoff* (Institution of Engineers 2001), t_c is estimated to be 10 minutes.

Design rainfall intensities

Adopt the values from Intensity–Frequency Duration (IFD) table for Ballarat:

t_c	Five-year	100-year
10 min	67 mm/hr	140 mm/hr

Design runoff coefficient

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

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

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

$f = (8750 \times 0.65 + 2125 \times 1 + 1125 \times 0.1)/12\ 000 = 0.66.$

$^{10}I_1 = 30.1$ mm/hr (Ballarat)

$C^1_{10} = 0.17\ C_{10} = 0.65$

$C_y = F_y\ C_{10}$

$C_5 = 0.95 \times 0.65 = 0.62.$

$C_{100} = 1.2 \times 0.65 = 0.78.$

Peak design flows

As it is a small catchment the peak design flows (Q) are calculated by using the Rational Method as follows:

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

$Q_5 = 0.002788 \times 0.62 \times 67 \times 1.2 = 0.14\ m^3/s$

$Q_{100} = 0.002788 \times 0.78 \times 140 \times 1.2 = 0.36\ m^3/s$

C = runoff coefficient

A = area (ha)

I = rainfall intensity (mm/hr)

8.6.3 Swale dimensions

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

8.6.4 Swale flow capacity

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

The worked example first considers the swale capacity using a grass surface with a vegetation height of 50 mm. An extension of the worked example is to investigate the consequence of using 300 mm high vegetation (e.g. sedges) instead of grass.

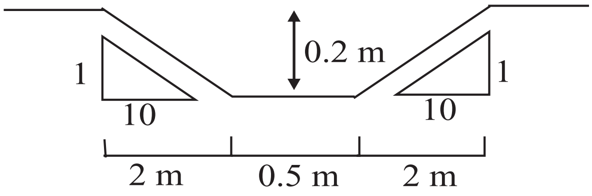


Figure 8.18 Cross section of an at-grade driveway crossing caption to be provided

8.6.4.1 Selection of Manning's *n*

A range of Manning's *n* values are selected for different flow depths appropriate for grass. It is first assumed that the flow height for a five-year ARI storm will be above the vegetation and, therefore, Manning's *n* is quite low. A figure of 0.04 is adopted. (The flow depth will need to be checked to ensure it is above the vegetation.) Thus,

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

Manning's equation $Q = (AR^{2/3}S_o^{1/2})/n$
$Q_{cap} = 0.50\ m^3/s >> Q_5\ (0.14\ m^3/s)$.

Therefore, the nominated swale has sufficient capacity to convey the required peak Q_5 flow without any requirement for an additional piped drainage system. The capacity of the swale ($Q_{cap} = 0.50\ m^3/s$) is also sufficient to convey the entire peak Q_{100} flow of $0.36\ m^3/s$ without impacting on the adjacent road and footpath.

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

Table 8.1 Manning's *n* and flow capacity variation with flow depth – turf

Flow depth (m)	Manning's *n*	Flow rate (m^3/s)
0.05	0.30	0.003
0.1	0.30	0.01
0.15	0.10	0.10
0.2	0.04	0.50

From Table 8.1, it can be seen that the five-year ARI flow depth is above the vegetation height and therefore the Manning's *n* assumption would seem reasonable.

8.6.4.2 Option 2 – assume higher vegetation

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

Table 8.2 presents the adopted Manning's *n* values and the corresponding flow capacity of the swale for different flow depths.

Table 8.2 Manning's *n* and flow capacity variation with flow depth – sedges

Flow depth (m)	Manning's *n*	Flow rate (m^3/s)
0.05	0.35	0.003
0.1	0.32	0.01
0.15	0.30	0.03
0.2	0.30	0.07

It can be seen in Table 8.2 that the swale with current dimensions is not capable of conveying a five-year discharge. Either the swale depth would need to be increased or overflow pits provided to convey a five-year ARI flow.

This worked example continues using grass for the remainder.

8.6.5 Inlet details

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

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

Flows from allotments will discharge into the base of the swale and localised erosion protection is provided with grouted rock at the outlet point of the pipe.

These are detailed in the construction drawings.

8.6.6 Velocity checks

Two velocity checks are performed to ensure vegetation is protected from erosion at high flow rates. The five-year and 100-year ARI flow velocities are checked and need to be kept below 0.5 m/s and 1.0 m/s, respectively.

Velocities are estimated using Manning's equation:

First, velocities are checked at the most downstream location (i.e. slope = 3%):

$$D_{5\text{-year}} = 0.16 \text{ m}$$

$$V_{5\text{-year}} = 0.44 \text{ m/s} < 0.5 \text{ m/s therefore OK.}$$

$$D_{100\text{-year}} = 0.19 \text{ m}$$

$$V_{100\text{-year}} = 0.70 \text{ m/s} < 1.0 \text{ m/s therefore OK.}$$

Second, velocities are checked at the bottom of the steeper section (i.e. slope = 6% with reduced catchment area):

$$D_{5\text{-year}} = 0.13 \text{ m } (Q_5 = 0.06 \text{ m}^3\text{/s})$$

$$V_{5\text{-year}} = 0.29 \text{ m/s} < 0.5 \text{ m/s therefore OK}$$

$$D_{100\text{-year}} = 0.15 \text{ m } (Q_{100} = 0.15 \text{ m}^3\text{/s})$$

$$V_{100\text{-year}} = 0.47 \text{ m/s} < 1.0 \text{ m/s therefore OK.}$$

8.6.6.1 Velocity check – safety

The velocity–depth product at both critical points (bottom of steep section and bottom of entire swale) needs to be less than 0.4 m^2/s during a 100-year ARI flow to meet pedestrian safety criteria.

At bottom of steep section:

$$V = 0.47 \text{ m/s}, d = 0.15 \text{ m; therefore, } V \times d = 0.07 \text{ m}^2\text{/s} < 0.4 \text{ m}^2\text{/s therefore OK.}$$

At bottom of swale:

$$V = 0.70 \text{ m/s}, d = 0.19 \text{ m; therefore } V \times d = 0.13 \text{ m}^2\text{/s} < 0.4 \text{ m}^2\text{/s therefore OK.}$$

8.6.6.2 Check dams

Given the steep slope of the upper part of the swale (6%), check dams are required to help to distribute flows across the base of the swale in the upper section. These are to be placed every

10 m along the steep part of the swale, be about 100 mm high and be constructed of stone. The check dams are to cross the base of the swale and merge into the batters.

8.6.7 Overflow structures

As the swale can carry a five-year ARI discharge, overflow structures are not required for this worked example. See Chapter 5 for an example including the design of an overflow pit.

8.6.8 Vegetation specification

To complement the landscape design of the area, a turf species is to be used. For this application a turf with a height of 50 mm has been assumed. The actual species will be selected by the landscape designer.

8.6.9 Calculation summary

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

Swales — CALCULATION SUMMARY

CALCULATION TASK	OUTCOME		CHECK
1 Identify design criteria			
Conveyance flow standard (ARI)	5	year	
Vegetation height	50	mm	✓
2 Catchment characteristics			
Upper area	4,800	m^2	
Total area	12,000	m^2	
Slope	3 and 6	%	
Fraction impervious			
f_{imp}	0.66		✓
3 Estimate design flow rates			
Time of concentration			
Estimate from flow path length and velocities	10	minutes	✓
Identify rainfall intensities			
Station used for IFD data:	Ballarat		
Major flood – 100-year ARI	140	mm/hr	
Minor flood – 5-year ARI	67	mm/hr	
Peak design flows			
Q_{minor}	0.14	m^3/s	
Q_{100}	0.36	m^3/s	✓
4 Swale design			
Manning's *n* below vegetation height	0.3		
Manning's *n* at capacity	0.04		✓
5 Inlet details			
Adequate erosion and scour protection?	rock pitching		
Flush kerb setdown?	50	mm	✓
6 Velocities over vegetation			
Velocity for 5-year flow (<0.5 m/s)	0.09	m/s	
Velocity for 100-year flow (<1.0 m/s)	0.49	m/s	
Safety: Vel x Depth (<0.4)	0.13	m^2/s	✓
7 Overflow system			
Spacing of overflow pits	not required		
Pit type			✓
8 Plant selection	turf		✓

8.6.10 Construction drawing

Figure 8.19 shows the construction drawing for the swale worked example.

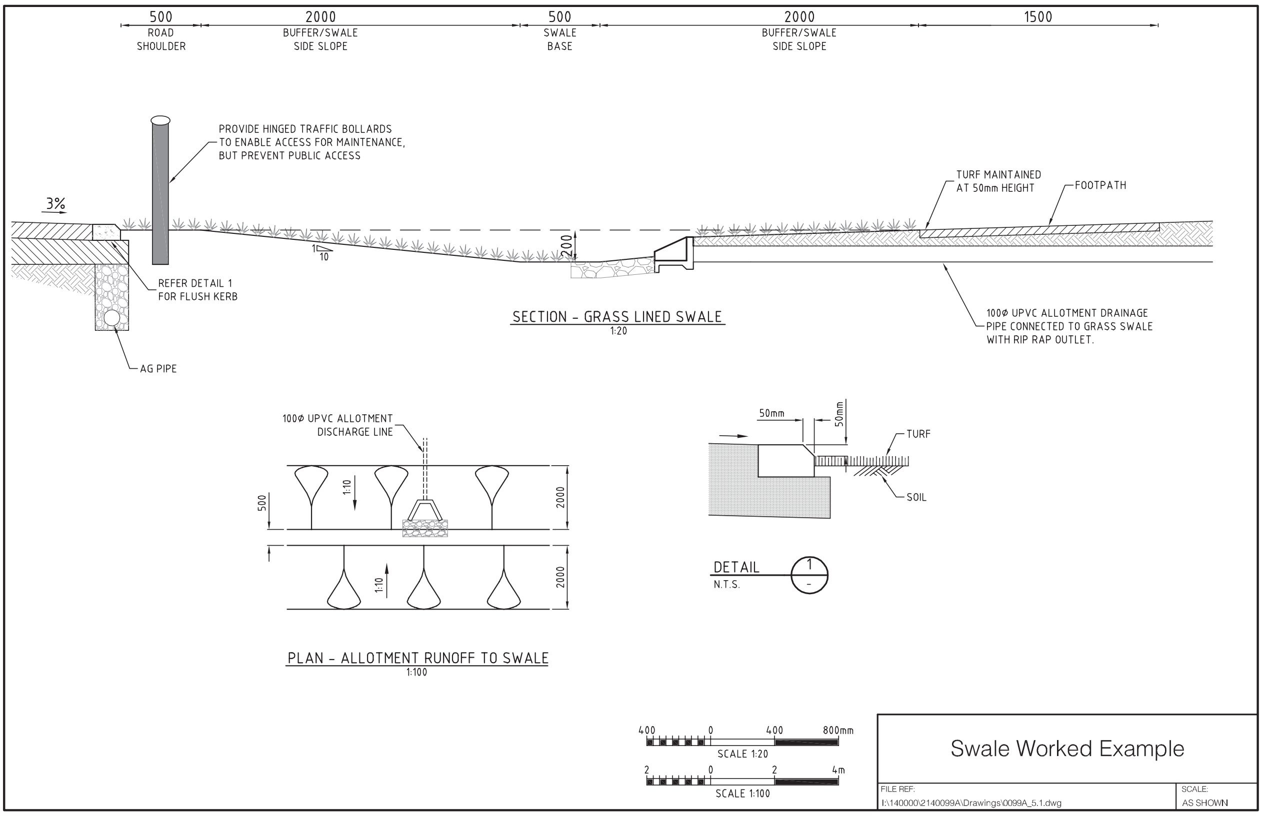


Figure 8.19 Swale worked example

8.7 References

Barling, R.D. and Moore, I.D. (1993). 'The role of buffer strips in the management of waterway pollution'. Paper presented at 'The role of buffer strips in the management of waterway pollution from diffuse urban and rural sources', The University of Melbourne.

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

Institution of Engineers, Australia (2001). *Australian Rainfall and Runoff – A Guide to Flood Estimation*, Revised edn, Pilgram, D.H. (Ed.), Institution of Engineers, Australia, Barton, ACT.

Chapter 9 **Constructed wetlands**

Constructed wetland in Lynbrook, Victoria.

9.1 Introduction

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

Wetlands generally consist of an **inlet zone** (**sediment basin** to remove coarse sediments), a **macrophyte** zone (a shallow, heavily vegetated area to remove fine particulates and uptake of soluble pollutants) and a high flow bypass channel (to protect the **macrophyte zone**) (e.g. Figure 9.1). They are designed primarily to remove stormwater pollutants associated with fine to colloidal particulates and dissolved contaminants.

Simulations using computer models are often undertaken to optimise the relationship between **detention time**, wetland volume and the **hydrologic effectiveness** of the constructed wetland to maximise treatment given the volume constraints of the wetland site. The relationship between detention time and pollutant removal efficiency is largely influenced by the settling velocity of the target particulate, although defining the settling velocity of fine to colloidal particulates is not a straight-forward exercise. Standard equations for settling velocities often do not apply for such fine particulates owing to the influence of external factors such as wind and water turbulence. Detention periods should notionally be about 72 hours to effectively remove nutrients in urban stormwater in Victoria.

The key operational design criteria for constructed wetlands may be summarised as to:

- promote **sedimentation** of particles larger than 125 μm within the inlet zone
- discharge water from the inlet zone into the macrophyte zone for removal of fine particulates and dissolved contaminants through the processes of enhanced sedimentation, filtration, adhesion and **biological uptake**
- ensure that the required detention period is achieved for all flow though the wetland system through the incorporation of a **riser outlet** system

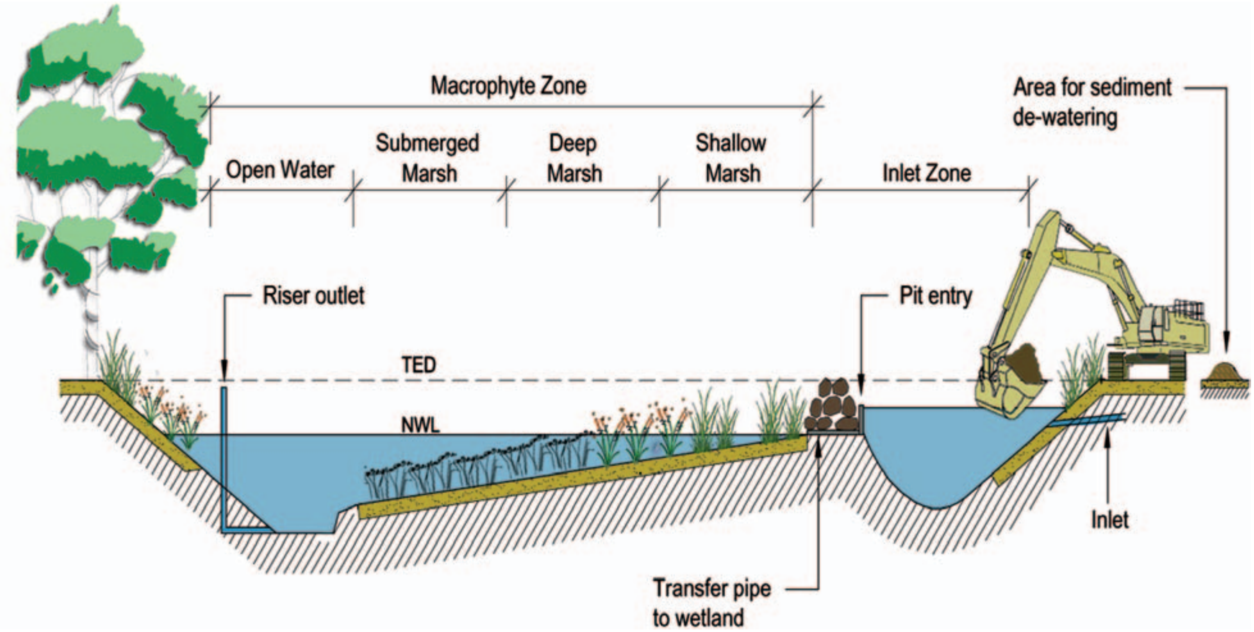


Figure 9.1 Layout of a constructed wetland system.

- ensure adequate flood protection of the macrophyte zone from scouring during above-design conditions by designing for bypass operation when inundation in the macrophyte zone reaches the design maximum **extended detention** depth.

Poor design of constructed wetlands has led to many of these urban wetlands and **ponds** systems becoming a long-term liability to the community. Common problems encountered include:

- accumulation of litter in some sections of the wetland
- accumulation of oil and scum at 'dead zones' in the wetland
- infestation of weeds or dominance of certain species of vegetation
- mosquito problems
- algal blooms
- scouring of sediment and banks, especially during high flows.

Many of the these problems can be minimised or avoided by good engineering design principles. Poor wetland hydrodynamics and lack of appreciation of the stormwater **treatment train** are often identified as major contributors to wetland management problems. A summary of desired hydrodynamic characteristics and design elements is presented in Table 9.1.

In many urban applications, wetlands can be constructed in the base of retarding basins, thus reducing the land required for stormwater treatment. In these situations, the wetland systems will occasionally become inundated to greater depths than the extended detention depth. However, the inundation is relatively short (hours) and is unlikely to affect the vegetation provided there is a safe pathway to drain following flood events that does not scour vegetation or banks.

Key design issues to be considered are:

1. verifying size and configuration for treatment
2. determining design flows
3. designing the inlet zone (see Design Procedure for Sedimentation Basin, Chapter 4)
4. layout of the macrophyte zone
   - zonation
   - longitudinal and cross sections
5. hydraulic structures:
   - macrophyte zone outlet structures
   - connection to the inlet zone
   - bypass **weir** and channel
6. recommending plant species and planting densities
7. providing maintenance.

Table 9.1 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	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)
	Morphological and outlet control design to match botanical layout design and the hydrology of the wetland	Inadequate attention to the inundation depth, wetness gradient of the wetland and the frequency of inundation at various depth range would lead to dominance of certain plant species especially weed species 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 high inflow rates

9.2 Verifying size for treatment

The curves (Figures 9.2–9.4) are based on the performance of the system in Melbourne with varying typical extended detention depths and were derived using the Model for Urban Stormwater Improvement Conceptualisation (**MUSIC**) (Cooperative Research Centre for Catchment Hydrology 2003)..To estimate an equivalent performance at other locations in Victoria, the hydrologic design region relationships should be used to convert the treatment area into an equivalent treatment area in Melbourne (reference site) (see Chapter 2). In preference to using the curves, local data should be used to model the specific treatment performance of the system.

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

- the inlet zone forms part of the wetland system sized to retain 125 μm sediment for flows up to the one-year ARI peak **discharge** and with provision for high flow bypass
- notional detention period of 72 hours.

The curves in Figure 9.2 to 9.4 can be used to check the expected performance of the wetland system for removal of Total Soluble Solids (TSS), Total Phosphorus (TP) and Total Nitrogen (TN). The X-axis on the curves is a measure of the size of the surface of the wetland (measured as the **permanent pool** area), expressed as a percentage of the contributing *impervious* catchment.

9.3 Design procedure: constructed wetlands

Major elements of constructed wetland systems are shown in Figure 9.5.

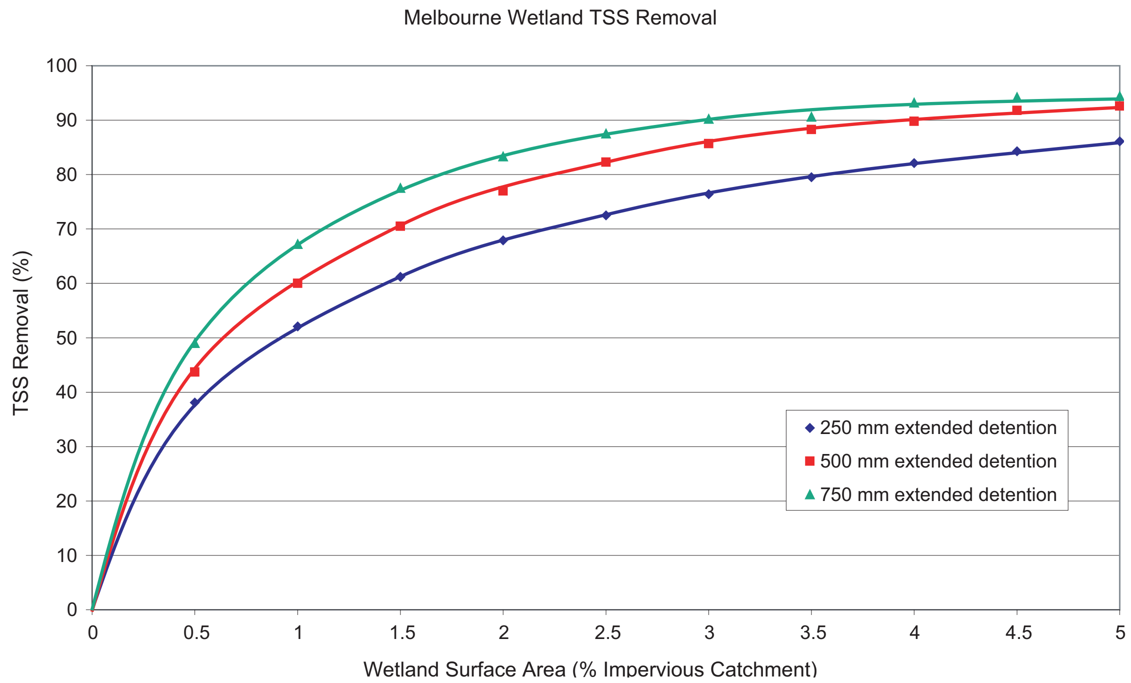


Figure 9.2 Performance of a wetland in removing Total Soluble Solids (TSS) in Melbourne.

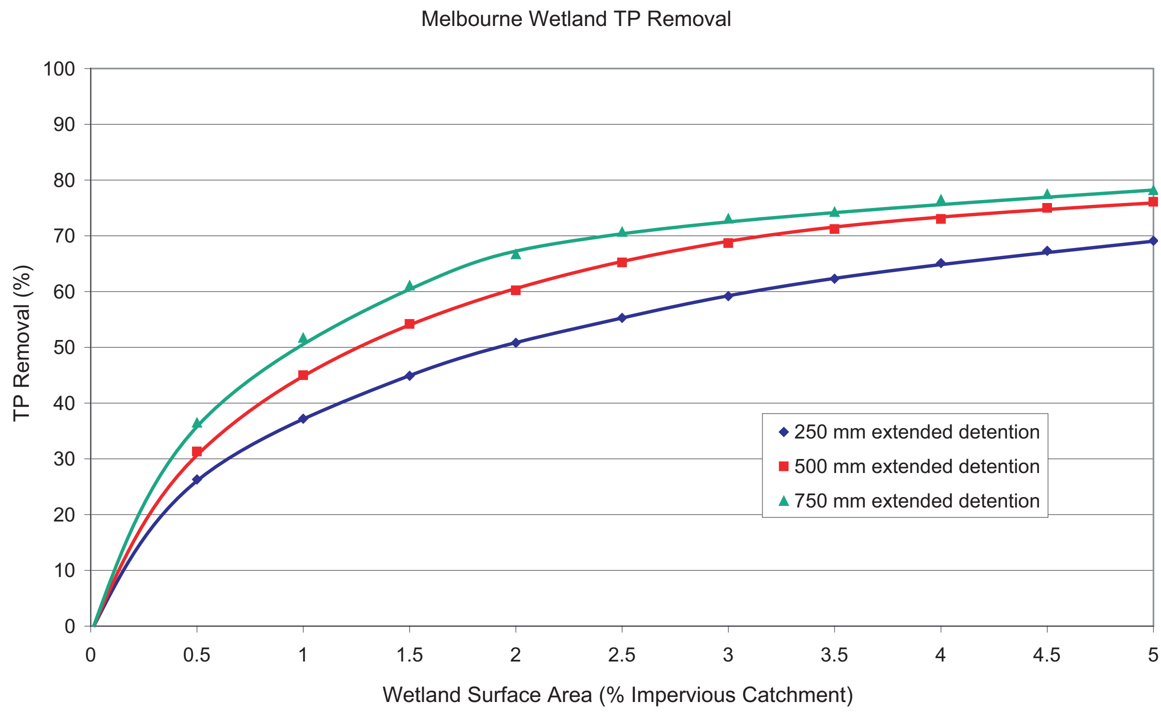


Figure 9.3 Performance of a wetland in removing Total Phosphorus (TP) in Melbourne.

Analyses to be undertaken during the detailed design phase of the inlet zone and the macrophyte zone of constructed wetland system include the following.

1. Design of the inlet zone as a sedimentation basin to target sediment of 125 μm or larger, including the:
   - inlet zone to operate as a flow regulator into the macrophyte zone during normal operation
   - inlet zone to operate for bypass of the macrophyte zone during above-design conditions
   - connection between the inlet zone and the macrophyte zone to be appropriately designed so that inlet conditions provide for energy dissipation and distribution of inflow into the macrophyte zone
- provision for sediment and debris removal.

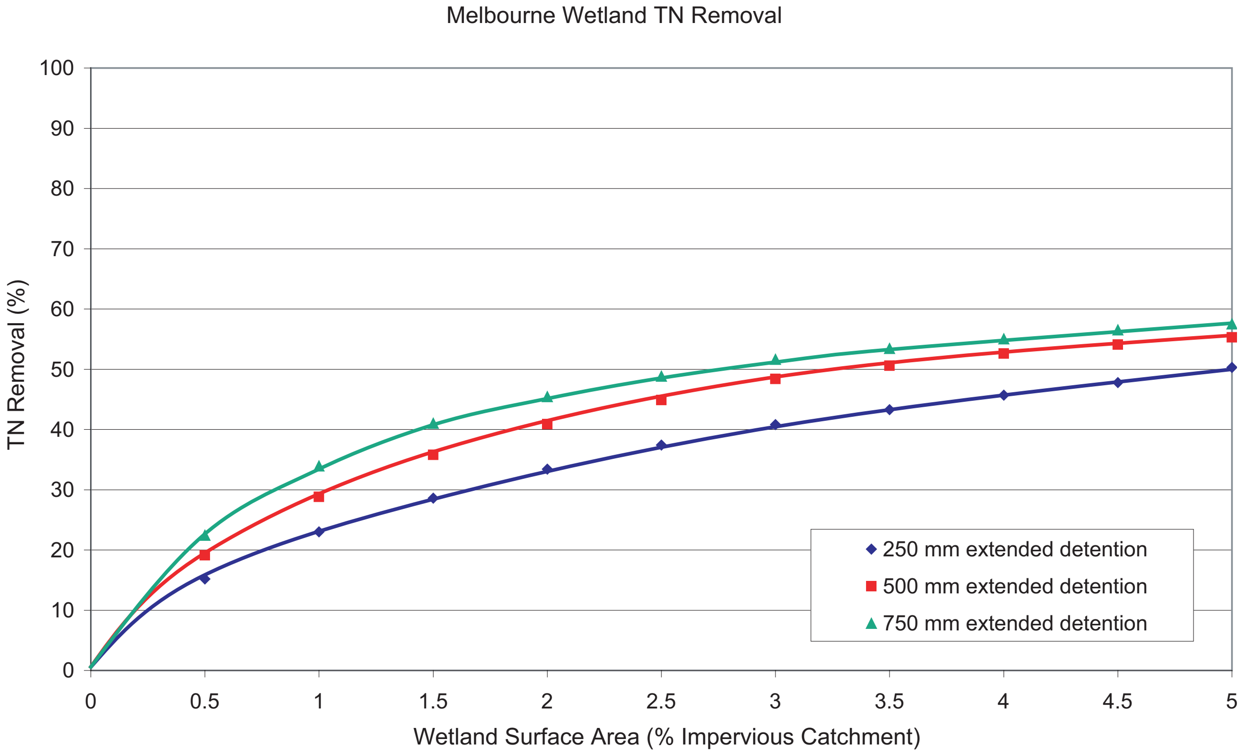


Figure 9.4 Performance of a wetland in removing Total Nitrogen (TN) in Melbourne.

2. Configure the layout of the macrophyte zone to provide an extended detention volume so that the system's **hydraulic efficiency** can be optimised, including design for the:
   - range of suitable extended detention depth to be between 0.25 m and 0.75 m, depending on the desired operation of the wetland and target pollutant
   - **bathymetry** of the macrophyte zone to promote a sequence of **ephemeral**, shallow marsh, marsh and submerged marsh systems in addition to a small, open water system near the outlet structure
   - placement of the inlet and outlet structures, the aspect ratio of the macrophyte zone and flow control features to promote a high hydraulic efficiency within the macrophyte zone, in particular
   - location and depth of permanent pools within the macrophyte zone
   - drainage of the macrophyte zone, if necessary.
3. Design the macrophyte zone outlet structure to provide for a 72 hour **notional detention time** for a wide range of flow depth. The outlet structure should include measures to trap debris to prevent clogging.

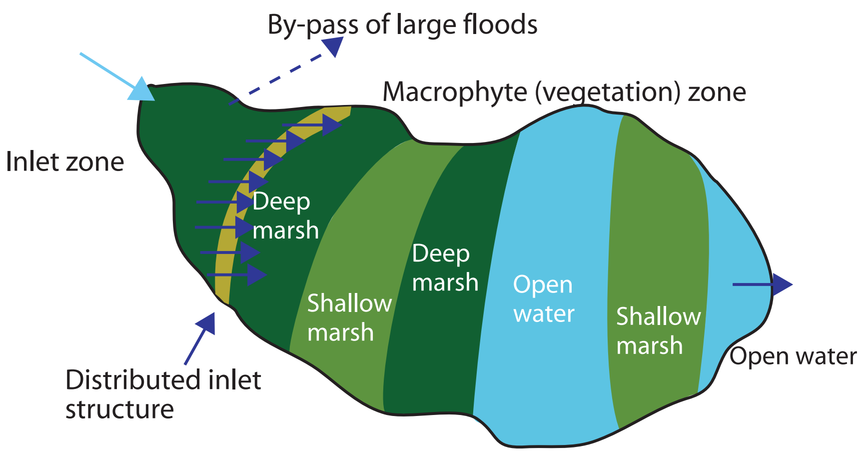


Figure 9.5 Elements of a constructed wetland system.

4. Provide landscape design, which requires:
   - macrophyte zone vegetation (including **littoral zone**)
   - terrestrial vegetation.

The following sections describe the design steps required for constructed stormwater wetland systems.

9.3.1 Estimating design flows

The hydrologic design objectives for the inlet zone are:

1. capacity to convey stormwater inflows up to the peak one-year ARI discharge into the macrophyte zone
2. capacity to convey above-design stormwater inflows to the by-pass system; design discharge capacity for the bypass system corresponds to, for example:
   - the minor system capacity (two-year or five-yearyear ARI) if overland flow path does not direct overland flow into the wetland
   - 100-year ARI peak discharge if the wetland system forms part of the major drainage system.

9.3.1.1 Minor and major flood estimation

A range of hydrologic methods can be applied to estimate design flows. If the typical catchment areas are relatively small, the **Rational Method** Design Procedure is considered to be a suitable method for estimating design flows. However, the use of the Rational Design Procedure should strictly be used only to size inlet hydraulic structures. A full flood routing computation method should be used in sizing the outlet hydraulic structures (e.g. outlet pipe, spillway and embankment height).

9.3.2 Inlet zone

The inlet zone of a constructed stormwater wetland serves two basic functions:

1 the pretreatment of inflow to remove gross pollutants and coarse to medium-sized sediment
2 the hydrologic control of inflows into the macrophyte zone and bypass of floods during 'above-design' operating conditions.

The inlet zone typically comprises a relatively deep, open waterbody (> 1.5 m) that operates essentially as a sedimentation basin. Often it may be necessary to install a **Gross Pollutant Trap (GPT)** at the inlet to this zone such that litter and large debris can be captured at the interface between the incoming waterway (or pipe) and the open water of the inlet zone.

For more information and guidance on the design of the inlet zone, see Chapter 4.

9.3.3 Macrophyte zone layout

9.3.3.1 Size and dimensions

To optimise hydraulic efficiency (i.e. reduce short circuits and dead zones), it is desirable to adopt a high length to width ratio. The ratio of length to width varies depending on the size of the system and the site characteristics. To simplify the design and earthworks smaller systems tend to have length to width ratios at the lower end of the range. This can often lead to poor hydrodynamic conditions within the macrophyte zone. Persson *et al* (1999) used the term 'hydraulic efficiency' to define the expected hydrodynamic characteristics for a range of configurations of stormwater detention systems. Engineers Australia (2003) present expected hydraulic efficiencies of detention systems for a range of notional shapes, aspect ratios and inlet/outlet placements within stormwater detention systems and recommends that the λ value for constructed wetland systems should not be less than 0.5, and should be designed to promote hydraulic efficiencies greater than 0.7 (see Figure 9.6).

The numbers in Figure 9.6 represent the values of λ. In Figure 9.6, 'o' in diagrams O and P represent islands in a waterbody and the double line in diagram Q represents a structure to distribute flows evenly.

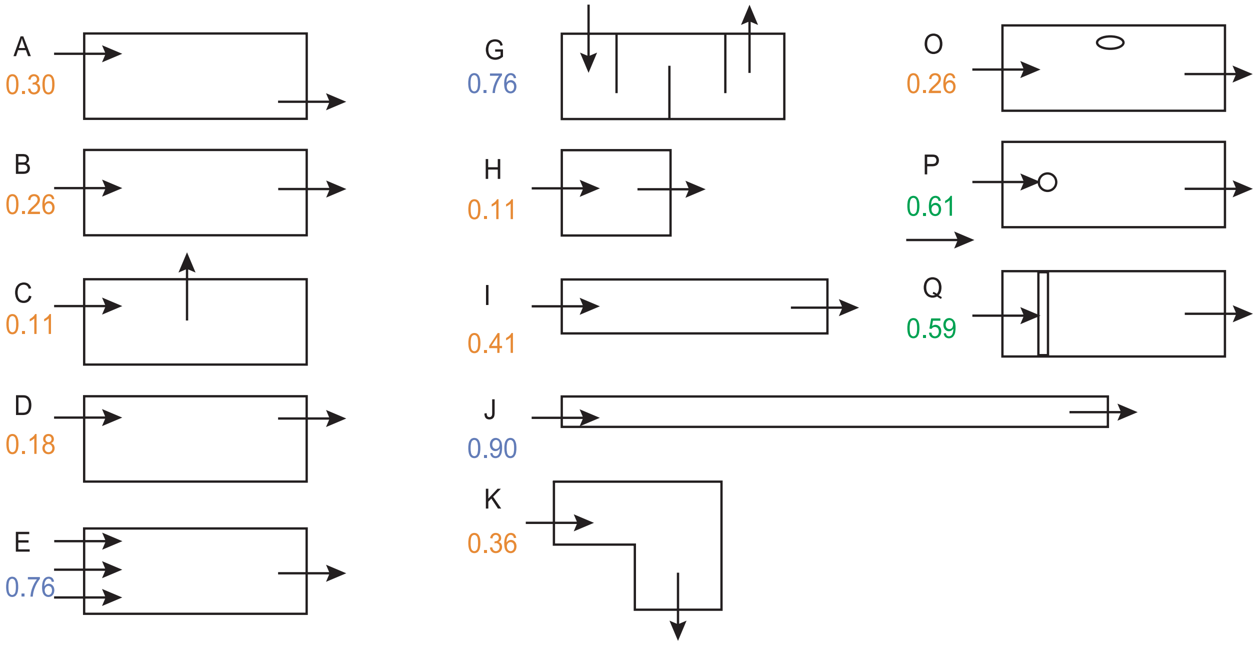


Figure 9.6 Hydraulic efficiency (λ) – a measure of flow hydrodynamic conditions in constructed wetlands and ponds; range is from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment (from Persson et al, 1999).

9.3.3.2 Zonation

A range of habitat areas within wetlands is needed to support a variety of plant species and ecological niches. The wetland is broadly divided into four macrophyte zones and an open water zone. The bathymetry across the four macrophyte zones is to vary gradually over the depth range, from 0.2 m above the permanent pool level to 0.5 m below the permanent pool level (see Table 9.2). The depth of the open water zone in the vicinity of the outlet structure is to be 1.0 m below the permanent pool level.

To ensure optimal hydraulic efficiency of the wetland for the given shape and aspect ratio, the wetland zones are arranged in bands of equal depth running across the flow path. The appropriate bathymetry coupled with uniform plant establishment ensures the cross section has equivalent hydraulic conveyance, thus preventing short-circuiting.

9.3.3.3 Long section

In defining a long section of a macrophyte zone, it is necessary to provide areas for habitat refuge. For this reason it is desirable to have permanent pools interconnected to prevent fauna being isolated in areas that dry out. This also reduces the piping required to drain the wetland for maintenance purposes.

An example bathymetry of a wetland system is shown in Figure 9.8. It illustrates gradual changes in depth longitudinally to create different vegetation areas as well as consistent zone banding across the wetland.

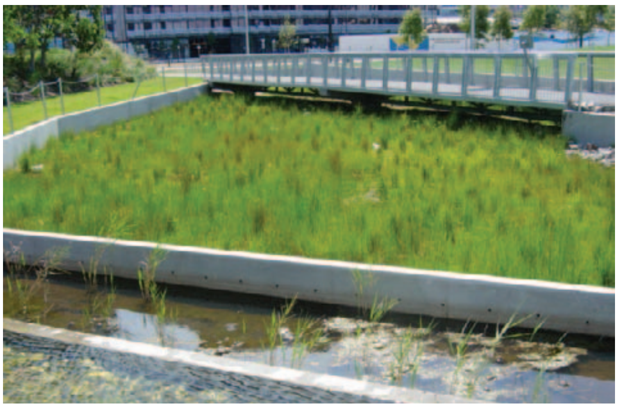


Figure 9.7 Zonation in wetland systems.

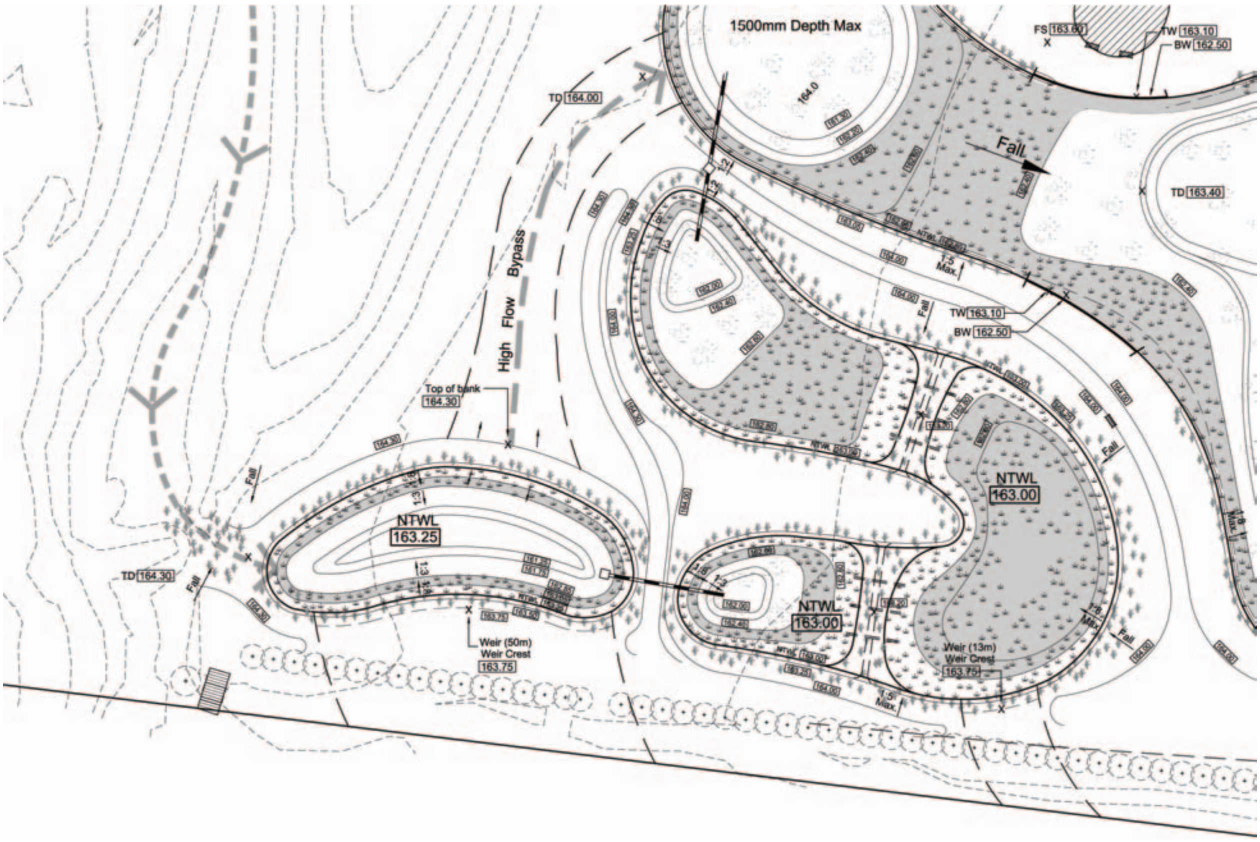


Figure 9.8 Example bathymetry of a constructed wetland system (Graeme Bentley Landscape Architects 2004).

9.3.3.4 Cross sections

The **batter slopes** on approaches and immediately under the permanent water level have to be configured with consideration of public safety (e.g. Figure 9.9).

A gentle slope to the water's edge and extending below the water line should 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 and built around the wetland.

Safety requirements for individual wetlands may vary from site to site, and it is recommended that an independent safety audit be conducted for each design. Safety guidelines are also provided by some local authorities (e.g. Melbourne Water 2003, and Royal Life Saving Society of Australia 2004) and these should be followed.

9.3.4 Macrophyte zone outlet structure

The macrophyte zone outlet structure forms two purposes. The first is to control discharges from the extended detention storage to ensure the wetland maintains a notional detention time

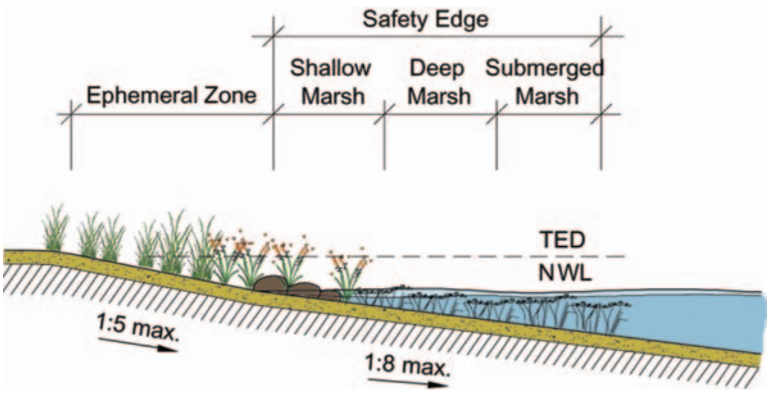


Figure 9.9 Example of edge design to a constructed wetland system.

of 72 hours. The outlet structure also needs to include features to allow the permanent pool to be drained for maintenance.

9.3.4.1 Maintenance drain

The permanent pool of the wetland should be able to be drained with a maintenance drain operated manually. A suitable design flow rate (Q) is one which can draw down the permanent pool within 12 hours (i.e. overnight).

The orifice discharge equation (Equation 9.1) is considered suitable for sizing the maintenance drain on the assumption that the system will operate under inlet control with its discharge characteristics determined as follows:

$$A_o = \frac{Q}{C_d\sqrt{2gh}} \qquad \text{(Equation 9.1)}$$

C_d = Orifice discharge coefficient (0.6)
h = Depth of water above the centroid of the orifice (m)
A_o = Orifice area (m^2)
Q = required flow rate to drain the volume of the permanent pool in 12 hours.

9.3.4.2 Riser outlet – size and location of orifices

The riser is designed to provide a uniform notional detention time over the full range of the extended detention depth. The target maximum discharge may be computed as the ratio of the volume of the extended detention to the notional detention time:

$$\text{Target maximum discharge } (m^3/s) = \text{extended storage volume } (m^3) \,/\, \text{detention time (s)} \qquad \text{(Equation 9.2)}$$

The placement of outlet orifices and determining their appropriate diameters is designed iteratively by varying outlet diameters and levels, using the orifice discharge equation (Equation 9.1) applied over discrete depths along the length of a riser up to the maximum detention depth. This can be performed with a spreadsheet as illustrated in the spreadsheet included on the CD.

As the outlet orifices can be expected to be small, the orifices need to be prevented from clogging by debris. Some form of debris guard is recommended (e.g. Figure 9.9).

An alternative to using a debris guard is to install the riser within a pit which is connected to the permanent pool of the macrophyte zone via a submerged pipe culvert. This connection should be adequately sized such that there is minimal water level difference between the water within the pit and the water level in the macrophyte zone. With the water entering into the outlet pit being drawn from below the permanent pool level, floating debris are prevented from entering the outlet pit while heavier debris would normally settled onto the bottom of the permanent pool.

9.3.4.3 Riser outlet – pipe dimension

While conservative, it is desirable to size the riser pipe such that it has the capacity to accommodate the one-year ARI peak discharge operating as a 'glory hole' spillway. Under normal operation, this flow would bypass the macrophyte zone when this zone is already operating at design capacity. Nevertheless, it is good practice to provide a level of contingency in discharge capacity for the riser outlet to prevent any overtopping of the embankment of the macrophyte zone. A minimum of a 0.3 m freeboard for the embankment (i.e. crest level of embankment above the top of the extended detention) is often required.

Significant attenuation of the peak one-year ARI inflow can be expected and some routing of the inflow hydrograph through the storage provided by the macrophyte zone is recommended.

The sharp-crested weir equation (Equation 9.3) can be used in defining the required perimeter (P) (and thus dimension) of the riser outlet. A weir coefficient of 1.7 (k_w sharp-crested weir) is recommended:

Figure 9.10 Debris guards on riser outlets.

$$P = \frac{Q_{des}}{C_w \times H^{1.5}} \qquad \text{(Equation 9.3)}$$

P = Perimeter of the outlet pit
C_w = Weir coefficient
H = Depth of water above the crest of the outlet pit
Q_{des} = Design discharge (m^3/s).

9.3.4.4 Discharge pipe

The discharge pipe of the wetland conveys the outflow of the macrophyte zone to the receiving waters (or existing drainage infrastructure). The conveyance capacity of the discharge pipe is to be sized to match the higher of the two discharges (i.e. maximum discharge from the riser or the maximum discharge from the maintenance drain).

9.3.5 Connection to the inlet zone

The pipe that connects the inlet zone to the macrophyte zone must have sufficient capacity to convey a one-year ARI flow, assuming the macrophyte zone is at the permanent pool level, without resulting in any flow in the bypass system. The configuration of the hydraulic structure connecting the inlet zone to the macrophyte zone would normally consist of an overflow pit connected to one or more pipes through the embankment separating these two zones.

Typical specifications of water and embankment levels are:

- bypass spillway level = top of extended detention in the macrophyte zone
- permanent pool level in inlet zone = 0.3 m above permanent pool level in macrophyte zone.

Velocity checks are to be conducted for when the wetland is full and when it is near empty. Velocities should ideally be less than 0.05 m/s.

The culvert connection between the inlet zone and the macrophyte zone can be sized using standard culvert equations that accounts for energy losses associated with the inlet and exit conditions and friction losses within the culvert. For most applications, the culvert will operate under outlet control with the inlet and outlet of the culvert being fully submerged. With relatively short pipe connections, friction loss is typically small and can be computed using Manning's equation. The total energy loss of the connection is largely determined by the inlet and outlet and outlet conditions and the total losses can be computed using the expression

$$\Delta H = (K_{in} + K_{out})\frac{v^2}{2g} + S_f \times L \qquad \text{(Equation 9.4)}$$

where K_{in} and K_{out} are the head loss coefficients for the inlet and outlet conditions (typically, and conservatively, assumed to 0.5 and 1.0 respectively), S_f is the friction slope (which is computed from Manning's equation or the Colebrook-White equation) and L is the length of the culvert (Chadwick and Morfett 1986).

Figure 9.11 Connection inlet zone.

9.3.6 High-flow route and bypass design

To protect the integrity of the macrophyte zone of the wetland, it is necessary to consider the desired above-design operation of the wetland system. This is generally provided for with a high flow route that bypasses the macrophyte zone during flow conditions that may lead to scour and damage to the wetland vegetation. A function of the inlet zone is to provide hydrologic control of inflow into the macrophyte zone (see Section 9.3.2 and Chapter 4). A bypass weir is to be included in the design of the inlet zone, together with a bypass floodway (channel) to direct high flows around the macrophyte zone.

Ideally, the level of the bypass weir should be set at the top of the extended detention level in the macrophyte zone. This would ensure that a significant proportion of catchment inflow will bypass the macrophyte zone once it has reached its maximum operating extended detention level. The width of the spillway is to be sized to safely pass the maximum discharge conveyed into the inlet zone or the 100-year ARI discharge (see Section 9.3.1) with the maximum water level above the crest of the weir to be defined by the top of embankment level (plus a suitable freeboard provision).

9.3.7 Vegetation specification

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 (see Appendix A). 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 (two years).

9.3.8 Designing to avoid mosquitos

Mosquitos are a natural component of wetland fauna and the construction of any waterbody will create some mosquito habitat. To reduce the risk of high numbers of mosquitos designs should function as balanced ecosystems with predators controlling mosquito numbers. Design considerations that should be addressed include:

- providing access for mosquito predators to all parts of the waterbody (do not have stagnant isolated area of water)
- providing areas of permanent water (even during long dry periods) that mosquito preditors can seek refuge
- maintaining natural water level fluctuations that disturb the breeding cycle of some mosquito species

- providing a bathymetry such that regular wetting and drying is achieved and water draws down evenly so isolated pools are avoided
- providing sufficient gross pollutant control at the inlet such that human-derived litter does not accumulate and provide breeding habitat
- ensuring maintenance procedures do not result in wheel rut and other localised depressions that create isolated pools when water levels fall.

Local agencies guidelines should also be consulted in regard to approaches for avoiding excessive numbers of mosquitos.

9.3.9 Design calculation summary

A *Constructed Wetland Calculation Summary* is included to aid the design process of key design elements of a constructed wetland.

Constructed Wetand	CALCULATION SUMMARY		
CALCULATION TASK	OUTCOME		CHECK
1 Identify design criteria			
Design ARI flow for inlet zone		year	
Target sediment size for inlet zone		mm	
Notional detention period for macrophyte zone		hrs	
Design ARI flow for bypass spillway		year	
Extended detention volume		m^3	☐
2 Catchment characteristics			
Residential		Ha	
Commercial		Ha	
Fraction impervious			
Residential			
Commercial			☐
3 Estimate design flow rates			
Time of concentration			
Estimate from flow path length and velocities		minutes	☐
Identify rainfall intensities			
Station used for IFD data:			
100-year ARI		mm/hr	
1-year ARI		mm/hr	☐
Design runoff coefficient			
C_1			
C_{100}			☐
Peak design flows			
Q_1		m^3/s	
Q_{100}		m^3/s	☐
4 Inlet zone			
Refer to *Sedimentation Basin Calculation Summary*			☐
5 Macrophyte zone layout			
Extend detention depth		m	
Area of macrophyte zone		m^2	
Aspect ratio		L:W	
Hydraulic efficiency			
Length		m	
Top width (including extended detention)		m	
Cross section batter slope		V:H	☐
6 Macrophyte zone outlet structures			
Maintenance drain			
Diameter of maintenance valve		mm	
Drainage time		hrs	☐
Riser			
Linear storage-discharge relationship for riser			☐
Discharge pipe			
Discharge capacity of discharge pipe		m^3/s	☐
7 Connection between inlet zone and macrophyte zone			
Discharge capacity of connection culvert		m^3/s	☐
8 Bypass weir			
Discharge capacity of bypass weir			

9.4 Checking tools

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

Checklists are provided for:

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

9.4.1 Design assessment checklist

The *Wetland Design Asset Checklist* 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, the design procedure should be assessed to determine the effect of the omission or error.

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

Land and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the *Asset Handover Checklist* (see Section 9.4.4).

9.4.2 Construction advice

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

Protection from existing flows

It is important to protect a wetland system from upstream flows during construction.. A mechanism to divert flows around a construction site, protect from litter and debris is required. This can be achieved by constructing a high flow bypass channel initially and then diverting all inflows along the channel until the wetland system is complete.

High flow contingencies

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

Erosion control

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

Inlet erosion checks

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

Tolerances

Tolerances are very important in the construction of wetlands (e.g. base, longitudinal and batters) – levels are particularly important for a well-distributed flow path and for establishing appropriate vegetation bands. As water levels reduce (e.g. for maintenance) areas need to drain back into designated pools and distributed shallow pools across the wetland are avoided. Generally a tolerance (plus or minus) of 50 mm is acceptable.

Wetland Design Assessment Checklist		
Wetland location:		
Hydraulics Minor flood: (m^3/s)	Major flood: (m^3/s)	
Area Catchment area (ha):	Wetland area (ha)	
Treatment	**Y**	**N**
Treatment performance verified from curves?		
Inlet zone	**Y**	**N**
Inlet pipe/structure sufficient for maximum design flow (Q_5 or Q_{100})?		
Scour protection provided at inlet?		
Configuration of inlet zone (aspect, depth and flows) allows settling of particles >125 µm?		
Bypass weir incorporated into inlet zone?		
Bypass weir and channel sufficient to convey >Q_1 <= maximum inlet flows?		
Bypass weir crest at macrophyte permanent pool level + extended detention depth?		
Bypass channel has sufficient scour protection?		
Structure from inlet zone to macrophyte zone enables energy dissipation/flow distribution?		
Structure from inlet zone to macrophyte zone enables isolation of the macrophyte zone for maintenance?		
Inlet zone permanent pool level above macrophyte permanent pool level?		
Maintenance access allowed for into base of inlet zone?		
Public access to inlet zone prevented through vegetation or other means?		
Gross pollutant protection measures provided on inlet structures (both inflows and to macrophyte zone)		
Macrophyte zone	**Y**	**N**
Extended detention depth >0.25 m and <0.75 m?		
Vegetation bands perpendicular to flow path?		
Vegetation bands of near uniform depth?		
Sequencing of vegetation bands provides continuous gradient to open water zones?		
Vegetation appropriate to selected band?		
Aspect ratio provides hydraulic efficiency >0.5?		
Velocities from inlet zone <0.05 m/s or scouring protection provided?		
Batter slopes from accessible edges shallow enough to allow egress?		
Maintenance access provided into areas of the macrophyte zone (especially open water zones)?		
Public access to macrophyte zones restricted where appropriate?		
Safety audit of publicly accessible areas undertaken?		
Freeboard provided above extended detention depth?		
Outlet structures	**Y**	**N**
Riser outlet provided in macrophyte zone?		
Orifice configuration allows for a linear storage-discharge relationship for full range of the extended detention depth?		
Riser diameter sufficient to convey Q_1 flows when operating as a 'glory hole' spillway?		
Maintenance drain provided?		
Discharge pipe from has sufficient capacity to convey the maintenance drain flows or Q_1 flows (whichever is higher)?		
Protection against clogging of orifice provided on outlet structure?		

Transitions

The detail of earthworks needs to be checked in order 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).

Inlet zone access

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

Inlet zone base

To aid maintenance it is recommended that the inlet zone is constructed with a hard (i.e. rock) bottom. This is important if maintenance is by 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.

Dewatering collected sediments

An area should be constructed that allows for dewatering of removed sediments from an inlet zone. This area should be located such that water from the material drains back into the inlet zone. Material should be allowed to drain for a minimum of overnight before disposal.

Timing for planting

Timing of planting vegetation depends on a suitable time of year (and potential irrigation requirements) as well as timing in relation to the phases of development. Temporary sediment controls should always be used prior to planting as lead times from earthworks to planting are often long.

Vegetation establishment

During the establishment phase water levels should be controlled carefully to prevent seedlings from being desiccated or drowned. This is best achieved with the use of maintenance drains. Once plants are established, water levels can be raised to operational levels (see Appendix A).

Bird protection

Protection against birds (e.g. using nets) should be considered for newly planted areas of wetlands as birds can pull out young plants and reduce plant densities.

9.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Wetlands

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

SITE: ______________________

CONSTRUCTED BY: ______________________

DURING CONSTRUCTION

Items inspected	Checked Y	Checked N	Satisfactory	Unsatisfactory		Checked Y	Checked N	Satisfactory	Unsatisfactory
Preliminary works									
1. Erosion and sediment control plan adopted					18. Concrete and reinforcement as designed				
					19. Inlets appropriately installed				
2. Limit public access					20. Inlet energy dissipation installed				
3. Location same as plans					21. No seepage through banks				
4. Site protection from existing flows					22. Ensure spillway is level				
5. All required permits in place					23. Provision of maintenance drain(s)				
Earthworks					24. Collar installed on pipes				
6. Integrity of banks					25. Low flow channel rocks are adequate				
7. Batter slopes as plans					26. Protection of riser from debris				
8. Impermeable (eg. clay) base installed					27. Bypass channel stabilised				
9. Maintenance access to whole wetland					28. Erosion protection at macrophyte outlet				
10. Compaction process as designed					**Vegetation**				
11. Placement of adequate topsoil					29. Vegetation appropriate to zone (depth)				
12. Levels as designed for base, benches, banks and spillway (including freeboard)					30. Weed removal prior to planting				
13. Check for groundwater intrusion					31.Provision for water level control during establishment				
14. Stabilisation with sterile grass					32. Vegetation layout and densities as designed				
Structural components									
15. Location and levels of outlet as designed					33. Provision for bird protection				
16. Safety protection provided					34. By-pass channel vegetated				
17. Pipe joints and connections as designed									

FINAL INSPECTION

1. Confirm levels of inlets and outlets					9. Check for uneven settling of banks				
2. Confirm structural element sizes					10. Evidence of stagnant water, short circuiting or vegetation scouring				
3. Check batter slopes									
4. Vegetation planting as designed					11. Evidence of litter or excessive debris				
5. Erosion protection measures working					12. Provision of removed sediment drainage area				
6. Pre-treatment installed and operational									
7. Maintenance access provided					13. Evidence of debris in high flow bypass				
8. Public safety adequate					14. Macrophyte outlet free of debris				

COMMENTS ON INSPECTION

ACTIONS REQUIRED

1.

2.

3.

4.

5.

6.

9.4.4 Asset handover checklist

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

9.5 Maintenance requirements

Wetlands treat runoff by filtering it through vegetation and providing extended detention to allow sedimentation to occur. In addition, they are used for flow management and need to be maintained to ensure adequate flood protection for local properties and protection of the wetland ecosystem.

Maintaining vibrant vegetation and adequate flow conditions in a wetland are 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 plant establishment (first two years) when weed removal and replanting may be required (see Appendix A). It is also the time when large loads of sediments could affect plant growth, particularly in developing catchments with poor building controls.

Other components of the system that require careful consideration are the inlet points. Inlets can be prone to scour and build-up of litter. Occasional litter removal and potential replanting may be required.

Maintenance is primarily concerned with:

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

Wetland Maintenance Checklist			
Inspection frequency: 3 monthly	Date of visit:		
Location:			
Description:			
Site visit by:			
Inspection items	Y	N	Action required (details)
Sediment accumulation at inflow points?			
Litter within inlet or macrophyte zones?			
Sediment within inlet zone requires removal (record depth, remove if >50%)?			
Overflow structure integrity satisfactory?			
Evidence of dumping (building waste, oils etc.)?			
Terrestrial vegetation condition satisfactory (density, weeds etc)?.			
Aquatic vegetation condition satisfactory (density, weeds etc.)?			
Replanting required?			
Settling or erosion of bunds/batters present?			
Evidence of isolated shallow ponding?			
Damage/vandalism to structures present?			
Outlet structure free of debris?			
Maintenance drain operational (check)?			
Resetting of system required?			
Comments:			

Vegetation maintenance will include:

- removal of noxious plants or weeds
- replacement 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. Inspection and removal of debris should be done regularly, but debris should be removed whenever it is observed on the site.

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

9.5.1 Operation and maintenance inspection form

The *Wetland Maintenance Checklist* 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 three months for the first year and then six-monthly thereafter. More detailed site specific maintenance schedules should be developed for major wetland systems and include a brief overview of the operation of the system and key aspects to be checked during each inspection.

9.6 Worked example

9.6.1 Worked example introduction

A sedimentation basin and wetland system is proposed to treat runoff from a residential and commercial area located in Shepparton, Victoria. The wetland will consist of an inlet zone designed to treat the larger pollutant sizes. Flow will than pass through into a macrophyte zone where a riser outlet will be used to control the system detention period to settle finer sediment particles. A bypass channel will enable large flood events to bypass the macrophyte zone during

Figure 9.12 Layout of proposed wetland system.

periods when the macrophyte zone is already operating at its design level. This worked example focuses only on the macrophyte zone component of the system with the design of the inlet zone (sedimentation basin) and bypass channel contained in an earlier worked example (see Chapter 4). An illustration of the site and proposed layout of the wetland is shown in Figure 9.12.

The contributing catchment area of the proposed wetland is 10 ha (with percentage imperviousness of 50%). The site is flat with the maximum fall of less than 0.5 m across the site. Stormwater from the catchment is conveyed by conventional stormwater pipes and discharges into the constructed wetland via a single 1000 mm diameter pipe. There are no site constrains with regard to the size of the wetland, as construction can extend into an adjacent park if required.

9.6.2 Design objectives

The design criteria for the wetland system are to:

- promote sedimentation of particles larger than 125 µm within the inlet zone
- optimise the relationship between detention time, wetland volume and the hydraulic effectiveness of the system to maximise treatment given the wetland volume site constraints – simulation using MUSIC has found that a wetland with an extended detention volume of about 650 m^3 will be sufficient to meet best practice water quality objectives, equivalent to a hydraulic effectiveness of 85% for a notional detention period of 72 hours.
- ensure that the required detention period is achieved for all flow though the wetland system by using a riser outlet system
- provide for bypass operation when the inundation of the macrophyte zone reaches the design maximum extended detention depth.
- configuring the layout of the macrophyte zone to provide an extended detention volume of 650 m^3 so that optimum hydraulic efficiency of the system can be achieved – this includes particular attention to the placement of the inlet and outlet structures, the aspect ratio of the macrophyte zone and the need to use bathymetry and other flow control features to promote a high hydraulic efficiency within the macrophyte zone; a key design consideration is the extended detention depth for the macrophyte zoneThis worked example focuses on the design of the macrophyte zone of the wetland system. Analyses to be undertaken during the detailed design phase of the macrophyte zone of the wetland system include the following:
- designing the provision to drain the macrophyte zone if necessary
- designing the connection between the inlet zone and the macrophyte zone appropriately so that inlet conditions provide for energy dissipation and distribution of inflow into the macrophyte zone
- designing the bathymetry of the macrophyte zone to promote a sequence of ephemeral, shallow marsh, and submerged marsh systems in addition to a small, open water system near the outlet structure
- designing the macrophyte zone outlet structure to provide for a 72 hour notional detention time, including a debris trap.

In addition, a landscape design will be need to be provided, including:

- macrophyte zone vegetation (including edge vegetation (littoral zone))
- terrestrial vegetation.

9.6.2.1 Confirming macrophyte zone area

As a basic check of the adequacy of the size of the wetland, reference is made to the performance curves presented in Section 9.2. According to Figures 9.2 to 9.4, the required wetland size to satisfy best practice environment management objectives for stormwater quality (based on 0.5 m extended detention depth) in Melbourne is the larger 2.3% (for 80% reduction in TSS); 1.0% (for 45% reduction of TP) and 2.5% (for 45% reduction of TN) of the impervious area (i.e. 2.5% of the impervious area is the critical size).

According to the **hydrologic region** analysis in Chapter 2, the **adjustment factor** for constructed wetlands in Shepparton is 1.21.

The required wetland area computed using the procedure presented in Chapter 4 is as follows:

Impervious area = 5 ha

Required wetland area (0.5 m extended detention) = 50 000 × 0.025 × 1.21 = 1500 m^2.

Extended detention volume required = 750 m^3 compared to 650 m^3 derived from MUSIC modelling.

The discrepancy between proposed extended detention volume derived from detailed modelling using MUSIC and the value determined from the simple procedure contained in Chapter 2 is within 20% and is considered acceptable.

Proposed required extended detention of 750 m^3 is within the expected size required to achieve best practice environmental management objectives for urban stormwater quality.

9.6.3 Design calculations

9.6.3.1 Estimating design flows

With the catchment area being relatively small, the Rational Method Design Procedure is considered to be an appropriate method for computing the design flows (Q).

Catchment area = 10 ha

t_c = ~ 10 min (Institution of Engineers 2001 methods)

Runoff coefficients (C) Institution of Engineers 2001 Book VIII

$^{10}I_1 = 38.2$ mm/hr

$F_{imp} = 0.5$

$C_{10} = 0.55$ ARR 1998

Runoff coefficients from Table 8.6 in Institution of Engineers 2001

$C_1 = 0.44$

$C_{100} = 0.66$

Institution of Engineers 2001 Rainfall intensities (Shepparton) t_c = 10 min

$I_1 = 38.2$ mm/hr

$I_{100} = 130$ mm/hr

Rational Method $Q = CIA/360$

$Q_1 = 0.47\ m^3/s$

$Q_{100} = 2.4\ m^3/s$

9.6.3.2 Inlet zone

The procedure for the design of the inlet zone follows that presented in Procedure 1 for sediment basin. In this worked example, design computation for the bypass weir and the connection to the macrophyte zone will be presented.

9.6.3.3 Macrophyte zone layout

Size and dimensions

The wetland has been sized to require an extended detention volume of 750 m^3. An extended detention depth of 0.5 m has been adopted requiring a surface area of 1500 m^2.

In this case it has been chosen to adopt a length (L) to width (W) ratio of 6:1. This aspect ratio represents a shape configuration in between Case G and Case I in Figure 9.6 and the expected hydraulic efficiency is 0.6. This is lower than ideal for a wetland; however, the space constraints of the site limit the available area for the macrophyte zone.

Aspect ratio is **6(L):1(W)**; hydraulic efficiency **~ 0.6**

To calculate the dimensions:

L = 6W

Wetland area = 6W × W = 1500

Proposed dimensions are 95 m × 16 m.

Notional macrophyte zone dimensions are **95 m (L) x 16 m (W).**

Zonation

The wetland is broadly divided into four macrophyte zones, an open water zone and a littoral zone. The bathymetry across the four macrophyte zones is to vary gradually over the depth range outlined below. The depth of the open water zone near the outlet structure is to be more than 1.0 m below the permanent pool level.

Table 9.2 Percentage of wetland surface allocated to macrophyte zones and open water

Zone	Depth range (m)	Percentage of macrophyte zone surface area (m)
Open Water	>1.0 below permanent pool	10%
Submerged Marsh	0.5–1.0 below permanent pool	10%
Deep Marsh	0.35–0.5 below permanent pool	25%
Marsh	0.2–0.35 below permanent pool	25%
Shallow Marsh	0.0–0.2 below permanent pool	25%
Littoral (edges)	+0.5–0.0 above permanent pool	5%

Each zone has varying depths, but within each zone there are bands of equal depth across the flow path. The appropriate bathymetry coupled with uniform plant establishment ensures the cross section has equivalent hydraulic conveyance, thus preventing short-circuiting.

Wetland consists of four macrophyte zones arranged in bands of equal depth running across the flow path.

Long section

Shepparton has a relatively low Mean Annual Rainfall (MAR, 563 mm), with much of the rainfall falling in winter and spring. The region also has high summer evaporation. It is, therefore, likely that water losses during summer will be high and it will be necessary to provide areas of habitat refuge. For this reason it is desirable to have areas of permanent pool interconnected to prevent fauna being isolated in areas that dry out. The proposed long section is for the bed of the wetland to gradually deepen over the four macrophyte zones (i.e. excluding edges). This profile also facilitates draining of the wetland.

Long section of the macrophyte zone is to be gradually deepening over the four macrophyte zones ranging from the permanent pool level (shallow marsh) to 1.0 m below the permanent pool (submerged marsh zone).

Cross sections

The batter slopes on approaches and immediately under the permanent water level have to be configured with consideration of public safety. A batter slope of 1(V):8(H) from the top of the extended detention depth to 0.3 m beneath the water line before steepening into a 1(V):3(H) slope is recommended as a possible design solution (see Figure 9.13). The safety requirements for individual wetlands may vary from site to site, and it is recommended that an independent safety audit be conducted of each design.

Cross section of macrophyte zone is trapezoidal in shape with a **base width of 8 m** and a **top width of 22.0 m**.

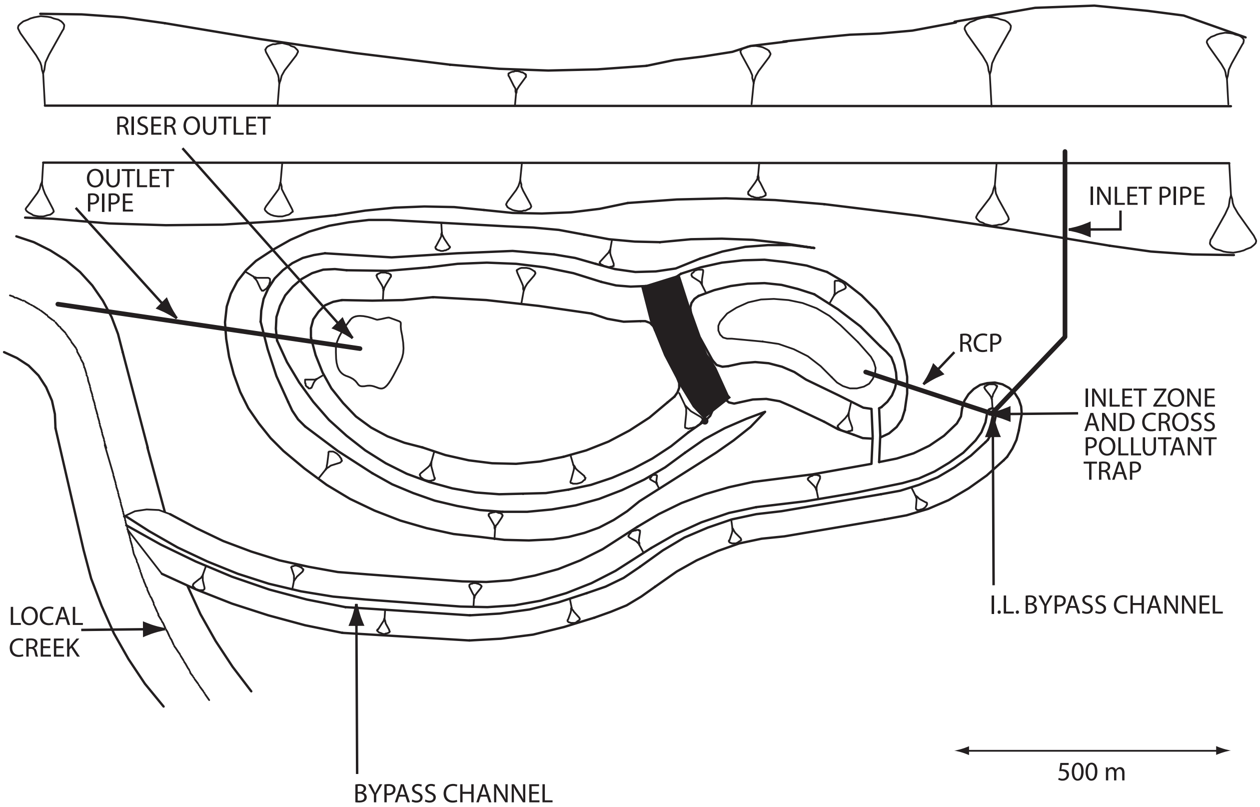


Figure 9.13 Typical cross section of macrophyte zone.

9.6.3.4 Macrophyte zone outlet structure

Maintenance drain

A maintenance drain will be provided to allow drainage of the system. Valves will be operated manually to drain the inlet zone and macrophyte zone independently.

The mean flow rate (Q) for the maintenance drain is selected to draw down the permanent pool over a notional 12 hours and is computed as follows:

Permanent pool volume ~ 375 m^3 (assuming approximate 0.25 m nominal depth)

$$Q = 375/(12 \times 3.6) = 9 \text{ L/s.} \qquad \text{(Equation 9.5)}$$

To determine the area of the orifice for the drain, it assumed that the valve orifice will operate under inlet control with its discharge characteristics determined by the orifice equation (Equation 9.1):

$$A_o = \frac{Q}{C_d\sqrt{2gh}}$$

$Q = 375/(12 \times 3600) = 0.009$ m^3/s
$C_d = 0.6$
$h = 0.33$ m (one-third of permanent pool depth)

Giving $A_o = 0.0018$ m^2 corresponding to an orifice diameter of 150 mm – adopt 150 mm.

Pipe valve to allow draining of the permanent pool for maintenance to be **at least 150 mm diameter.**

Riser outlet – size and location of orifices

The riser is designed to provide a uniform notional detention time over the full range of the extended detention depth.

Target Q_{max} = extended storage volume/detention time
= 750/(72 × 3.6) = 2.9 L/s

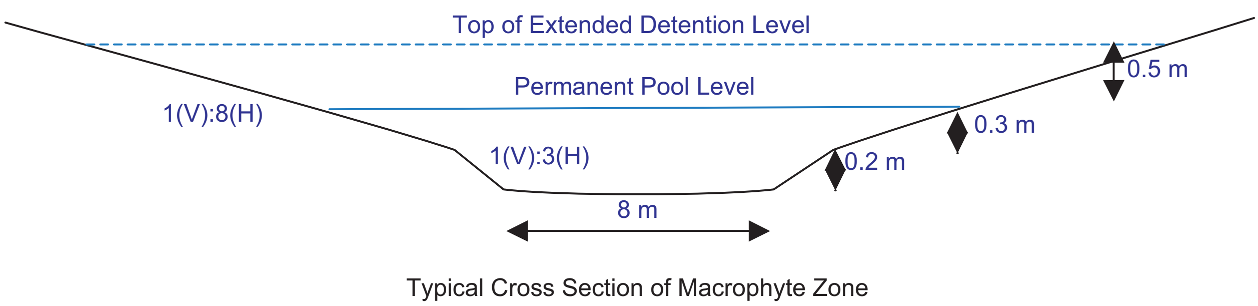


Figure 9.14 Typical cross section.

Outlet orifices along the riser are located at 0.167 m intervals along the length of the riser (i.e. at 0 m, 0.167 m and 0.333 m above the permanent pool level). A standard orifice diameter of 20 mm was selected and the numbers required at each level were determined iteratively using a spreadsheet (Table 9.3) and applying the orifice equation (Equation 9.1) applied over discrete depths along the length of the riser up to the maximum detention depth. The results of the design are summarised in Table 9.3. The stage–discharge relationship of the riser is plotted in Figure 9.14 and shows that the riser maintains a linear stage–discharge relationship.

Table 9.3 Determination of orifice positions

		Q_1	Q_2	Q_3	Total flow (L/s)	Notional detention time (hr)
Orifice positions (invert level)		0	0.167	0.333		
Orifice diameter (mm)		20	20	20		
number		2	1	1		
Water depth (m)	**Volume**					
0	0	0			0	
0.167	139	0.662			0.662	58.16
0.333	317	0.945	0.327		1.272	69.29
0.5	549	1.169	0.475	0.334	1.978	77.09

As the wetland is relatively small, the required orifices are small, and it is necessary to include measures to prevent blocking of the orifices.

The riser is to be installed within an outlet pit with a culvert connection to the permanent pool of the macrophyte zone. The connection is via a 300 mm diameter pipe. The pit is accessed via the locked screen on top of the pit.

The riser pipe should not be smaller than the pipe conveying the outflow from the wetland to the receiving waters (see 'Discharge pipe', p. 178).

Table 9.4 Outlet risers

Outlet riser consist of three rows of orifices of 20 mm diameter located as follows:	
Depth above permanent pool (m)	**No. of 20 mm diameter orifices**
0.000	2
0.167	1
0.333	1

Riser outlet – pipe dimension

As designed, high flows would bypass around the macrophyte zone when this zone is already operating at design capacity (i.e. when the water level in the macrophyte zone reaches the top of its extended detention). A notional riser pipe diameter of 150 mm is thus sufficient.

Riser pipe to be **150 mm diameter.**

Discharge pipe

The discharge pipe of the wetland conveys the outflow of the macrophyte zone to the receiving waters (or existing drainage infrastructure). Under normal operating conditions, this pipe will need to have sufficient capacity to convey the larger of the discharges from the riser or the maintenance drain:

- the maximum discharge from the riser = 1.3 L/s
- the maximum discharge through the maintenance pipe occurs for depth of 1.0 m (i.e. depth of open water zone); the maximum discharge through the 90 mm diameter valve is computed to be 17 L/s.

The required pipe diameter for the outlet should thus be larger than 90 mm. Since the diameter of the riser has been selected to be 150 mm, it is appropriate to also use this dimension for the outlet pipe connecting the wetland to the adjoining creek.

Outlet pipe for wetland for discharge to receiving waters (or existing drainage infrastructure) is to be **150 mm** diameter.

9.6.3.5 Connection to the inlet zone

The configuration of the hydraulic structure connecting the inlet zone to the macrophyte zone consists of an overflow pit (in the inlet zone) and a pipe with the capacity to convey the one-year ARI peak discharge of 0.47 m^3/s.

Design specifications:

Bypass spillway level = top of extended detention in the macrophyte zone

Permanent pool level in inlet zone = 0.3 m above permanent pool level in macrophyte zone

In designing the culvert connecting the inlet zone to the macrophyte zone, the following conditions apply:

Headwater level = 0.5 m above macrophyte permanent pool level

Tail water level = permanent pool level

Design flow = 0.47 m^3/s.

Assume culvert under outlet control; $K_{in} = 0.5$, $K_{out} = 1$, $n = 0.015$

Try 1 by 450 mm diameter capacity = 0.41 m^3/s Too small

Try 3 Nos 300 mm diameter capacity = 0.55 m^3/s OK.

Culverts connecting inlet zone to macrophyte zone is to be **3** Nos 300 mm diameter.

Velocity checks are to be conducted for when the wetland is full and when it is at permanent pool level. For the velocity checks, the maximum inflow corresponding to the one-year ARI peak discharge is used (i.e. 0.47 m^3/s).

Flow check $V = Q/A$

When full:

$A = 15 \times 0.5 = 7.5\ m^2$; $V = 0.06$ m/s no risk of scour

When at permanent pool level:

$A = 15 \times 0.1 = 1.5\ m^2$; $V = 0.3$ m/s no risk of scour.

9.6.3.6 High-flow route and bypass design

The bypass weir level at the inlet zone is set to match the top of the extended detention level in the macrophyte zone. The length of the spillway (L) is to be sized to safely pass the 100-year ARI discharge with a water level over the weir of 0.3 m (i.e. top of wetland embankment).

The 100-year ARI peak discharge = 2.4 m^3/s

Crest level = 0.5 m above macrophyte permanent pool

Freeboard (top of wetland embankment) = 0.3 m

$k_w = 1.7$ (sharp-crested weir)

$$\text{Weir flow, } Q = k_w \times L \times H^{1.5} \qquad \text{(Equation 9.6)}$$

Therefore, $L = Q / k_w \times H^{1.5}$

$Q = 2.4\ m^3/s$ (100-year ARI flow from contributing catchment)

$H = 0.30$ m

$L = 8.6$ m

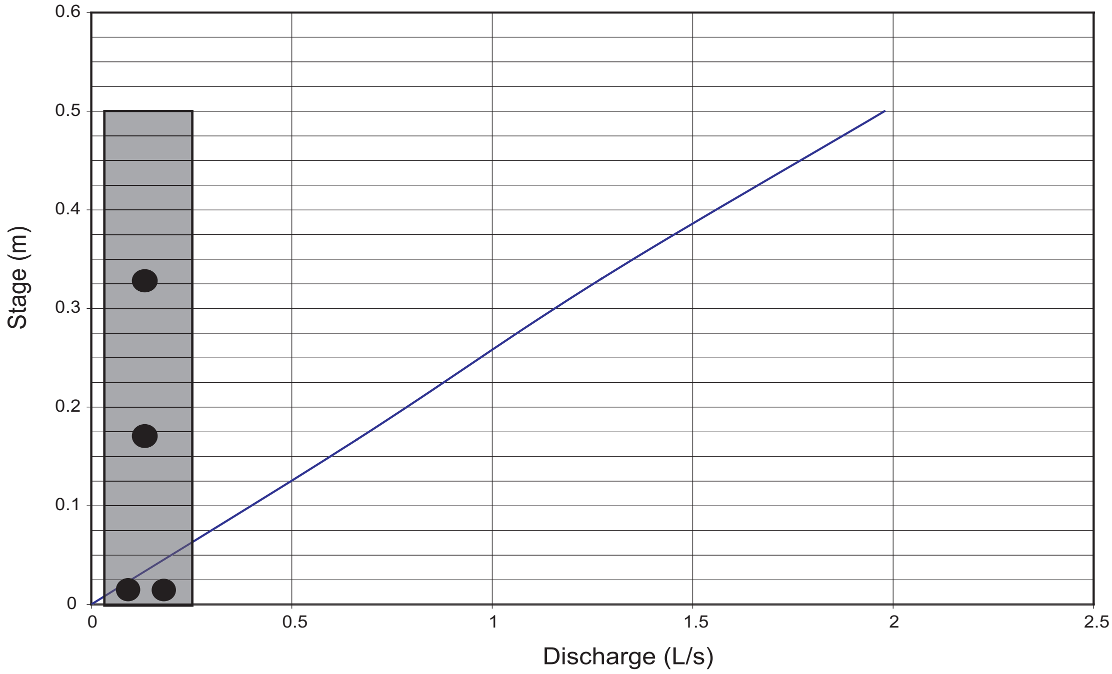


Figure 9.15 Positioning of orifice outlets (see attached CD).

The spillway length is to be 9.0 m set at a crest level 0.5 m above the permanent pool level of the macrophyte zone.

9.6.3.7 Vegetation specifications

The vegetation specification and recommended planting density for the macrophyte zone are summarised in Table 9.5 (see Appendix A for further discussion and guidance).

Table 9.5 Vegetation specifications

Zone	Plant species	Planting density (plants/m^2)
Littoral berm	*Persicaria decipens*	3
Ephemeral marsh	*Blechnum minus*	6
Shallow marsh	*Cyperus lucidus*	6
Marsh	*Bolboschoenus caldwellii*	4
Deep marsh	*Juncus ingens*	8

9.6.4 Design calculation summary

The completed *Constructed Wetland Calculation Summary* shows the results of the design calculations.

Constructed Wetand — CALCULATION SUMMARY

CALCULATION TASK	OUTCOME		CHECK
1 Identify design criteria			
Design ARI flow for inlet zone	1	year	
Target sediment size for inlet zone	0.125	mm	
Notional detention period for macrophyte zone	72	hr	
Design ARI flow for bypass spillway	100	year	
Extended detention volume	750	m^3	✓
2 Catchment characteristics			
Residential	7	ha	
Commercial	3	ha	
Fraction impervious			
Residential	0.4		
Commercial	0.7		✓
3 Estimate design flow rates			
Time of concentration			
Estimate from flow path length and velocities	10	minutes	✓
Identify rainfall intensities			
Station used for IFD data:	Shepparton		
100-year ARI	130	mm/hr	
1-year ARI	38.2	mm/hr	✓
Design runoff coefficient			
C_1	0.44		
C_{100}	0.66		✓
Peak design flows			
Q_1	0.47	m^3/s	
Q_{100}	2.400	m^3/s	✓
4 Inlet zone			
Refer to *Sedimentation Basin Calculation Summary*			✓
5 Macrophyte zone layout			
Extend detention depth	0.5	m	
Area of macrophyte zone	1500	m^2	
Aspect ratio	6(L):1(W)	L:W	
Hydraulic efficiency	0.6		
Length	95	m	
Top width (including extended detention)	16	m	
Cross section batter slope	1(V):8(H)	V:H	✓
6 Macrophyte zone outlet structures			
Maintenance drain			
Diameter of maintenance valve	90	mm	
Drainage time	12	hr	✓
Riser			
Linear storage-discharge relationship for riser			✓
Discharge pipe			
Discharge capacity of discharge pipe	0.75	m^3/s	✓
7 Connection between inlet zone and macrophyte zone			
Discharge capacity of connection culvert	0.55	m^3/s	✓
8 Bypass weir			
Discharge capacity of bypass weir	2.4	m^3/s	✓

9.6.5 Construction drawings

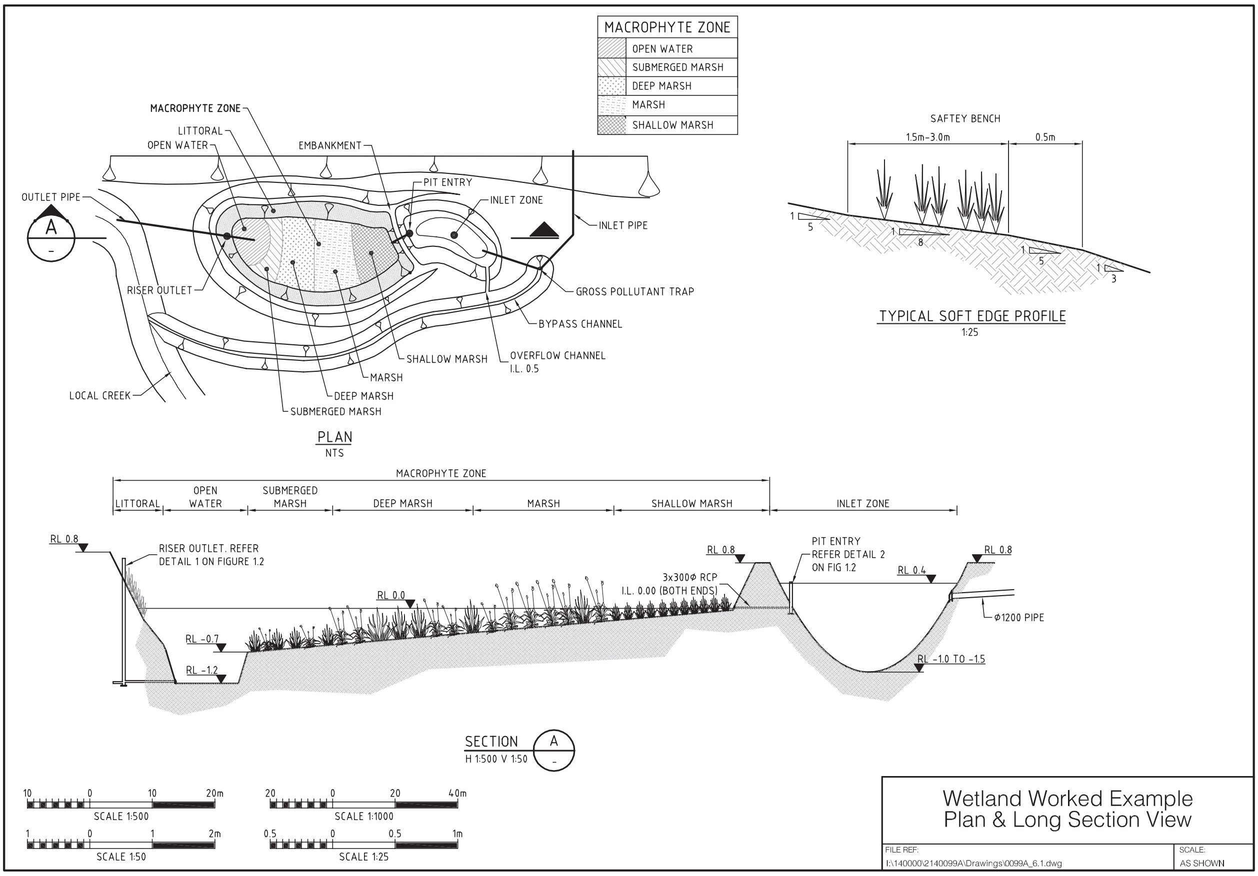


Figure 9.16 Wetland worked example plan and long section view

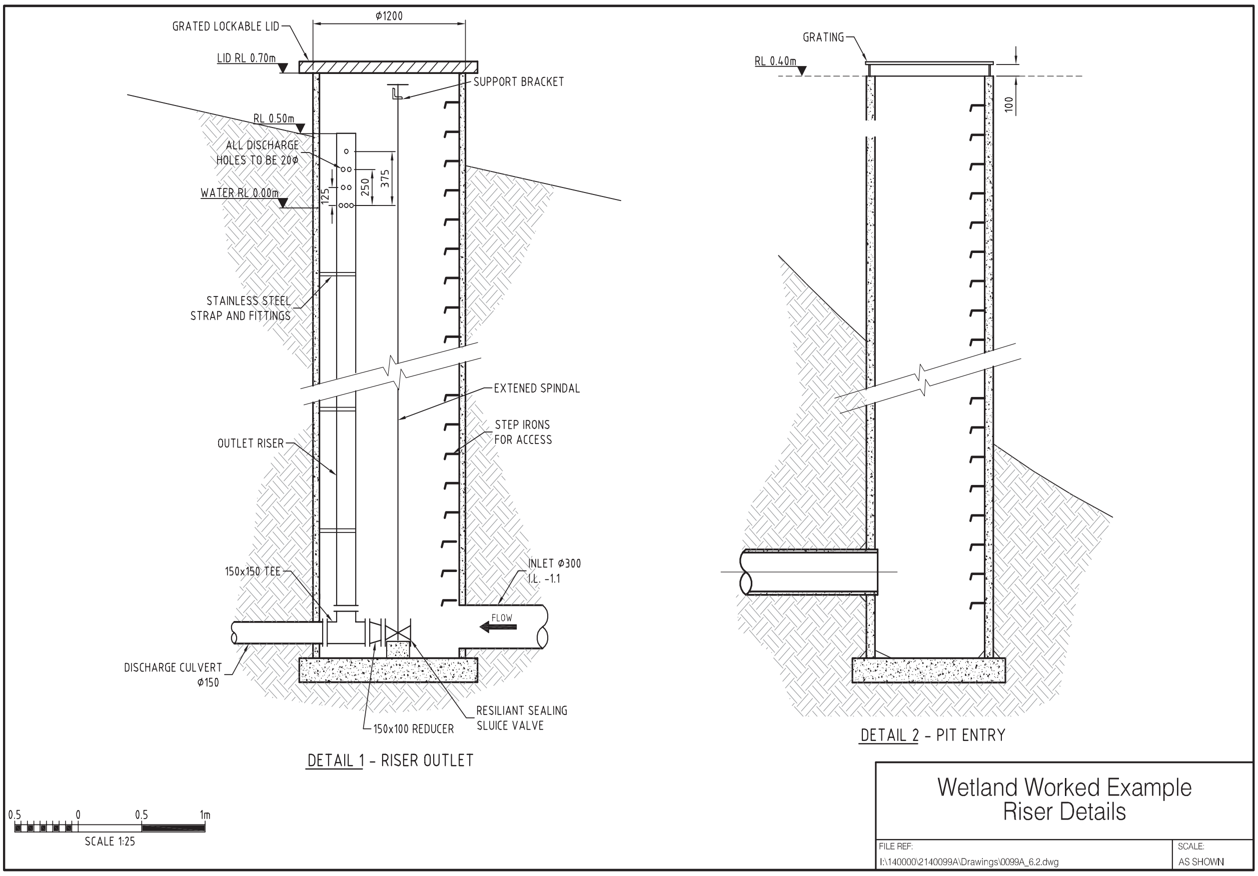


Figure 9.17 Wetland worked example riser details

9.6.6 Example inspection and maintenance schedule

An example inspection and maintenance schedule, the *Wetland Maintenance Checklist*, is included for a constructed wetland showing local adaptation to incorporate specific features and configuration of each individual wetland. The *Shepparton Wetlands Maintenance Form* is an inspection sheet developed for the Shepparton wetland, modified from the generic *Wetland Maintenance Checklist*.

SHEPPARTON WETLANDS – MAINTENANCE FORM	
Location	
Description	Constructed wetland and sediment forebay

SITE VISIT DETAILS	
Site visit date:	____________________
Site visit by:	____________________
Weather	____________________

Purpose of the site visit	Tick Box	Complete Sections
Routine inspection	☐	Section 1 only
Routine maintenance	☐	Section 1 and 2
Cleanout of sediment	☐	Section 1, 2 and 3
Annual inspection	☐	Section 1, 2, 3 and 4

SECTION 1 – INSPECTION	
Depth of sediment:	__________ m
Cleanout required if depth of sediment >1.0 m	Yes/No
Any weeds or litter in wetland (If Yes, complete Section 2 – Maintenance)	Yes/No
Any visible damage to wetland or sediment basin? (If Yes, completed Section 4 – Condition)	Yes/No
Inspection comments:	

SECTION 2 – MAINTENANCE	
Are there weeds in the wetland?	Yes/No
Were the weeds removed this site visit?	Yes/No
Is there litter in the wetland or forebay?	Yes/No
Was the litter collected this site visit?	Yes/No

SECTION 3 – CLEANOUT OF SEDIMENT		
Have the following been notified of cleanout date?	Yes	No
Coordinator – open space and/or drainage	☐	☐
Local residents	☐	☐
Other (specify …………………………………………)	☐	☐
Method of cleaning (excavator or eductor)		
Volume of sediment removed (approximate estimate)	m^3	
Any visible damage to wetland or sediment forebay? (If yes, complete Section 4 – Condition)	Yes/No	

SECTION 4 – CONDITION

Component	Checked?		Condition OK?		Remarks
	Yes	**No**	**Yes**	**No**	
Inlet weir or pipes					
Outlet riser/s and weir/s					
Sediment forebay					
Bypass channel (if constructed)					
Wetland vegetation					
Wetland banks and batter slopes					
Wetland floor					
Wetland diversion bunds (if constructed)					
Retaining walls					
Surrounding landscaping					
Comments:					

Shepparton Wetlands Maintenance Form

9.7 References

Chadwick, A.J. and Morfett, J.C. (1986). *Hydraulics in Civil Engineering*, Allen and Unwin, London.

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

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

Graeme Bentley Landscape Architects (GbLA) (2004). *Preliminary drawings for Mernda Wetland*, Report for Stockland.

Institution of Engineers, Australia (2001). *Australian Rainfall and Runoff – A Guide to Flood Estimation*, Revised edn, Pilgram, D.H. (Ed.), Institution of Engineers, Australia, Barton, ACT.

Melbourne Water (2003), *Constructed Wetland Systems*, *Design Guidelines for Developers*, Melbourne Water, Melbourne.

Royal Life Saving Society of Australia (2004). *Guidelines for Water Safety in Urban Water Developments*, Royal Life Saving Society of Australia, Broadway, NSW.

Persson, J., Somes, N.L.G. and Wong T.H.F. (1999). Hydraulic efficiency and constructed wetland and ponds, *Water Science and Technology*, 40(3), 291–289.

Chapter 10 **Ponds and Lakes**

10.1 Introduction

Ponds and lakes are artificial bodies of open water usually formed by a simple dam wall with a **weir** outlet structure or created by excavating below natural surface levels. The depth of water in these waterbodies is typically greater than 1.5 m and there is usually a small range of water level fluctuation although newer systems may have riser style outlets allowing for **extended detention** and longer temporary storage of inflows. Aquatic vegetation has an important function for water quality management in ponds and lakes. Emergent **macrophytes** are normally restricted to the margins because of water depth, although submerged plants may occur in the open water zone. Submerged plants are important to the maintenance of both biological processes and water quality. They provide a surface for the absorption of dissolved nutrients and provide food and shelter for zooplankton which may graze on algal species. The oxygen released during photosynthesis is important in maintaining oxygen saturation in the water column which is depleted by animal respiration and decomposing organic matter. Vegetation can also help stabilise sediments and reduce the release of sediment-bound nutrients arising from resuspension processes. Ponds are seldom used as 'stand-alone' stormwater treatment measures and are often combined with **constructed wetlands** as a treatment forebay to the open waterbody. In many cases, these ponds ultimately become the ornamental waterbody that require water quality protection.

Ponds and lakes often form part of a flood-retarding system and design requirements are generally associated with hydraulic structures for flow conveyance and flood attenuation. These are not covered in this document and only design elements associated with the water quality function of the system is presented.

There have been cases where water quality problems in ornamental ponds and lakes have been caused by poor inflow water quality, especially high organic load, infrequent waterbody 'turnover' and inadequate mixing. Detailed modelling may be necessary to track the fate of nutrients and consequential algal growth in the waterbody during periods of low inflow (and thus long detention period). As a general rule, it is recommended that the turnover period for

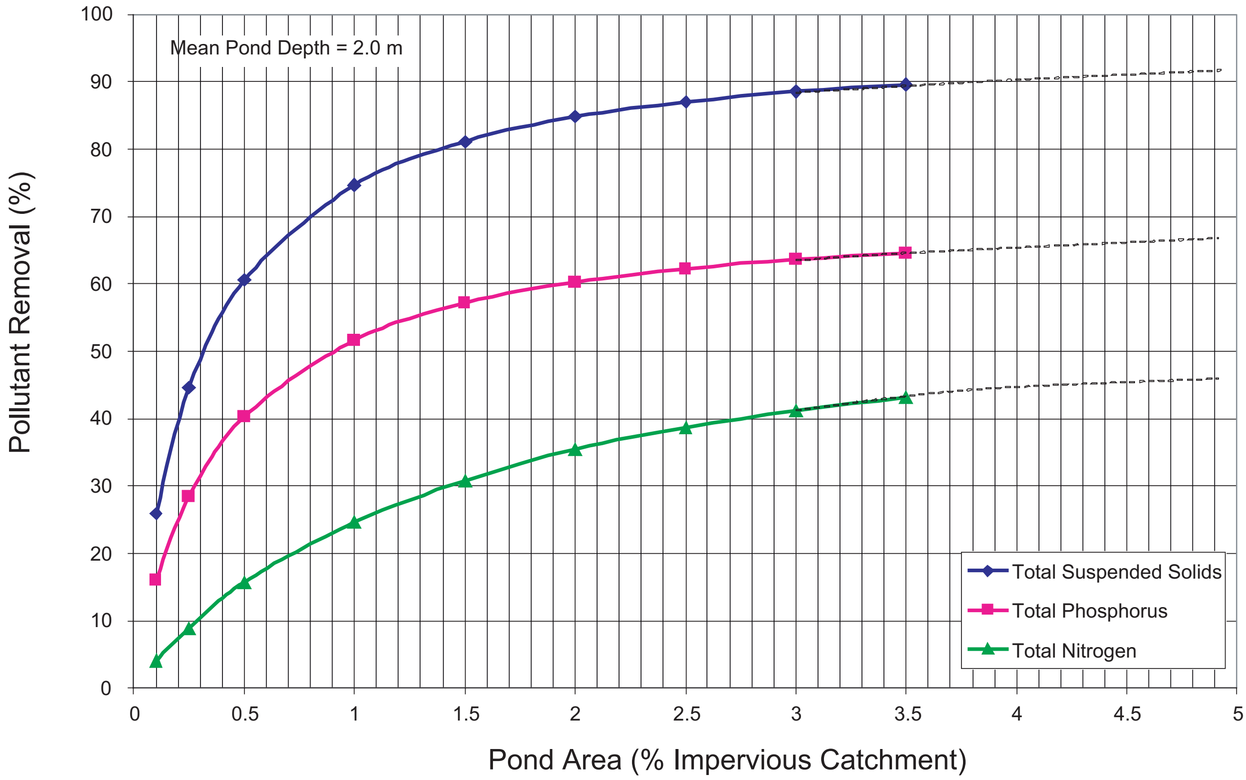


Figure 10.1 Performance of ponds and lakes in removing Total Suspended Solids (TSS), Total Phosphorus (TP) and Total Nitrogen (TN) in Melbourne.

lakes is between 20 and 50 days (depending on water temperatures) at least 80% of the time (see Appendix D). If these turnover times can not be met, it may be necessary to introduce a lake management plan to reduce the risk of algal blooms during the dry season. This design procedure outlines design elements for large waterbodies associated with the design of a constructed lake, an associated **wetland** forebay (or **inlet zone**) and water recirculation scheme (if required) to maintain water quality in the pond. Further investigations need to be undertaken to finalise the design from that presented in the worked example to address issues such as the embankment stability and detailed design. These are discussed in the worked example.

The design procedure is primarily concerned with the risk of cyanobacterial blooms and therefore the health risk of these systems. Nuisance green algal blooms may occur more frequently than cyanobacterial blooms and possibly affect the amenity of the system, particularly in residential areas. Acknowledgement of these issues is essential to any waterbody design, construction and handover process. A number of additional steps can be taken in the planning and design of open waterbodies to minimise algal growth:

- Ensure that pre-treatment of stormwater is adequate to prevent large nutrient 'spikes' entering the system.
- Submerged macrophytes should include a minimum of 50% area cover and 50% volumetric coverage of the lake. A greater cover is highly recommended.
- The lake should be oriented to the dominant winds to facilitate mixing, particularly for summer and autumn; edge treatments should be designed to minimise wave damage.

Residents and managing authorities must be aware that as the lake system ages, it has a greater chance of problem algal growth and algal blooms. It is possible that a lake system may require desilting and a total 'reset' if algal blooms become a recurrent and persistent problem.

10.2 Verifying size for treatment

The curves shown in Figure 10.1 describe the pollutant removal performance expected for constructed pond and lake systems in Melbourne (reference site) for Total Suspended Solids

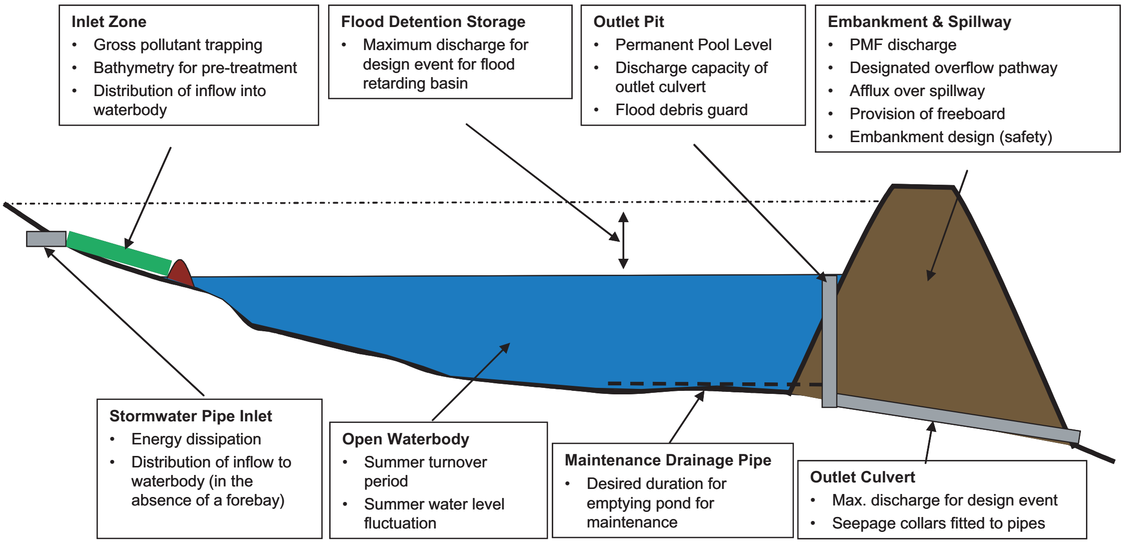


Figure 10.2 Pond or lake design elements and design considerations.

(TSS), Total Phosphorus (TP) and Total Nitrogen (TN). The curves were derived assuming the systems receive direct runoff (i.e. no other Water Sensitive Urban Design, WSUD, elements upstream) and have the following characteristics:

- mean depth of 2.0 m
- outflow from the system is via an overflow weir.

These curves can be used, together with the **adjustment factors** derived from the hydrologic regionalisation procedure discussed in Chapter 2, to check the expected performance of the wetland system for removal of TSS, TP and TN.

10.3 Design procedure: ponds and lakes

Design considerations include the following:

1. computations to ensure that the pond volume is not excessively large or too small in comparison to the hydrology of the **catchment**
2. configuration of the layout of the pond and inlet zone such that the system's hydraulic efficiency can be optimised, including a transition structure between the inlet zone and the open waterbody
3. design of hydraulic structures, including
   - inlet structure to provide for energy dissipation of inflows up to the 100-year Average Recurrence Interval (ARI) peak **discharge**
   - design of the pond outlet structure for the pond
4. landscape design, including
   - edge treatment
   - recommended plant species and planting density
5. maintenance provisions.

Figure 10.2 summarises the pond/lake design elements. The following sections describe the design steps required for ponds and lakes.

10.3.1 Hydrology

The hydrologic operation of a pond or lake is to safely convey stormwater inflows up to the peak 100-year ARIdischarge into the pond or lake system with discharge from the pond or lake being via a combination of pipe (low flow) culvert and overflow spillway.

10.3.1.1 Flood estimation

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

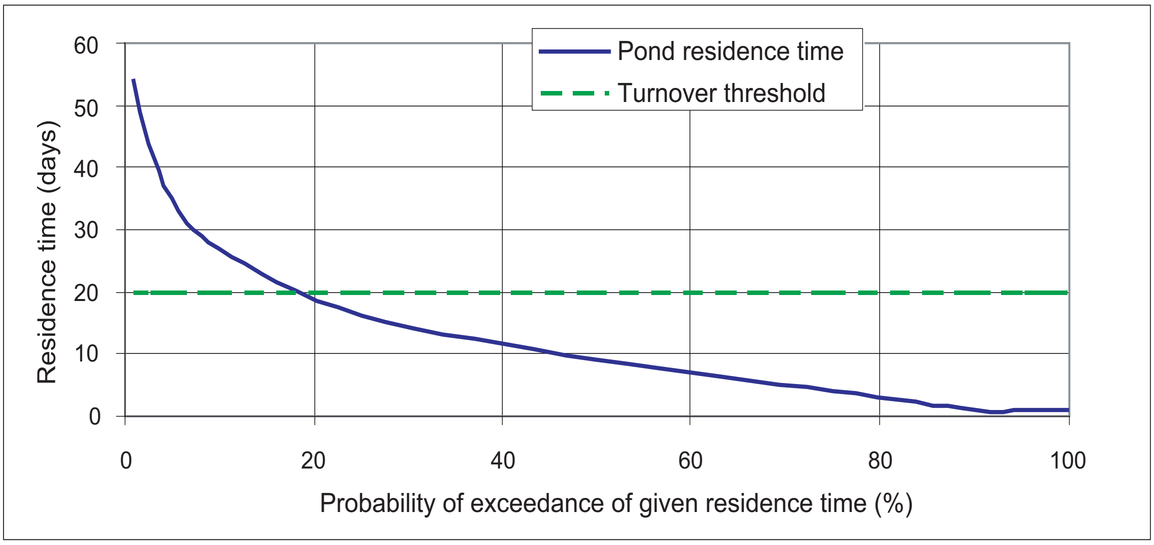


Figure 10.3 Results of residence time analysis for a waterbody in Melbourne.

method for estimating design flows. However, the use of the Rational Method Design Procedure should strictly be used only to size inlet hydraulic structures A full flood routing computation method should be used in sizing the outlet hydraulic structures (e.g. outlet pipe, spillway and embankment height).

10.3.1.2 Waterbody residence time

A combination of inflow water quality, organic load and water circulation characteristics influence the water quality in the pond. Water quality problems for large lakes exhibiting relatively small upstream catchments typically arise because the water body receives insufficient water inflows to circulate and/or displace the water stored in the lake. Under long residence times blooms of cyanobacteria (blue-green algae) can occur.

Experience with management of many open waterbodies suggests that a large number of these incidences of algal blooms are preceded by extended periods of no or minimal inflows. Waterbody residence time (or turnover frequency) analysis can often be a very useful indicator as to whether the waterbody is at significant risk of water quality problems (especially associated with algal growth) (see Appendix D).

Turnover analysis can be undertaken using probabilistic monthly evaporation and rainfall data or daily historical rainfall data, with the latter providing a more rigorous analysis. Average residence times are calculated by modelling continuous simulation of flows into and out of a lake. Estimates of daily outflows are then summed (in arrears) to give an estimate of the average residence time of the lake for each day of the simulation.

Seasonal distribution of rainfall and the relative volume of the waterbody to the mean annual runoff will determine the range of residence periods for the waterbody. For example, a small waterbody with a large catchment will have small residence times because the volume of the waterbody is a small fraction of the mean annual runoff volume of the catchment. However, the residence times of a larger waterbody will be more sensitive to seasonality of rainfall and thus be at a higher risk of long periods of water detention and associated water quality problems.

A cumulative probability distribution of exceedance versus waterbody residence time can be derived using the modelled outflows from a lake (e.g. Figure 10.3).

Algal growth can occur rapidly under favourable conditions. Growths (blooms) of cyanobacteria (blue-green algae) can occur in both natural and constructed water bodies. In constructed water bodies it is important to ensure that designs include measures to restrict cyanobacterial growth. Cyanobacterial blooms can have adverse effects on aquatic ecosystem function, aesthetics and public amenity. Some species of cyanobacteria are of particular concern because of their potential to produce toxins.

Many factors influence cyanobacterial growth including (Sherman et al. 1998; Mitrovic et al. 2001; Tarczynska et al. 2002; Reynolds 2003):

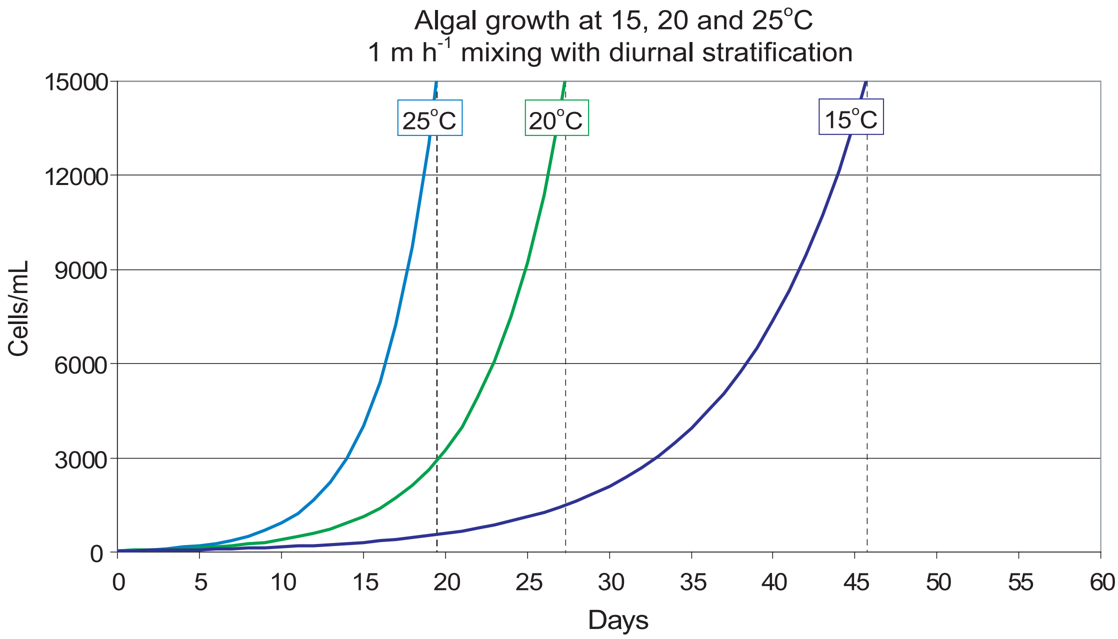


Figure 10.4 Growth curves illustrating modelled times for cyanobacterial populations to reach bloom proportions under different temperature conditions and 1 m/h mixing conditions with diurnal stratification. Based on growth rates of *Anabaena. circinalis* measured *in situ* (Westwood and Ganf 2004) adjusted for temperature, a Q_{10} 2.9, and assuming starting concentrations of 50 cells/mL.

- light intensity
- water temperature
- nutrient concentration
- hydrodynamics
- stratification
- catchment hydrology
- zooplankton grazing
- parasitism.

Excessive growth of cyanobacterial species is considered an 'Alert Level 1 Algal Bloom' when concentrations reach 15 000 cells/mL (Government of Victoria 1995) (see Appendix D).

Assuming adequate light and nutrient availability, a model of algal growth can be developed using a simple relationship between time and growth rate at various temperatures (see Appendix D). This simple model can be used to determine how long it will take for an algal population to reach bloom proportions (15 000 cells/mL) and hence inform the development of guidelines on water body hydraulic **detention time**.

Modelling conducted and based on reasonable assumptions suggests the following times (Figures 10.4 and 10.5) under ideal conditions for blooms to occur depending on mixing conditions (Appendix D). Figures 10.4 and Figure 10.5 were derived assuming a 'best practice' design of a pond. This includes a pond having a shallow depth, a flat bottom and being well mixed. A reasonable assumption is that the hydrodynamic conditions in a best-management practice design varies somewhere between fully mixed and diurnally, partially mixed.

The curves represent three temperature zones in Victoria relating to summer water temperature as follows:

- 15°C Use for upland sites in the Eastern and Western Ranges
- 20°C Use for lowland sites south of the Great Dividing Range
- 25°C Use for lowland sites north of the Great Dividing Range.

The modelling approach taken is considered to be reasonably conservative. For example, it adopts:

- non-limiting conditions for nutrient and light availability
- growth rates for a known nuisance species (*Anabaena circinalis*)

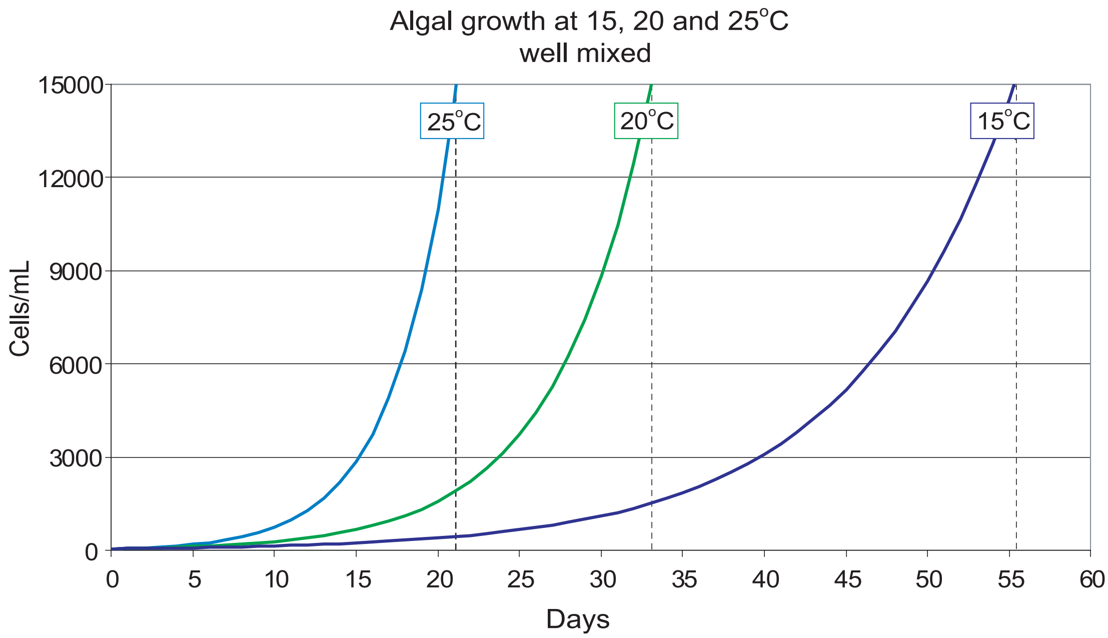


Figure 10.5 Growth curves illustrating modelled times for cyanobacterial populations to reach bloom proportions under different temperature conditions and well mixed conditions. Based on growth rates of *Anabaena circinalis* measured *in situ* (Westwood and Ganf 2004) adjusted for temperature, Q_{10} 2.9, and assuming starting concentrations of 50 cells/mL.

- summer temperature values (the main risk period)
- high starting population concentrations (50 cells/mL).

As a result, a probabilistic approach to the use of detention time criteria is recommended. A 20% exceedance is suggested as an acceptable risk to compensate for the occurrence of all other risk factors being favourable for algal growth. The 20% exceedance of a specific detention time objective does not indicate that a bloom will occur; just that detention time (for a given temperature range) is long enough for exponential growth to achieve a bloom alert level of 15 000 cells/mL if all other risk factors were favourable. The 20% exceedance value is an interim value chosen as a relatively conservative estimate of the general variation in ecological factors in the Australian environment.

10.3.1.3 Turnover design criteria

The following guidelines for detention times are recommended. For water bodies with summer water temperatures in the following ranges, the 20th percentile detention times should not exceed:

- 50 days (15°C)
- 30 days (20°C)
- 20 days (25°C).

These values are broadly consistent with detention time values published in the literature that are considered to be protective against the risk of cyanobacterial blooms (Reynolds 2003, Wagner-Lotkowska et al. 2004) and consistent with current industry experience.

10.3.1.4 Lake water level fluctuation analysis

Analysis of the fluctuation in water levels is another important analysis that needs to be undertaken as these levels may have a significant influence on the landscape design of the lake's edge. As in the waterbody turnover analysis, lake water level analysis can be undertaken using probabilistic monthly evaporation and rainfall data or daily historical rainfall data, with the latter providing a more rigorous analysis. A variety of models can be used to predict water levels from continuous simulations (e.g. Model for Urban Stormwater Improvement Conceptualisation, **MUSIC**) (Cooperative Research Centre for Catchment Hydrology 2003). A typical analysis may be to determine the 10th percentile, 50th percentile and 90th percentile water depths in a lake during summer (e.g. Figure 10.6).

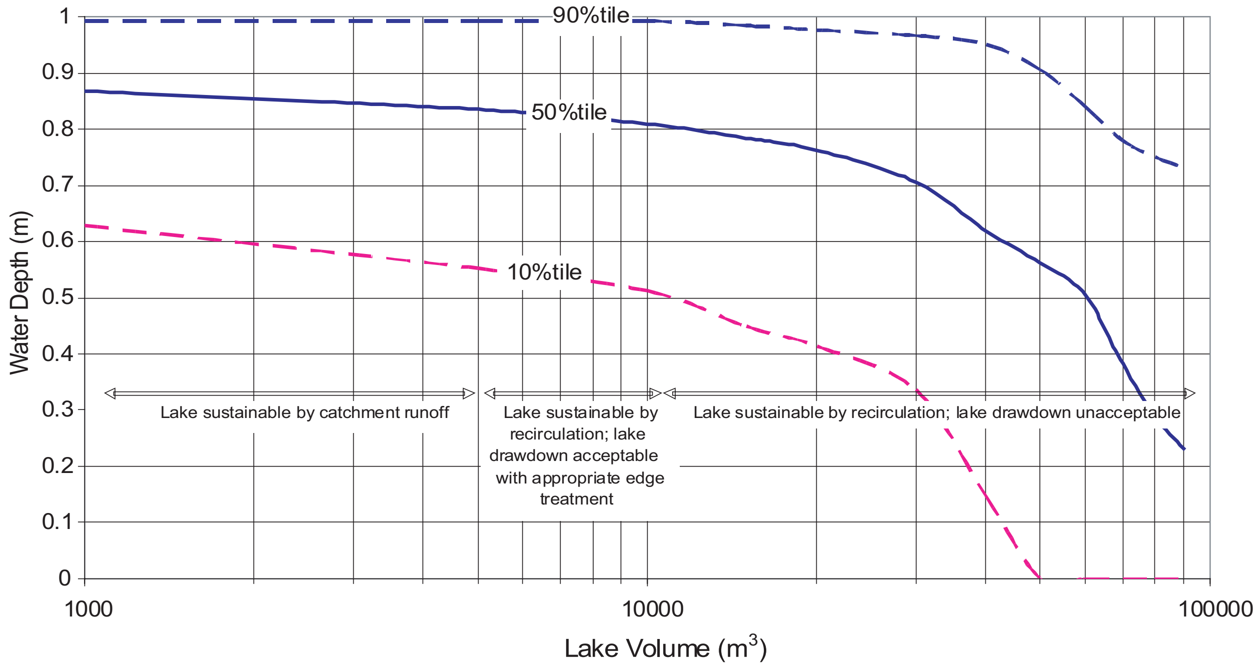


Figure 10.6 Analysis of probabilistic summer water depth with different lake volume for a proposed lake in Shepparton, Victoria.

10.3.1.5 Option for a larger waterbody

Often much larger open waterbodies are proposed by landscape and urban designers to be converted to ornamental lakes. This can mean further design and operation considerations necessary to maintain a healthy waterbody, to provide an acceptable low level of risk of algal growth.

If an analysis indicates that a waterbody is at significant risk of algal blooms (i.e. the turnover design criteria are not met) a lake turnover strategy will need to be developed. In addition, a lake management plan may be required and involve more detailed modelling using such models as the Cooperative Research Centre for Freshwater Ecology's Pond Model.

10.3.2 Pond layout

10.3.2.1 Size and dimensions

To optimise hydraulic efficiency (i.e. reduce short circuits and dead zones), it is desirable to adopt a high length to width ratio and to avoid zones of water stagnation. The ratio of length to width varies depending on the size of the system and the site characteristics wheras inlet and outlet conditions as well as the general shape of the pond can influence the presence and extent of water stagnation zones. To simplify the design and earthworks, smaller systems tend to have length to width ratios at the lower end of the range. This can often lead to poor hydrodynamic conditions.

Persson et al. (1999) used the term 'hydraulic efficiency' to define the expected hydrodynamic characteristics for a range of configurations of stormwater detention systems. Engineers Australia (2003) present expected hydraulic efficiencies of detention systems for a range of notional shapes, aspect ratios and inlet/outlet placements within stormwater detention systems and recommends that the λ value for such systems should not be less than 0.5 and should be designed to promote hydraulic efficiencies greater than 0.7 (see Figure 10.7). The value for λ is estimated from the configuration of the basin according to Figure 10.7.

The numbers in Figure 10.7 represent the values of λ that are used to estimate the turbulence parameter $\boldsymbol{n}$ for Equation 4.2 (see Chapter 4) or Equation 10.2 (see Section 10.3.2.2). In Figure 9.6, 'o' in diagrams O and P represent islands in a waterbody and the double line in diagram Q represents a structure to distribute flows evenly.

There can often be multiple inlets into the waterbody and the locations of these inlets to the outlet structure can influence the hydraulic efficiency of the system. A design for inlet structures that reduces localised water eddies and promotes good mixing of water within the immediate

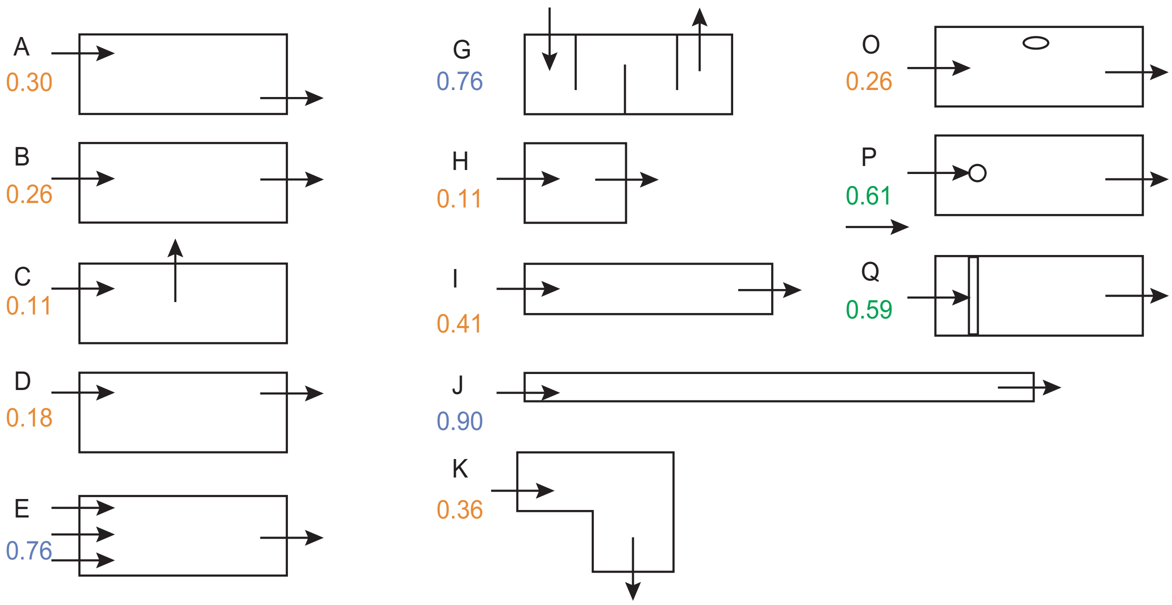


Figure 10.7 Hydraulic efficiency – λ – a measure of flow hydrodynamic conditions in constructed wetlands and ponds; range is from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment (Persson et al. 1999).

vicinity of the inlet may be necessary and the use of an inlet zone is a common approach to inlet design.

10.3.2.2 Inlet zone

It is good design practice to provide pretreatment of stormwater to ponds and lakes for removal of sediment, organic matter and nutrients. The inlet zone can take many forms, ranging from systems that function as a **sedimentation basin** to that of a shallow **ephemeral** wetland. They are a transitional zone into the deeper waters of a pond. Some inlet zones are constructed with a porous embankment at its transition with the deeper water zone to promote a wider distribution of inflow water across the open water body.

The **bathymetry** across the inlet zone is to vary gradually from 0.2 m above the permanent pool level to 0.3 m below the permanent pool level over a distance of between 10 m and 20 m.

There is generally little need for any hydraulic structures to separate an inlet zone of a pond to the open water section, although a designer may consider the use of a porous embankment to promote better flow distribution into the open water zone. A low flow vegetated **swale** should be provided to convey dry weather flow and low flows to the open waterbody.

The notional required inlet zone area can be computed by the use of **sedimentation** theory (see Chapter 4), targeting the 125 μm sediment (settling velocity of 11 mm/s) operating at the one-year ARI peak discharge.

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

$$R = 1 - \left(1 + \frac{1}{n}\frac{v_s}{Q/A}\right)^{-n} \qquad \text{(Equation 10.1)}$$

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

The above expression for sedimentation is applied with n being a turbulence parameter. Figure 10.7 provides guidance on selecting an appropriate n value (according to the configuration of the basin). A value of n is selected using the following relationship:

$$\lambda = 1 - 1/n; \; n = \frac{1}{1-\lambda} \qquad \text{(Equation 10.2)}$$

Figure 10.8 Open water with edge vegetation.

Equation 10.1 is strictly applicable for systems with no permanent pool, and will generally overestimate the required area of a sedimentation basin. This equation is thus often considered to provide an upper limit estimate of the required size for sedimentation basins.

Good practice in the design of inlet zone will include a permanent pool to reduce flow velocities and provide storage of settled sediment. The presence of a permanent pool reduces flow velocities in the sedimentation basin and thus increases detention times. Owing to the outlet structure being located some distance above the bed of a sedimentation basin, it is also not necessary for sediment particles to settle to the bed of the basin to effectively retain the sediments. It is envisaged that sediments need only settle to an effective depth which is less than the depth to the bed of the sediment. This depth is considered to be about 1 m below the permanent pool level. Equation 10.1 can thus be re-derived to account for the effect of the permanent pool storage as follows:

$$R = 1 - \left[1 + \frac{1}{n} \times \frac{v_s}{Q/A} \times \frac{(d_e + d_p)}{(d_e + d^*)}\right]^{-n} \qquad \text{(Equation 10.3)}$$

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

Table 10.1 list the typical settling velocities of sediments. d^* is the depth below the permanent pool level that is sufficient to retain the target sediment (m) – adopt 1.0 or d_p whichever is lower.

10.3.2.3 Cross sections

Batter slopes on approaches and immediately under the water line have to be configured with consideration of public safety. Both hard and soft edge treatments can be applied to complement the landscape of the surrounding area of a pond or lake. Soft edge treatments involve using gentle slopes to the water's edge and extending below the water line for a distance before the batter slopes steepen into deeper areas (Figure 10.9).

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

Figure 10.10 shows an option for a hard edge detail using a vertical wall, and has an associated handrail for public safety. This proposal uses rock to line the bottom of the pond to prevent vegetation (particularly weed) growth.

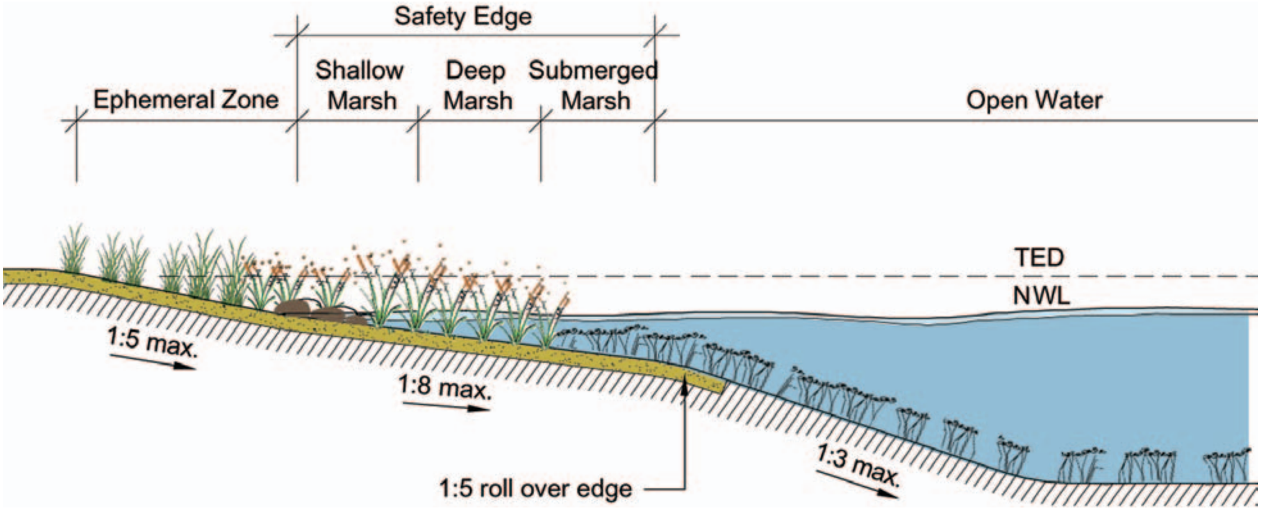


Figure 10.9 Illustration of a soft edge treatment for ponds and lakes (Graeme Bentley Landscape Architects 2004).

Table 10.1 Settling velocities under ideal conditions

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

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

10.3.3 Hydraulic structures

Hydraulic structures are required at the inlet and outlet of a pond or lake. Their function is essentially one of conveyance of flow with provisions for (i) energy dissipation at the inlet structure(s) and (ii) extended detention (if appropriate) at the outlet.

10.3.3.1 Inlet structure

Discharge of stormwater into the open waterbody of a pond or lake may be via an inlet zone or direct input. In both cases it will be necessary to ensure that inflow energy is adequately dissipated so as not to cause localised scour near the pipe outfall. Design of stormwater pipe outfall structures are common hydraulic engineering practice (see e.g. Chow 1959; Henderson 1966).

Litter control is normally required at the inlet structure and it is generally recommended that some form of **gross pollutant trap, GTP** be installed as part of the inlet structure. Several proprietary products are available for capture of gross pollutants (see Engineers Australia 2003, Chapter 7). The storage capacity of GPTs should be sized to ensure that maintenance (clean-out) frequency is not greater than once every three months.

10.3.3.2 Outlet structure

The outlet structure of a pond or lake can be configured in many ways and depends on the specified operation of the system during periods of high inflows. Many ponds form part of a **flood retarding basin** in which case the outlet structure consists of two components: an outlet pit and an outlet culvert. The computation of the required outlet culvert is an essential element

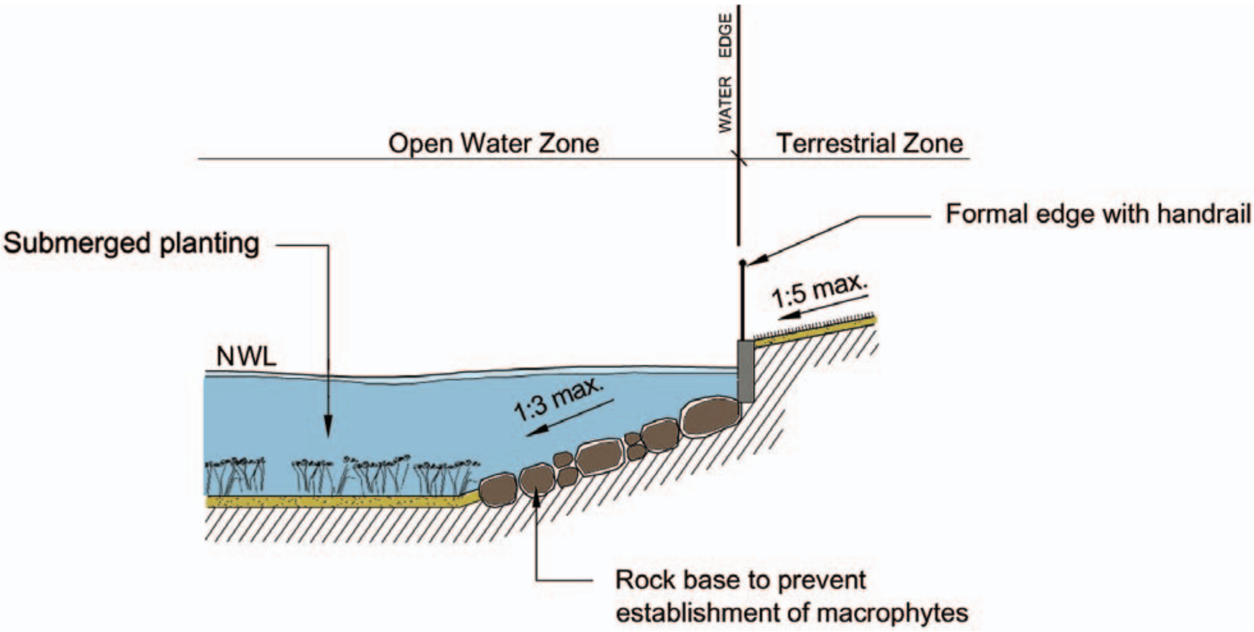


Figure 10.10 Illustration of hard edge treatment for open waterbodies (Graeme Bentley Landscape Architects 2004).

of the retarding basin design and will be based on flood routing computation as outlined in ARR (Institution of Engineers 2001). The main function of the inlet pit is to maintain the desired permanent pool level and to provide a means of connecting the maintenance pipe to the outlet culvert. Design considerations of the outlet pit include:

- ensuring that the crest of the pit is set at the permanent pool level of the lake or pond
- ensuring that the dimension of the pit provides discharge capacity that is greater than the discharge capacity of the outlet culvert or pipe
- protection against clogging by flood debris.

The dimension of an outlet pit is determined by considering two flow conditions: weir and orifice flow (Equations 10.4 and 10.5).

A blockage factor (B) is also used to account for any debris blockage; a value of 50% blockage is recommended. Generally, the discharge pipe from the inlet zone (and downstream water levels) controls the maximum flow rate from the area. It is therefore less critical if the outlet pit is oversized to allow for blockage.

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

$$P = \frac{Q_{des}}{B \times C_w \times H^{1.5}} \qquad \text{(Equation 10.4)}$$

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 – this equation is applied when the inlet pit is completely submerged (corresponding to conditions associated with larger flood events):

$$A_o = \frac{Q_{des}}{B \times C_d \sqrt{2gH}} \qquad \text{(Equation 10.5)}$$

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

Use of whichever equation that results in the larger size of required pipe should be adopted. It is important that an outlet pit is prevented from blockage by debris. Design consideration needs to include means of preventing blockage of the outlet structure.

Outlet culvert of pipe capacity is estimated using the orifice discharge equation (Equation 10.5) without a blockage factor.

10.3.3.3 Maintenance drain

The waterbody should be able to be drained for maintenance with manual operation. A suitable design flow rate is one which can draw down the permanent pool within seven days although, depending on the volume of the waterbody, this may not be realistic.

The orifice discharge equation (Equation 10.5) is considered suitable for sizing the maintenance drain (without a blockage factor) on the assumption that the system will operate under inlet control.

10.3.4 High-flow route design

The provision of a high flow route is standard design practice to ensure that overflow from the dam embankment can be safely conveyed either by the use of a spillway or ensuring that the embankment is designed to withstand overtopping. This issue requires specialised design inputs and is not discussed in this document.

10.3.5 Vegetation specification

Vegetation planted along the **littoral zone** of a pond or lake serves the primary function of inhibiting public access to the open waterbody. Terrestrial planting beyond the littoral zone may also be recommended to screen areas and provide an access barrier to uncontrolled areas of the stormwater treatment system.

Plant species for the inlet zone area will be predominantly those of ephemeral wetlands (see Appendix A).

10.3.6 Design calculation summary

A *Ponds and Lakes Calculation Summary* is included to aid the design process of key design elements of a pond or lake.

10.4 Checking tools

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

Checklists are provided for:

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

10.4.1 Design assessment checklist

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

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

Ponds and Lakes		CALCULATION CHECKLIST	
CALCULATION TASK	**OUTCOME**		**CHECK**
1 Identify design criteria			☐
Design ARI flow for inlet hydraulic structures		year	
Design ARI flow for outlet hydraulic structures			
Design ARI for emergency hydraulic structures		year	
80%tile turnover period		days	
Probabilistic summer water level – 10%tile		m	
Probabilistic summer water level – 90%tile		m	
Flood detention storage volume (from flood routing analysis)		m^3	
Outlet pipe dimension (from flood routing analysis)		mm	
2 Catchment characteristics			☐
Residential		ha	
Commercial		ha	
Fraction impervious			☐
Residential			
Commercial			
3 Estimate design flow rates			
Time of concentration			
Estimate from flow path length and velocities		minutes	☐
Identify rainfall intensities			
Station used for IFD data:			
Design rainfall intensity for inlet structure(s)		mm/hr	☐
Design runoff coefficient			
Inlet structure(s)			☐
Peak design flows			☐
Inlet structure(s)		m^3/s	
Outlet structure(s)		m^3/s	
4 Forebay zone layout			☐
Area of forebay zone			
Aspect ratio			
Hydraulic efficiency			
5 Lake residence time			☐
6 Pond layout			☐
Area of open water		m^2	
Aspect ratio		L:W	
Hydraulic efficiency			
Length		m	
Width		m	
Cross section batter slope		V:H	
		m	
7 Hydrualic structures		m	
Inlet structure			☐
Provision of energy dissipation			
Outlet structure			☐
Pit dimension		L x B	
		mm diam	
Discharge capacity of outlet pit		m^2/s	
Provision of debris trap			
Maintenance drain			☐
Diameter of maintenance valve		mm	
Drainage time		days	
Discharge pipe			☐
Discharge capacity of discharge pipe		m^3/s	
8 Emergency spillway			☐
Discharge capacity of emergency spillway		m^3/s	

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

Land and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the *Asset Handover Checklist* (see Section 10.4.4).

Pond and Lake Design Assessment Checklist				
Lake location:				
Hydraulics	Minor flood: (m³/s):	Major flood: (m³/s):		
Inlet zone			**Y**	**N**
Inlet pipe/structure sufficient for maximum design flow (Q_5 or Q_{100})?				
Scour protection provided at inlet structures?				
Configuration of forebay zone (aspect, depth and flows) allows even distribution of inflow into open water zone?				
Maintenance access provided?				
Public access to forebay zone managed through designated pathways?				
Gross pollutant protection measures provided on inlet structures?				
Open water zone			**Y**	**N**
Depth of open water > 1.5 m?				
Aspect ratio provides hydraulic efficiency >0.5?				
Depth of permanent water >1.5 m?				
20% probability of exceedance in accordance with guidelines (i.e. 20, 30 or 50 days)				
Edge treatment – batter slopes from accessible edges shallow enough to allow egress?				
Edge treatment – provision of littoral zone planting with 1:8 batter slopes to 0.2 m below the waterline ?				
Edge treatment – vertical fall to shallow bench?				
Maintenance access provided?				
Public access to open zones restricted to designated pathways with appropriate safety considerations?				
Embankment height > flood detention depth?				
Lake turnover management plan developed (if turnover is inadequate)?				
Probabilistic summer water level fluctuation within desired range and edge treatment developed to suit?				
Outlet structures			**Y**	**N**
Outlet pit set at permanent water level?				
Discharge capacity of outlet pit > computed discharge capacity of outlet pipe? (checked against weir flow and orifice flow operating conditions)				
Maintenance drain provided?				
Protection against clogging of outlet pit provided?				

10.4.2 Construction advice

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

Protection from existing flows

It is important to protect lakes and ponds from upstream flows during construction. A mechanism to divert flows around a construction site, protect from litter and debris is required.

This can be achieved by constructing a high flow bypass channel initially and then diverting all inflows along the channel until the pond system is complete.

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

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

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

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

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

Dewatering collected sediments
An area should be constructed that allows for dewatering of removed sediments from an inlet zone. This area should be located such that water from the material drains back into the inlet zone. Material should be allowed to drain for a minimum of overnight before disposal.

Timing for planting
Timing of planting vegetation depends on a suitable time of year (and potential irrigation requirements) as well as timing in relation to the phases of development. Temporary sediment controls should always be used prior to planting as lead times from earthworks to planting are often long.

Vegetation establishment
During the establishment phase water levels should be controlled carefully to prevent seedlings from being desiccated or drowned. This is best achieved with the use of maintenance drains. Once plants are established, water levels can be raised to operational levels (see Appendix A).

Bird protection
Protection against birds (e.g. using nets) should be considered for newly planted area of wetlands as birds can pull out young plants and reduce plant densities.

Trees on embankments
The size of trees planted on embankments needs to be considered as root systems of larger trees can threaten the structural integrity of embankments.

10.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Ponds and Lakes

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

SITE: ______________________

CONSTRUCTED BY: ______________________

DURING CONSTRUCTION

Items inspected	Checked Y	Checked N	Satisfactory	Unsatisfactory	**Structural components**	Checked Y	Checked N	Satisfactory	Unsatisfactory
Preliminary works	Y	N			15. Location and levels of outlet as designed	Y	N		
1. Erosion and sediment control plan adopted					16. Safety protection provided				
					17. Pipe joints and connections as designed				
2. Limit public access					18. Concrete and reinforcement as designed				
3. Location same as plans					19. Inlets appropriately installed				
4. Site protection from existing flows					20. Inlet energy dissipation installed				
5. All required permits in place					21. No seepage through banks				
Earthworks					22. Ensure spillway is level				
6. Integrity of banks					23. Provision of maintenance drain(s)				
7. Batter slopes as plans					24. Collar installed on pipes				
8. Impermeable (eg. clay) base installed					26. Protection of riser from debris				
9. Maintenance access for inlet zone					**Vegetation**				
10. Compaction process as designed					29. Vegetation appropriate to zone (depth)				
11. Placement of adequate topsoil (edges)					30. Weed removal prior to planting				
12. Levels as designed for base, benches, banks and spillway (including freeboard)					31.Provision for water level control during establishment				
13. Check for groundwater intrusion					32. Vegetation layout and densities as designed				
14. Stabilisation with sterile grass									

FINAL INSPECTION

1. Confirm levels of inlets and outlets					9. Check for uneven settling of banks				
2. Confirm structural element sizes					10. Evidence of stagnant water or short circuiting				
3. Check batter slopes									
4. Vegetation planting as designed					11. Evidence of litter or excessive debris				
5. Erosion protection measures working					12. Provision of removed sediment drainage area				
6. Pre-treatment installed and operational									
7. Maintenance access provided					13. Outlet free of debris				
8. Public safety adequate									

COMMENTS ON INSPECTION

ACTIONS REQUIRED

1.									
2.									
3.									
4.									
5.									

10.4.4 Asset handover checklist

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

10.5 Maintenance requirements

Pond and lakes treat runoff by providing extended detention and allowing sedimentation to occur. In addition, they are used for flow management and need to be maintained to ensure adequate flood protection for local properties.

Most lake maintenance is associated with the inlet zone (and GPT if installed). Weeding, planting and debris removal are the dominant tasks. In addition, if artificial turnover of the lake is required (because of long residence times) a mechanical system will need to be employed and will require specific maintenance.

Edge vegetation will also require maintenance, including weed removal and replanting. Other components of the system that 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.

Maintenance is primarily concerned with:

- flow to and through the system
- maintaining vegetation
- removal of accumulated sediments
- litter and debris removal.

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

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

10.5.1 Operation and maintenance inspection form

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

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

10.6 Worked example

10.6.1 Worked example introduction

As part of a residential development in Portland, a permanent waterbody is proposed to treat runoff from a residential area of 110 ha (45% catchment imperviousness) and provide landscape amenity as an integral component of the public open space. The residential development is to have several stormwater quality improvement measures within the streetscape. Modelling using MUSIC has indicated that a pond area of 3000 m^2 of 2 m mean depth is required to provide the final component of the treatment train strategy for the development. The pond is expected to reduce the nitrogen load from the catchment by 10%.

This pond is to be nested within the site of a flood retarding basin. The site for the retarding basin is 4.2 ha and is quadrangle in shape (Figure 10.11). A combination of active and passive open space (e.g. urban forestry andpond) functions are to be incorporated into the site.

Stormwater is conveyed by stormwater pipes (up to the 10-year ARI event) and by designated floodways (including roadways) for events larger than the 10-year ARI event. There are four subcatchments discharging into the retarding basin. During the design 100-year ARI event, the maximum discharge from the retarding basin is 4.1 m^3/s.

10.6.2 Design considerations

Key design issues to be considered include:

1. verifying the size of the pond (depth and area)
2. computation to ensure that the pond volume is not excessively large in comparison to the hydrology of the catchment
3. configuring the layout of the pond such that the system's **hydraulic efficiency** can be optimised, including the transition structure between the inlet zone and the open waterbody
4. design of hydraulic structures, including
   - inlet structure to provide for energy dissipation of inflows up to the 100-year ARI peak discharge
   - design of the outlet structure for the pond and retarding basin.
5. designing the landscape, including
   - edge treatment
   - recommending plant species and planting density
6. providing maintenance.

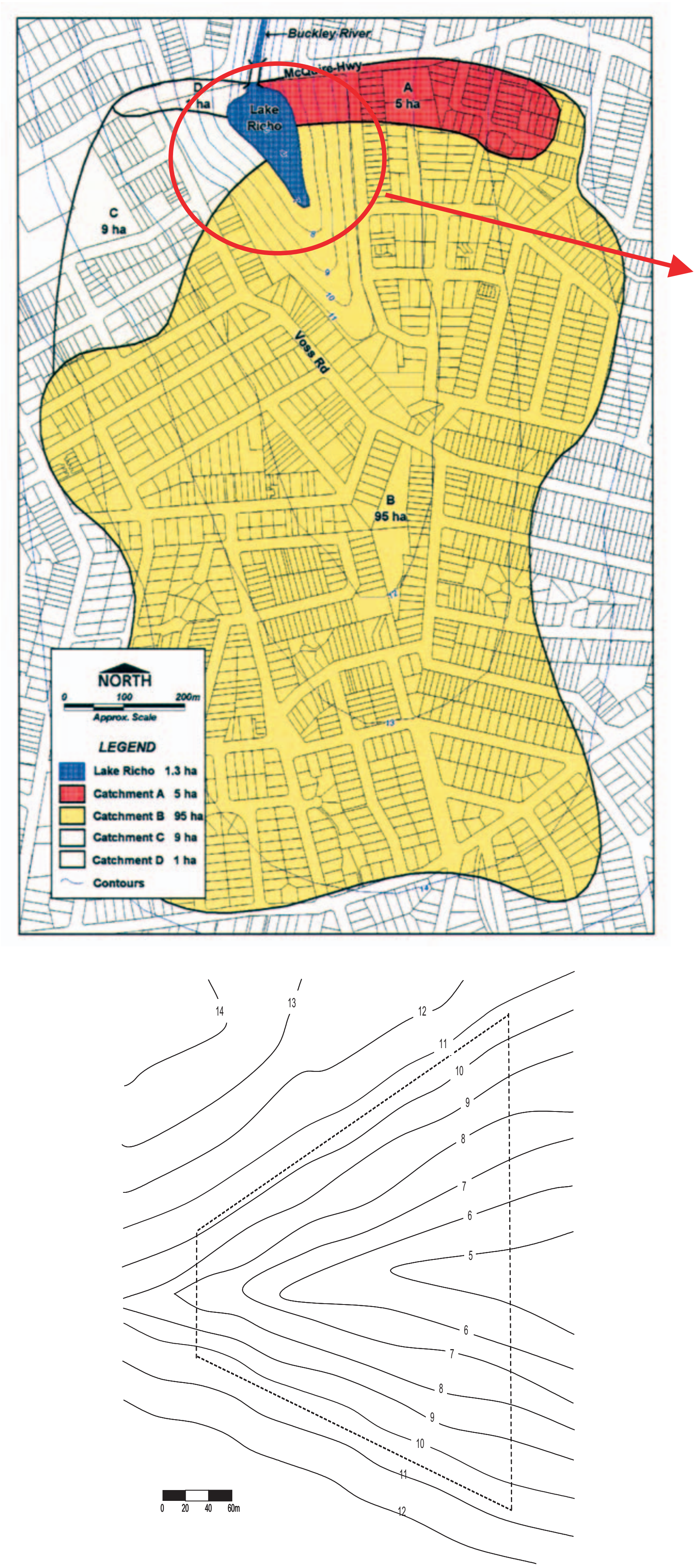


Figure 10.11 Proposed site for retarding basin and pond.

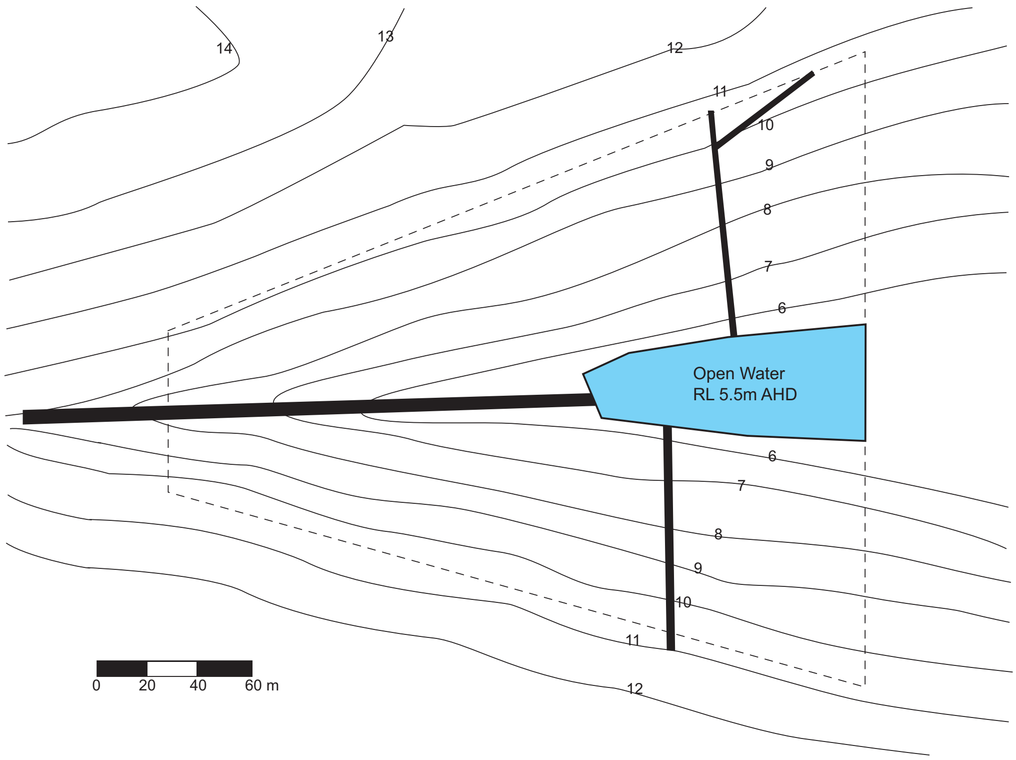


Figure 10.12 Layout of proposed pond.

10.6.3 Confirming pond area

As a basic check of the adequacy of the size of the lake, reference is made to the performance curves presented in Section 10.2. According to Figure 10.1, the required lake area necessary to reduce TN load by 10% is about 0.3% of the impervious area of the catchment.

According to the **hydrologic region** analysis in Chapter 2 (see Section 2.4), the adjustment factor for ponds and lakes in Portland is 1.38.

The required lake area computed from the simple procedure presented in Chapter 2 is as follows:

$$\text{Impervious area} \sim 110 \text{ ha} \times 0.45 \sim 50 \text{ ha}$$

$$\text{Required lake area (mean depth of 2 m)} = 500\,000 \times 0.003 \times 1.38 = 2070 \text{ m}^2.$$

The proposed lake area is 3000 m^2, which is larger than the value determined from the simple procedure contained in Chapter 2, Section 2.4, and is thus acceptable.

The proposed permanent pool level is 5.5 m AHD (Australian Height Datum) with a maximum depth of 2.5 m and a depth range between 1.5 m and 2.5 m. The volume of the proposed lake waterbody is about 6 ML (i.e. 0.3 ha × 2 m depth). The layout of the proposed waterbody is shown in Figure 10.12.

Proposed pond area is 3000 m^2 is confirmed as larger than the expected size required to achieve the 10% reduction in TN proposed
Permanent pool level is set at 5.5 m AHD
Lake volume ~ 6 ML

10.6.4 Design calculations

10.6.4.1 Lake hydrology

Analysis of waterbody residence time

An analysis of waterbody residence time should be undertaken using a continuous simulation approach with the use of historical rainfall data with historical potential evaporation data or estimates of probabilistic monthly potential evaporation (see Section 10.3.1). A 'simplified

Table 10.2 Meteorological data for Portland

	Jan	Feb	Mar	Apr	May	Jun	
Mean Rainfall (mm)	35.2	33.5	43	65.3	88.9	100.1	
Median Rainfall (mm)	25.9	25.8	36.9	61.9	82.1	95.5	
Decile 9 Rainfall (mm)	74.7	72.6	83.6	110.2	145.6	153.7	
Decile 1 Rainfall (mm)	9.4	5.9	12.1	25.6	36	56.4	
Mean no. of Raindays	8.7	8	11.4	14.7	18.4	19.6	
Monthly Area PET (mm)	150	120	100	85	40	30	
	Jul	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Annual**
Mean Rainfall (mm)	108.5	107.6	85.1	70.4	53.2	44.7	835.5
Median Rainfall (mm)	102.8	102.6	81.4	67.5	48.9	38.6	834.9
Decile 9 Rainfall (mm)	167.2	165.1	126.6	111.9	93.4	81.8	1001.4
Decile 1 Rainfall (mm)	57	55.5	53.4	29.2	22	13.8	656.6
Mean no. of Raindays	21.1	21.2	18.5	16.2	13	11.3	182.1
Monthly Area PET (mm)	30	45	70	100	125	135	1000

approach' may be undertaken as a preliminary assessment of the adequacy of waterbody turnover in the first instance. This is outlined below.

The statistics of the monthly rainfall and areal potential evapotranspiration data for Portland are summarised (Table 10.2).

From the above meteorological data, a simple assessment of the waterbody residence times for the 10th percentile, 50th percentile and 90th percentile summer meteorological conditions can be done. The ratio of net summer inflow to the lake volume can be computed, and the number of days subsequently divided over the summer period (92 days) with this ratio (Table 10.3).

The 20th percentile residence time can be estimated by interpolating between the 10th percentile value and the 50th percentile value. The interpolation is best undertaken using log-normal probability paper (Figure 10.13).

The analysis undertaken indicated that the proposed pond has a 20th percentile probabilistic residence time of about 28 days. This is just within the guidelines for sustainable ecosystem health of a waterbody of 30 days and it is advisable that a continuous simulation of pond residence time be undertaken to confirm that the pond has a low risk of eutrophication.

Continuous rainfall data was not available for Portland for this case study and the closest available **pluviographic** station is Mortlake. Mortlake, although in the same hydrologic region, is located further inland compared with Portland and its mean annual rainfall is 776 mm compared to 836 mm for Portland. Thus, continuous simulation using Mortlake rainfall data will tend to overestimate the probabilistic residence time of ponds in Portland.

Rainfall data for 1976 to 1990 was used in MUSIC to simulate the hydrology of the proposed pond. For water balance simulation, a daily time step was used. The results of the simulation are plot as a residence time frequency plot (Figure 10.14). The 20% probability of exceedence residence time was estimated to be 26 days and is consistent with the findings of the simplified method.

There is no significant risk of summer blue-green algal bloom with the proposed pond volume.

Table 10.3 Probabilistic residence time

	Summer rainfall (mm)	Net summer inflow (ML)[A]	Net summer inflow/ Lake volume	Summer probabilistic residence time (approx. no. of days)
10%tile	29.1	13.3	2.2	41
50%tile	90.3	43.6	7.3	13
90%tile	229	112	18.7	5

[A]Catchment inflow (~rainfall × impervious area) – net evaporation (~[evaporation – rainfall] × lake area)

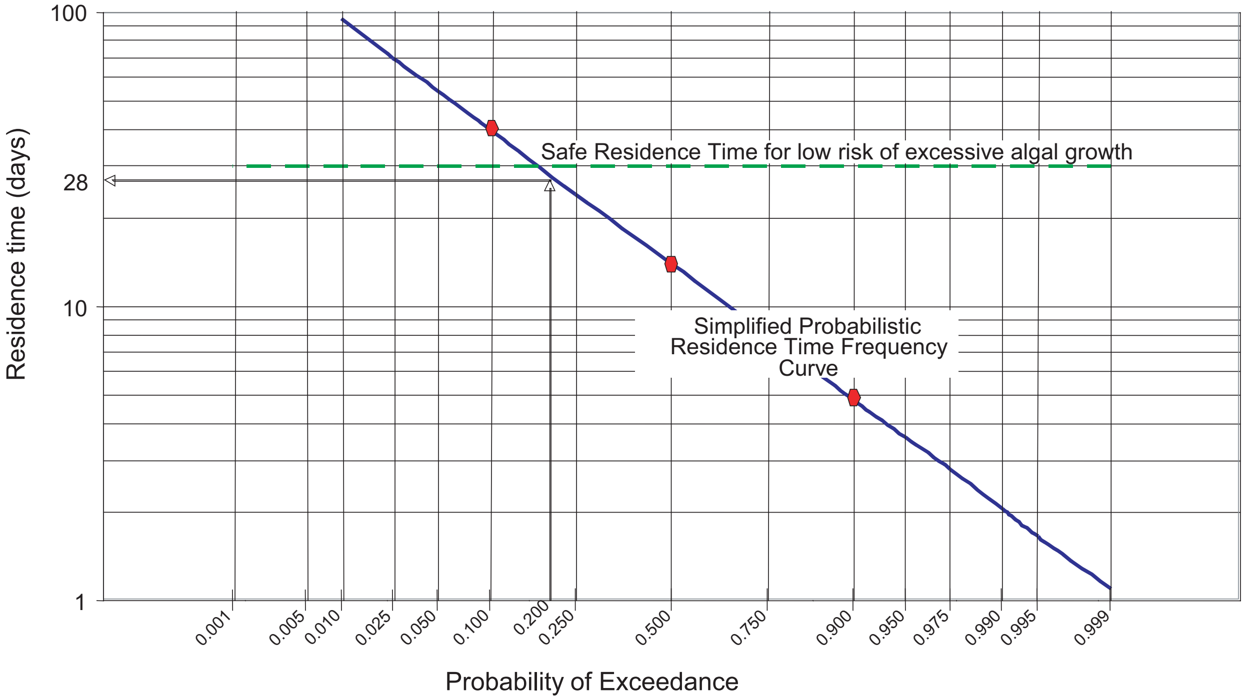


Figure 10.13 Simplified log-normal probability plot of probabilistic residence time of pond water in summer.

Probabilistic summer water levels

Water level fluctuation over the summer is influenced by catchment inflow and evaporation from the lake waterbody. As is the case for the waterbody turnover analysis, a rigorous approach to determination of the probabilistic summer water level fluctuation is through a continuous simulation approach using a daily time step.

A 'simplified approach' to determine if water level fluctuation is excessive within the waterbody can be undertaken by examining the 10th percentile monthly water balance (Figure 10.14). The adoption of the average monthly evaporative losses are not expected to significantly underestimate the evaporative loss corresponding to a 10th percenttile hydrologic scenario.

The analysis shows that monthly catchment inflow exceeds evaporative losses in all months indicating that even for the 10th percentile rainfall scenario, the lake level can be expected to be full at least once each month. The maximum fluctuation in water level (corresponding to the

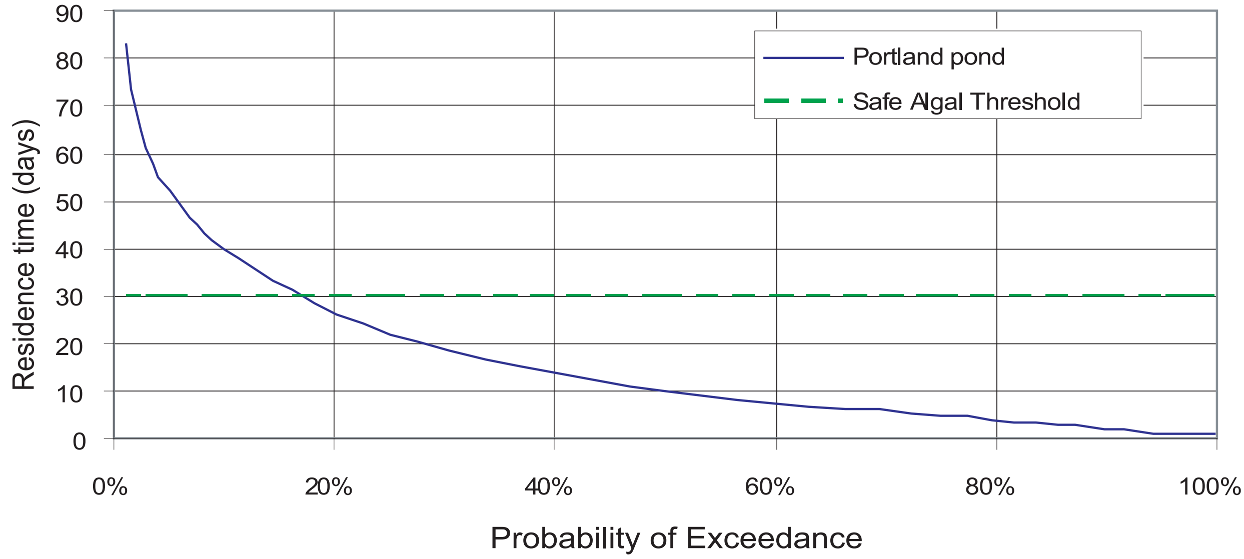


Figure 10.14 Plot of probabilistic residence time determined from continuous simulation using 25 years of rainfall record.

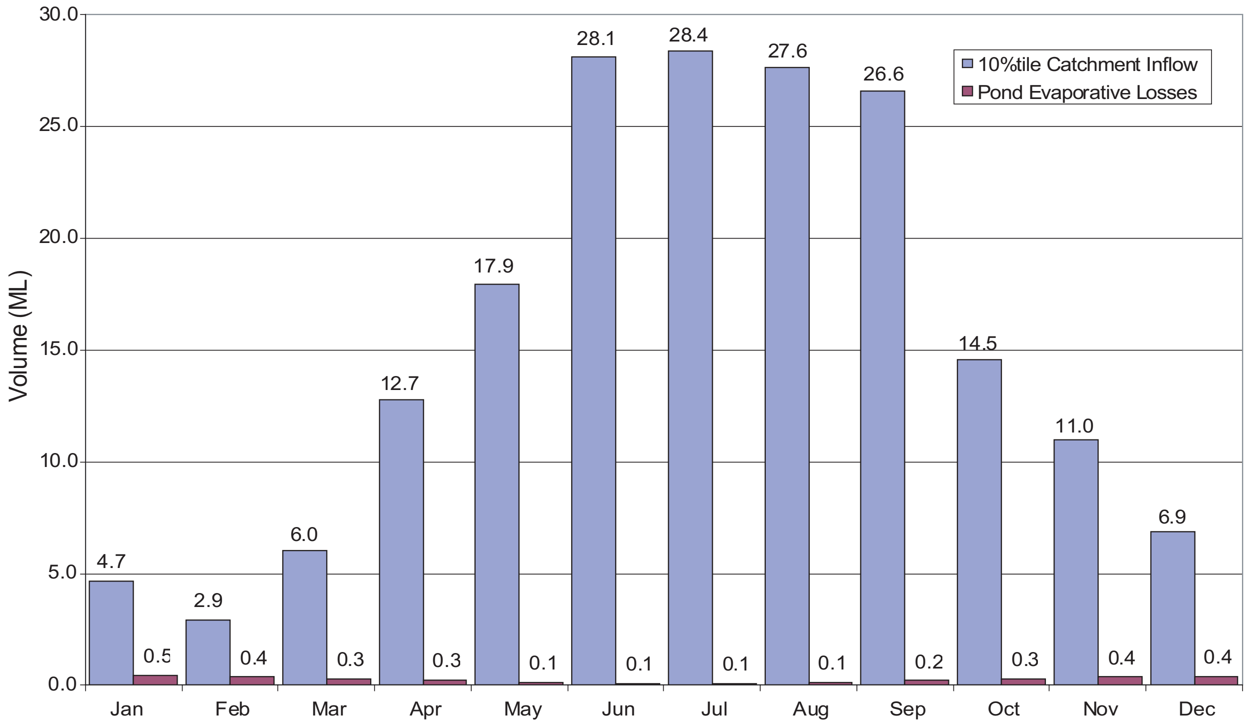


Figure 10.15 Lake water budget showing 10th percentile catchment inflow and pond evaporative losses.).

January/February period) can be conservatively computed to be the sum of the expected evaporation losses of these two months (i.e. about 250 mm).

Lake water level fluctuation is not expected to be a significant aesthetic issue for the proposed lake.

Estimating design flows

Times of concentration (t_c) have been assessed by assuming pipe and overland flow velocities of 1 m/s and estimating flow paths. In smaller catchments, a minimum time of concentration of six minutes has been adopted to allow for lot scale impacts. The characteristics of each catchment are summarised in Table 10.4.

Rainfall intensities were estimated using IFD intensities for Portland and are also summarised in Table 10.4.

10.6.4.2 Open water zone layout

Size and dimensions

The open water zone will be quadrangular in shape to conform to the natural terrain of the site. The general dimension is a mean width of 30 m and 100 m along the long axis, giving an aspect ration of 3(L) to 1(W). With the largest of the catchments discharging into the lake from one end of the longer axis, the expected hydraulic efficiency of the open water body can be of the

Table 10.4 Catchment characteristics (*C*), rainfall intensities (*I*) and design discharges (*Q*)

Subcatchment	Area (ha)	Flow path length (m)	t_c (min)	I_1	C_1	Q_1	I_{10}	C_{10}	Q_{10}	I_{100}	C_{100}	Q_{100}
A	5	220	7	34	0.59	0.28	63	0.74	0.65	144	0.88	1.78
B	95	1400	30	17	0.39	1.73	30	0.49	3.84	64	0.59	9.93
C	9	200	7	34	0.59	0.50	63	0.74	1.17	144	0.88	3.20
D	1	150	7	34	0.59	0.06	63	0.74	0.13	144	0.88	0.36

Note: Runoff coefficients for the one-year, 10-year and 100-year ARI events (each with a 0.45 fraction impervious) were calculated in accordance to the procedure in AR&R 1998 (Book VIII)

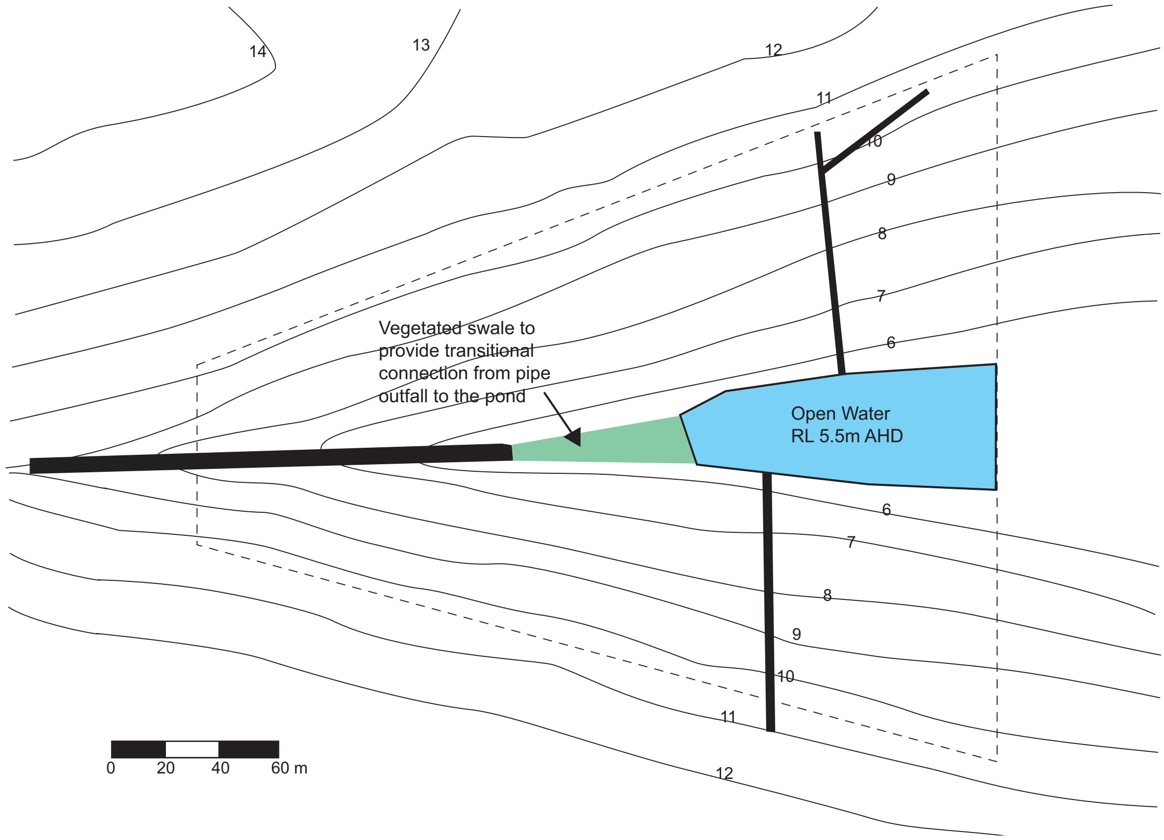


Figure 10.16 Vegetated swale recommended to provide flow transition from pipe outfall to foreshore of pond.

order of 0.34 (Figure 10.7) unless the outlet from subcatchment B can be designed such that outflow is uniformly distributed across the 20 m wide foreshore of the pond. This can be achieved by designing a vegetated swale transition between the pipe outfall and the forebay of the pond (Figure 10.16).

Aspect ratio is **3(L) to 1(W)**;
Hydraulic efficiency **~0.76** with distributed inflow.

Section

The long and cross sections of the pond will follow the natural terrain with limited requirement for earthworks to form the bathymetry of the pond.

The batter slopes on approaches and immediately under the permanent water level have to be configured with consideration of public safety. A batter slope of 1(V):8(H) from the littoral zone to 0.3 m beneath the water line before steepening into a 1(V):3(H) slope is recommended as a possible design solution (e.g. Figure 10.17).

Cross section of littoral zone to below the water line consists of a 1:8 batter slope to 0.3 m below the permanent pool level.

10.6.4.3 Pond outlet structure

Maintenance drain

A maintenance drain will be provided to allow drainage of the system. Valves will be operated manually to drain the permanent waterbody. The drawdown period should be about 24 hours if practical.

The mean flow rate (Q) for the maintenance drain is selected to drawdown the permanent pool over 24 hours and is computed as follows:

$$\text{Permanent pool volume} \sim 6000\ \text{m}^3$$

$$Q = 6000/(1 \times 24 \times 3600) = 0.07\ \text{m}^3/\text{s} = 70\ \text{L/s}. \qquad \text{(Equation 10.6)}$$

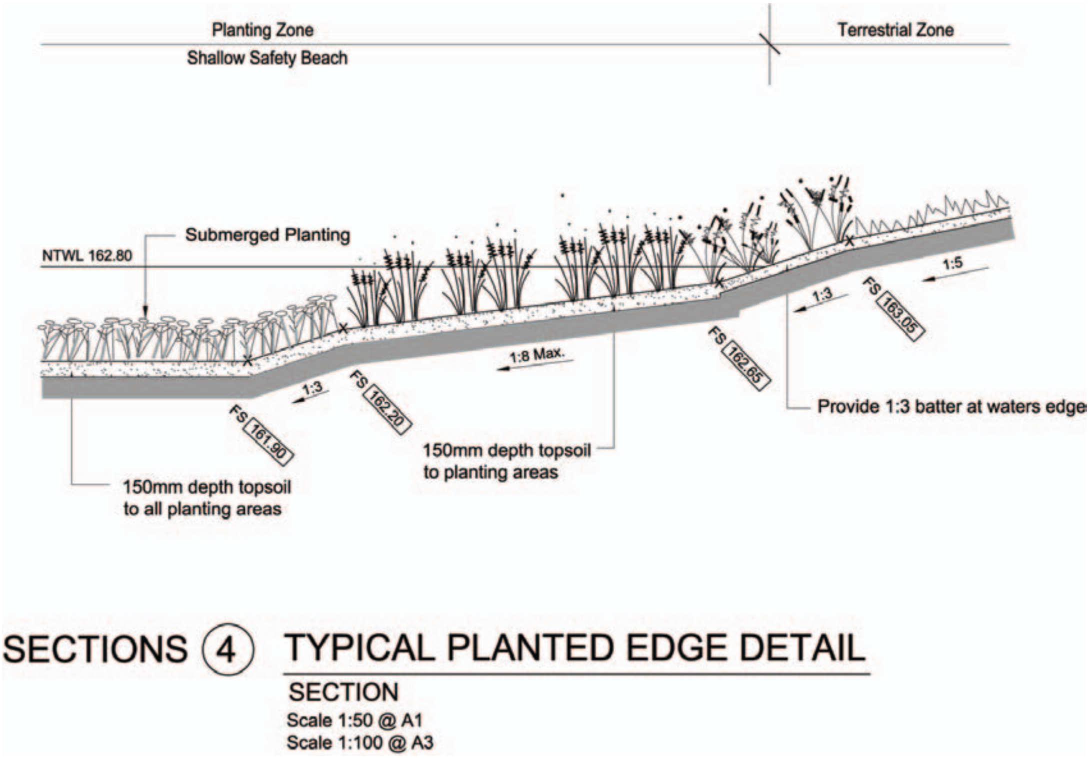


Figure 10.17 Planted edge detail

To determine the area of the orifice for the drain, it assumed that the valve orifice will operate under inlet control with its discharge characteristics determined by the orifice equation (Equation 10.5):

$$A_o = \frac{Q_{des}}{C_d\sqrt{2gH}} \qquad \text{(Equation 10.7)}$$

$Q = 0.07$ m^3/s
$C_d = 0.6$
$H = 0.67$ m (two-thirds of maximum permanent pool depth)

Giving $A_o = 0.02$ m^2 corresponding to an orifice diameter of 161 mm – adopt 200 mm maintenance pipe.

Pipe valve to allow draining of the permanent pool for maintenance to be **at least 200 mm diameter.**

Outlet pit

The outlet pit is to be set at a crest level at the nominated permanent pool level of 5.5 m AHD. The discharge capacity of the outlet pit must be at least equal but preferably higher than the design retarding basin outflow.

During the 100-year ARI operation of the retarding basin, the outlet pit will be completely submerged and the required dimension of the outlet pit to discharge 4.1 m^3/s can be computed using the orifice flow equation (Equation 10.5):

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

C_d = Orifice Discharge Coefficient (0.6)
$H = 3.5$ m
A_o = Orifice area (m^2)
$Q_{des} = 4.1$ m^3/s.

The computed plan area of the overflow pit is 0.825 m^2. The nominal pit dimension to ensure adequate discharge capacity is 1.0 m × 1.0 m although maintenance access may require the pit to be larger.

Outlet pit dimension is 1.0 m × 1.0 m

10.6.4.4 High-flow route and spillway design

The spillway weir level is set at reduced level (RL) 11.0 m AHD and the retarding basin embankment height is about 7 m. The spillway needs to be designed with adequate capacity to safely convey peak discharges up to the probable maximum flood level. This requires specialist hydrological engineering input involving flood estimation and flood routing calculations.

The spillway needs to be designed to safely convey discharges up to the probable maximum flood level.

10.6.4.5 Vegetation specifications

The vegetation specification and recommended planting density for the littoral and open water zone are summarised in Table 10.5 (see Appendix A for further discussion and guidance).

Table 10.5 Vegetation specifications

Zone	Plant species	Planting density (plants/m^2)
Littoral berm	*Persicaria decipens*	3
Open water zone	*Vallisneria spiralis*	4

10.6.4.6 Design calculation summary

The completed *Ponds and Lakes Calculation Summary* shows the results of the design calculations.

Ponds and Lakes — CALCULATION SUMMARY

CALCULATION TASK		OUTCOME		CHECK
1 Identify design criteria				✓
	Design ARI Flow for inlet hydraulic structures	10	year	
	Design ARI Flow for outlet hydraulic structures	100		
	Design ARI for emergency hydraulic structures	PMF	year	
	80%tile turnover period	>>110	days	
	Probabilistic summer water level – 10%tile	7.2	m	
	Probabilistic summer water level – 90%tile	7.5	m	
	Flood detention storage volume (from flood routing analysis)	150000	m^3	
	Outlet pipe dimension (from flood routing analysis)	750	mm	
2 Catchment characteristics				✓
	Residential	110	Ha	
	Commercial	0	Ha	
Fraction impervious				✓
	Residential	0.45		
	Commercial	N/A		
3 Estimate design flow rates				
Time of concentration				
	Estimate from flow path length and velocities	7 to 30	minutes	✓
Identify rainfall intensities				
	Station used for IFD data:	Portland		
	Design rainfall intensity for inlet structure(s)	30 to 63	mm/hr	✓
Design runoff coefficient				
	Inlet structure(s)	0.49 to 0.74		✓
Peak design flows				✓
	Inlet structure(s)	0.13 to 3.84	m^3/s	
	Outlet structure(s)	4.100	m^3/s	
4 Forebay zone layout				✓
	Area of forebay zone	15 to 125	m^2	
	Aspect ratio	2(L):1(W)	L:W	
	Hydraulic efficiency	0.4		
5 Lake residence time				✓
	Is wetland forebay for recirculation required	Y		
	Area of wetland forebay for water recirculation	10000	m^2	
	Detention time during reciculation of wetland forebay	5	days	
	Lake water recirculation pump rate	17	L/s	
6 Pond layout				✓
	Area of open water	22000	m^2	
	Aspect ratio	2(L):1(W)	L:W	
	Hydraulic efficiency	0.76		
	Length	200	m	
	Width	50 to 150	m	
	Cross section batter slope	1(V):8(H)	V:H	
7 Hydrualic structures				
Inlet structure				✓
	Provision of energy dissipation	Y		
Outlet structure				✓
	Pit dimension	1 x 1	L x B	
			mm diam	
	Discharge capacity of outlet pit	4.1	m^2/s	
	Provision of debris trap	Y		
Maintenance drain				✓
	Diameter of maintenance valve	200	mm	
	Drainage time	7	days	

10.6.5 Example maintenance schedule

The Portland Lake Maintenance Form is an inspection sheet developed for a lake at Portland showing local adaptation to incorporate specific features and configuration of individual lakes. It was developed from the generic *Pond Maintenance Checklist*.

PORTLAND LAKE – MAINTENANCE FORM	
Location	
Description	Constructed lake and sediment forebay

SITE VISIT DETAILS

Site visit date: ______

Site visit by: ______

Weather ______

Purpose of the site visit	Tick Box	Complete Sections
Routine inspection	☐	Section 1 only
Routine maintenance	☐	Section 1 and 2
Cleanout of sediment	☐	Section 1, 2 and 3
Annual inspection	☐	Section 1, 2, 3 and 4

SECTION 1 – INSPECTION

Gross pollutant load cleanout required?	Yes/No
Depth of sediment in forebay:	______ m
Cleanout required if depth of sediment >1.0 m	Yes/No
Any weeds or litter in wetland (If Yes, complete Section 2 – Maintenance)	Yes/No
Any visible damage to wetland or sediment basin? (If Yes, completed Section 4 – Condition)	Yes/No
Inspection comments:	

SECTION 2 – MAINTENANCE

Are there weeds in the wetland forebay and littoral zone?	Yes/No
Were the weeds removed this site visit?	Yes/No
Is there litter in the lake or forebay?	Yes/No
Was the litter collected this site visit?	Yes/No

SECTION 3a – CLEANOUT OF GROSS POLLUTANTS

Have the following been notified of cleanout date?	Yes	No
Coordinator – open space and/or drainage	☐	☐
Local residents	☐	☐
Other (specify ....)	☐	☐
Method of cleaning (excavator or eductor)		
Volume of gross pollutant and sediment removed (approximate estimate)	m^3	
Any visible damage to gross pollutant trap? (If yes, complete Section 4 – Condition)	Yes/No	

SECTION 3b – CLEANOUT OF SEDIMENT

Have the following been notified of cleanout date?	Yes	No
Coordinator – open space and/or drainage	☐	☐
Local residents	☐	☐
Other (specify ....)	☐	☐
Method of cleaning (excavator or eductor)		
Volume of sediment removed (approximate estimate)	m^3	
Any visible damage to wetland or sediment forebay? (If yes, complete Section 4 – Condition)	Yes/No	

SECTION 4 – CONDITION

Component	Checked?		Condition OK?		Remarks
	Yes	No	Yes	No	
Inlet structures					
Outlet structures					
Sediment forebay					
Spillway and spillway channel					
Forebay and littoral zone vegetation					
Banks and batter slopes					
Forebay bunds or porous embankment (if constructed)					
Retarding basin embankment					
Surrounding landscaping					

Comments:

10.7 References

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

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

Fair G.M. and Geyer J.C. (1954). *Water Supply and Waste Disposal*, Vol. 2, John Wiley, New York.

Graeme Bentley Landscape Architects (GbLA) (2004). *Preliminary drawings for Mernda Wetland*, Report for Stockland.

Government of Victoria (1995). *Blue-Green Algae*, *Information Brochure*, Victorian Project Team on Blue-Green Algae, Department of Conservation and Natural Resources, Melbourne.

Institution of Engineers, Australia (2001). *Australian Rainfall and Runoff – A Guide to Flood Estimation*, Revised edn, Pilgram, D.H. (Ed.), Institution of Engineers, Australia, Barton, ACT.

Mitrovic, S.M., Bowling, L.C. and Buckney, R.T. (2001). Vertical disentrainment of *Anabaena circinalis* in the turbid, freshwater Darling River, Australia: quantifying potential benefits from buoyancy, *Journal of Plankton Research*, 23(1), 47–55.

Persson, J., Somes, N.L.G. and Wong T.H.F. (1999). Hydraulic efficiency and constructed wetland and ponds, *Water Science and Technology*, 40(3), 291–289.

Reynolds, C.S. (2003). The development of perceptions of aquatic eutrophication and its control, *Ecohydrology and Hydrobiology*, 3(2), 149–163.

Sherman, B.S., Webster, I.T., Jones, G.J., Oliver, R.L. (1998). Transitions between Aulacoseira and Anabaena dominance in a turbid river weir pool, *Limnology and Oceanography*, 43(8), 1902–1915.

Tarczynska, M., Frankiewicz, P. and Zalewski, M. (2002). The regulation and control of hydrologic and biotic processes within reservoirs for water quality improvement, In *Guidelines for the Integrated Management of the Watershed – Phytotechnology and Ecohydrology*, Freshwater Management Series No. 5, UNEP-IETC and UNESCO-IHP.

Wagner-Lotkowska, I., Izydorczyk, K., Jurczak, T., Tarczynska, M., Frankiewicz, P. and Jorgensen, S.E. (2004). Reservoir and lake management: improvement of water quality, In Zalewski, M. (Ed.), *Integrated Watershed Management – Ecohydrology and Phytotechnology – Manual*, United Nations Publication, United Nations Environment Programme,

Westwood, K.J. and Ganf, G.G. (2004). Effect of cell flotation on growth of *Anabaena circinalis* under diurnally stratified conditions, *Journal of Plankton Research*, JPR Advance Access, June 17, 2004.

Chapter 11 **Infiltration measures**

Infiltration system in Adelaide, showing the overflow trench

11.1 Introduction

Stormwater infiltration systems encourage stormwater to infiltrate into surrounding soils (eg Figure 11.1). 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. They result in a reduction in the volume and magnitude of peak **discharges** from impervious areas. *Australian Runoff Quality Guidelines* (Engineers Australia 2003) provides a detailed discussion of procedures for sizing stormwater infiltration systems. This chapter outlines the engineering design of such systems following the selection of a required detention storage volume associated with infiltration.

Not all areas are suited to infiltration systems. Careful consideration of the type of runoff area from which the runoff originates is important to ensure the continued effective operation of these schemes. Australian experience highlights the importance of good design of these systems and the position of these systems in a stormwater **treatment train**. Poor consideration of **catchment** pollutant types and characteristics and site conditions is often the main cause for deteriorating infiltration effectiveness over time because of clogging and lack of appropriate maintenance.

Pretreatment to remove sediments is a vital component in the treatment train and infiltration systems should be positioned as the final element, with its primary function being the discharge of treated stormwater into the surrounding soils and groundwater system.

Soils with low hydraulic conductivities do not necessarily preclude the use of infiltration systems even though the required infiltration/storage area may become unfeasible. However, these soils are likely to render them more susceptible to clogging and require enhanced pretreatment. In addition, standing water for a long period of time may promote algal growth that increases the risk of clogging of the infiltration media. Thus, it is recommended that soil saturated hydraulic conductivities exceeding 1×10^{-5} m/s (36 mm/hr) are most suited for infiltration systems.

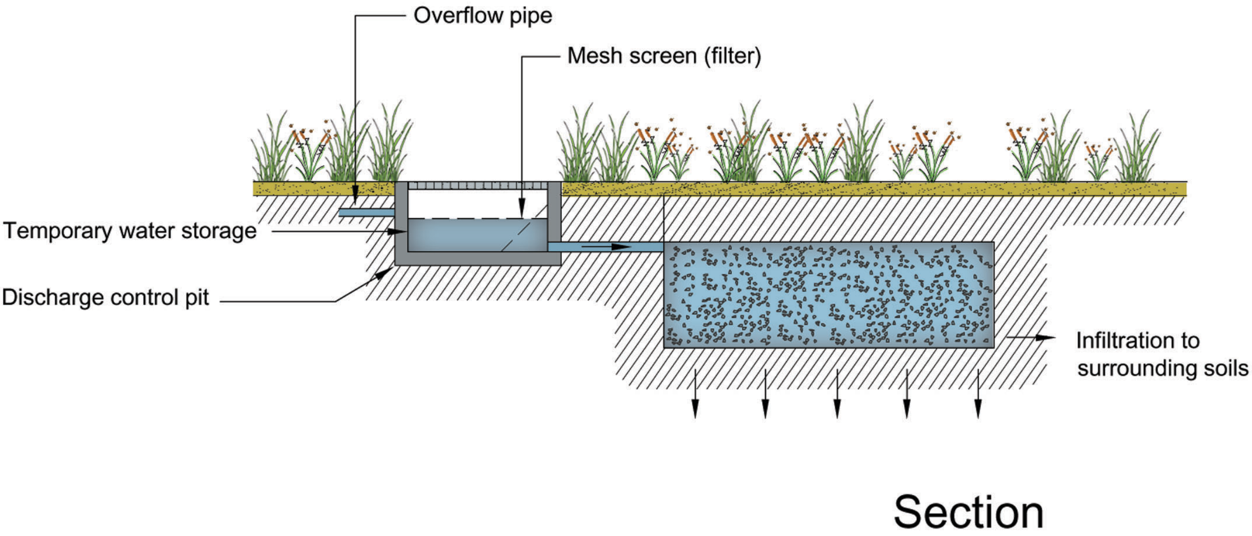


Figure 11.1 Operation of a gravel filled 'soak-away' pit-style infiltration system.

Key factors influencing the operation of an infiltration system are the relationship between infiltration rate, the volume of runoff discharged into the infiltration system, depth to groundwater or bedrock and the available detention storage, that is:

- infiltration rate Q_{inf}) is a product of the infiltration area (A) and the hydraulic conductivity of the *in situ* soil (K_h), i.e. $Q_{inf} = A \times K_h$ m^3/s – therefore, different combinations of infiltration area and hydraulic conductivity can produce the same infiltration rate
- volume of runoff discharged into an infiltration system is a reflection of the catchment area of the system and the meteorological characteristics of the catchment
- detention storage provides temporary storage of inflow to optimise the volume of runoff that can be infiltrated.

The **hydrologic effectiveness** of an infiltration system defines the proportion of the mean annual runoff volume that infiltrates. For a given catchment area and meteorological condition, the hydrologic effectiveness of an infiltration system is determined by the combined effect of the soil hydraulic conductivity, infiltration area and available detention storage. As outlined in Engineers Australia (2003), there are four basic types of detention storages used for promoting infiltration, these being:

- single-size gravel or crushed concrete trenches
- upstand slotted pipes forming 'leaky wells'
- 'milk-crate' type trenches or 'soakaways'
- infiltration basins.

11.2 Verifying size for treatment

The curve (Figure 11.2) shows the relationships between the hydrologic effectiveness, infiltration area and detention storage for a range of soil hydraulic conductivities using Melbourne meteorological conditions. These charts can be used to verify the selected size of a proposed infiltration system.

11.3 Design procedure: infiltration measures

11.3.1 Checking field conditions

Key factors influencing a site's capability to infiltrate stormwater are the soil permeability, soil reactivity to frequent wetting, presence of groundwater and its environmental values, and site terrain.

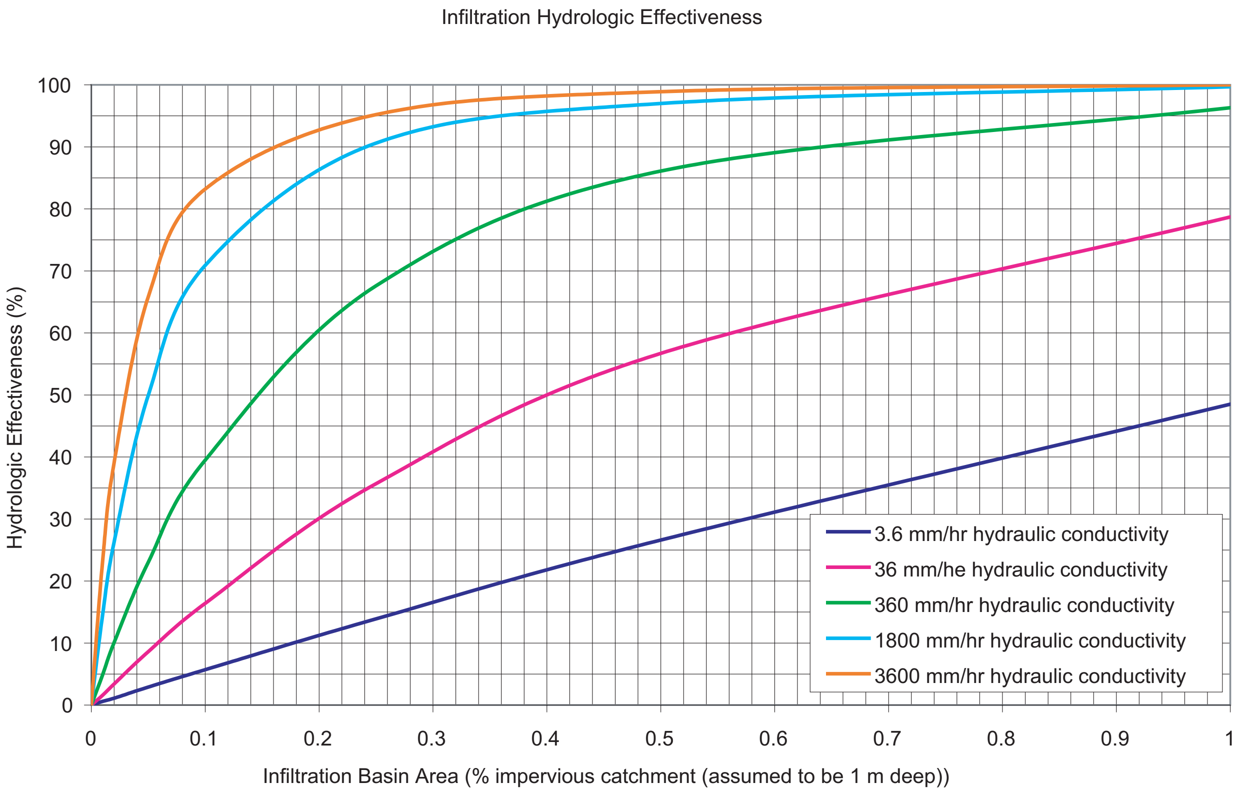


Figure 11.2 Hydrologic effectiveness of detention storages for infiltration systems in Melbourne.

11.3.1.1 Site terrain and soil salinity

A combination of poor soil conditions (e.g. sodic and dispersive soils), steep terrain and shallow saline groundwater can render the use of infiltration systems inappropriate. Dryland salinity is caused by a combination of factors, including leaching of infiltrated water and salt at 'break-of-slope' terrain and the tunnel erosion of dispersive soils. Soil with high sodicity is generally not considered to be suited for infiltration as a means of managing urban stormwater.

Infiltration into steep terrain can result in the stormwater re-emerging onto the surface at some point downstream. The likelihood of this pathway for infiltrated water depends on the soil structure, with duplex soils and shallow soil over rock being situations where re-emergence of infiltrated water to the surface is most likely to occur. This occurrence does not necessarily preclude infiltrating stormwater, unless leaching of soil salt is associated with this process. The provision for managing this pathway will need to be taken into consideration at the design stage.

11.3.1.2 Hydraulic conductivity

Field hydraulic conductivity tests must be undertaken to confirm assumptions of soil hydraulic conductivity adopted during the concept design stage. Field soil hydraulic conductivity (K_h) can be determined using the falling head augerhole method of Jonasson (1984). The range of soil hydraulic conductivities typically determined from a 60-minute falling head period is as follows:

Sandy soil: $K_{60} = 5 \times 10^{-5}$ m/s (180 mm/hr)

Sandy clay: K_{60} = between 1×10^{-5} and 5×10^{-5} m/s (36–180 mm/hr)

Medium clay: K_{60} = between 1×10^{-6} and 1×10^{-5} m/s (3.6–36 mm/hr)

Heavy clay: K_{60} = between 1×10^{-8} and 1×10^{-6} m/s (0.036–3.6 mm/hr)

where K_{60} is the 60-minute value of hydraulic conductivity.

Saturated hydraulic conductivity (K_{sat}) is the hydraulic conductivity of a soil when it is fully saturated. The K_{60} is considered to be a reasonable estimate of K_{sat} for design purposes and can be measured in the field.

Soil is inherently non-homogeneous and field tests can often misrepresent the areal hydraulic conductivity of a soil into which stormwater is to be infiltrated. Field experience has suggested that field tests of 'point' soil hydraulic conductivity can often underestimate the areal hydraulic conductivity of clay soils and overestimate the value for sandy soils. To this end, Engineers

Table 11.1 Moderation factors to convert point to areal conductivities (after Engineers Australia 2003)

Soil type	Moderation factor (U) (to convert 'point' K_h to areal K_h)
Sandy soil	0.5
Sandy clay	1.0
Medium and heavy clay	2.0
K_h = soil hydraulic conductivity	

Australia (2003) recommends that moderation factors (U) for hydraulic conductivities determined from field tests be applied (Table 11.1).

11.3.1.3 Groundwater

Two groundwater issues need to be considered when implementing an infiltration system. The first relates to the environmental values of the groundwater (i.e. the receiving water) and it may be necessary to achieve a prescribed water quality level before stormwater can be discharged. A second design factor is to ensure that the base of an infiltration system is always above the groundwater table and consideration of the seasonal variation of groundwater levels is essential if a shallow groundwater table is likely to be encountered. This investigation should include groundwater mounding (i.e. higher levels very close to the infiltration system) that in shallow groundwater areas could cause problems with nearby structures.

11.3.2 Estimating design flows

11.3.2.1 Design discharges

Two design flows are required for infiltration systems:

- peak inflow to the infiltration system for design of an inlet structure
- major flood rates for design of a bypass system.

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

Design discharge for the bypass system is often set at the 100-year ARI event or the discharge capacity of the stormwater conveyance system directing stormwater runoff to the infiltration system.

11.3.2.2 Minor and major flood estimation

A range of hydrologic methods can be applied to estimate design flows. With typical catchment areas discharging to infiltration measures being relatively small (< 1 ha), the **Rational Method** Design Procedure is considered to be a suitable method for estimating design flows.

Figure 11.3 shows an assumed shape of an inflow hydrograph that can be used to estimate the temporary storage volume for an infiltration system. The flow rate shown on the diagram represents a linear increase in flow from the commencement of runoff to the time of concentration (t_c), then this peak flow rate is maintained for the storm duration. Following the

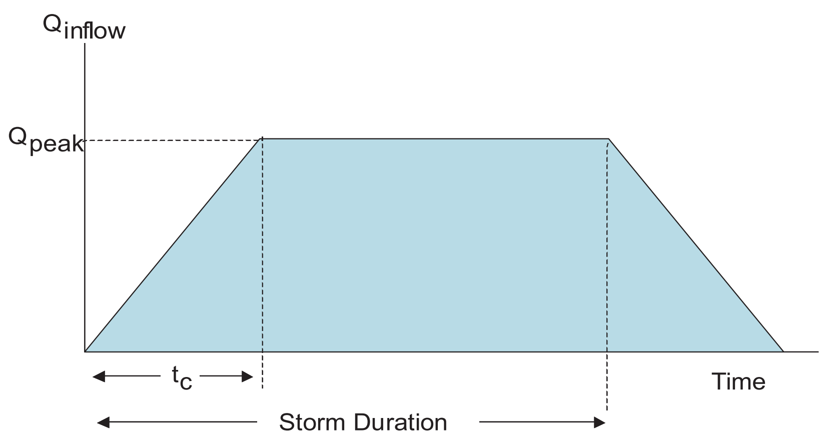


Figure 11.3 Generalised shape of inflow hydrograph.

Table 11.2 Minimum set-back distances
(after Engineers Australia 2003)

Soil type	Saturated hydraulic conductivity m/s (mm/hr)	Minimum distance from structures and property boundaries (m)
Sand	> 5 x 10^{-5} (180)	1.0
Sandy clay	1 x 10^{-5} to 5 x 10^{-5} (36–180)	2.0
Weathered or fractured rock	1 x 10^{-6}–1 x 10^{-5} (3.6–36)	2.0
Medium clay	1 x 10^{-6}–1 x 10^{-5} (3.6–36)	4.0
Heavy clay	1 x 10^{-8}–1 x 10^{-6} (0.036–3.6)	5.0

storm duration the flow rate decreases linearly over the time of concentration. This is a simplification of an urban hydrograph for the purposes of design.

11.3.3 Location of infiltration systems

Infiltration systems should not be placed near building footings to remove the influence of continually wet subsurface or greatly varying soil moisture contents on the structural integrity of these structures. Engineers Australia (2003) recommends minimum distances from structures (and property boundaries to protect possible future buildings in neighbouring properties) for different soil types (Table 11.2).

Identifying suitable sites for infiltration systems should also include avoidance of steep terrain and areas of shallow soils overlying largely impervious rock (non-sedimentary rock and some sedimentary rock such as shale). An understanding of the seasonal variation of the groundwater table is also an essential element in the design of these systems.

11.3.4 Source treatment

Treatment of source water for the removal of debris and sediment is essential and storm runoff should never be conveyed directly into an infiltration system. Pretreatment measures include the provision of leaves and roof litter guards along the roof gutter, sediment sumps, vegetated **swales**, bioretention systems or sand filters.

11.3.5 Sizing the detention storage

There are generally two different methods for determining the size of the detention storage of an infiltration system, i.e. continuous simulation and event-based approaches.

The continuous simulation approach to determining the detention storage volume of an infiltration basin is most suited to meeting design objectives associated with mean annual pollutant load and stormwater runoff volume reduction. This approach uses the hydrologic effectiveness curves as typically shown in Figure 11.1 to determine the size of the infiltration basin on the basis of the percentage of the mean annual runoff infiltrated (hydrologic effectiveness). The design parameters for areas other than Melbourne can be determined from applying the adjustment curves for bioretention systems.

The event-based approach is most suited when the design criteria is based on achieving peak flow reductions as well as volume reduction for pre-specified probabilistic events. The methodology follows that of Argue (2004). The sections below and the worked example present this methodology in greater detail.

11.3.5.1 Storage volume

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

$$\text{Inflow volume } (v_i) \text{ (for storm duration } D, \text{m}^3) = (C \times I \times A \times D)/1000 \qquad \text{(Equation 11.1)}$$

where C is the runoff coefficient as defined in ARR (Institution of Engineers 2001) Book VIII
I is the probabilistic rainfall intensity (mm/hr)
A is the contributing area connected to the infiltration system (m^2)
D is the storm duration (hours).

Outflow from the infiltration system is via the base and sides of the infiltration system and depends on the area and depth of the infiltration system. In computing the infiltration from the walls of an infiltration system, Engineers Australia (2003) suggests that pressure is hydrostatically distributed and thus equal to half the depth of water over the bed of the infiltration system, that is:

$$\text{Outflow volume } (v_o) \text{ (for storm duration } D, \text{m}^3) = \{[(A_{inf}) + (P \times d)/2]\} \times U \times K_h \times D/1000 \qquad \text{(Equation 11.2)}$$

where K_h is the 'point' saturated hydraulic conductivity (mm/hr)
A_{inf} is the infiltration area (m^2)
P is the perimeter length of the infiltration area (m)
d is the depth of the infiltration system (m)
U is the 'point' soil hydraulic conductivity moderating factor (see Table 11.1)
D is the storm duration (hours)

Approximations of the required storage volumes of an infiltration system can be computed as follows:

$$\text{Required storage } (\text{m}^3) = \{(C \times I \times A) - [(A_{inf}) + (P \times d/2)] \times U \times K_h\} \; D/1000 \qquad \text{(Equation 11.3)}$$

Computation of the required storage will need to be carried out for the full range of probabilistic storm durations, ranging from six minutes to 72 hours. The critical storm event is the one which results in the highest required storage. A spreadsheet application is the most convenient way of doing this.

11.3.5.2 Emptying time

Emptying time is defined as the time taken to fully empty a detention storage associated with an infiltration system following the cessation of rainfall. This is an important design consideration as the computation procedure associated with Equation 11.3 assumes that the storage is empty prior to the commencement of the design storm event. Engineers Australia (2003) suggest an emptying time of the detention storage of infiltration systems to vary from 12 hours to 84 hours, depending on the Average Recurrence Interval (ARI) of the design event with the former being more appropriate for frequent events (1 in 3-month ARI) and the latter to less frequent events of 50 years or longer ARI.

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

11.3.6 Hydraulic structures

Two checks of details of the inlet hydraulic structure are required for infiltration systems (i.e. provision of energy dissipation and bypass of above-design discharges). Bypass can be achieved in several ways, most commonly a surcharge pit, an overflow pit or discharge into an overflow pipe connected to a drainage system (see Chapters 5, 6 and 8 for designing a surcharge pit).

11.3.7 Design calculation summary

An *Infiltration System Calculation Checklist* is included to aid the design process of key design elements of an infiltration system.

11.4 Checking tools

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

Checklists are provided for:

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

11.4.1 Design assessment checklist

The *Infiltration Design Assessment Checklist* presents the key design features that should be reviewed when assessing the design of an infiltration system. These considerations include configuration, safety, maintenance and operational issues that should be addressed during the design phase.

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

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

Infiltration System — CALCULATION CHECKLIST

CALCULATION TASK	OUTCOME	CHECK
1 Identify design criteria		☐
Design ARI event to be infiltrated (in its entirety)		year
Or		
Design hydrologic effectiveness		%
ARI of bypass discharge		year
2 Site characteristics		☐
Catchment area connected to infiltration system		m^2
Impervious area connected to infiltration system		m^2
Site hydraulic conductivity		mm/hr
Areal hydraulic conductivity moderating factor		
3 Estimate design flow rates		
Time of concentration		
Estimate from flow path length and velocities		minutes ☐
Identify rainfall intensities		
Station used for IFD data:		
Design rainfall intensity for inlet structure(s)		mm/hr
Design rainfall intensity for overflow structure(s)		mm/hr ☐
Design runoff coefficient		
Inlet structure(s)		☐
Peak design flows		☐
Inlet structure(s)		m^3/s
Bypass structure(s)		m^3/s
4 Detention Storage		☐
Volume of detention storage		m^3
Dimensions		L:W
Depth		m
Emptying time		hr
5 Provision of Pre-treatment		☐
Receiving groundwater quality determined		
Upstream pre-treatment provision		
6 Hydraulic Structures		
Inlet structure		☐
Provision of energy dissipation		
Bypass structure		☐
Weir length		m
Afflux at design discharge		m
Provision of scour protection		
Discharge pipe		☐
Capacity of discharge pipe		m^3/s

Infiltration Design Assessment Checklist					
Bioretention location:					
Hydraulics	Minor flood: (m^3/s)		Major flood: (m^3/s)		
Area	Catchment area (ha):		Infiltration area (ha)		
Treatment				**Y**	**N**
Pretreatment system sufficient to protect groundwater?					
Infiltration storage volume verified from curves?					
Inlet zone/hydraulics				**Y**	**N**
Station selected for IFD appropriate for location?					
Overall flow conveyance system sufficient for design flood event?					
Velocities at inlet and within infiltration system will not cause scour?					
Bypass sufficient for conveyance of design flood event?					
Basin				**Y**	**N**
Maximum ponding depth will not impact on public safety?					
Maintenance access provided to base of infiltration (where reach to any part of a basin >6 m)?					

Land and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the *Asset Handover Checklist* (see Section 11.4.4).

11.4.2 Construction advice

General advice is providedfor the construction of infiltration systems. It is based on observations from construction projects around Australia.

Building phase damage

Protection of infiltration media and vegetation is critical during the building phase as uncontrolled building site runoff is likely to cause excessive **sedimentation**, introduce litter and require replacement of media.

Traffic and deliveries

Ensure traffic and deliveries do not access infiltration areas during construction. Traffic can compact the filter media, cause preferential flow paths and clogging of the surface, deliveries and wash down material can also clog **filtration media**. Infiltration areas should be fenced off during the building phase and controls implemented to avoid washdown wastes.

Timing for engagement

It is critical to ensure that the pretreatment system for an infiltration device is fully operational before flows are introduced into the infiltration media. This will prolong the life of the infiltration system and reduce the risk of clogging.

Inspection wells

It is good design practice to install inspection wells at numerous locations in an infiltration system. This allows water levels to be monitored during and after storm events and infiltration rates can be confirmed over time.

Clean drainage media

Ensure drainage media is washed prior to placement to remove fines and prevent clogging.

11.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Infiltration measures

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

SITE: ____________________

CONSTRUCTED BY: ____________________

DURING CONSTRUCTION									
Items inspected	Checked		Satisfactory	Unsatisfactory		Checked		Satisfactory	Unsatisfactory
Preliminary works	Y	N			**Structural components**	Y	N		
1. Erosion and sediment control plan adopted					10. Location and levels of overflow points as designed				
2. Traffic control measures					11. Pipe joints and connections as designed				
3. Location same as plans					12. Concrete and reinforcement as designed				
4. Site protection from existing flows					13. Inlets appropriately installed				
Earthworks					14. Observation wells appropriately installed				
5. Excavation as designed					**Infiltration system**				
6. Side slopes are stable					15. Correct filter media used				
Pre-treatment					16. Fines removed from filter media				
7. Maintenance access provided					17. Inlet and outlet as designed				
8. Invert levels as designed									
9. Ability to freely drain									
FINAL INSPECTION									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of surface				
2. Traffic control in place					7. No surface clogging				
3. Confirm structural element sizes					8. Maintenance access provided				
4. Filter media as specified					9. Construction generated sediment and debri removed				
5. Confirm pre-treatment is working									

COMMENTS ON INSPECTION

ACTIONS REQUIRED

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

11.4.4 Asset handover checklist

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

11.5 Maintenance requirements

Maintenance for infiltration systems is focused on ensuring the system does not clog with sediments and that an appropriate infiltration rate is maintained. The most important consideration during maintenance is to ensure the pretreatment is operating as designed.

In addition to checking and maintaining the pretreatment, the *Infiltration Maintenance Checklist* is designed to be used during routine maintenance inspections.

Infiltration Maintenance Checklist			
Inspection frequency: 3 monthly	**Date of visit:**		
Location:			
Description:			
Site visit by:			
Inspection items	**Y**	**N**	**Action required (details).**
Sediment accumulation in pretreatment zone requires removal?			
Erosion at inlet or other key structures?			
Evidence of dumping (e.g. building waste)?			
Evidence of extended ponding times (eg. algal growth)?			
Weeds present within device?			
Clogging of drainage points (sediment or debris)?			
Damage/vandalism to structures present?			
Surface clogging visible?			
Drainage system inspected?			
Resetting of system required?			
Comments:			

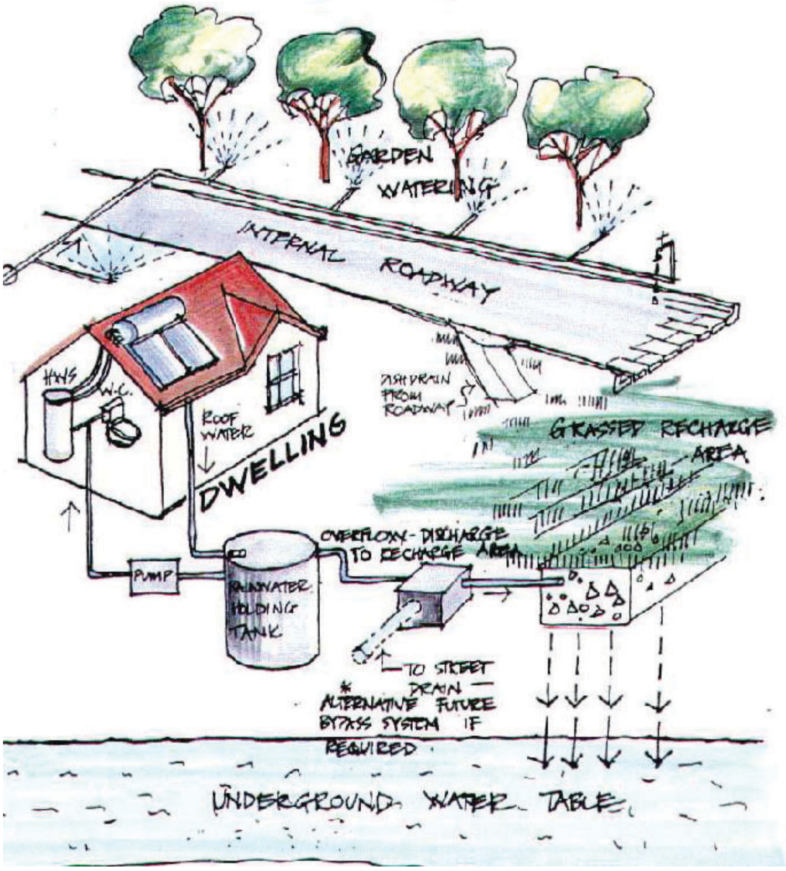


Figure 11.4 A design of an allotment stormwater management scheme (from Urban Water Resource Centre, University of South Australia; http://www.unisa.edu.au/uwrc/ham.htm).

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 in Venus Bay. As discussed in Engineers Australia (2003), pretreatment of stormwater prior to discharge into the ground via infiltration is essential to ensure sustainable operation of the infiltration system and protection of groundwater. Suspended solids and sediment are the key water quality constituents requiring pretreatment 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 'pretreatment'. Stormwater runoff from paved areas will be directed to a pretreatment 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 11.4.

The allotment in question in this worked example is 1000 m^2 on a rectangular site with an overall impervious surface area of 500 m^2. The site layout is shown in Figure 11.5.

Of the impervious surfaces, roof areas make up a total of 210 m^2, while onground impervious surfaces make up the remaining 290 m^2. There is no formal stormwater drainage system, with stormwater runoff discharging into a small table drain in the front of the property. The design objective of the infiltration system is retention of stormwater runoff from the allotment for events up to, and including, the two-year ARI event. Stormwater flows in excess of the two-year ARI peak discharges are directed towards the road table drain at the front of the property.

Roof runoff is directed to a 5 kL rainwater tank. **Rainwater tanks** can provide significant peak discharge reduction owing to available storage capacity prior to the occurrence of a storm event. In this worked example, the design of the infiltration system involves an assumption that the 5 kL tank will be full in the event of a two-year storm event (owing to the extended period

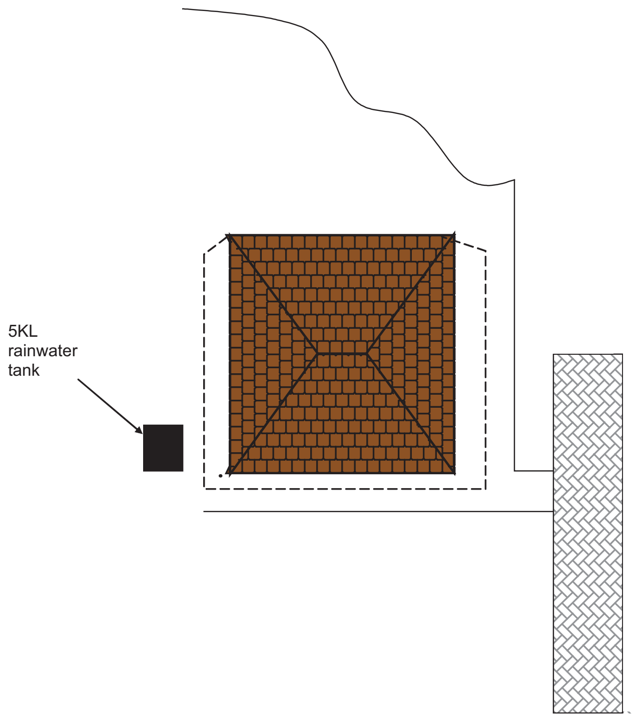


Figure 11.5 Site layout of an infiltration system to treat stormwater runoff from a residential allotment in Venus Bay.

and frequency in which this property will be uninhabited such that the tank water level will not be drawn down as regularly as one associated with a residence that has permanent residents).

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 bypass of stormwater when the infiltration system is operating at its full capacity.

This worked example focuses on the design of the infiltration system and associated hydraulic structures. Analyses to be undertaken during the detailed design phase of the infiltration trench will be based on the procedure outlined in Engineers Australia (2003, chapter 10).

11.6.1.1 Design objectives

The design objectives for an infiltration system are to:

- size infiltration trench to retain the entire runoff volume from the critical (volume) two-year ARI storm event
- design the inlet and outlet structures to convey the peak two-year ARI flow from the critical (flow rate) storm event, ensuring the inlet configuration includes provision for stormwater bypass when the infiltration system is full
- configure the layout of the infiltration trench and associated inlet/bypass structures
- pretreat stormwater runoff
- design appropriate ground cover and terrestrial vegetation over the infiltration trench.

11.6.1.2 Site characteristics

The property is frequently uninhabited and the 5 kL tank will be full for a more significant proportion of time than typical installations. It is assumed that the 5 kL tank will be full at the commencement of the design event. The site characteristics are:

- catchment area 210 m^2 (roof)
  290 m^2 (ground level paved)
  500 m^2 (pervious)
  1000 m^2 (Total)

- landuse/surface type pervious area is either grassed or landscaped with garden beds
- overland flow slope lot is 25 m wide, 40 m deep, slope = 3%
- soil type sandy clay
- saturated hydraulic conductivity (K_h) = 360 mm/hr.

11.6.2 Checking field conditions

Boreholes were drilled at two locations within the site and the results are as follows:

Field tests found the soil to be suitable for infiltration, consisting of fine sand with a saturated hydraulic conductivity of between 360 mm/hr and 1800 mm/hr.

The moderating factor to convert this to the representative areal hydraulic loading is 0.5.

11.6.3 Estimating design flows

The rational method is used to calculate design flows:

- Catchment area = 1000 m^2
  Time of concentration (t_c) ~ 6 min (Institution of Engineers 2001, methods)
- runoff coefficients (Institution of Engineers 2001, Book VIII)

$$^{10}I_1 = 25.6 \text{ mm/hr}$$

$$F_{imp} = 0.5$$

$$C_{10}^{1} = 0.1 + (0.7 - 0.1) \times (^{10}I_1 - 25)/(70 - 25) = 0.11$$

$$C_{10} = 0.9 \times f + C_{10}^{1} \times (1 - f) = 0.50$$

- Runoff coefficients – (Institution of Engineers 2001, Table 8.6)
  C_2 = 0.43
  C_{100} = 0.60.
- Rainfall intensities (Institution of Engineers 2001, Venus Bay)
  t_c = 6 min
  I_2 = 56.4 mm/hr
  I_{100} = 155 mm/hr
- Rational method

$Q = C.I.A/360$ [A = 0.1 ha]
Q_2 = 0.007 m^3/s
Q_{100} = 0.026 m^3/s.

Design discharges Q_2 = 0.007 m^3/s

Q_{100} = 0.026 m^3/s

11.6.4 Location of infiltration systems

With a sandy soil profile, the minimum distance of the infiltration system from structures and the property boundary is 1 m. As the general fall of the site is to the front of the property, it is proposed that the infiltration system be sited near the front of the property with paved area runoff directed to grassed **buffers** and a feature vegetated landscaped area adjacent to the infiltration system.

Overflow from the infiltration system will be directed to the table drain of the street in front of the property.

The infiltration system is to be located near the front of the property set back by at least 1 m from the property boundary.

11.6.5 Source treatment

Roof runoff is directed to a rainwater tank. Although the tank may often be full, it nevertheless serves a useful function as a sedimentation basin. This configuration is considered sufficient to provide the required sediment pretreatment for roof runoff.

Calculation of dimensions of soakaways

Location	Venus Bay			
Catchment area	1000 m^2	Infiltration area	16 m^2	
Volumetric runoff coefficient	0.55	Perimeter of infiltration area	20 m	
Soil K_h	360 mm/hr	Emptying time	1 hour	OK
Moderating factor	0.5			
Width of infiltration area	2 m			
Length of infiltration area	8 m			
Depth of storage	1 m			
Porosity	0.35			

Storm duration	Storm mean intensity	Volume in	Volume out (during storm duration period)	Storage volume required	Percentage of storage provided	
(minutes)	(mm/hr)	(m^3)	(m^3)	(m^3)	%	
6	56.39	3.101	0.468	2.633	213%	OK
12	42.29	4.652	0.936	3.716	151%	OK
18	34.87	5.754	1.404	4.350	129%	OK
30	26.71	7.345	2.340	5.005	112%	OK
45	21.27	8.774	3.510	5.264	106%	OK
60	17.97	9.884	4.680	5.204	108%	OK
90	14.11	11.641	7.020	4.621	121%	OK
120	11.84	13.024	9.360	3.664	153%	OK
180	9.22	15.213	14.040	1.173	477%	OK
240	7.72	16.984	16.984	0.000		OK
300	6.72	18.480	18.480	0.000		OK
360	6.01	19.833	19.833	0.000		OK
480	5.03	22.132	22.132	0.000		OK
600	4.39	24.145	24.145	0.000		OK
720	3.92	25.872	25.872	0.000		OK
840	3.53	27.181	27.181	0.000		OK
960	3.22	28.336	28.336	0.000		OK
1080	2.98	29.502	29.502	0.000		OK
1200	2.77	30.470	30.470	0.000		OK
1320	2.59	31.339	31.339	0.000		OK
1440	2.44	32.208	32.208	0.000		OK
2160	1.83	36.234	36.234	0.000		OK
2880	1.48	39.072	39.072	0.000		OK
3600	1.24	40.920	40.920	0.000		OK
4320	1.07	42.372	42.372	0.000		OK

Figure 11.6 Spreadsheet for calculating required storage volume of infiltration system (spreadsheet included on CD).

Stormwater runoff from paved areas is directed to a combination of grass buffer areas and a landscape vegetated area which is slightly depressed to provide for trapping of suspended solids conveyed by stormwater. Stormwater overflows from the landscaped area into a grated sump pit and then into the infiltration system.

Pretreatment for sediment removal is provided by:

1. collection of roof runoff into a rainwater tank
2. runoff from the paved area being conveyed to a combination of grassed buffer areas and a landscaped vegetated depression.

11.6.6 Sizing the detention storage

Estimating the required storage volume of the infiltration system involves the computation of the difference in the volumes of stormwater inflow and infiltration outflow according to Equation 11.3. A gravel-filled trench will be used, with a proposed depth of 1 m.

Figure 11.6 shows the spreadsheet developed to undertake the calculations to determine the required dimension of a gravel-filled soakaway trench for the range of probabilistic two-year ARI storm durations. By varying the size (and perimeter) of the infiltration system, at least 100% of required storage is provided for all storm durations.

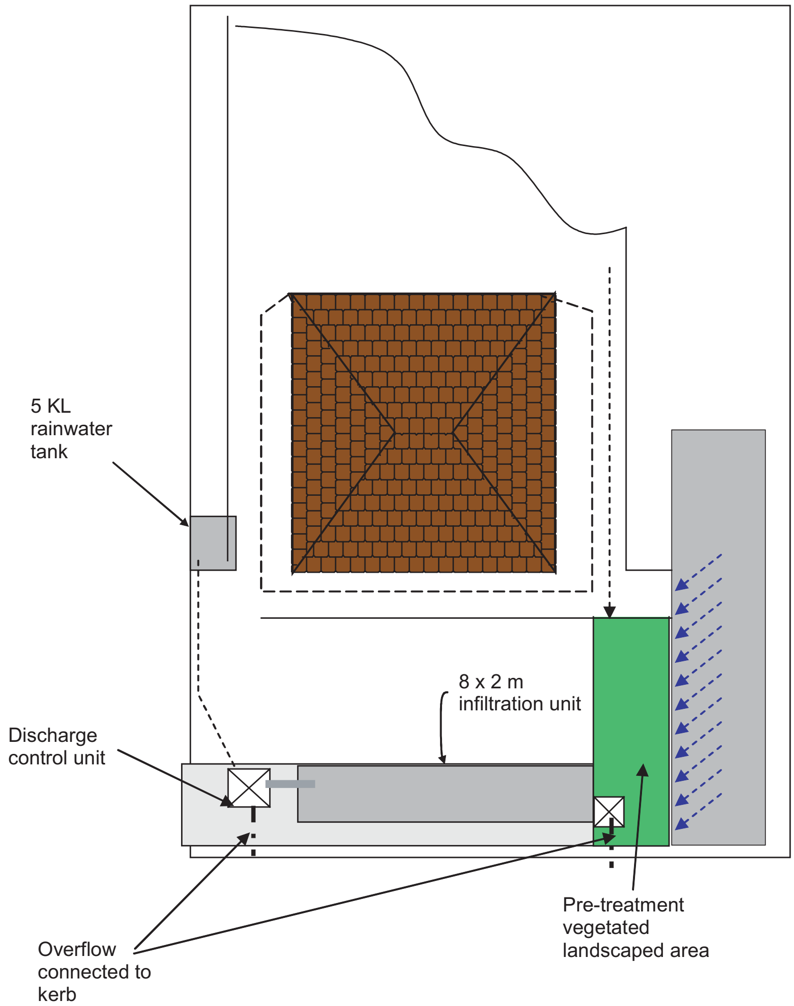


Figure 11.7 Layout of a proposed stormwater infiltration system.

As shown in Figure 11.7, the storm duration that provides the lowest percentage of required storage (above 100%) is a storm duration of 45 minutes (the dimensions of the infiltration device in the spreadsheet have been altered until the storage is greater than 100% for each storm duration). The critical storm duration is 45 minutes and the storage volume requirement 5.3 m^3. With a porosity of a gravel-filled trench estimated to be 0.35, the required dimension of the soakaway is 2 m (width) by 8 m (length) by 1 m (depth). The proposed layout of the infiltration system is shown in Figure 11.7.

11.6.7 Hydraulic structures

11.6.7.1 Inlet design

There are several mechanisms that need to be designed:

- peak two-year ARI design flow
- inlets to the infiltration system
- pipe connections.

Peak two-year ARI design flow is 0.007 m^3/s (calculated in Section 11.6.3) with about 0.003 m^3/s discharging from the rainwater tank overflow and 0.004 m^3/s from other paved areas.

There are two inlets to the infiltration system: from the rainwater tank; and from the driveway (Figure 11.7). These inlets are to be designed to discharge flows up to 0.004 m^3/s each into the infiltration trench with overflows directed to the table drain on the street in front of the property.

Pipe connections from the inlet pits to the infiltration system and street table drain are computed using the orifice flow equation (Equation 11.4)

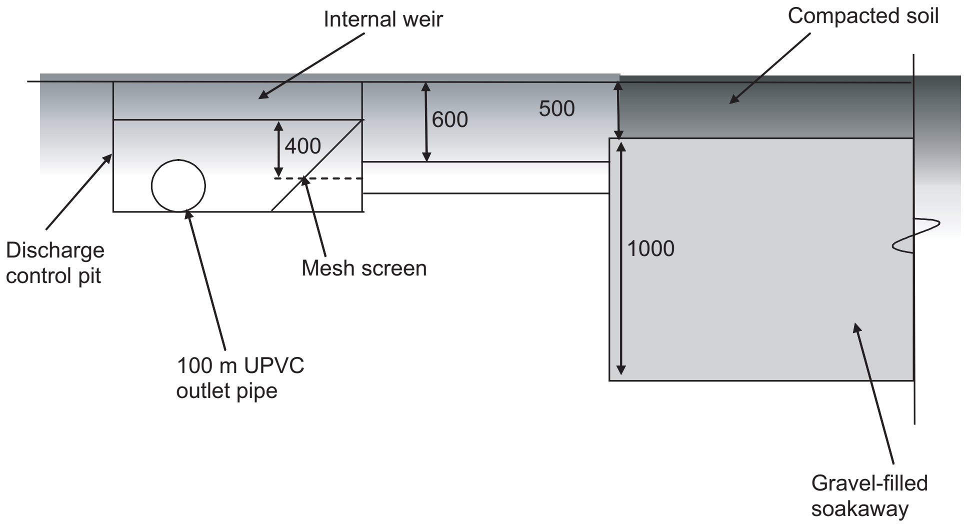


Figure 11.8 Inlet design

$$A_o = \frac{Q}{C_d\sqrt{2gh}} \qquad \text{(Equation 11.4)}$$

C_d = orifice discharge coefficient = 0.6
H = depth of water above the centroid of the orifice (m)
A_o = orifice area (m^2)

For pipe connections to the infiltration system, adopt h = 0.40 m; Q = 0.004 m^3/s. This gives an orifice area of 0.002 m^2, equivalent to a 55 mm diameter pipe → adopt 100 mm diameter uPVC (rigid PVC) pipe (Figure 11.8).

11.6.7.2 Bypass design

An overflow **weir** (internal weir) separates two chambers in the inlet pits: connecting to the infiltration system; and conveying overflows (in excess of the two-year ARI event) to the street table drain. The overflow internal weirs in discharge control pits are to be sized to convey the peak 100-year ARI flow:

Q_{100} = 0.5 × 0.026 m^3/s (two inlet pits) = 0.013 m^3/s

The weir flow equation is used to determine the required weir length:

$$L = \frac{Q}{C_w \Delta H^{1.5}} \qquad \text{(Equation 11.5)}$$

Adopting C =1.7 and H = 0.05 gives
L = 0.7.

Overflow weir will provide at least 150 m freeboard during the peak 100-year ARI flow.

For pipe connection to the street table drain, adopt h = 0.40 m; Q = 0.013 m^3/s. This gives an orifice area of 0.008 m^2, equivalent to a 100 mm diameter pipe → adopt 100 mm diameter uPVC pipe (Figure 11.9).

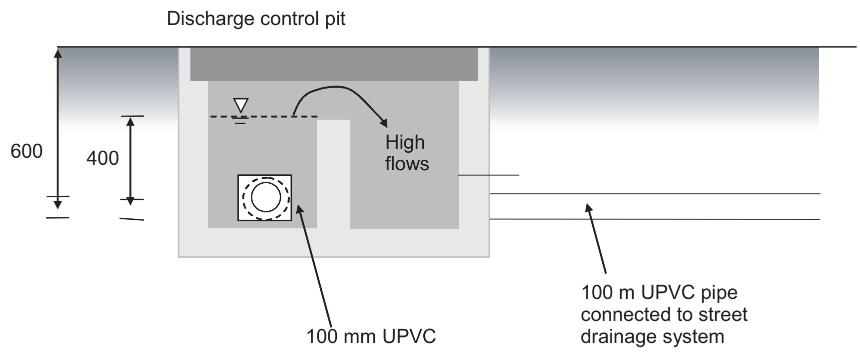


Figure 11.9 Bypass design

11.6.8 Design calculation summary

The completed *Infiltration System Calculation Summary* shows the results of the design calculations.

Infiltration System CALCULATION SUMMARY			
CALCULATION TASK	**OUTCOME**		**CHECK**
1 Identify design criteria			✓
Design ARI event to be infiltrated (in its entirety)	2	year	
Design hydrologic effectiveness	N/A	%	
ARI of bypass discharge	100	year	
2 Site characteristics			✓
Catchment area connected to infiltration system	1000	m^2	
Impervious area connected to infiltration system	500	m^2	
Site hydraulic conductivity	360	mm/hr	
Areal hydraulic conductivity moderating factor	0.5		
3 Estimate design flow rates			
Time of concentration			
Estimate from flow path length and velocities	6	minutes	✓
Identify rainfall intensities			
Station used for IFD data:	Venus Bay		
Design rainfall intensity for inlet structure(s)	56.4	mm/hr	
Design rainfall intensity for overflow structure(s)	155	mm/hr	✓
Design runoff coefficient			
Inlet structure(s)	0.43 to 0.60		✓
Peak design flows			✓
Inlet structure(s)	0.004	m^3/s	
Bypass structure(s)	0.013	m^3/s	
4 Detention Storage			✓
Volume of detention storage	5.3	m^3	
Dimensions	8 m x 2 m	L:W	
Depth	1	m	
Emptying time	1	hr	
5 Provision of Pre-treatment			✓
Receiving groundwater quality determined	Y		
Upstream pre-treatment provision	Y		
6 Hydraulic Structures			
Inlet structure			✓
Provision of energy dissipation	Y		
Bypass structure			✓
Weir length	0.70	m	
Afflux at design discharge	0.05	m	
Provision of scour protection	Y		
Discharge pipe			✓
Capacity of discharge pipe	0.013	m^3/s	

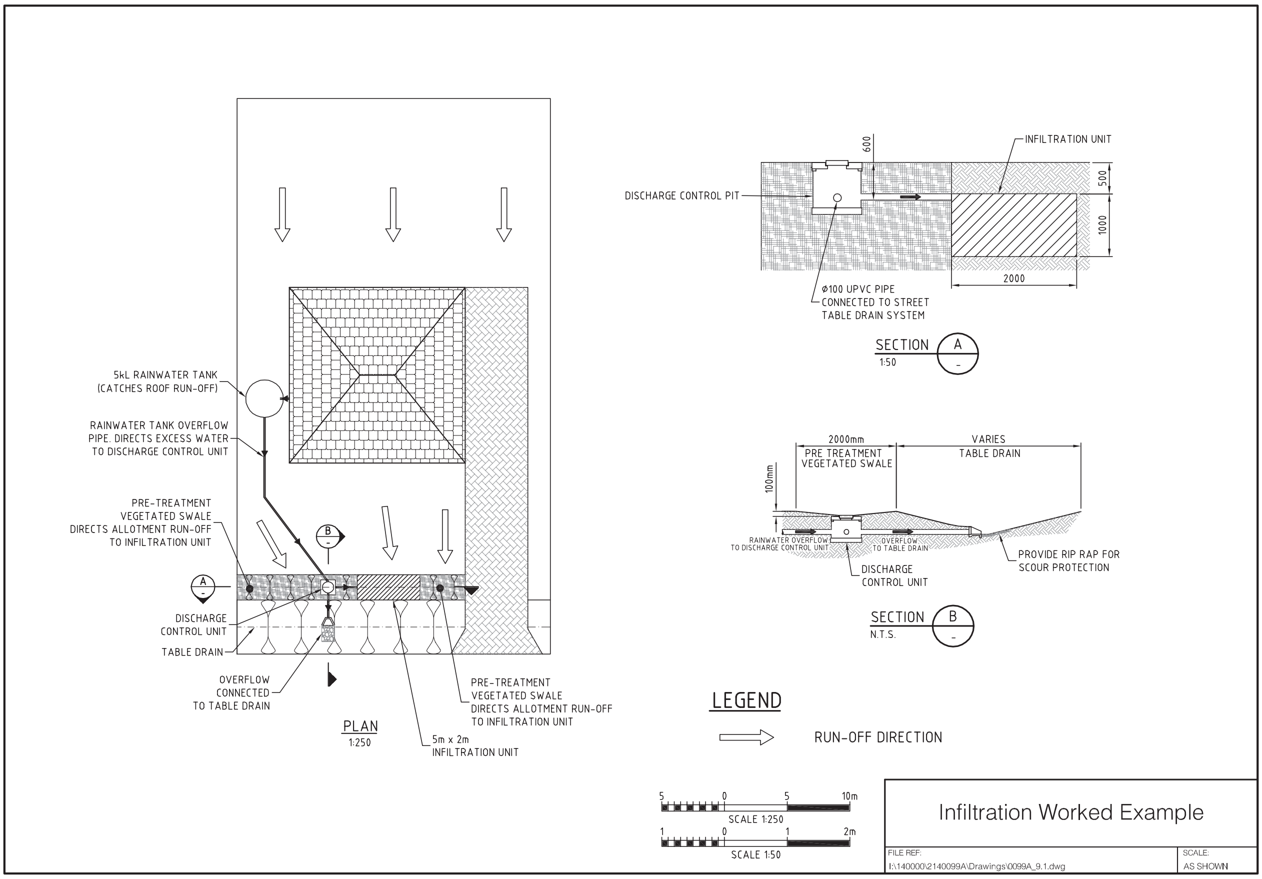


Figure 11.10 Infiltration worked example.

11.6.9 Construction drawing

Figure 11.10 shows the construction drawing for the worked example.

11.7 References

Argue, J.R. (ed) (2004). *Water Sensitive Urban Design: Basic Procedures for 'Source Control' of Stormwater*, Urban Water Resources Centre, University of South Australia.

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

Institution of Engineers, Australia (2001). *Australian Rainfall and Runoff – A Guide to Flood Estimation*, Revised edn, Pilgram, D.H. (Ed.), Institution of Engineers, Australia, Barton, ACT.

Jonasson, S.A. (1984). 'Determination of infiltration capacity and hydraulic conductivity', Proceedings of the 3rd International Conference on Urban Storm Drainage, IAWQ/IAHR, Gothenburg, Sweden.