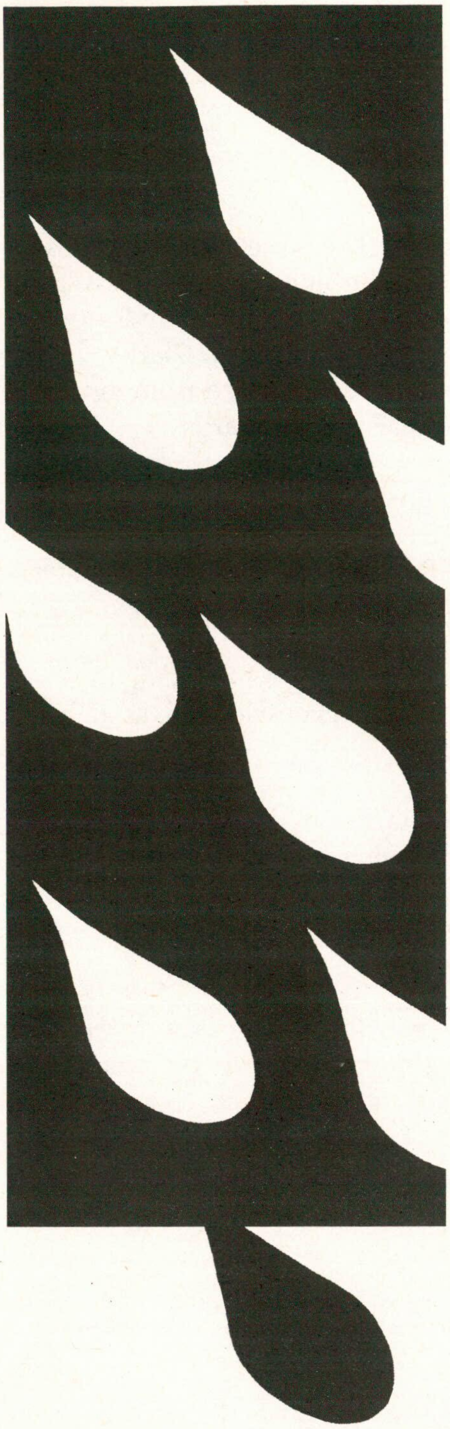
Treatment Techniques

Managing Urban Stormwater

EPA
NSW



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For technical information about this report, contact:
Principal Project Officer—Stormwater
Environment Protection Authority
BOC Gases Building
799 Pacific Highway
PO Box 1135
Chatswood 2057
Phone: (02) 9795 5000
Fax: (02) 9325 5678

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Environment Protection Authority
BOC Gases Building
799 Pacific Highway
PO Box 1135
Chatswood 2057
Phone: (02) 9795 5000 (main switchboard)
Phone: 131 555 (publications and information requests)
Fax: (02) 9325 5678
http://www.epa.nsw.gov.au

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FOREWORD

Urban stormwater management is a complex and challenging issue. There is no single answer to our stormwater management problems and we need to derive innovative approaches using a mix of strategies. The effective management of urban stormwater is also a shared responsibility, requiring the active involvement of many State Government agencies, local councils, the private sector and the community.

The preparation of catchment-based stormwater management plans provides an opportunity to involve all stakeholders in the development of an appropriate and coordinated suite of strategies to address the specific management issues for each catchment. These plans will highlight the significant conceptual shift that has occurred in stormwater management, focusing on issues that affect the health and amenity of our waterways rather than perpetuating the limited and traditional focus on flood mitigation and drainage. Many of the solutions will be found by looking upstream to find ways we can manage a range of day-to-day activities, rather than focusing on managing the impacts downstream.

The NSW Government recognises the need for a whole-of-government approach to stormwater management and for an effective partnership between State and Local Government. As part of this partnership, the Government has committed funding of $60 million over the next three years to tackle stormwater pollution throughout NSW. This funding will be allocated for:

- assisting councils and certain State Government agencies either individually or in groups, to pilot innovation in stormwater management or to undertake remedial activities
- providing assistance to councils for the preparation of stormwater management plans, and
- a State-wide education program to be coordinated by the Environment Protection Authority.

To provide further assistance to local councils and other organisations, the NSW Government is releasing a series of *Managing Urban Stormwater* documents to improve our urban stormwater management practices.

Pam Allan MP
Minister for the Environment

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List of Abbreviations

ARC	Auckland Regional Council
ARI	Average recurrence interval
ASCE	American Society of Civil Engineers
BMP	Best management practice
BOD	Biochemical oxygen demand
CDM	Camp Dresser and McKee
CIRIA	Construction Industry Research and Information Association
CRC	Cooperative Research Centre
DLWC	Department of Land and Water Conservation
DUAP	Department of Urban Affairs and Planning
EMC	Event mean [pollutant] concentration
EPA	Environment Protection Authority (NSW)
EPAV	Environment Protection Authority (Victoria)
FC	Faecal coliforms
FDER	Florida Department of Environmental Regulation (USA)
GPT	Gross pollutant trap
MDE	Maryland Department of the Environment (USA)
NSW	New South Wales
OMEE	Ontario Ministry of Environment and Energy (Canada)
SCCG	Sydney Coastal Councils Group
SKM	Sinclair Knight Merz
SS	Suspended solids
STM	Stormwater treatment measure
TP	Total phosphorus
TN	Total nitrogen
UDFCD	Urban Drainage and Flood Control District (Denver, USA)
USBR	United States Bureau of Reclamation
US EPA	United States Environmental Protection Agency
WEF	Water Environment Federation

1 INTRODUCTION

1.1 Managing Urban Stormwater

This document is part of a package of documents on managing urban stormwater published by NSW Government agencies. The other components of the package are currently:

- *Managing Urban Stormwater: Council Handbook,* which provides guidance to Councils on the preparation of stormwater management plans. The Environment Protection Authority (EPA) will publish this document.
- *Managing Urban Stormwater: Source Control,* which contains a range of techniques for managing stormwater at the source, including education and Council operations. This document will be published by the EPA and the Department of land and Water Conservation.
- *Managing Urban Stormwater: Soils & Construction,* which describes urban soil management and stormwater management of construction sites. Landcom and the Department of Housing will publish this document.

It is important that this document on treatment techniques be seen in the context of the information contained in the companion documents, particularly the *Council Handbook.*

1.2 Purpose of this document

Aim

The aim of this document is to provide guidance to stormwater planners and designers on the selection and functional (or conceptual) design of a range of structural stormwater quality management practices, known as stormwater treatment measures (STMs). These STMs are intended for application in existing and new urban residential areas, and can be selected during the preparation of a stormwater management plan.

Stormwater treatment objectives are not contained in this document. These objectives will be specified in catchment-based stormwater management plans. Further details are contained in *Managing Urban Stormwater: Council Handbook.*

This document is not an exhaustive selection of techniques, but more of a 'source book'. It is not intended to provide detailed design information for stormwater treatment measures (STMs) or to be a definitive 'design manual'. It is also not the intention to stifle the development or application of innovative management practices. It is recognised that stormwater quality management is a rapidly developing field and innovation is strongly encouraged.

Applicability to non-urban (residential) areas

These techniques are not specifically intended to apply to major road or freeway projects or industrial sites. These sites often have specific stormwater treatment requirements that differ from those applicable to residential areas, although some of the techniques in this document might be appropriate. Further details on stormwater quality management from industrial sites can be obtained from US EPA (1993) and CDM (1993).

Uncertainty

It is important for the user to note that there is currently a degree of uncertainty associated with the pollutant retention predictions of these STMs. This is due to the complexity of the pollutant retention processes (refer to section 2) and other factors such as inflow characteristics and STM design details. This highlights the need for further long-term performance monitoring and research, with refinements to design techniques expected to follow over time. In addition, some techniques could have incidental environmental impacts that have only recently been recognised (discussed in section 3.4).

Selecting a Stormwater Treatment Measure

There will not be a single STM that will apply to all situations. A suite of measures will generally be appropriate to suit the characteristics of a site. Appropriate treatment measures can be selected during the development of a stormwater management plan, which can identify the site constraints and the need for stakeholder involvement (particularly the community) in the design development process.

Additional Information

Australian and overseas publications on stormwater treatment techniques that can further help in the planning and design process include:

- *Urban Stormwater: Standard Engineering Practices* (ACT Government 1994)
- *Planning and Management Guidelines for Water Sensitive Urban (Residential) Design* (Whelans et al 1994)
- *Stormwater Treatment Devices: Design Guideline Manual* (Auckland Regional Council 1992)
- *Stormwater Management Practices Planning and Design Manual* (Ontario Ministry of Environment and Energy 1994)
- *California Storm Water Best Management Practice Handbooks: Municipal* (Camp Dresser and McKee 1993)
- *Design and Construction of Urban Stormwater Management Systems* (ASCE&WEF 1992)
- *Fundamentals of Urban Runoff Management: Technical and Institutional Issues* (Horner et al 1994)
- *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs* (Schueler 1987)
- *Urban Storm Drainage Criteria Manual—Volume 3: Best Management Practices* (UDFCD 1992)
- *The Florida Development Manual: A Guide to Sound Land and Water Management* (FDER, 1989)

1.3 Stormwater Treatment in Stormwater Management

Stormwater treatment is only one of a number of factors that should be considered in stormwater management, including streamflow, riparian vegetation and aquatic habitat management. Further, the context of stormwater quality management can be based on the following hierarchy:

- Preserve and restore (if required) existing valuable elements of the stormwater system (e.g. natural channels, wetlands, riparian vegetation).
- Manage the quality and quantity of stormwater at or near the source, which will involve a significant component of public education and community involvement.
- Install 'structural' stormwater management practices, such as stormwater treatment measures and retarding basins, for water quality and streamflow control.

Further details are contained in *Managing Urban Stormwater: Council Handbook*.

2 POLLUTANT TRAPPING PROCESSES

A range of physical, chemical and biological processes that result in the trapping of various pollutants can occur within STMs. These processes, summarised in Table 2.1, are highly complex, often interact, and many are currently not well understood. The nature of a particular process can depend on a range of factors, including the inflow water quality and geochemistry, and the physical and biological characteristics of the STM. Some processes can also result in the release of pollutants from an STM, particularly during low flow conditions.

Table 2.1 - Potential physical, chemical and biological processes in a treatment measure

Name	Process	Pollutant affected
Physical trapping	Trapping floating litter behind a barrier (e.g. litter rack, boom)	Litter
Volatilisation	Evaporation and aerosol formation	Hydrocarbons, mercury
Sedimentation	Gravity settling of particles and adsorbed pollutants	Sediments, hydrocarbons, heavy metals, nutrients
Re-suspension	Re-mobilisation of particles by wind or hydraulic turbulence	Sediments, hydrocarbons, heavy metals, nutrients
Filtration	Mechanical filtration of particles through substrate, aquatic flora or fauna	Sediments
Adsorption	Bonding of ions etc. to sediments or organic matter (generally colloidal)	Hydrocarbons, phosphorus, nitrogen, heavy metals
De-sorption	Release of ions from sediments under adverse conditions (e.g. low pH, anaerobic)	Phosphorus, heavy metals
Oxidation/ Reduction	Oxidation of organic matter by microbial organisms, reduction of metals and nutrients	Hydrocarbons, metals, nitrogen, phosphorus
Complexation/ Chelation	Formation of a complex ion by combining a metal ion with an inorganic ion, etc.	Metals, phosphorus
Precipitation	Formation or co-precipitation of insoluble compounds	Hydrocarbons, metals, phosphorus
Fixation	Fixation of atmospheric nitrogen to ammonia by microbial organisms and chemical fixation	Nitrogen
Nitrification	Microbial conversion of ammonia to nitrate, then to nitrite	Nitrogen
Denitrification	Microbial conversion of nitrate to atmospheric nitrogen	Nitrogen
Biological uptake	Uptake of ions from soil by aquatic plants through root system, limited uptake directly from water; uptake by algae	Metals, phosphorus, nitrogen
Decomposition	Decomposition of organic matter by aquatic invertebrates and microorganisms	Organic matter
Disinfection	Disinfection of pathogens (e.g. bacteria) by ultraviolet light	Pathogens
Aeration	Exchange of oxygen from the atmosphere to the water body	Oxygen demanding substances
Dislocation	Movement of organic matter and algae downstream during high flows	Organic matter, nutrients

Source: after Cullen (1992), Harper et al (1986), Manahan (1991), Lawrence (1996)

Different processes remove different pollutants, and hence some STMs are more effective at removing some pollutants than other measures. Particulate pollutants, or pollutants bound to particulates, and can be removed effectively by STMs that rely on physical settling or filtration as their primary pollutant removal mechanism. Other pollutants are dissolved in stormwater, and must be treated by STMs that facilitate chemical adsorption or biological uptake.

Figure 2.1 shows a hierarchy of stormwater treatment levels based on the dominant treatment processes. At the primary level, physical screening (or trapping) and rapid sedimentation are the dominant processes. This will result in the removal of a proportion of the inflow litter and coarse sediment. At the secondary level, sedimentation and filtration will dominate. This will improve the removal of suspended solids and offer some removal of nutrients and metals. The dominant tertiary level processes are enhanced sedimentation, filtration, biological uptake and adsorption. This will result in the improved retention of nutrients and heavy metals. This is broadly similar to the classification system used in sewage treatment.

In most situations, the use of a combination of different STMs that reduce pollutants through different processes should provide the best overall treatment of runoff.

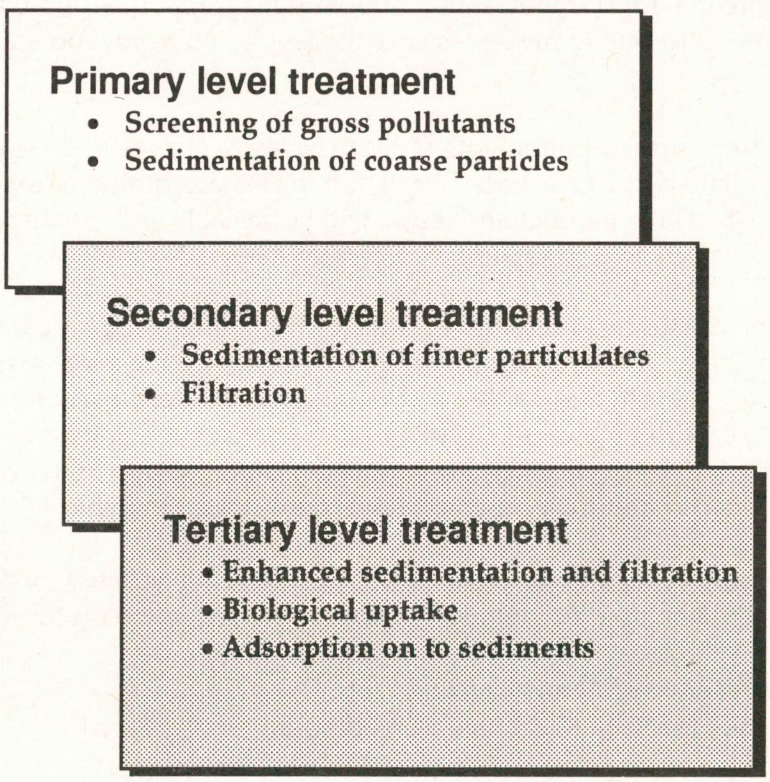


Figure 2.1 - Levels of stormwater treatment
(adapted from EPAV (1996))

3 SELECTING A STORMWATER TREATMENT MEASURE

3.1 Selection Approach

To help with the selection of a STM for a particular site, a number of 'screening tools' are provided in tabular form. The selection of an STM will also be heavily influenced by local climatic conditions, particularly rainfall and evaporation, which are not addressed by these tools.

Table 3.1 indicates the potential pollutant removal for a range of STMs. It should be noted that the range of removal rates in this table are based on optimal design conditions and considerably lower rates can be expected if the design is below optimum. The pollutant retention characteristics for infiltration trenches and basins is based on an assessment of the effective treatment of the proportion of the mean annual runoff that is expected to be treated by the STM. It should be noted that the monitoring of STM performance has not been extensive to date and a number of the removal rates have been estimated based on an understanding of STM designs and pollutant removal processes.

In many circumstances, a 'treatment train' approach will be appropriate to optimise the pollutant removal offered by the treatment system. This involves a series of STMs, which may include primary, secondary and tertiary treatment measures in series.

A number of the STMs require upstream pre-treatment (primarily for coarse sediment removal) to optimise their efficiency, which is also noted in this table. This table also indicates the potential for re-mobilisation of pollutants trapped within the STM. This re-mobilisation can occur due to processes such as desorption from, and scouring of, sediments.

The relative maintenance requirements for each STM are also included in this table. This is expressed as a relative measure, based partly on the expected ratio of capital to maintenance costs. These maintenance costs will be variable and are discussed further in Section 3.6.

The physical site constraints that might influence the choice of a particular STM are summarised in Table 3.2. The STMs that might be influenced by catchment area considerations are indicated in Table 3.3. The potential influence of soil type on STMs is noted in Table 3.4, with further details of soil characteristics contained in *Managing Urban Stormwater: Soils & Construction*. The potential environmental attributes of particular STMs and their associated community amenity value are outlined in Table 3.5.

There are a number of considerations when STMs are being selected for a particular site. A rating system could be used to identify the optimum STM based on these considerations, which include:

- ability to meet any regulatory requirements
- effectiveness of the STM to achieve desired pollutant retention
- capital costs
- operations and maintenance costs
- compatibility with any site constraints (e.g. land availability, services)
- likely public acceptance of the STM (e.g. aesthetics, safety)
- ability to satisfy multiple objectives (e.g. habitat, recreation)

Stormwater Treatment Measure:	Pollutant:								Potential for pollutant re-mobilisation
	Litter and gross pollutants	Coarse sediment (>2 mm)	Suspended solids	Total phosphorus	Total nitrogen	Oxygen demanding substances	Oil and grease	Bacteria	
Litter baskets and pits	Moderate	Low (est.)	Negligible (est.)	Negligible (est.)	Negligible (est.)	Low (est.)	Negligible (est.)	Negligible (est.)	Low
Litter racks	Low	Low (est.)	Negligible (est.)	Negligible (est.)	Negligible (est.)	Low (est.)	Negligible (est.)	Negligible (est.)	Moderate
Sediment traps	Negligible (est.)	High (est.)	Low (est.)	Negligible (est.)	Negligible (est.)	Low (est.)	Negligible (est.)	Negligible (est.)	Moderate
Gross pollutant traps	Moderate	High (est.)	Low (est.)	Negligible (est.)	Negligible (est.)	Low (est.)	Negligible (est.)	Negligible (est.)	Moderate
Litter booms	Low	Negligible (est.)	Negligible (est.)	Negligible (est.)	Negligible (est.)	Low (est.)	Negligible (est.)	Negligible (est.)	Low
Catch basins	Low (est.)	High (est.)	Low (est.)	Negligible (est.)	Negligible (est.)	Low (est.)	Negligible (est.)	Negligible (est.)	High
Oil/grit separators	Low (est.)	Moderate (est.)	Low	Negligible	Negligible	Low	Low	Negligible	High
Filter strips	Moderate (est.)	High (est.)	High	Moderate	Moderate	Low	Low	High (est.)	Low
Grass swales	Moderate (est.)	High (est.)	High	Moderate	Moderate	Low	High	High (est.)	Low
Extended detention basins*	Negligible (est.)	Moderate (est.)	Moderate	Low	Low	Low	Low (est.)	High	Moderate
Sand filters*	Negligible (est.)	Negligible (est.)	High	Moderate	Moderate	Moderate	Moderate (est.)	Moderate (est.)	Low
Infiltration trenches*	Negligible (est.)	Negligible (est.)	High (est.)	Moderate (est.)	Moderate (est.)	High (est.)	Low (est.)	High	Low
Infiltration basins*	Negligible (est.)	Negligible (est.)	High (est.)	Moderate (est.)	Moderate (est.)	High (est.)	Low (est.)	High	Low
Porous pavements	Negligible (est.)	Negligible (est.)	High	Moderate	Low	High	Moderate (est.)	High (est.)	Low
Constructed wetlands	Low (est.)	High (est.)	High	Moderate	Low	Low	Low (est.)	High	Moderate

Legend:

- Negligible [0–10% removal]
- Low [10–50% removal]
- Moderate [50–75% removal]
- High [75–100% removal]
- * Pre-treatment reqd (sediment/litter)
- (shaded) Estimated (limited monitoring data)

Table 3.1 - Potential pollutant removal and maintenance requirements screening tool
(adapted from Schueler (1987), Horner et al (1994) and Mudgway et al (1997))

Stormwater Treatment Measure:	Potential Constraint:								
	Steep site/catchment slope	High water table	Shallow bedrock	Land availability limitation	Installation underground is reqd	High sediment input	Requires pre-treatment	Hydraulic head loss limitation	Installation in tidal system
Litter baskets and pits	●	●	●	●	●	○	●	○	●
Litter racks	●	●	●	●	○	○	●	⊘	○
Sediment traps	●	●	○	○	●	○	○	○	○
Gross pollutant traps	●	●	○	○	○	○	○	⊘	○
Litter booms	●	●	●	●	●	●	●	●	○
Catch basins	●	●	●	●	●	●	●	●	○
Oil/grit separators	●	●	●	●	●	●	●	⊘	○
Filter strips	⊘	⊘	○	⊘	⊘	●	●	●	⊘
Grass swales	⊘	⊘	○	⊘	⊘	●	●	●	⊘
Extended detention basins	○	○	○	⊘	⊘	⊘	●	○	○
Sand filters	●	●	●	●	●	○	●	⊘	○
Infiltration trenches	⊘	⊘	⊘	○	●	⊘	●	○	⊘
Infiltration basins	⊘	⊘	⊘	⊘	○	⊘	●	○	⊘
Porous pavements	⊘	⊘	⊘	⊘	●	⊘	●	○	⊘
Constructed wetlands	⊘	○	○	⊘	⊘	○	●	○	○

Legend:

⊘ Constraint may preclude the use of this STM

○ Constraint may be overcome with appropriate STM design

● Generally not a constraint

Table 3.2 - Potential site constraints screening tool
(adapted from Schueler (1987) and Horner et al (1994))

Stormwater Treatment Measure:	**Catchment area:**									
	< 1 ha	1 – 2 ha	2 – 4 ha	4 – 6 ha	6 - 8 ha	8 – 10 ha	10 – 15 ha	15 – 20 ha	20 – 40 ha	> 40 ha
Litter baskets and pits	■	▒	▒	▒						
Litter racks			▒	▒	▒	■	■	■	▒	
Sediment traps			▒	▒	▒	■	■	■	▒	▒
Gross pollutant traps			▒	▒	▒	■	■	■	▒	▒
Litter booms							▒	▒	■	■
Catch basins	■	▒								
Oil/grit separators	■									
Filter strips	■	■	▒							
Grass swales	■	■	▒							
Extended detention basins			▒	▒	■	■	■	■	■	■
Sand filters	▒	■	■	■	▒	▒	▒	▒		
Infiltration trenches	■	■								
Infiltration basins		▒	■	■	▒					
Porous pavements	■	■	▒							
Constructed wetlands			▒	▒	■	■	■	■	■	■

Legend:

□ Catchment area may preclude use of this STM

▒ Catchment area may limit use of this STM (appropriate STM design may be requd)

■ Catchment area will generally not limit the use of this STM

Table 3.3 - Catchment area screening tool
(adapted from Schueler (1987), Schueler et al (1992), OMEE (1994))

Stormwater Treatment Measure:	Site soil type (typical infiltration rate):									
	Sand (210 mm/h)	Loamy sand (60 mm/h)	Sandy loam (25 mm/h)	Loam (13 mm/h)	Silty loam (7 mm/h)	Sandy-clay loam (4.5 mm)	Clay loam (2.5 mm/h)	Sandy clay (1.3 mm/h)	Silty clay (1.0 mm/h)	Clay (0.5 mm/h)
Litter baskets and pits	■	■	■	■	■	■	■	■	■	■
Litter racks	■	■	■	■	■	■	■	■	■	■
Sediment traps	■	■	■	■	■	■	■	■	■	■
Gross pollutant traps	■	■	■	■	■	■	■	■	■	■
Litter booms	■	■	■	■	■	■	■	■	■	■
Catch basins	■	■	■	■	■	■	■	■	■	■
Oil/grit separators	■	■	■	■	■	■	■	■	■	■
Filter strips	▒	▒	■	■	■	■	■	▒	▒	▒
Grass swales	▒	▒	■	■	■	■	■	▒	▒	▒
Extended detention basins	▒	▒	■	■	■	■	■	■	■	■
Sand filters	▒	▒	■	■	■	■	■	■	■	■
Infiltration trenches		▒	■	■	▒					
Infiltration basins		▒	■	■	▒					
Porous pavements		▒	■	■	▒					
Constructed wetlands	▒	▒	▒	■	■	■	■	■	■	■

Legend:

- □ Site soils may preclude use of this STM
- ▒ Site soils may limit use of this STM (appropriate STM design may be required)
- ■ Site soils will generally not limit the use of this STM

Table 3.4 - Site soils screening tool
(adapted from Schueler (1987), Horner et al (1994))

Stormwater Treatment Measure:	Environmental and community amenity:									
	Groundwater recharge	Peak flow management	Aquatic habitat provision	Wildlife habitat provision	Water temperature increase	Landscape enhancement	Recreational benefits	Aesthetic benefits	Public safety provisions	Likely community acceptance
Litter baskets and pits	⊘	⊘	⊘	⊘	⊘	⊘	⊘	○	○	●
Litter racks	⊘	⊘	⊘	⊘	⊘	⊘	⊘	⊘	○	⊘
Sediment traps	⊘	⊘	○	⊘	○	⊘	⊘	⊘	○	○
Gross pollutant traps	⊘	⊘	○	⊘	○	⊘	⊘	○	○	⊘
Litter booms	⊘	⊘	⊘	⊘	⊘	⊘	⊘	⊘	○	⊘
Catch basins	⊘	⊘	⊘	⊘	⊘	⊘	⊘	⊘	○	●
Oil/grit separators	⊘	⊘	⊘	⊘	⊘	⊘	⊘	○	○	●
Filter strips	○	●	⊘	○	○	○	○	○	●	●
Grass swales	○	●	⊘	○	○	○	○	○	●	●
Extended detention basins	⊘	●	⊘	○	○	○	○	○	○	○
Sand filters	●	○	⊘	⊘	⊘	⊘	⊘	⊘	○	●
Infiltration trenches	●	●	⊘	⊘	⊘	⊘	⊘	⊘	●	●
Infiltration basins	●	●	⊘	○	⊘	○	○	○	○	○
Porous pavements	●	●	⊘	⊘	⊘	⊘	⊘	⊘	●	●
Constructed wetlands	○	○	●	●	●	●	●	●	○	●

Legend:

⊘ Generally not provided [or does not occur]

○ Can be provided with appropriate STM design [or can occur]

● Usually provided [or usually occurs]

Table 3.5 - Environment and community amenity screening tool
(adapted from Schueler (1987) and Horner (1994))

- capability of mitigating other impacts of urbanisation (i.e. reducing runoff volumes or peak flows)
- incidental environmental impacts.

Evaluation of costing could be undertaken on a life-cycle cost basis. Further, for existing areas, availability of funding could preclude the installation of a larger STM (i.e. a number of smaller STMs might need to be installed over a number of years).

3.2 Steps in Design Development

The following steps can be followed in the design of STMs, although a number of these steps may have already been addressed if a stormwater management plan has been prepared:

1. Establish and rank the site-specific objectives for the design, including water quality requirements, recreation, aesthetics, habitat creation and retention, education and maintenance.
2. Define existing conditions at the site, in the catchment, and the receiving waters.
3. Determine the design parameters for the required level of stormwater treatment.
4. Identify site constraints that compromise the optimum design.
5. Evaluate trade-offs between site constraints and desired objectives, review objectives and revise design.
6. Develop the final design.
7. Prepare operations, maintenance and inspection/monitoring plans.

Site constraints may affect the design and require a compromise between effectiveness and practicality. In existing urban areas, compromises will commonly be required. These site-specific factors can include:

- space limitations
- topography (e.g. steep slopes)
- geology (e.g. depth to bedrock or instability)
- soils (e.g. erosivity, permeability)
- groundwater (e.g. geochemistry and water table depth)
- valuable aquatic or riparian habitats
- services and other obstructions (expensive to relocate)
- climatic conditions (e.g. rainfall distribution, evaporation rates)
- community perceptions.

It should be noted that there will rarely be a single, universal approach to selecting and designing a STM that will apply to all sites. Innovation is therefore encouraged.

There are however, usually certain limits that should not be exceeded or key features that need to be included to avoid seriously impairing the pollutant retention performance of the STM Key features that can be expected to enhance the performance of an STM are noted within the text describing each STM.

It is important to note that STMs are generally not intended effectively to treat all catchment runoff. The common goal is to provide effective treatment up to a certain flow level, with lower or negligible treatment provided for flows beyond this level. It is important that the majority of the pollutants trapped during flows less than the design level are not lost during higher flows.

3.3 Locating Stormwater Treatment Measures

There are two broad approaches to locating STMs within a catchment (or subcatchment). The first and more traditional approach is to locate a single large STM at the outlet to treat all runoff. The second is a distributed approach, where a number of smaller STMs are installed throughout the catchment. These are indicated schematically in Figure 3.1.

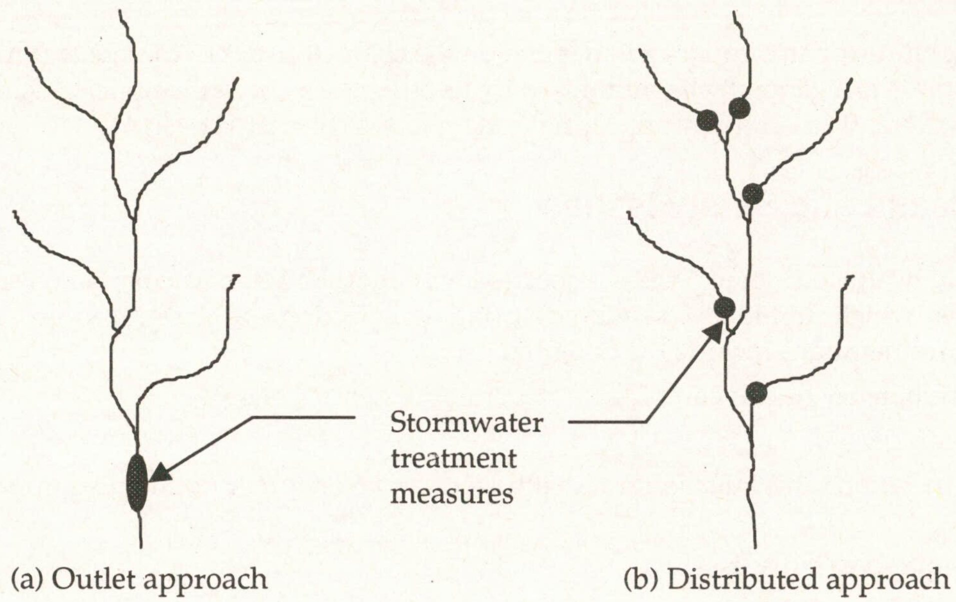


Figure 3.1 – Location of stormwater treatment measures

Potential advantages of the distributed approach include:

- water quality protection of a greater length of waterway
- lower capital costs of individual STMs, partly due to lower flow rates (and hence spillway costs for some STMs). Total system costs can, however, be higher
- lower operations and maintenance costs of individual STMs, although total system maintenance costs can be higher
- improved efficiency, partly due to lower flows
- lower environmental impacts (facilitates aquatic fauna and sediment movement downstream of the STMs)
- potential for improved safety and aesthetics if STMs are sufficiently small to be installed underground
- lower risk of overall system failure (i.e. the failure of a single STM is not likely to have a significant impact on total catchment loads)
- compatibility with a staged approach to implementation (particularly appropriate for developing or existing urban areas).

3.4 Environmental Considerations

A number of STMs have the potential to create adverse environmental impacts at the installation site, which can include:

- inhibiting the passage of aquatic fauna
- retention of sediment to below pre-development levels, potentially causing sediment starvation and downstream watercourse erosion
- loss of aquatic habitat and riparian vegetation
- increased downstream water temperature
- reduced downstream dissolved oxygen concentrations
- outflows from STMs under low-flow conditions having poorer water quality than inflows.

The magnitude of any impact will depend on the characteristics of the site and the STM. Any impacts can generally be minimised by locating the STM in the upper reaches of a catchment or off-line, and by appropriate site selection and STM design.

3.5 Health and Safety Issues

There are health and safety issues associated with most STMs, particularly above-ground measures, which should be considered during the selection and design process. These safety considerations include the potential for:

- physical injury due to climbing on structures (e.g. litter racks)
- drowning
- skin irritations and infection from pathogens and from other pollutants trapped in the STM
- mosquito-borne diseases
- odours
- health and safety risks to maintenance staff
- overtopping of embankments for some STMs during floods in excess of the spillway design flood.

These safety issues should be addressed during the design stage and could require:

- installation of warning signs
- relatively flat side slopes (to a certain depth) in structures liable to inundation
- fencing of any vertical drops or steep side slopes
- strategic placement of structures or vegetation to prevent or hinder access
- spillways designed to meet the requirements of the Dam Safety Committee
- development of occupational health and safety procedures for operations and maintenance staff.

3.6 Operation and Maintenance Considerations

Appropriate maintenance is essential to ensure the long-term pollutant trapping efficiency of all STMs. Maintenance considerations should also be addressed during the design and implementation stage of a STM, including:

- Designing the STM to facilitate maintenance. This can involve additional capital costs but can significantly reduce operation and maintenance costs. Maintenance staff could be involved in reviewing designs to assist with this process.
- Maintenance staff should follow occupational health and safety procedures.
- Avoiding direct human contact with debris and pollutants trapped in an STM (e.g. maintenance staff could wear gloves).
- Routine maintenance should generally not be undertaken during storm events, although emergency maintenance (e.g. unblocking an outlet structure) could be required.
- A maintenance procedure should be developed during the design process.
- Monitoring of pollutant build-up in the STM could be undertaken to enable maintenance to be undertaken before the STM becomes overloaded.
- Disposal facilities for debris and liquid pollutants will need to be arranged before undertaking maintenance.
- Additional, non-programmed, maintenance could be required if problems arise (e.g. odours).

An indication of the capital and relative operation/maintenance costs of STMs is presented in Table 3.6. There are difficulties in comparing capital costs, as the actual cost of an STM will be dependent on the site characteristics and the design of the STM. Actual maintenance costs can also be difficult to determine, as they depend on the nature of the inflows, the type of maintenance equipment and the STM design. Further, there has been limited monitoring of maintenance costs in Australia.

The operation/maintenance costs in Table 3.6 are expressed relative to the capital cost, with an indication of the annual costs being:
- Low – less than 10% of the capital cost
- Medium – between 10 and 70% of the capital cost
- High – greater than 70% of the capital cost.

Maintenance costs are also commonly higher, on a relative basis, for smaller STMs, due to the relatively constant establishment costs.

3.7 Details of Stormwater Treatment Measures

Sections 4 to 6 of this document contain descriptions of a range of STMs that might be suitable for treating runoff from urban residential areas. A summary page is provided for each of the STMs; it includes:
- a schematic section of the STM (not drawn to scale)
- a brief description of the measure
- a summary of the criteria that can be used to select the measure and potential advantages
- an indication of the relative pollutant trapping efficiency of the measure
- the potential limitations and disadvantages of the measure, some of which can be addressed by appropriate site selection and STM design
- cost considerations, both capital and operations/maintenance.

Table 3.6 - Indicative Capital and Operation/Maintenance Costs

Stormwater Treatment Measure	Indicative capital Cost	Indicative relative operation/maintenance cost
Litter baskets and pits	L	H
Litter racks	M	H
Sediment traps	M	M
Gross pollutant traps	H	H
Litter booms	L	H
Catch basins	L	H
Oil/grit separators	M	H
Filter strips	L	L
Grass swales	L	L
Extended detention basins*	M	L
Sand filters*	M	M
Infiltration trenches*	M	M
Infiltration basins*	M	M
Porous pavements	H	M
Constructed wetlands	H	L

The noted potential (optimum) pollutant trapping efficiency uses the following notation (also used for Table 3.1):

H high efficiency (75–100 % removal)

M moderate efficiency (50–75% removal)

L low efficiency (10–50% removal)

N negligible (0–10% removal)

The subsequent pages contain:

- additional information on the measure
- key design factors
- functional design considerations
- inspection/monitoring considerations
- maintenance considerations
- references for further information.

The suggested minimum catchment areas noted in these sections are a general guide. The actual minimum areas will be site-specific, depending on factors such as rainfall distribution, evaporation, catchment land use, soil characteristics and the design of the STM.

The selection of water quality STMs that reduce runoff volumes by infiltration and provide storage to reduce downstream peak discharges can help reduce the impacts of changes in the hydrological regime due to increasing the impervious fraction of the catchment, hence increasing the effectiveness of downstream STMs.

4 PRIMARY STORMWATER TREATMENT MEASURES

A number of primary stormwater treatment measures have been developed, including proprietary products (which are not included in this document). These types of STMs, (which are sometimes generically classified as gross pollutant traps) include:

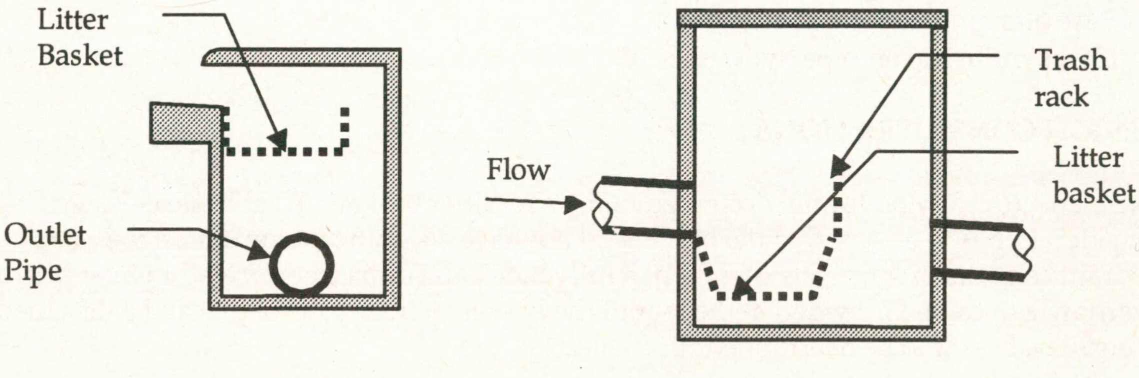
- litter basket—a basket installed within an inlet pit to collect rubbish directly entering the stormwater system from road surfaces
- litter (control) pit—a basket is located in a stormwater pit, where it collects litter from the upstream piped drainage system
- litter (trash) rack—a vertical rack installed across a stormwater channel (or at the downstream end of a sediment trap), generally with vertical bars
- sediment trap (forebay)—structure placed within the stormwater system or upstream of other STMs to trap coarse sediment, being either a formal 'tank' or a less formal pond
- gross pollutant trap (GPT)—sediment trap with a litter rack, usually located at the downstream end of the trap
- litter boom—floating device installed in channels and waterways to collect floating litter and oil
- catch basin—drainage pit with depressed bases to collect sediment. Conventional catch basins might have limited applicability in Australia
- oil/grit separators (water quality inlets)—generally comprise three underground retention chambers designed to remove coarse sediment and hydrocarbons. Conventional oil/grit separators might have limited applicability in Australia.

There has been limited comprehensive performance monitoring of primary STMs, and consequently most of the design information is based on theoretical considerations.

The estimation of coarse sediment and litter loads from catchments is difficult, largely due to the limited comprehensive monitoring of catchment yields of these pollutants. Therefore the primary STM could be designed to meet a nominated treatment objective, discussed further in *Managing Urban Stormwater: Council Handbook.*

Designs of primary STMs are evolving relatively rapidly and this innovation is encouraged.

4.1 Litter Baskets and Pits



Litter basket

Litter pit

Source: after NCTCOG (1993) and Cooper (1992)

DESCRIPTION

A wire or plastic 'basket' installed in a stormwater pit to collect litter from a paved surface (litter basket) or within a piped stormwater system (litter pit).

SELECTION CRITERIA/ADVANTAGES

- Can be retrofitted in existing areas.
- Applicable in areas with high litter loads (e.g. shopping centres) or deciduous street trees.
- Reduce downstream maintenance requirements
- Can be used to pre-treat stormwater for other STMs (e.g. constructed wetlands).
- Installed underground to minimise visual impacts.
- Litter baskets are generally applicable for small catchments (<1–2 ha); litter pits can be used for larger catchments (,150 ha)

POLLUTANT TRAPPING EFFICIENCY

Litter	M-17	Sediment	L	Nutrients	N
Oxygen demanding material	M	Oil and grease	N	Bacteria	N

LIMITATIONS/DISADVANTAGES

- Potential for litter pits to aggravate upstream flooding if blocked by litter and vegetation. (loss of pit inlet capacity)
- Potential for litter baskets to reduce pit inlet capacity if located close to inlet
- Hydraulic head loss occurs, particularly for litter pits.
- Potential loss of pit inlet capacity due to litter basket, particularly on steeper slopes.
- Possible odour problems.
- Previously caught material can be re-mobilised if overtopping occurs.

KEY PERFORMANCE FACTORS

- Provision for overtopping.
- Retention of trapped litter.
- Low to moderate approach velocities for inlet pits.
- Ease of maintenance.
- Check influence on pipe hydraulics.

DESIGN CONSIDERATIONS

There are currently no formal design guidelines for litter baskets. The primary design consideration is to ensure that the baskets do not have a significant impact on the hydraulics of the pit or pipe system when fully blocked. For baskets located in inlet pits, an overflow gap can be provided at the rear of the basket. A litter control pit can be designed to enable a basket to be overtopped (Brownlee 1995).

INSPECTION/MONITORING

Performance monitoring is unlikely to be appropriate for litter baskets. The primary monitoring activity is likely to be related to maintenance. This can involve assessing the frequency of maintenance required to minimise maintenance costs without reducing trapping efficiency.

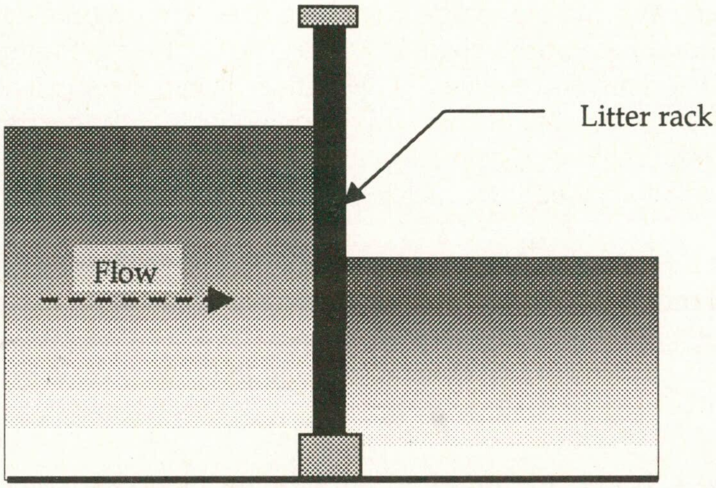
MAINTENANCE

Litter baskets require regular maintenance, particularly after major storm events and in autumn in areas with deciduous trees. Litter baskets in pits can be maintained by a vacuum truck, while the baskets in a litter control pit can be removed by a small crane mounted on the back of a truck. Sediment might also need to be removed from the control pit.

REFERENCES FOR FURTHER INFORMATION

Cooper (1992), Brownlee (1995), NCTCOG (1993), Allison et al (1997).

4.2 Litter Racks



DESCRIPTION

Litter racks (or trash racks) are a series of metal bars located across a channel or pipe to trap litter and debris.

SELECTION CRITERIA/ADVANTAGES

- Litter racks can be used to trap litter upstream of other stormwater treatment measures (possibly to prevent blocking at basin outlets) or waterways.
- They might be appropriate for retrofitting into existing areas.
- Reduced downstream maintenance requirements.
- They are generally applicable for catchments between 8 and 20 ha.

POLLUTANT TRAPPING EFFICIENCY

Litter	L	Sediment	L	Nutrients	N
Oxygen demanding material	L	Oil and grease	N	Bacteria	N

LIMITATIONS/DISADVANTAGES

- Racks have a tendancy to be blocked by debris
- Potential to aggravate upstream flooding when.
- Sediment is generally trapped upstream of the rack.
- Collected litter can move upstream along a tidal channel due to tidal influence.
- Appearance of the rack and trapped litter can be obtrusive.
- Potential odours and health risk to workers when handling litter.
- Possible safety risk when installed in channels.
- Some litter will break down after approximately 2 weeks, releasing pollutants
- Previously caught material can be re-mobilised when overtopping occurs.
- Difficult to clean and maintain

ADDITIONAL INFORMATION

Litter racks are commonly used at the downstream end of gross pollutant traps but are increasingly being used as stand-alone treatment measures. The original design for litter racks involved a vertical rack across a channel with vertical bars (e.g. Phillips 1992, Willing and Partners 1992a). Beecham and Sablatnig (1994) investigated the efficiency of 24 litter racks, and identified six arrangements that had the potential to improve trapping rates and reduce maintenance requirements, given particular site conditions. Freeman (1995) proposed a stepped and staggered litter rack to increase the trapping efficiency of the rack.

A major limitation of litter racks is blockage by debris. The water pressure against the rack and the roughness of the bars commonly stops the movement of debris up or along the bars. Although a number of self-cleaning rack schemes have been proposed (e.g. Beecham and Sablatnig 1994), these factors have generally precluded self-cleansing in practice (Sim 1997).

Litter racks can be located either on-line or off-line. On-line litter racks are placed within the existing channel or drainage system. This can apply in established urban areas where insufficient land is available to locate racks off-line. Off-line arrangements consist of a flow diversion mechanism, whereby low and medium flows are directed into the litter rack and high flows bypass the structure. This enables all litter from the majority of flows to be retained, whereas on-line structures can be overtopped during high flows and a significant proportion of the pollution collected since they were last cleaned can be lost (Freeman 1995).

Litter racks can also be installed at the headwalls of piped stormwater drainage systems. This is illustrated schematically in Figure 4.1. Alternatively, they can be located at the upstream end of culvert headwalls.

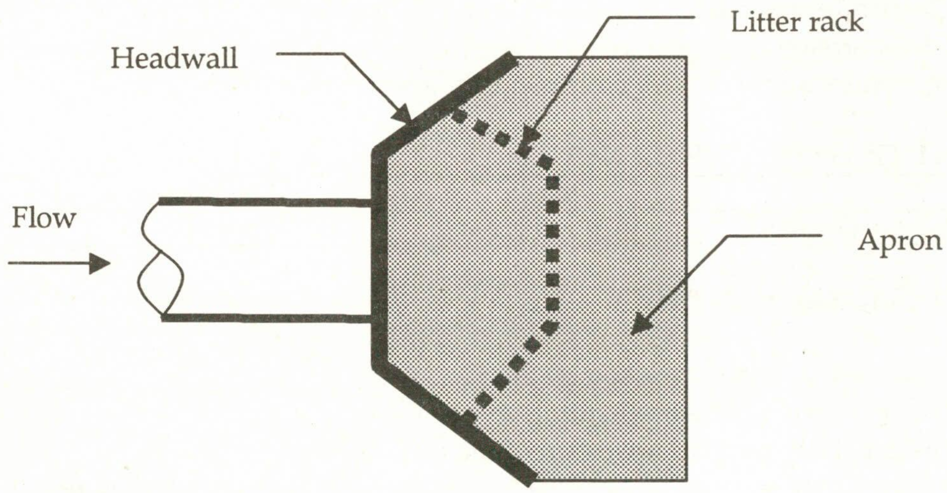


Figure 4.1 – Litter rack at a pipe headwall.

Accumulation of coarse sediment commonly occurs upstream of litter racks. This appears to be due to the reduction in flow velocity upstream of the trap, the filtering effect of debris on the trap and the barrier formed by the small concrete kerb on which the rack is commonly installed.

As litter racks commonly overtop during storm events, the rack acts as a weir, with overflows directed towards the downstream base of the rack. This can result in scouring at the base of the rack in watercourses that are not adequately protected.

The design of litter racks is evolving to address the identified problems, and monitoring of these techniques is encouraged (e.g. Sim et al 1992) to enable firmer recommendations on rack design to be made.

KEY PERFORMANCE FACTORS

- Retention of trapped litter.
- Downstream scour protection.
- Provision for maintenance.
- Check influence on channel/waterway hydraulics.

DESIGN CONSIDERATIONS

Rack arrangements

The original design of the litter rack was a vertical rack constructed perpendicular to the direction of flow in the channel (ACT Government 1994). A range of alternative layouts has been proposed, including angling the screen across the channel and a staggered arrangement (Freeman 1995). This is illustrated in Figure 4.2.

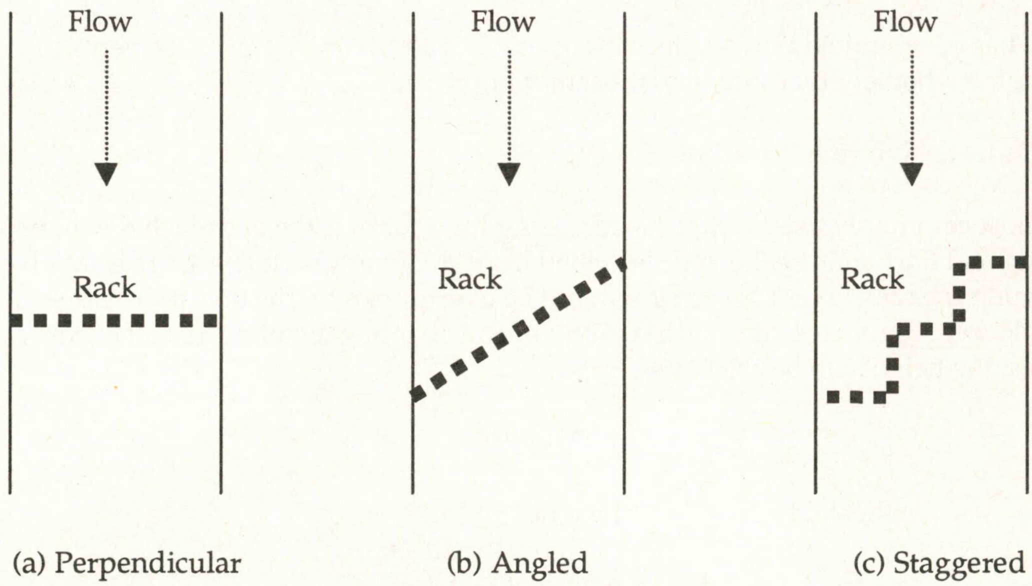


Figure 4.2 - Litter rack layouts

A number of alternative designs to vertical racks have been proposed, with the goal of retaining trapped litter when the rack is overtopped. Freeman (1995) proposed a staggered rack, both in plan and section, with the lowest section of the rack located at the upstream end. This is intended to overtop first, with the higher, downstream sections retaining litter. Another possible arrangement includes a horizontal lip on top of a vertical rack, retaining litter beneath the lip. A horizontal rack located over a drop structure has also been proposed, whereby flow drops through the rack and a lip at the end of the trap retains debris. A further arrangement involves a flexible basket located on a ledge with an upstream opening. The basket can be removed during maintenance. These are indicated schematically in Figure 4.3.

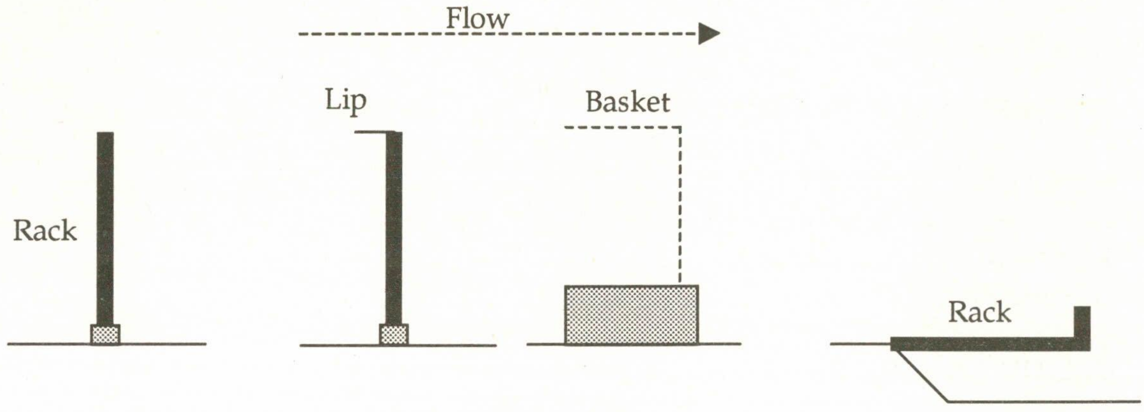


Figure 4.3 – Litter rack sections

The litter rack should be designed so that flows do not escape the confines of a channel during overtopping of the litter rack. A bypass can also be provided for high flows (to avoid overtopping). If the side walls of the channel are lower than the top of the rack, flows will bypass part of the rack and potentially scour the sides of the channel adjacent to the rack.

There has been limited performance monitoring of alternate rack arrangements to determine whether efficiency is significantly improved

Hydraulic design considerations

The most commonly used technique for sizing litter racks is the approach developed by Willing and Partners (1992a) and described in ACT Government (1994). This involves designing the rack height so that it will not be overtopped by the one-year ARI event when 50% blocked. For a rack constructed with vertical 10 mm galvanised flat steel bars at 60 mm centres, the height can be calculated by:

$$H = 1.22\,(Q/L)^{2/3}$$

where H = rack height (m)
Q = design flow (m^3/s)
L_r = length of rack (m)

This height is based on a free outfall and needs to be adjusted if submergence effects from downstream controls (e.g. culvert crossing) are present. If an alternative rack geometry is used, Willing and Partners (1992a) give details of the derivation of this sizing technique, which could be modified to suit the desired rack geometry. Where a rack is easily accessible, ACT Government (1994) recommends a minimum rack height of 1.2 m.

Metcalf and Eddy (1991) provide an equation for calculating the head loss through an unblocked rack.

Due to the tendency of litter racks to block, it is important to conduct a hydraulic analysis (backwater profile) of the waterway assuming complete blockage of the rack (i.e. acting as a weir), to assess the impact on water levels of the designated flood (e.g. 100-year ARI). This analysis might result in a reduction in the rack height to avoid exacerbating existing flooding conditions.

Structural design considerations

The litter rack should be designed to be of adequate strength to withstand the flow from a major storm (e.g. 100-year ARI event) when fully blocked. Alternatively, the rack could be designed to 'fail' in a controlled manner during a major storm, for subsequent re-installation. The rack can also be able to withstand impact loading by both debris and accidents during cleaning.

An apron might be required downstream of the rack to prevent scouring during overtopping. If the rack is not located in a concrete-lined channel, scour protection might also be required at the side of the rack to minimise scouring during overtopping.

Maintenance considerations

Vehicular access to both sides of the structure is generally appropriate for maintenance. This might be required for hand cleaning, mechanical raking or high suction vacuuming of the rack, and collection of material. If this is not practical or appropriate, safe access for maintenance staff should be provided. A hard base can also be provided upstream and downstream of the rack to facilitate maintenance.

INSPECTION/MONITORING

Inspections and monitoring activities that might be undertaken include the following:
- Monitor the removal efficiency of the rack for design litter size, possibly by litter located downstream of the trap (ie litter not retained by the rack)
- Inspect the structural integrity of the rack.
- Look for evidence of overflows and blockage.

MAINTENANCE

The rack and surrounds should inspected after significant rainfall events and on a regular basis. Debris could be removed when identified by inspection, or on a programmed basis.

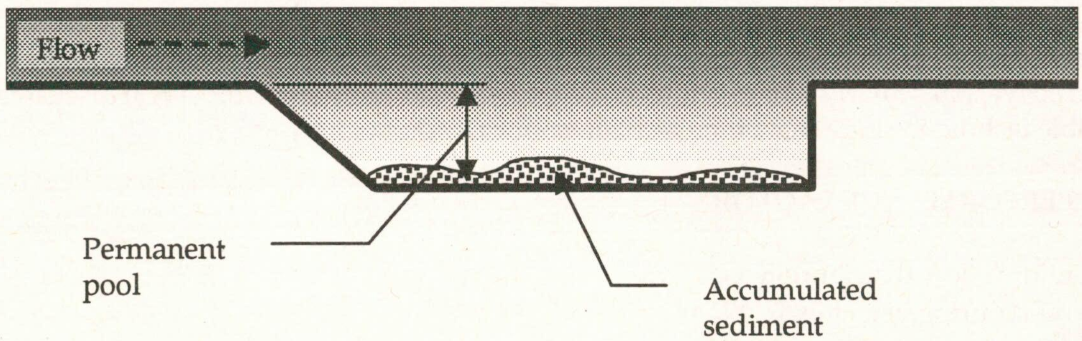
Cleaning methods might depend on the structure's size and the amount of sediment trapped behind the rack. Possible cleaning methods include:
- manually, using a rake, truck
- vacuum/eductor truck
- bobcat or front-end loader
- a combination of manual and mechanical techniques.

Appropriate disposal of the debris should be arranged before starting maintenance.

REFERENCES FOR FURTHER INFORMATION

ACT Government (1994), Phillips (1992), Willing and Partners (1992a), Sim and Webster (1992), Beecham and Sablatnig (1994), Allison et al (1997)

4.3 Sediment Traps



DESCRIPTION

Sediment traps (sometimes known as sediment basins or sediment forebays) are designed to trap coarse sediment and can take the form of a formal 'tank' or a less formal pond.

SELECTION CRITERIA/ADVANTAGES

- Trap coarse sediments upstream of a treatment measure such as a wet basin or constructed wetland.
- Reduce coarse sediment loads to stormwater systems or receiving waters.
- Can be installed underground.
- Generally applicable for catchments greater than 5 ha.

POLLUTANT TRAPPING EFFICIENCY

Litter	N	Sediment	H	Nutrients	N
Oxygen demanding material	L	Oil and grease	N	Bacteria	N

LIMITATIONS/DISADVANTAGES

- Limited removal of fine sediment or soluble pollutants.
- Above-ground sediment traps (particularly in tank form) can be visually unattractive.
- Trapping of excessive sediment can result in downstream channel erosion.
- Pollutants can be re-mobilised from sediments.
- Potential for mosquito breeding.

ADDITIONAL INFORMATION

A sediment trap generally takes two forms, namely a tank or pond. The former is a formal 'box' or 'tank' structure; smaller traps can be installed underground. They are similar to gross pollutant traps (GPTs), but without a downstream litter rack. The pond-type sediment traps (sometimes known as sediment forebays) usually take a less formal shape and can be located upstream of a constructed wetland.

Tank-type sediment traps, incorporated in GPTs, have often been regarded as visually unattractive, particularly where reinforced concrete walls are used. DoP (1993) discusses possible techniques for enhancing their appearance.

KEY PERFORMANCE FACTORS

- Uniform flow distribution.
- Non-scouring velocities.
- Sufficient length for particulate settling.

DESIGN CONSIDERATIONS

Trap surface area

There are two components that need to be sized for a sediment basin, namely the surface area and depth, in addition to length: width ratios.

The surface area is commonly sized on the basis of settling theory. There are four common classifications of settling behaviour:
- Class I—unhindered settling of discrete particles
- Class II—settling of flocculent particles
- Class III—hindered settling and zone settling
- Class IV—compression settling (compaction)

At the relatively low sediment concentrations that occur in stormwater, Class I settling is generally the most appropriate theoretical behaviour. Flocculation can occur in waters with high cation concentrations (e.g. saline waters), where Class II settling behaviour can be more applicable. However, given the short residence time, comparatively high turbulence and coarse design particle, this is probably not a significant aspect of sediment basin design.

The settling velocity of discrete particles under ideal settling conditions (Class I settling) is presented in Table 4.1.

In practice, ideal settling conditions rarely occur, due a range of factors including:
- sediment concentrations, where particles interfere with the settling of other particles
- sediment shape, where non-spherical particles settle at a slower rate
- sediment particle size variability. The settling of larger particles results in the formation of currents that inhibit the settling of smaller particles. Conversely, smaller particles effectively increase the fluid density, inhibiting large particle settling. Litter and other debris can also be expected to influence settling behaviour.
- density of the particles, which can vary according to geology and organic matter content
- turbulence and non-uniform flow distribution

Table 4.1 – Settling velocities under ideal conditions

Classification of Particle Size Range	Particle Diameter (μm)	Settling Velocity (mm/s)
	2000	200
Very coarse sand		
	1000	100
Coarse sand		
	500	53
Medium sand		
	250	26
Fine sand		
	125	11
Very fine sand		
	62	2.6
Coarse silt		
	31	0.66
Medium silt		
	16	0.18
Fine silt		
	8	0.04
Very fine silt		
	4	0.011
Clay		

Source: MDE (1987)

- flocculation and coagulation, which can occur during inter-event periods and increase the removal of finer particles.

As a consequence, the settling velocities that can be expected within sediment traps will be lower than those predicted by ideal settling behaviour. Barnes et al (1981) noted that the design of sediment tanks for wastewater treatment could incorporate a factor of safety, based on an assumption that the settling velocities will be 40–60% of the theoretical values. OMEE (1994), using data collected in the Nationwide Urban Runoff Program (US EPA 1983, 1986), estimated that settling velocities in stormwater were about 2% of the theoretical values.

Due to the difficulties in estimating actual settling velocities, which are expected to be site-specific, the design could be based on ideal settling characteristics. There will need to be a recognition of the lower velocities that will occur in practice, which can be expected to result in lower trapping for the design particle during the design flow. However, for lower flow rates, trapping of particles finer than the design particle will occur (e.g. Whytecross et al 1989).

Under constant flow conditions, settling theory (e.g. Barnes et al 1981) indicates that:

$$v_s = Q/A$$

where v_s = settling velocity (m/s)
Q = flow rate (m^3/s)
A = surface area of the sediment basin (m^2)

Therefore, for an ideal sedimentation basin, the smallest particle that will be retained has a settling velocity of Q/A. This ratio is also known as the overflow rate or surface loading rate. This equation can be rearranged to determine the theoretical length of the sediment basin:

$$L = (rQ/v_s)^{0.5}$$

where L = length of the basin (m)
r = length/width ratio of the basin

Length:width ratios commonly exceed 2:1 to 3:1 for sediment traps (OMEE 1994; Willing and Partners 1992b) to minimise short circuiting. This sizing technique assumes that the flow rate remains constant during the settling period, which will not occur for a sediment basin located in a stormwater system subject to dynamic flow conditions.

An alternative design technique has been proposed by Wong (in press).

Overall, there are limitations to the design techniques currently available for sediment basin sizing, particularly due to the non-ideal settling characteristics and dynamic flow conditions.

Trap depth

There are two primary considerations for determining the depth of a sediment basin, namely re-suspension of particles and maintenance frequency.

The most common technique for estimating the scouring velocity for particular particle sizes is based on channel erosion studies (Camp 1946, cited in Metcalf and Eddy 1991). Table 4.2 presents estimates of the velocities that will initiate scour for various particle sizes (derived using average values for the erosion equation constants suggested by Metcalf and Eddy (1991)).

Table 4.2 – Estimated scouring velocities

Particle Diameter (μm)	Scouring Velocity (m/s)
2000	0.72
1000	0.51
500	0.36
250	0.25
125	0.18
62	0.13
31	0.09
16	0.06

Source: after Metcalf and Eddy (1991)

This scouring velocity can be used to estimate the depth of the basin to avoid re-suspension of the design particle size during the design storm event. Some scouring is probably inevitable at the inlet to the basin, due to the jet action of the inflows, although this can be minimised with appropriate design.

Assessment of the scouring velocity could be based on the cross-sectional averaged velocity, which would be more applicable at the downstream end of the basin. The depth for this calculation could be based on the depth of water in the trap and the flow depth above this level. An estimate of the flow depth could be made assuming broad crested weir flow occurs at the downstream end of the trap. An appropriate weir flow coefficient would need to be adopted (e.g. 1.5) and any submergence effects included. Submergence effects only become significant when the downstream water depth is greater than about 90% of the upstream depth (Bradley 1978).

It is worth noting that the velocity will probably need to be higher than the scouring velocity if the particle is to be scoured from the basin, as there will need to be a vertical component of the velocity.

The depth of sediment retained within the trap will be related to the design of the trap, the inflow sediment characteristics and the frequency of maintenance. These are difficult parameters to estimate, particularly given the limited comprehensive monitoring of sediment export (particularly bed load) from urban catchments and sediment (or gross pollutant) trap performance. Further, sediment deposition within traps tends to be non-uniform.

An allowance for sediment storage of at least an additional 30–50% of the trap depth estimated by the scouring velocity technique could be provided.

Construction considerations

Sediment traps can be constructed from a range of materials, including reinforced concrete, masonry block walls, gabions and battered banks. Reinforced concrete is durable, strong and low maintenance but has a high construction cost and commonly has an unattractive appearance. Masonry brick walls have a lower construction cost than reinforced concrete but are also generally unattractive. Gabions and reno mattresses commonly have the lowest lining costs and can be the most aesthetically pleasing structures. However, litter and organic matter can become trapped in the mesh, stones can be removed from gabions by children and the public, and the structure is susceptible to damage during maintenance. Gabios could also harbour vermin. The less formal pond-type traps commonly have unlined sides. However, they are susceptible to erosion and difficult to grass due to the varying water levels, although macrophytes could be used to reduce erosion.

Maintenance considerations

Because sediment traps need to be maintained relatively frequently and are costly to maintain, considering maintenance issues during the design can be expected significantly to reduce long-term maintenance costs. The following issues can be considered during the design of a sediment trap:

- Vehicular access to the sediment trap for removal of sediment. For large traps, a ramp for vehicular access might be appropriate.
- Concrete or hard stand base to facilitate removal of debris and sediment.
- A sediment drying area could be provided for de-watering of the sediments before transport, with water from this area draining back to the trap.
- A marker can be used to indicate the level of sediment and debris accumulation for clean out (note that sediment will generally not settle evenly over the trap floor).

- A low-flow bypass could be provided to divert flow around the GPT during maintenance.
- Facilities for de-watering of the trap could be provided, particularly for above-ground traps. The floor of the trap could be graded to fall towards a sump or a corner to facilitate de-watering.

Safety considerations

- The wet component of an above-ground trap could be fenced, with any fences aligned parallel to the direction of flow to reduce their impact on flood levels.
- The slope of any side batter above the side walls of a above ground trap could be designed for safety and maintenance (e.g. slopes flatter than 4:1 to 6:1).
- Appropriate warning and information signs could be installed.

Aesthetic considerations

Enhancement of visual amenity of a trap could be made by landscaping (e.g. screen planting), mounding and selection of construction materials (e.g. rock walls above the pool level). This is discussed further in DoP (1993).

MAINTENANCE

Sediment and other debris will need to be removed from the trap on a regular basis, and possibly after large storms. Above-ground traps can be drained before sediment removal. Testing of the overlying water should be undertaken before de-watering to determine whether pollutant concentrations are sufficiently low for the water to be discharged to the downstream waterway. It might be appropriate for these waters to be discharged to an adjacent sewer, subject to approval from the operating authority. If this is not feasible, it might be appropriate to use a tanker to transport the water off site for approved disposal.

An eductor (vacuum) truck could be used to remove sediment and debris from underground traps.

INSPECTION/MONITORING

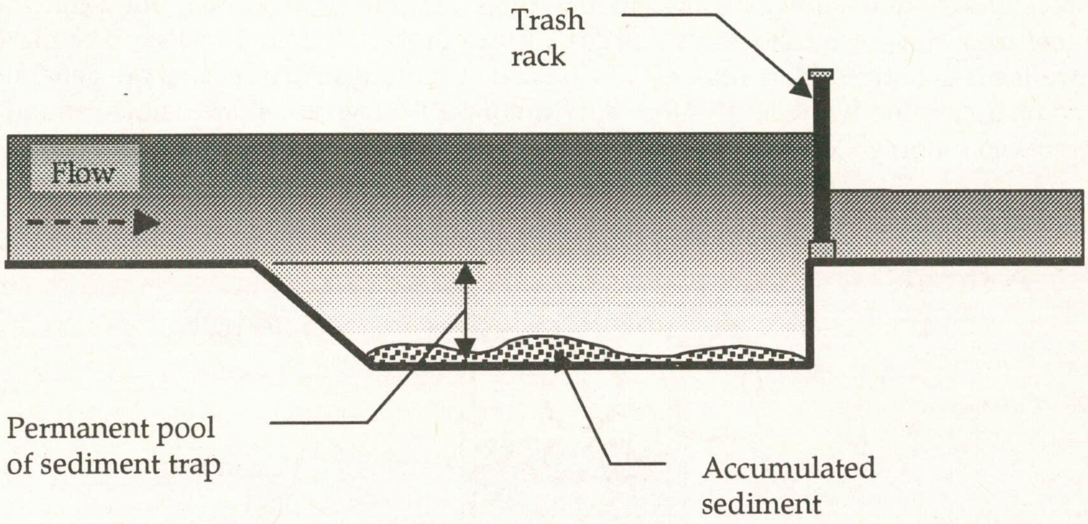
Regular inspections, and extra inspections following a large storm event, might be appropriate for:

- structural integrity
- upstream and downstream erosion
- sediment accumulation
- mosquito breeding
- odour.

REFERENCES FOR FURTHER INFORMATION

ACT Government (1994), Freeman (1995), Willing and Partners (1989), Willing and Partners (1992a), Willing and Partners (1992b), Schueler (1987), Schueler et al (1992), OMEE (1994), ARC (1992).

4.4 Gross Pollutant Traps



DESCRIPTION

A gross pollutant trap (GPT) is a sediment trap with a litter (or trash) rack, usually located at the downstream end.

SELECTION CRITERIA/ADVANTAGES

- GPTs trap coarse sediments before they enter a wetland, pond, or other stormwater treatment device, thereby preserving capacity and pond/wetland shape.
- They concentrate litter at a single location for ready removal.
- They may be appropriate for retrofitting into existing urban areas.
- Small traps can be located underground, minimising visual impacts.
- They are generally suitable for catchments greater than approximately 6–8 ha.

POLLUTANT TRAPPING EFFICIENCY

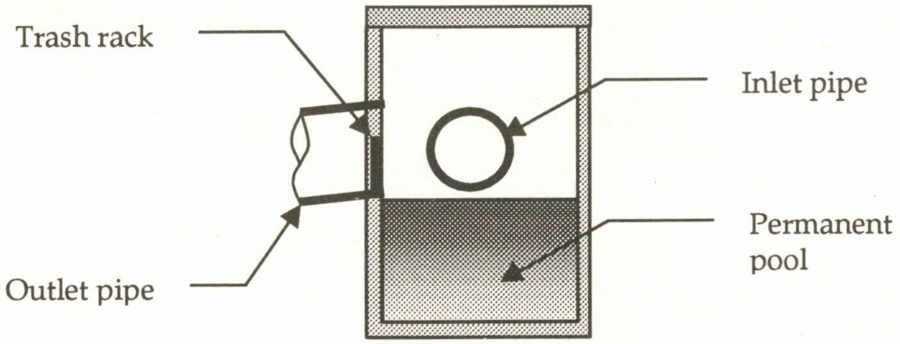
Litter	L	Sediment	M–H	Nutrients	L
Oxygen demanding material	L	Oil and grease	N	Bacteria	L

LIMITATIONS/DISADVANTAGES

- Litter rack has a tendency for blockage
- Potential to aggravate upstream flooding if the litter rack becomes blocked by debris.
- The appearance of the trap and litter can be obtrusive.
- Potential odours and health risk to workers when handling litter.
- Possible safety risk when installed in channels
- Previously caught material can be re-mobilised when overtopping occurs.
- Litter can move upstream if installed in a tidal channel.
- Difficult and expensive to clean
- Potential break-down of material in trap, possibly creating odour problems.

ADDITIONAL INFORMATION

GPTs are designed to retain coarse sediment and litter. GPTs typically consist of a metal litter rack located at the downstream end of a single cell above- or below-ground concrete sediment trap. However, innovative designs are encouraged and litter racks can be placed in any effective configuration. Major GPTs are above-ground structures that are generally located on major floodways and waterways. Minor GPTs (Figure 4.4) are below-ground structures commonly located within or at the outlet of piped drainage systems.



Source: after ACT Government (1994)

Figure 4.4 – Minor gross pollutant trap

KEY PERFORMANCE FACTORS

- Uniform flow distribution.
- Non-scouring velocities.
- Sufficient length for particulate settling.
- Retention of trapped litter.
- Downstream scour protection.
- Provision for maintenance.
- Check influence on channel/waterway hydraulics.

DESIGN CONSIDERATIONS

The design of GPTs could be based on the design techniques for sediment traps (section 4.3) and litter racks (section 4.2).

MAINTENANCE

The maintenance strategies for sediment traps and litter racks would apply to gross pollutant traps. However, litter racks might have to be cleaned more frequently than sediment traps.

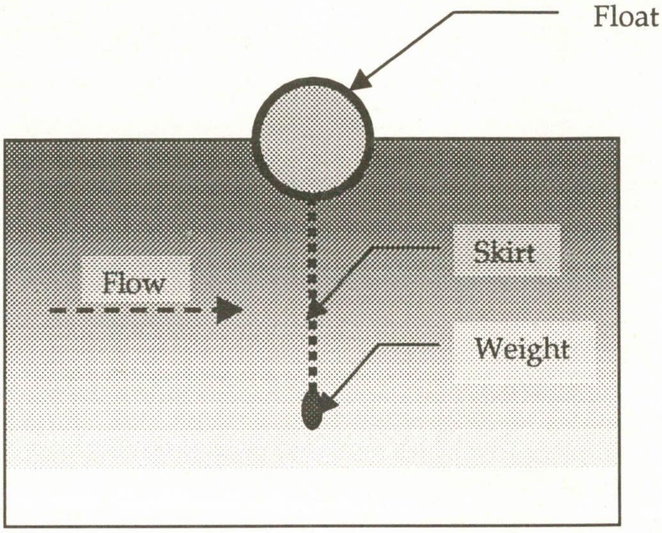
INSPECTION/MONITORING

The inspection and monitoring strategies for sediment traps and litter racks would apply to gross pollutant traps.

REFERENCES FOR FURTHER INFORMATION

ACT Government (1994), Willing and Partners (1992), Freeman (1995), DoP (1993).

4.5 Litter Booms



Source: after Freeman (1995)

DESCRIPTION

Litter booms are floating booms with mesh skirts placed in channels or creeks to collect floating litter and debris.

SELECTION CRITERIA/ADVANTAGES

- Used to remove floating litter.
- Enhance aesthetic appeal and recreational potential of downstream waterways.
- Mobile and can be appropriate for retrofitting into existing areas.
- Collect litters at a single location.
- No hydraulic head loss.
- Boom can rise and fall with changing water level
-

POLLUTANT TRAPPING EFFICIENCY

Litter	L	Sediment	N	Nutrients	N
Oxygen demanding material	N	Oil and grease	L	Bacteria	N

LIMITATIONS/DISADVANTAGES

- Only traps floating litter and debris (may be a small proportion of the total load)
- Floating and neutrally buoyant litter can be swept under the skirt during high flows.
- Impacts from large objects such as branches or boats can reduce boom effectiveness.
- Litter can be blown over the boom's collar in high winds.
- Maintenance can be difficult as most booms must be cleaned by boat.
- Potential for vandalism.
- Possibility of sinking due to marine growth.
- Collected litter can move upstream along a tidal channel due to tidal flows.
- Low visual amenity

ADDITIONAL INFORMATION

The booms are typically installed across a waterway (pond, channel or creek) to collect floating and partly submerged litter and debris. They are composed of a series of connected floating elements that can incorporate a weighted skirt submerged beneath the element. These elements are attached to the bank of the waterway.

Litter booms collect only floating litter. Waterlogged and neutrally buoyant material, such as plastic bags, can be dragged under the boom by the flow velocity. Their success to date has been mixed, partly because of the use of oil booms that do not include skirts.

KEY PERFORMANCE FACTORS

- Low flow velocities.
- Installation in deep water.
- Restraint/attachment devices that allow the boom to move vertically.
- Inclusion of a weighted skirt.

DESIGN CONSIDERATIONS

Horton et al (1995) investigated a range of boom types for litter collection. It was found that the optimum performance was achieved for a boom with a deep skirt (approximately 0.5 m or greater) with an open mesh. To avoid excessive velocities beneath the skirt, Freeman (1995) recommend that they be located in deep water where the reduction in hydraulic cross section is minimal. To minimise loss of litter, there should be no gaps at the edges of the boom.

Traditionally, litter boom installations have the boom attached at points on opposite sides of the channel with sufficient slack to allow the boom to form a semi-circle under normal flow conditions. This shape results in the collected litter accumulating at the centre of the boom, which is also the centre of the channel and the region of highest velocity. High flow velocities can drag collected litter under the boom and/or allow the boom to twist and become less effective at trapping and retaining litter and debris.

Boom performance can be improved by angling the boom across the channel to allow the collected litter to accumulate on one side of the channel, away from the high velocity area. Horton et al (1995) and Freeman (1995) recommended that the boom be angled across the channel (approximately 45°) with minimum slack (Figure 4.5). In addition, Freeman (1995) suggested that the litter could be collected in a mesh container that will retain litter during high flows, and attached to the side of the channel within easy reach of the bank for cleaning.

Booms can be installed in larger navigable waterways in a staggered chevron arrangement from opposite banks. In this way, the entire width of the waterway can be treated but still enable boat passage.

The boom and its restraints need to be designed for the hydraulic load on the boom. These loads can be considerable and can break the boom or damage the restraining device. Further details are contained in Horton et al (1995).

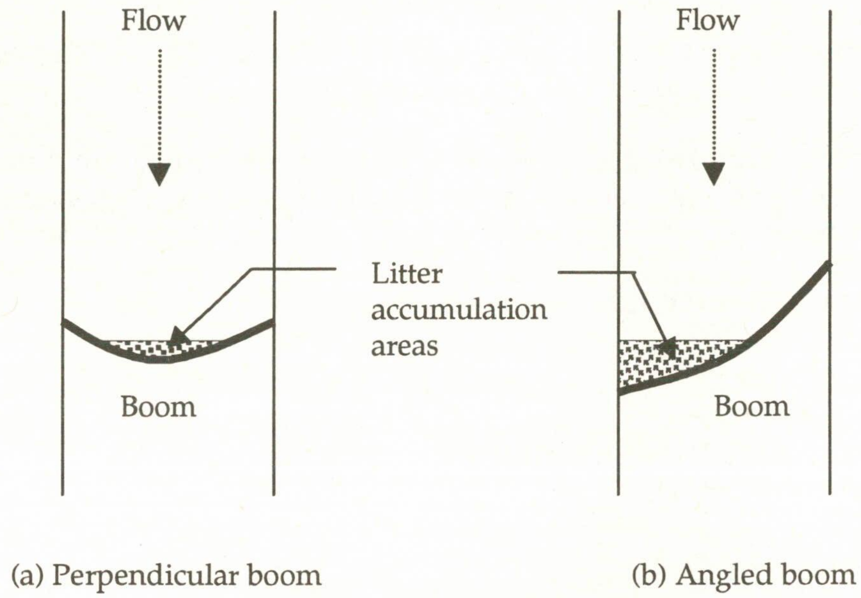


Figure 4.5 - Boom layouts

Maintenance considerations

Maintenance considerations that should be assessed during the design include:

- Vehicular access to the boom location for removal of collected material.
- Boat/pontoon access to boom.
- Ability to retain captured litter off-line.

INSPECTION/MONITORING

Inspections of the boom can be made on regular basis and after large storm events for:

- Accumulation of litter behind the boom.
- Structural integrity of the boom, skirt and restraints.

MAINTENANCE

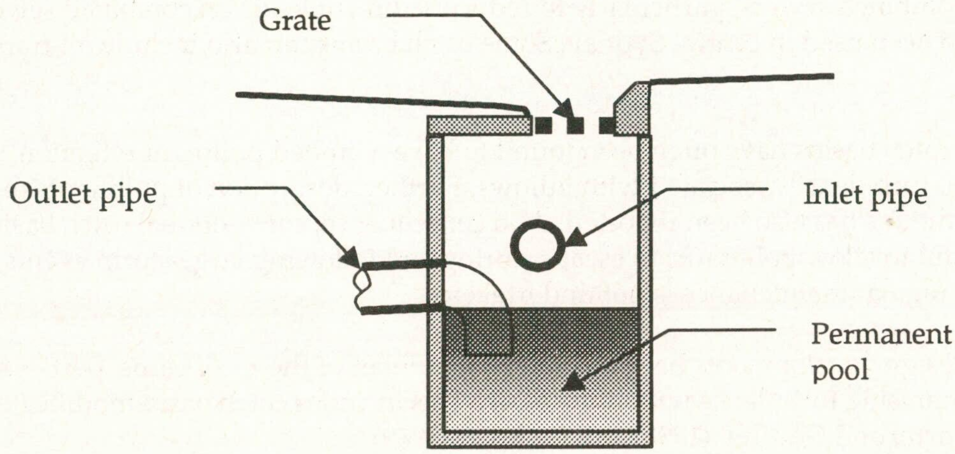
The following maintenance activities might be required:

- Regular removal of litter.
- Repair of any breaks in the boom.
- Removal of marine growth from the skirt.

REFERENCES FOR FURTHER INFORMATION

Horton et al (1995), Freeman (1995).

4.6 Catch Basins



(Source: after Grottker 1989)

DESCRIPTION

A catch basin is a stormwater pits with a depressed base that accumulates sediment.

SELECTION CRITERIA/ADVANTAGES

- Can be used upstream of other stormwater treatment measures to enhance performance.
- Can be appropriate for retrofitting into existing areas, particularly on roads with high traffic volumes.
- Installed below ground and therefore unobtrusive.
- Generally apply to small catchments (less than 1–2 ha)

POLLUTANT TRAPPING EFFICIENCY

Litter	L	Sediment	L–M	Nutrients	N
Oxygen demanding material	L	Oil and grease	L	Bacteria	N

LIMITATIONS/DISADVANTAGES

- Potential re-suspension of sediments (depending on design).
- Potential release of nutrients and heavy metals from sediments.
- Need regular maintenance.

COST CONSIDERATIONS

Low capital cost, moderate to high maintenance costs.

ADDITIONAL INFORMATION

Catch basins have been used widely in Europe and North America for both stormwater systems and combined sewers, particularly to reduce sediment loads on combined sewers. They have also been used in central Sydney. Some catch basins can also include oil-trapping features.

Conventional catch basins have often been found to have a limited pollutant retention capacity due to turbulence associated with inflows. Further, desorption of pollutants under anaerobic conditions has also been reported. As a consequence, conventional catch basins have been found to allow pollutants to escape during and following large storm events, particularly if regular maintenance is not undertaken.

More recent design developments have addressed a number of these concerns, particularly relating to minimising turbulence within the basin. This includes catch basin modifications described by Jarret and Godfrey (1995) and Grottker (1989).

KEY PERFORMANCE FACTORS

- Minimise turbulence.
- Regular maintenance.

DESIGN CONSIDERATIONS

There are no formal guidelines for catch basin sizing, although OMEE (1994) provides the following guidelines for a proprietary system:
- Maximum catchment area of 1 ha.
- A wet pool volume of 15 m^3 per impervious hectare.

MAINTENANCE

Regular maintenance of catch basins is necessary to maximise their pollutant retention, with an eductor truck commonly used for this purpose.

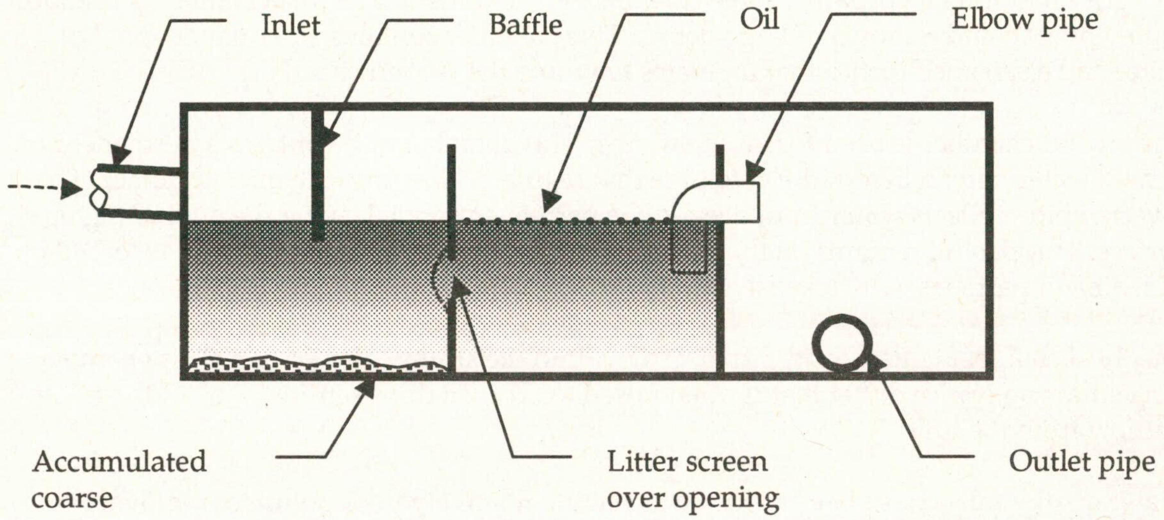
INSPECTION/MONITORING

Catch basins can be monitored to assess their pollutant retention efficiency, particularly to determine whether they act as a 'source' of pollution rather than as a 'sink'.

REFERENCES FOR FURTHER INFORMATION

Jarrett and Godfrey (1995), Grottker (1989), Evernden (1995), OMEE (1994).

4.7 Oil/Grit Separators



(Source: after Schueler 1987)

DESCRIPTION

Oil/grit separators, also known as water quality inlets, generally consist of three underground retention chambers designed to remove coarse sediment and hydrocarbons.

SELECTION CRITERIA/ADVANTAGES

- Appropriate for treating stormwater from areas expected to have significant vehicular pollution (e.g. parking lots), particularly hydrocarbons. They can also trap litter.
- Can also be used for treating stormwater from areas storing or handling petroleum products (e.g. service stations and petroleum depots).
- Can be appropriate for retrofitting into existing areas.
- Installed below ground and therefore unobtrusive.
- Applicable for small catchments (generally less than 2,500 m^2).

POLLUTANT TRAPPING EFFICIENCY

Litter	L–M	Sediment	M	Nutrients	L
Oxygen demanding material	L	Oil and grease	M	Bacteria	L

LIMITATIONS/DISADVANTAGES

- Limited removal of fine sediment or soluble pollutants.
- When turbulent stormwater enters the chambers, this action may re-suspend particulates or entrain floating oil. A high flow bypass is required.
- Trapped debris is likely to have high concentrations of pollutants, possibly toxicants.
- Need to be regularly cleaned to achieve design objectives.
- Potential safety hazard to maintenance personnel.
- Potential release of nutrients and heavy metals from sediments.

ADDITIONAL INFORMATION

Oil/grit separators incorporate three chambers for treatment. The first chamber is used for sedimentation and removal of large debris. This chamber contains a permanent pool of water and a screened orifice that regulates flow into the second chamber.

The second chamber is used for oil separating. This chamber also contains a permanent pool of water and an inverted elbow pipe that regulates flow into the third chamber. The inverted pipe collects water from deep in the permanent pool, leaving the oil floating on the surface. Trapped oil remains on the surface of the water until it is removed or absorbed on to sediment particles, which subsequently settle.

The third chamber collects and disperses flow into the stormwater system. This chamber can contain an orifice outlet that is often raised to create a third settling pool and regulate outflow from the unit.

Water quality inlets have been reported as having relatively poor pollutant removal performance, which has been attributed to infrequent maintenance and the passage of high flows (Galli 1992). They have often been found to be cost-ineffective due to their high maintenance requirements (OMEE 1994) and have not been widely used in Australia.

DESIGN CONSIDERATIONS

OMEE (1994) provides the following guidelines for the design of water quality inlets:

- The permanent pool volume should be 30 m^3 per impervious hectare with 50–70% of this volume in the first chamber.
- The separator can be designed to accept flows from the design event only, with a high flow bypass provided.
- The length:width ratio should be greater than 3:1.
- Ensure that an effective screen is used to protect the orifice allowing flow into the second chamber. It should be easily accessible and removable for cleaning.
- The inverted elbow separating the second and third chambers should extend a sufficient distance into the second chamber's pool to ensure the oil is adequately separated.
- Access needs to be made easy for inspection and cleaning. Each chamber should have its own manhole entrance with step rings to the bottom of the chamber.
- Only runoff from areas that are likely to have contaminated runoff (e.g. filling areas on a service station site) should be directed to the separator. This will reduce the size of the separator required. These areas can be bunded to prevent contamination of clean runoff. This will ensure clean runoff does not overload the system and contaminated runoff does not escape the system.
- Consideration can be given to venting, because oil in the system can be present for some time. In some situations, using vapour retention baffles might be desirable to stop vapours being drawn down stormwater pipes.

MAINTENANCE

Oil/grit separators need to be cleaned frequently (e.g. by an eductor truck) to prevent loss of oil and sediment. If any standing water is removed, it could be replaced with clean water to prevent oil escaping to the outlet weir or orifice.

INSPECTION/MONITORING

Oil/grit separators can be monitored to:

- Assess sediment and oil levels
- assess outflow oil concentrations.

REFERENCES FOR FURTHER INFORMATION

OMEE (1994), Schueler (1987), Schueler et al (1992), Horner et al (1994), Galli (1992), MDE (1991), Whelans et al (1994), CDM (1993), ARC (1993).

5 SECONDARY STORMWATER TREATMENT MEASURES

Secondary treatment measures principally remove sediment, with some retention of nutrients and bacteria also occurring. These types of treatment measures are currently not widely used in Australia. Available techniques include:

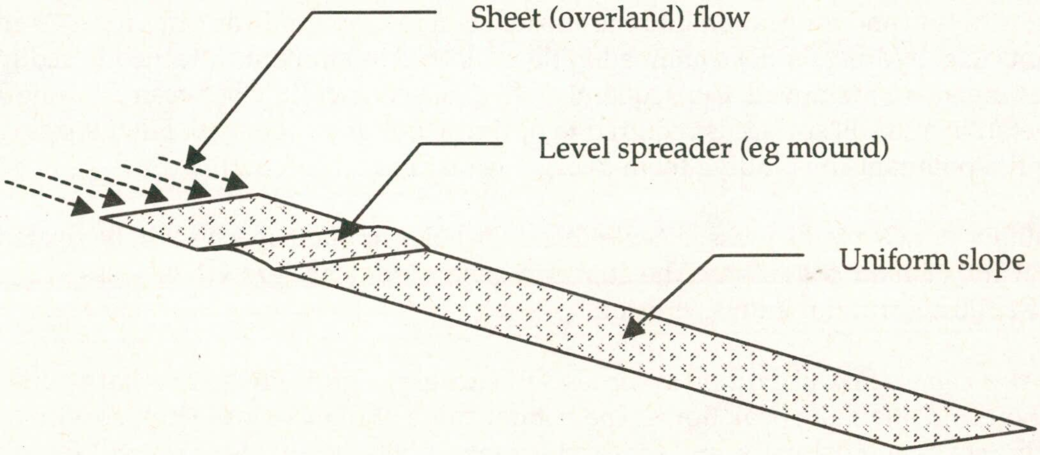
- filter strips (also known as buffer zones or buffer strips)—grassed or vegetated areas that treat overland flow, often adjacent to watercourses
- grass swales—grass-lined channels for conveying runoff from roads and other impervious surfaces
- extended detention (dry) basins—basins that store runoff for 1–2 days and drain to an essentially dry condition between storm events. These differ from conventional dry retarding basins, which generally store runoff for up to a few hours.
- sand filters—beds of sand (or other media) through which runoff is passed. The filtered runoff is then collected by an underdrain system.
- infiltration trenches—shallow, excavated trenches filled with gravel into which runoff drains to groundwater
- infiltration basins—open excavated basins designed to infiltrate runoff through their floors
- porous pavements—pavements that allow runoff to drain through a coarse graded concrete/asphalt pavement or open concrete blocks, subsequently to infiltrate to the underlying soil

Swales and filter strips are currently the most commonly used devices. Due to the high reported failure rates of infiltration basins, trenches and porous pavements, their use needs to be approached with some caution.

There has been a moderate amount of performance monitoring of secondary STMs. This monitoring has commonly indicated the range of pollutant retention and the factors likely to enhance retention, although it has generally been insufficient to derive comprehensive design techniques.

There has been limited development of proprietary secondary treatment measures to date, although some proprietary primary treatment measures have been reported as achieving similar pollutant retention to non-propritrary secondary measures.

5.1 Filter Strips



DESCRIPTION

Filter strips, also known as buffer zones or buffer strips, are grassed or vegetated areas that treat overland (sheet) flow, often adjacent to watercourses.

SELECTION CRITERIA/ADVANTAGES

- Appropriate for treating shallow overland flow.
- Can reduce runoff volumes (by infiltration) and delay runoff flow rates.
- Most effective at removing particulate matter and associated pollutants.
- Can be used to pre-treat runoff for other STMs.
- Generally apply to catchments smaller than 2 ha.

POLLUTANT TRAPPING EFFICIENCY

Litter	M	Sediment	H	Nutrients	M
Oxygen demanding material	L	Oil and grease	H	Bacteria	H

LIMITATIONS/DISADVANTAGES

- Limited removal of fine sediment or dissolved pollutants (e.g. dissolved nutrients).
- Requires considerable land areas and restrictions to vehicular access.
- Adequate sunlight is required—heavy or prolonged shading should be avoided.
- Vegetation needs to be maintained all year.
- Reduced effectiveness for concentrated flows and high flow depths.
- Generally applicable for slopes of up to 5%.
- High failure rates (largely erosion) have been reported—attributed to poor maintenance and vegetation cover, and difficulties achieving sheet flow and avoiding channelisation.

ADDITIONAL INFORMATION

Filter strips are primarily intended to remove sediment and, to a lesser extent, hydrocarbons from shallow overland flow. Pollutant removal is achieved primarily by settling, filtration and infiltration into the subsoil. Soil microorganisms can process some pollutants (e.g. hydrocarbons) contained in the runoff. Contaminants attached to sediment particles can also be removed. Consequently, adequate contact time between the runoff and the vegetation and soil surface is required to optimise pollutant removal. Filter strips can immobilise pollutants by binding them to organic matter and soil particles.

As pollutant uptake by the grass is negligible, the choice of plant species will be related to local soil and climatic conditions. The goal is to generate a dense growth of grass to maximise filtration and minimise erosion.

Filter strips can also reduce runoff volumes and encourage groundwater recharge due to infiltration, and attenuate peak flows. The habitat value of filter strips increases with vegetative cover and diversity, and they can provide a habitat corridor for wildlife. Additional benefits can include landscape, aesthetics, recreation and increased biodiversity.

Runoff from adjacent impervious areas should be evenly distributed across the upstream end of filter strips as sheet flow. This can be achieved by a flow spreader such as a shallow weir, rip-rap mattress, stilling basin or perforated pipe located across the width of the strip. A strip of turf could be placed immediately downstream of the level spreader to assist with flow spreading during the establishment period of the downstream seeded area. Filter strips should not receive flow until the vegetation is established. Alternatively, the strips could be established with turfing to reduce the time between establishment and use.

Filter strips should not receive discharges directly from stormwater pipes without an energy dissipator and flow spreader.

Natural filter strips or buffer zones can also achieve good pollutant retention (e.g. Woodfull et al (1992).

KEY PERFORMANCE FACTORS

- Adequate contact/residence time.
- Non-scouring velocities.
- Dense grass growth.
- Appropriate soil types (refer to *Managing Urban Stormwater: Soils & Construction*)

DESIGN CONSIDERATIONS

There are no comprehensive guidelines on the design of filter strips available, and a number of 'rules of thumb' have been developed (e.g. Schueler et al 1992). There have also been a limited number of studies that have monitored the performance of filter strips in urban areas.

Horner et al (1994) cite a technique developed in Seattle, USA, for sizing filter strips and grass swales. This is based on the results of studies indicating that the optimum pollutant retention occurs for a hydraulic residence time of nine minutes. Performance was found to deteriorate noticeably when residence time fell below five minutes.

Based on a nine-minute average residence time, the reported retention of pollutants is summarised in Table 5.1.

Table 5.1 - Average pollutant retention of filter strips

Pollutant	Retention (%)	Pollutant	Retention (%)
Suspended solids	83	Lead	67
Oil and grease	75	Total phosphorus	29
Iron	72	Total nitrogen	Negligible

Source: Horner et al (1994)

The following steps are involved in designing a filter strip using the technique described by Horner et al (1994):

1. Estimate the design flow for the design storm event.
2. Determine the slope of the filter strip.
3. Set the design flow depth.
4. Solve Manning's equation to determine the required width of flow. A Manning's 'n' value of 0.20 for mowed filter strips and 0.24 for natural grasses or infrequently mowed strips was suggested.
5. Calculate the flow area based on the calculated flow width and established depth.
6. Calculate the resulting velocity. If the velocity exceeds 0.3 m/s (the velocity where most grass will be knocked over), reduce the flow, increase the flow width or reduce the depth of flow
7. Using the resulting velocity, calculate the flow length to achieve a residence time in the filter strip of 9 minutes (5 minutes at an absolute minimum).

A maximum depth of flow over the filter strip of 12 mm was recommended.

The minimum length of a filter strip will generally be 6 m to maintain sheet flow. A shorter filter strip (3 m at an absolute minimum) could be used if good sheet flow is established and maintained (CDM 1993).

The performance of grass filter strips has been found to be reduced if they are located on grades exceeding 5%, particularly if slopes exceeded 15% (Schueler et al 1992). If slopes are lower than 2%, consideration could be given to installing a subsoil drainage system to ensure effective infiltration.

The slope of the filter strip should be uniform and the cross section should be level, requiring particular attention during the construction phase.

The integrity of the filter strip can be impaired if flows greater than the design event enter the strip. Velocities exceeding the design velocity can be expected to reduce the pollutant removal efficiency of the strip until the grass has recovered and can result in scouring of the strip. A bypass for high flows could be installed.

The depth to groundwater should be considered when designing a filter strip. If the water table is shallow, the grass species will need to tolerate this situation. Further, there is a possibility of pollution entering shallow groundwater due to the shallow soil depth for pollutant retention.

INSPECTION/MONITORING

Inspections of the integrity of the filter strip should be undertaken on a regular basis, particularly during the establishment period. Additional inspections could be undertaken after large storm events. The following items could be inspected:

- channelisation and erosion
- vigour and density of vegetation
- weed inundation
- integrity of level spreading device (if applicable)
- inappropriate access and wear.

MAINTENANCE

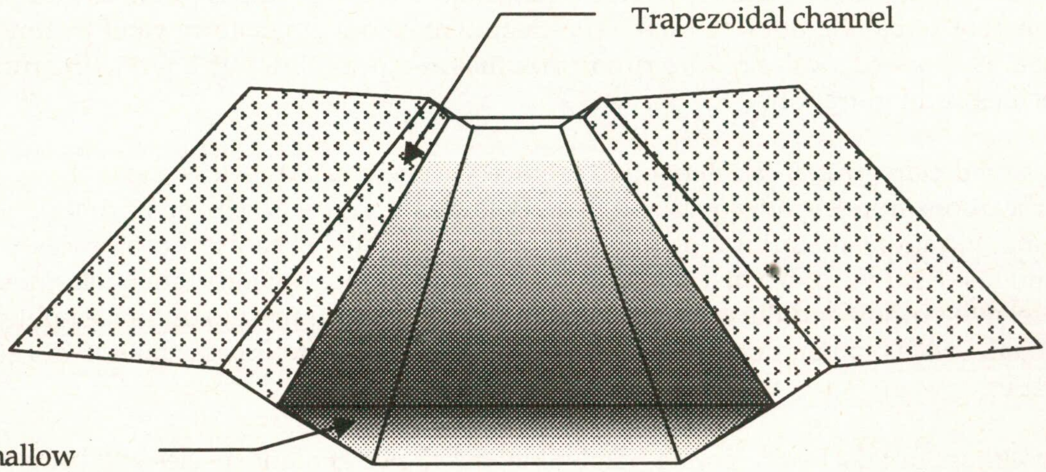
The following items of maintenance could be required:

- repair of any erosion (e.g. channelisation) across the filter strip
- watering, reseeding and careful fertilisation (if appropriate) might be required to establish and maintain a dense, vigorous growth of vegetation
- maintenance of the level spreader might be required to ensure that it is not concentrating flows (i.e. no localised low points)
- judicious access control might be required (e.g. preventing car parking).

REFERENCES FOR FURTHER INFORMATION

Horner et al (1994), Schueler (1987), Schueler et al (1992), CDM (1993), OMEE (1994), Whelans et al (1994), ARC (1992), Woodfull et al (1992).

5.2 Grass Swales



DESCRIPTION

Grass swales are grass-lined channels for conveying runoff from roads and other impervious surfaces.

SELECTION CRITERIA/ADVANTAGES

- Can reduce runoff volumes (by infiltration) and delay runoff flow rates.
- Most effective at removing particulate matter and associated pollution.
- Can be used to pre-treat runoff for other STMs.
- More aesthetically appealing than kerb and gutter.
- Generally applicable for catchments less than 2 ha and for lower density urban areas.

POLLUTANT TRAPPING EFFICIENCY

Litter	M	Sediment	H	Nutrients	M
Oxygen demanding material	L	Oil and grease	H	Bacteria	H

LIMITATIONS/DISADVANTAGES

- Limited removal of fine sediment or dissolved pollutants (e.g. dissolved nutrients).
- Require larger land areas than kerb and gutter, and limited access (e.g. car parking).
- Adequate sunlight is required, and heavy or prolonged shading should be avoided.
- Vegetation needs to be maintained all year.
- Reduced effectiveness for concentrated flows and high flow depths.
- Generally applicable for slopes of up to 5%.
- High failure rates have been reported—attributed to poor maintenance, poor vegetation cover and difficulties in achieving uniform flow and avoiding channelisation.

ADDITIONAL INFORMATION

Swales are grass-lined channels often running adjacent to a road pavement as an alternative to concrete kerb and guttering or as a pre-treatment before other stormwater treatment measures. Grassed swales reduce runoff volumes and peak flows by attenuating runoff velocities and infiltration.

Grass swales are primarily intended to remove sediment and, to a lesser extent, hydrocarbons from shallow overland flow. Pollutant removal is achieved primarily by settling, filtration and infiltration into the subsoil. Soil microorganisms can process some pollutants (e.g. hydrocarbons) contained in the runoff. Consequently, adequate contact time between the runoff and the vegetation and soil surface is required to optimise pollutant removal. Swales can immobilise pollutants by binding them to organic matter and soil particles.

As pollutant uptake by the grass is negligible, the choice of plant species will be related to local soil and climatic conditions. The goal is to generate a dense growth of grass to maximise filtration and minimise erosion. If the swale intercepts groundwater, the grass species should be tolerant of the groundwater chemistry.

Grassed swales are primarily intended for the enhancement of water quality and infiltration. This differs from grass waterways, which are designed primarily for conveyance of floodwaters. Swales can be used in lower density residential developments as an alternative to kerb and gutter. They can also be used in road medians, road verges, and areas receiving runoff from car parks, parks and recreation areas. Swales adjacent to roads might be compacted by vehicular traffic and could have reduced infiltration rates.

Runoff from adjacent impervious areas should be evenly distributed across the upstream end of the swale as sheet flow. This can be achieved by a flow spreader such as a shallow weir, rip-rap mattresses, stilling basin or perforated pipe located across the width of the swale. A strip of turf could be placed immediately downstream of the level spreader to assist with flow spreading during the establishment period of the downstream seeded area.

Swales should not receive flow until the vegetation is established. Alternatively, swales can be established with turfing or other form of stabilisation (e.g. erosion control matting) to reduce the time between establishment and use.

Swales should not receive discharges directly from stormwater pipes without an energy dissipater and flow spreader. Careful consideration is required when designing road and driveway crossings to avoid concentrating flows.

KEY PERFORMANCE FACTORS

- Adequate contact/residence time.
- Non-scouring velocities.
- Dense grass growth.
- Appropriate soil types (refer to *Managing Urban Stormwater: Soils & Construction*).

DESIGN CONSIDERATIONS

There are no comprehensive guidelines on the design of swales, and a number of 'rules of thumb' have been developed (e.g. Schueler et al 1992). There have also been a limited number of studies that have monitored the performance of swales in urban areas.

Horner et al (1994) cite a technique developed in Seattle, USA, for sizing filter strips and grass swales. This technique is described in Section 5.1 (Filter Strips).

A maximum depth of flow equal during the design storm to one-third of the grass height in infrequently mowed swales or half of the height in regularly mowed swales, to a maximum of 75 mm, was recommended. Greater flow depths would be appropriate in grassed waterways designed to convey floodwaters (e.g. 100 year ARI event).

The swales should be trapezoidal, with Horner et al (1994) recommending a minimum bottom width of 0.6 m and a maximum width of 2.5 m. If a wider base is required, the flow could be divided into more than one swale or the base could be hand-finished to obtain a completely flat bottom. The side slopes should generally not be steeper than 3:1. If steeper slopes (up to 2:1) are used, permanent stabilisation might be required (Terrene Institute 1996). Triangular cross-sections are not recommended as flow can become channelised in the bottom of the swale.

The minimum length of a swale recommended by Horner et al (1994) is 30 m to minimise flow short-circuiting. If this cannot be accommodated on the site, a large-radius curved alignment could be adopted.

The grass swales should be located on grades of 4% or less. Slopes of up to 6% can be adopted if small check dams (mounds) are located in the swale every 15 to 30 m to reduce flow velocities (Horner et al 1994). If slopes are lower than 2%, consider installing a subsoil drainage system to ensure effective drainage and infiltration.

The longitudinal grade of the swale should be uniform, or any grade changes should be gradual. The base of the swale should be level, and particular attention should be paid to these requirements during the construction phase.

The integrity of the swale can be impaired if flows greater than the design event enter the swale at a velocity sufficient to damage the grass or cause erosion. Velocities exceeding the design velocity can be expected to reduce the pollutant removal efficiency of the swale until the grass has recovered. A bypass for high flows or stormwater drain inlets could be installed to prevent large concentrated flows from eroding the swale. Alternatively, the velocity during the high-flow event can be calculated to determine whether it is less than the scouring velocity.

The depth to groundwater should be considered when designing a swale. If the water table is shallow, the grass species will need to tolerate this situation. Further, there is a possibility of pollution entering shallow groundwater, as there is a short distance for pollutant retention within the soil.

INSPECTION/MONITORING

Inspections of the integrity of the swale should be undertaken on a regular basis, particularly during the establishment period. Additional inspections could be undertaken after large storm events. The following items could be inspected:

- channelisation and erosion
- vigour and density of vegetation
- weed inundation
- integrity of the level spreading device (if applicable)
- inappropriate access and wear
- if the swale is not adequately maintained, nuisance problems such as mosquitoes, weed infestation, boggy base and erosion can arise.

MAINTENANCE

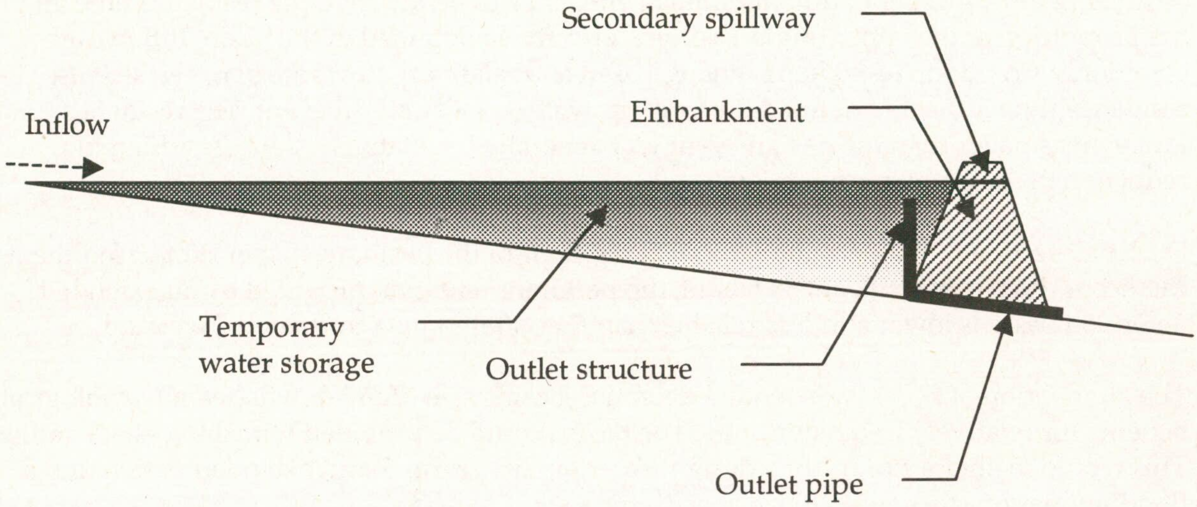
The following items of maintenance could be required:

- repair of any erosion (e.g. channelisation) across the swale
- watering, re-seeding and careful fertilisation (if appropriate) might be required to establish and maintain a dense, vigorous growth of vegetation
- maintenance of any level spreaders might be required to ensure that it is not concentrating flows (i.e. no localised low points)
- judicious access control might be required (e.g. preventing car parking)
- removal of sediment behind any check dams (if applicable)
- removal of litter and debris from the swale surface.

REFERENCES FOR FURTHER INFORMATION

Horner et al (1994), Schueler (1987), Schueler et al (1992), CDM (1993), OMEE (1994), Whelans et al (1994), ARC (1992).

5.3 Extended Detention Basins



DESCRIPTION

Extended detention basins commonly store runoff for 1–2 days and drain to an essentially dry basin between storm events.

SELECTION CRITERIA/ADVANTAGES

- Principal objective is the retention of particulates.
- Can be appropriate in areas where runoff is insufficient or too unreliable, evaporation rates are too high or soils are too permeable to sustain the use of constructed wetlands.
- Could be used where the site area precludes the use of a constructed wetland.
- Potential for multiple use if basin drains between storm events (e.g. sport field or park).
- Detention of runoff reduces the frequency of erosive flows downstream.
- Generally appropriate for catchments over 3-6 ha (depends on minimum outlet size).

POLLUTANT TRAPPING EFFICIENCY

Litter	L	Sediment	M	Nutrients	L
Oxygen demanding material	L	Oil and grease	L	Bacteria	H

LIMITATIONS/DISADVANTAGES

- Limited removal of fine sediment or dissolved pollutants (e.g. dissolved nutrients).
- Potentially lower efficiency for events smaller than the design event.
- Outlet structures are prone to clogging (if no litter removal pre-treatment is provided).
- Potential for erosion and resuspension of deposited sediment in the basin floor.
- Possible safety hazard due to intermittent nature of flooding.
- Possible maintenance problems and mosquito breeding from the frequently wetted floor.

ADDITIONAL INFORMATION

Extended detention basins achieve their pollutant removal function primarily through sedimentation, with the pollutant removal efficiency depending on the residence time and the proportion of the annual runoff volume effectively detained in the basin. Inflows are commonly stored for 24–40 hours and released at a relatively slow rate between storms. The residence time is usually defined for a design water quality storm event. The residence time for events smaller than the design event will generally be relatively short, resulting in a reduction in the pollutant retention.

Due to the absence of a permanent pool in the floor of the basin, re-suspension of sediment can occur during storm events. Overall, the pollutant retention provided by an extended detention basin is lower and less reliable than that offered by a constructed wetland.

The attenuation of flows by extended detention basins also offers downstream hydrological benefits for relatively frequent events. The basins could be provided with a two-stage outlet. This would facilitate storage of a design water quality storm for an extended period and a flood mitigation storm (e.g. 100 year ARI) for a short period.

KEY PERFORMANCE FACTORS

- Appropriate retention time for sedimentation (for design storm and other events).
- Relatively shallow depth (for sedimentation).
- Uniform flow through the basin.
- Relatively low velocities (to maximise sedimentation and minimise re-suspension).
- Design outlet to minimise risk of blockage.
- Free-draining basin floor.
- Safety considerations (including appropriate side slopes and inlet/outlet structures).

DESIGN CONSIDERATIONS

Sizing

There are currently no techniques available for predicting the pollutant retention offered by extended detention basins. Optimum reported performance occurs for a retention time of 24–40 hours, with Stahre and Urbonas (1990) quoting the retention rates summarised in Table 5.2 for a 40-hour retention.

Table 5.2 – Pollutant Retention Rates for Extended Detention Basins

Pollutant	Retention (%)	Pollutant	Retention (%)
Suspended solids	50 – 70	Total phosphorus	10 – 20
Oil and grease	50 – 70	Total nitrogen	10 – 20
Lead	75 – 90	Bacteria	50 – 90
Zinc	30 – 60	Chemical oxygen demand	20 – 40

Source: Stahre and Urbonas (1990)

A retention period of 24 hours was suggested by Schueler et al (1992), while a 40 hour period was recommended for California to settle finer clay particles (CDM 1993).

Outlet Design

The proper design of the outlet from an extended detention basin is critical to its performance. Two common problems have been reported to occur with outlets:

- the outlet is too large, resulting in only partial filling of the basin for the design storm event. This will reduce the residence time and hence pollutant retention.
- the outlet becomes blocked by litter and other debris, extending the detention time and resulting in boggy conditions on the floor of the basin. This has been found to be a common problem in the USA (Galli 1992).

Potential outlet types include:

- **weir.** The use of a weir will reduce the chance of blockage, particularly if it is protected by an upstream litter rack. Difficulties can be experienced achieving low release rates at low heads with a V-notch weir. An alternative is a proportional discharge weir.
- **perforated riser.** The outlet structure consists of vertical riser pipe perforated with holes 12–25 mm in diameter. The flow through the riser can be controlled either by an orifice plate located at the bottom of the riser structure, or by the perforations within the riser. Gravel can be placed around the riser pipe to act as a filter, or holes smaller in diameter than the orifice plate can be used.
- **reverse slope pipe.** The outlet is applicable where a small pool is located at the outlet. The reverse sloped pipe can drain to an outlet chamber located in the basin embankment. The outlet chamber could also act as an outlet for flood attenuation purposes. A gate valve could be attached to the reverse sloped pipe in the outlet chamber. This will allow the extended detention drawdown time to be modified to improve pollutant removal if the pond is found to be operating outside the design criteria.

Further details on outlets are provided in UDFCD (1992), Schueler (1987) and OMEE (1994).

A small pool (0.3–1.0 m deep) could be constructed at the outlet to prevent clogging of the orifice. ARC (1992) recommends a minimum orifice diameter of 75 mm to minimise clogging. For a perforated riser outlet, the orifice is protected by the smaller perforations in the riser and a minimum orifice size of 50 mm might be acceptable. An anti-vortex device could be used on a riser outlet.

Stage (depth)–discharge relationships can be established for these outlet structures using conventional weir flow, orifice flow and pipe hydraulic equations. The equation for the adopted outlet type can be specified in a rainfall–runoff model of the basin, which also includes a stage–storage volume relationship. The design storm event can then be routed through the storage to determine the residence time. The residence time for an extended detention basin can be defined as the time taken to empty the basin (i.e. the drawdown time).

If an orifice outlet is adopted, CDM (1993) note that care must be taken with the selection of the orifice coefficient ('c'). It was noted that:

- for thin material (thickness less than or equal to the orifice diameter): c=0.66
- for thick material (greater than the orifice diameter): c=0.80.

An approximate technique for sizing the outlet, cited in (CDM 1993), is applicable when side slopes are uniform and an orifice outlet is used. The equation is based on the drawdown time of a falling head orifice:

$$a = \frac{2A(\Delta h)^{0.5}}{3600ct(2g)^{0.5}}$$

where a = area of orifice (m^2)
A = average surface area of pond (m^2)
Δh = difference between full and empty levels (m)
c = discharge coefficient
t = draw down time (hr)
g = gravitational acceleration constant ($9.81 m/s^2$)

The sizing of the basin for a single design event does not guarantee that adequate residence time will be achieved for smaller events, which may pass through the basin in a period too short for effective treatment. CDM (1993) noted that this applied particularly for basins with a 24-hour design residence time, and considered that a 40-hour design residence time might be sufficient to provide adequate treatment for smaller storms. As small events comprise the majority of the annual runoff, additional investigation of the hydraulic performance of the basin could be undertaken. A continuous rainfall–runoff model could be established to investigate whether the residence time for small storms is adequate. Alternatively, the hydraulics of the outlet could be checked to determine if the basin takes approximately 24 hours to drain if half full.

If the residence time for small events is found to be inadequate, the design can be modified or a two-stage outlet used. This could involve a second orifice at half of the basin height, with the lower orifice designed to drain half of the basin in 24 hours and the two orifices designed to drain the entire basin in 40 hours. Alternatively, a pool could be installed at the outlet, with the outlet structure located above the bottom of the pool. This pool could dry up between events, but will store the runoff from small events.

Geometry and layout

As sedimentation is the primary pollutant removal mechanism for extended detention basins, features that can be considered during the design to optimise performance include:

- **effectiveness of the residence time.** It is important that the inflow volume is uniformly distributed within the basin volume, and that short-circuiting is minimised. This can be achieved by a length to width ratio of between 3:1 and 5:1. The inlet to the basin should also generally be located as far from the basin outlet as possible. To increase the length to width ratio or to overcome problems with the inlet being in close proximity to the outlet, berming can be included to redirect flows.
- **velocity distribution.** Sedimentation will be enhanced by low flow velocities, so strong flow jets at the inlet to the basin should be avoided. This can be minimised by installing energy dissipaters at the inlets.
- **depth.** The sedimentation process will also be enhanced by a relatively shallow depth, which reduces the distance for settling particles to fall. An average depth of 1–2 m is generally appropriate.

Features that minimise the re-suspension of deposited material include:

- locating the basin off-line, bypassing high flows above the design storm flow. Alternatively, flood storage can be provided above the water quality storage, thus reducing flow velocities through the basin.

- designing to achieve low flow velocities through the basin (e.g. less than 0.3 m/s during a design storm). Energy dissipaters should be installed at the inlet.
- incorporating a small pool at the outlet to retain any eroded sediment.
- incorporating a stabilised low flow path into the basin to minimise scouring during frequent events.
- grassing the basin floor, to filter the sediment and help bind it to the basin floor.

Other aspects of the geometry of the basin can include:
- For grassed basins, the basin side slopes should be designed for mowing and safety considerations. Maximum slopes of between 5:1 (h:v) and 8:1 might be appropriate. Steeper side slopes could be used if the areas are not grassed (i.e. retaining walls or shrub beds) and safety fences are installed.
- The basin floor should be designed to drain freely (e.g. slope steeper than 1–2%). Flat slopes can result in difficulties with grass mowing and can result in mosquito breeding. Grass species planted on the basin floor should be tolerant of frequent inundation. Subsoil drains could be provided to address this problem.
- Depth to groundwater should be considered during the design. If the water table is high, or the basin is excavated deeply into the subsoil, problems can be experienced with grass mowing. Further, the grass species should to be tolerant of the groundwater or subsoil chemistry.
- Vehicular access for maintenance should be provided.
- An energy dissipater should be considered at the downstream end of the outlet pipe from the basin, to minimise erosion of downstream waterways.

INSPECTION/MONITORING

Monitoring activities for extended detention basins might include inspections on a regular basis and after large storm events for:
- ponding of water and any other indication of clogging of the outlet
- sediment accumulation
- subsidence/cracking of the embankment
- integrity of the spillway
- downstream erosion.

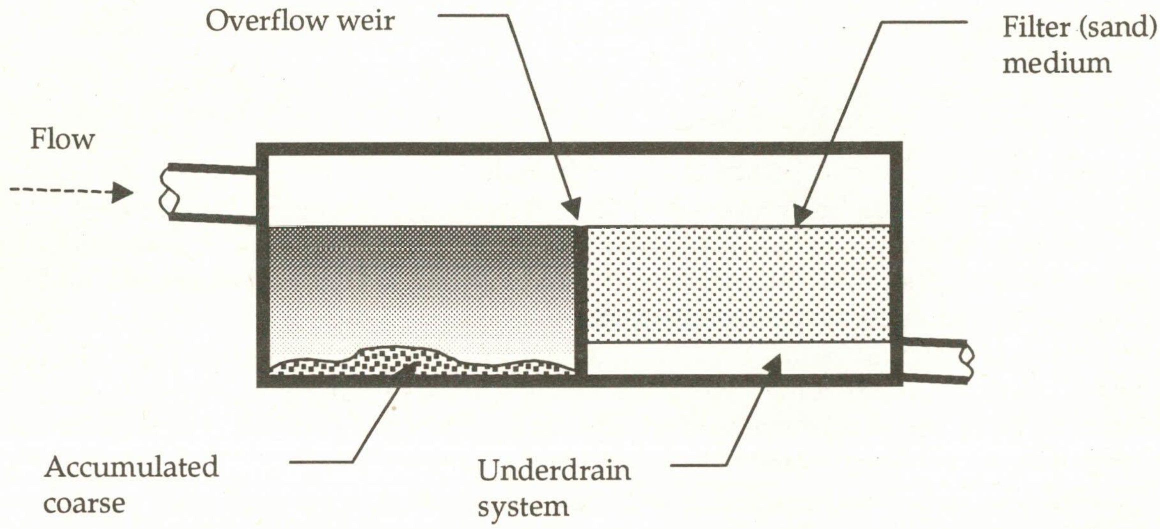
MAINTENANCE

The following maintenance activities might be required for extended detention basins:
- removal of debris and rubbish after significant storm events
- restoring any erosion problems
- unclogging the outlet structure
- removing accumulated sediment
- mowing of grass.

REFERENCES FOR FURTHER INFORMATION

CDM (1993), Schueler (1987), Schueler et al (1992), Schueler and Helfrich (1989), OMEE (1994), MDE (1987), ARC (1992), Whelans et al (1994), Horner et al (1994).

5.4 Sand Filters



DESCRIPTION

Sand filters comprise a bed of sand (or other medium) through which runoff is passed. The filtered runoff is collected by an underdrain system.

SELECTION CRITERIA/ADVANTAGES

- Principal objective is the retention of particulates.
- Can be appropriate in areas where runoff is insufficient or too unreliable, evaporation rates are too high or soils are too pervious to sustain the use of constructed wetlands.
- Appropriate for retrofitting, sites with space limitations and underground installation.
- Generally suitable for stabilised and largely impervious catchments up to 25 ha.

POLLUTANT TRAPPING EFFICIENCY

Litter	L	Sediment	M	Nutrients	M
Oxygen demanding material	M	Oil and grease	M	Bacteria	H

LIMITATIONS/DISADVANTAGES

- Limited removal of dissolved pollutants (e.g. dissolved nutrients).
- Upstream litter and coarse sediment removal is required to minimise clogging.
- Easily clogged, and effectiveness is highly dependent upon frequent maintenance.
- High head loss and relatively low flow rates through the filter.
- Large sand filters without grass cover can be unattractive in residential areas.

COST CONSIDERATIONS

Moderate to high capital cost, moderate to high maintenance costs.

ADDITIONAL INFORMATION

Sand filters are a form of infiltration system often constructed within a formal tank. The filter medium (commonly sand, although peat, limestone and topsoil have been used) overlies an underdrain system. Runoff is diverted on to the filter medium, where it ponds and flows through the medium, and is collected in the underdrain and discharged. It is a variation on the sand filters used for water treatment purposes.

Sand filters are provided with an upstream pre-treatment system to remove coarse sediment and to distribute the inflow evenly across the sand filter. The pre-treatment system (sedimentation) is generally intended to trap sand and gravel while the filter can remove finer silt and clay particles.

There are two broad scales of sand filters:

- large sand filters suitable for catchments of up to 25–50 ha, which include a pre-treatment basin for settling. These filters can have topsoil and grass cover. They can treat flow from floodways or piped systems.
- small sand filters in underground pits/chambers generally applicable for highly impervious catchments of up to 2 ha. They are usually installed within the piped drainage system.

Pollutant removal processes are sedimentation within the filter (or the upstream pre-treatment facility) and infiltration through the filter media. This removes finer particulate material and any associated pollutants.

The actual performance of a sand filter will depend on the characteristics of the inflow sediments (e.g. grading), which can relate to catchment geology and soil type. For example, clay soils might require a greater filter size, although the influence of soil type can be expected to decrease with catchment impervious fraction (ARC 1992).

Care should be taken with the installation/operation of a sand filter if the upstream catchment is generating considerable sediment loads (i.e. construction activities or erosive pervious areas). These sediment loads can clog the filter, resulting in the need to replace the filter media.

The performance monitoring of sand filters has been relatively limited, although the results to date (summarised in Table 5.3) indicate comparatively high removal rates for most pollutants (with the notable exception of oxidised nitrogen). Overall, removal rates are similar to those for constructed wetlands.

Table 5.3 – Pollutant Retention Rates for Sand Filters

Pollutant	Retention (%)	Pollutant	Retention (%)
Suspended solids	60 – 90	Total phosphorus	35 – 80
Total nitrogen	40 – 70	Oxidised nitrogen	-110 – 0
Lead	65 – 90	Biochemical oxygen demand	60 – 80
Zinc	10 – 80	Chemical oxygen demand	35 – 70

Source: Mudgway et al (1997)

KEY PERFORMANCE FACTORS

- Pre-treatment to remove coarse sediment and other debris (to minimise clogging of the filter).
- Appropriate filtration period.
- Uniform flow across filter.

DESIGN CONSIDERATIONS

Large sand filters

Sizing

There are two components to be sized for large sand filters, namely the upstream settling (or pre-treatment) basin and the filter. These components can be designed on the basis of a design storm event and high flows in excess of the design storm can be designed to bypass the filter.

The upstream settling basin should be designed for a removal efficiency that avoids rapid clogging of the filter. The approach suggested by CDM (1993) is an extended detention basin based on a water quality design storm that achieves 60-75% of the suspended solids retention objective and a drawdown time of 24 hours. A perforated riser pipe can be used as the outlet for the basin, as described in Section 5.3 (Extended Detention Basins).

ARC (1992) recommend a settling basin with a permanent pool. This pool could be sized to achieve the same retention as the extended detention settling basin, using settling velocity theory or the retention curves contained in Section 6.1 (Constructed Wetlands). Alternatively, the pool could be sized as for a sediment trap, described in Section 4.3.

A decision on whether an extended detention or permanent pool should be adopted could be based on whether local climatic conditions enable a permanent pool to be sustained. A permanent pool system is likely to result in a smaller volume than an extended detention system for the same sediment retention. However, sediment removal might be easier for an extended detention system, which would drain between storm events.

The surface area of the filter can be derived from the following equation (after ARC 1992):

$$A = \frac{Vd}{Kt(h+d)}$$

where

A	=	surface area of filter (m^2)
V	=	volume to be infiltrated (m^3)
K	=	hydraulic conductivity (m/h)
t	=	drainage time (h)
h	=	average head above filter [half the storage depth] (m)
d	=	depth of filter (m)

ARC (1992) recommends a hydraulic conductivity of 0.033 m/h, which corresponds to a system with partial and full pre-treatment (City of Austin 1988). This is less than the typical conductivity of new sand media and hence accounts for some clogging. ARC (1992) recommends a minimum media depth of 0.4 m. A filtration time of 24 hours is recommended by CDM (1993) and City of Austin (1988), while ARC (1992) recommend a 16 hour period, corresponding to one-third of the mean inter-event period. This criterion was adopted so that the filter has a dry period between events to maintain aerobic conditions and hence long term infiltration capability. The filtration period could therefore be based on rainfall patterns at the proposed site, based on the underlying criteria adopted by ARC (1992).

Geometry

Other characteristics of the settling basin that can enhance efficiency include (CDM 1993; City of Austin, 1988; ARC 1992):
- energy dissipation at the inlet
- flow velocities to minimise re-suspension (e.g. <0.3 m/s)
- effective use of the storage volume (i.e. minimising short circuiting). A length to width ratio of at least 3:1 to 5:1 could be adopted.
- access for maintenance for sediment removal
- litter rack at the outlet from the settling basin.

Characteristics of the filter that optimise efficiency include (CDM 1993; City of Austin, 1988; ARC 1992):
- use of a flow spreader to achieve a uniform flow distribution over the filter. A saw-tooth weir could be used for this purpose.
- a geotextile fabric over a coarse gravel layer above the under-drain.

CDM (1993) adopt a sand size of between 0.5 and 1.0 mm, and the City of Austin (1988) adopt 0.25 to 0.5 mm. ARC (1992) recommends 10% should pass a 63 μm sieve and 90% should pass a 500 μm sieve.

A peat–sand media has been proposed by Galli (1991), where the peat has an enhanced adsorption capacity to remove dissolved pollutants. Other types of organic media could be used, although the hydraulic conductivity might be lower, requiring an larger filter.

Small sand filters

The upstream sedimentation chamber and the filter also need to be sized for small sand filters. These components can be designed on the basis of a design storm event and high flows in excess of the design storm can be designed to bypass the filter. The sedimentation chamber can be designed using the sediment trap sizing technique (section 4.3), and the filter designed using the technique noted above. Shaver (1996) presents an example of a small sand filter design.

Sand filters should be located in areas accessible for inspection and maintenance, including access by trucks required for maintenance.

INSPECTION/MONITORING

Sand filters can be monitored on a regular basis and after every large storm event for:
- ponding, clogging and blockage of the filter media
- depth of sediment in the settling tank/sedimentation chamber
- blockage of the outlet from the settling tank/sedimentation chamber to the filter.

MAINTENANCE

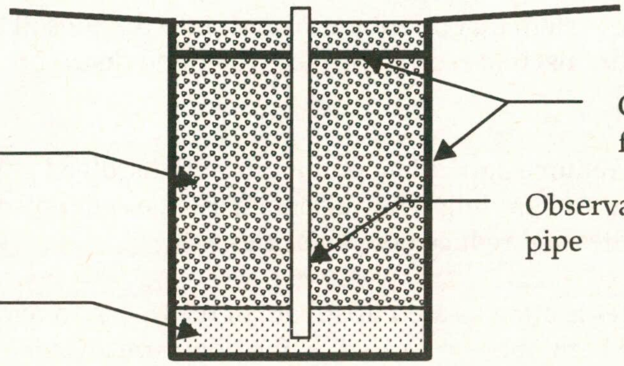
The following maintenance activities might be required for sand filters:

- The sediment and litter should be removed from the settling basin/sedimentation chamber, with drying of the sediment possibly required before disposal.
- The filter surface could be regularly raked to remove sediment and to break up any crusts (to improve infiltration).
- The top layer (50–100 mm) of the filter media can be removed and replaced.
- If the filter is not cleaned frequently, the entire filter media might need to be replaced due to migration of sands within the media. This can result in frequent maintenance being more cost-effective in the long term.
- Contaminated sand and other material removed from the filter or the sedimentation chamber (if applicable) can be removed to landfill.

ADDITIONAL REFERENCES

Shaver (1996), CDM (1993), Truong and Phua (1995), Horner et al (1994), Schueler (1987), Schueler et al (1992), OMEE (1994), Galli (1992).

5.5 Infiltration Trenches



(Source: after OMEE 1994)

DESCRIPTION

An infiltration trench is a shallow, excavated trench filled with gravel into which runoff drains to subsoil.

SELECTION CRITERIA/ADVANTAGES

- Principal objective is the removal of particulates and some dissolved pollutants
- Reduces peak runoff rates and volumes, and recharges groundwater.
- Appropriate for areas with moderate permeability soils and underground installations.
- Generally applicable for urban residential catchments < 2 ha, particularly roofs.

POLLUTANT TRAPPING EFFICIENCY

Litter	N	Sediment	H	Nutrients	M
Oxygen demanding material	H	Oil and grease	L	Bacteria	H

LIMITATIONS/DISADVANTAGES

- Risk of sediment clogging the gravel and infiltration surface (pre-treatment is required).
- Should not be used if sediment yields are high or until catchment has been stabilised.
- Risk of groundwater contamination and low dissolved pollutant removal in coarse soils.
- Potential for metals accumulation in the trench.
- Cannot be placed on steep slopes, fill or unstable areas without appropriate design.
- Inadequate maintenance has been a cause of high failure rates for these devices.

ADDITIONAL INFORMATION

There are three main types of infiltration trench systems (Horton et al, 1994):
1. Shallow excavated trenches filled with rock that receive surface runoff. The runoff then infiltrates into the underlying soil. This is the most common type of infiltration trench.
2. A perforated pipe system that distributes runoff into the subsoil through a gravel trench.
3. Drains ('French' drains) that receive surface water and distribute it to the surrounding soils.

Infiltration trenches remove particulate pollutants and dissolved pollutants, depending on the soil geochemistry and grading. They also increase the soil moisture levels and groundwater flow rates and reduce flow velocities.

A grass buffer or filter is often located upstream of the trench to remove coarse particulate matter. An overflow berm may be located on the downstream side of the trench to encourage ponding of water over the trench to increase infiltration.

The major problems with infiltration trenches are (CDM 1993):
- clogging of the rock fill or base of the trench
- groundwater contamination due to insufficient pollutant removal by the trench and underlying soil and/or high groundwater levels
- accumulation of heavy metals in the sediments retained in the trench or underlying soils.

In appropriate cases, roof water can be directed to on-site infiltration trenches. As this water is relatively clean, pre-treatment is generally not required.

The use of any infiltration system should be approached with caution in areas with urban salinity problems.

Managing Urban Stormwater: Soils & Construction contains information on soil characteristics that might influence the choice of infiltration practices.

There has been limited comprehensive performance monitoring of infiltration trenches. Table 5.4 summarises the expected removal rates.

Table 5.4 - Pollutant Retention Rates for Infiltration Trenches

Pollutant	Retention (%)	Pollutant	Retention (%)
Suspended solids	71 – 99	Total phosphorus	50 – 75
Total nitrogen	60 – 70	Bacteria	75 – 98
Lead	25 – 99	Biochemical oxygen demand	70 – 90
Zinc	51 – 99		

Source: Mudgway et al (1997), Schueler (1987)

SITE SELECTION

Due to the reported problems with infiltration systems, site selection criteria have been developed in the US to identify potentially suitable sites. Horner et al (1994) presented the following criteria to reduce the potential for failure, minimise groundwater pollution and achieve water quality improvement:

- The bed of the facility should be at least 1.0 to 1.5 m above the seasonal high water table, bedrock or a relatively impermeable layer.
- The subsoil percolation rate should be at least at least 0.8 to 1.3 mm/h.
- The soil should not have more than 30% clay or more than 40% clay and silt combined.
- When the facility will drain to groundwater, the maximum infiltration rate should be 60 mm/h.
- Generally, only loams, sandy loams and loamy sands are eligible for infiltration for stormwater quality purposes.
- The facility should not be constructed on fill material or on a slope exceeding 15%.
- Baseflows should not enter the facility.

The decision on the minimum percolation rate for a site will depend on the available infiltration area, as a higher area can accommodate a lower percolation rate for the same infiltration volume.

CDM (1993) presented a point system for evaluating potential infiltration sites, which is presented in Table 5.5. A site that obtains less than 20 points is considered unsuitable, with a site earning more than 30 points being considered excellent.

Table 5.5 - Point System for Evaluating Potential Infiltration Sites.

Item	Condition	Points
Ratio between the directly connected imperious area (DCIA) and the infiltration area (IA)	IA > 2 DCIA	20
	DCIA < IA < 2 DCIA	10
	0.5 DCIA < IA < DCIA	5
Nature of the surface soil layer	Coarse soil and low organic material fraction	7
	Normal humus soil	5
	Fine grained soils and high organic matter fraction	0
Underlying soils (if finer than surface soils; otherwise use surface soils classification)	Gravel or sand	7
	Silty sand or loam	5
	Fine silt or clay	0
Slope of the infiltration surface	S < 7%	5
	7% < S < 20%	3
	S > 20%	0
Catchment vegetation cover	Healthy natural vegetation	5
	Well established lawn	3
	New lawn	0
	No vegetation (bare soil)	-5
Degree of traffic on infiltration surface	Little foot traffic	5
	Average foot traffic (e.g. park, lawn)	3
	Considerable foot traffic (e.g. playing fields)	0

Source: CDM (1993)

Argue (1995) has developed site selection criteria for Australian conditions.

KEY PERFORMANCE FACTORS

- Pre-treatment to remove coarse sediment (to reduce the likelihood of blockage).
- Moderate underlying soil permeability.

- Deep water table.
- Appropriate infiltration period.
- Infiltration of a significant portion of the annual runoff.
-

DESIGN CONSIDERATIONS

Sizing

The pollutant retention achieved by an infiltration trench directly related to the amount of runoff captured and infiltrated by the trench. The greater the percentage of the annual runoff that is captured, the higher the long term removal rates.

There are currently no techniques available for predicting the pollutant retention offered by an infiltration trench. Optimum performance is expected to depend on the underlying soil permeability, grading and geochemistry, in addition to the infiltration rate (i.e. the amount of time that runoff is in contact with the soil).

The sizing of an infiltration trench can be based on infiltrating the runoff from a design storm event, and high flows in excess of the design storm can be designed to bypass the trench.

There are two possible techniques for sizing an infiltration trench, based on a maximum allowable drain time and flow through porous media.

The first and relatively simple technique (CDM 1993, ARC 1992, OMEE 1994) estimates the base area of the infiltration trench by:

$$A = V_e/d$$

where A = area of infiltration surface (m^2)
V_e = effective volume of infiltration trench (m^3)
d = depth of trench (m)

The effective volume of the trench is the design storm runoff volume, accounting for the volume of rock within the trench (commonly occupying 30–40% of the trench volume).

The depth of the trench can be estimated from:

$$d = I.t.Fs$$

where I = infiltration rate (m/h)
t = infiltration time (h)
Fs = factor of safety

Due to the difficulty in obtaining reliable estimates of percolation rates, the Washington Department of Ecology (1992) recommends making several site measurements and adopting the lowest value, in addition to adopting a factor of safety of 2. More accurate and comprehensive field measurements could result in a lowering of the factor of safety. Estimates of the infiltration rate can be obtained from soils texts, based on soil textural classes. If this approach is used, a higher factor of safety might be appropriate.

The choice of an infiltration period is related to the inter-event period and the need to minimise the creation of anaerobic conditions in the underlying soil or, during warmer periods, the growth of algae that might clog the soil. Reducing the infiltration time results in a smaller volume but higher surface area. Pollutant removal is enhanced by increasing the surface area of the bottom of the trench; this also reduces the risk of clogging

Infiltration periods of 24–72 hours have been recommended by CDM (1993) and Schueler (1987), with the lower periods applying when the inter-storm period in the wet season is relatively short. ARC (1992) adopt an infiltration period for the mean storm of at least 50% of the mean inter-storm period. As the majority of the infiltration occurs during the inter-event period, an appropriate infiltration period could be determined from an analysis of historical rainfall data at the site. Similar criteria to those of ARC (1992) could then be applied.

Horner et al (1994) describe a technique for calculating the surface area and infiltration volume based on Darcy's law, which describes flow through porous media. This is potentially a more accurate technique but requires more information than the simple technique described above (refer to section 5.4).

Configuration

Pre-treatment of runoff entering an infiltration trench is generally necessary for the removal of coarse particulates to minimise clogging. This pre-treatment can consist of a grass filter strip, grassed swales, a sand filter or gross pollutant trap.

The length and width of the trench can be determined by the characteristics of the site. If stormwater is conveyed to the trench as uniform sheet (overland) flow, the length of the trench perpendicular to the flow direction can be maximised. If runoff is conveyed as channel flow, the length of the trench parallel to the direction of flow can be maximised. Alternatively, the stormwater could be conveyed to the trench by a drainage system, with the geometry of the trench being less critical. The base of the trench should be level for uniform hydraulic head over the infiltration bed.

For flows in excess of the design storm, infiltration trenches can be designed with overflow pipes to drain excess water following filling of the trench. Trenches can be designed to pond excess water above the trench for delayed infiltration.

Clean, washed stone aggregate (or similar material)—commonly 25–75 mm in diameter—can be used as fill. A sand layer or geotextile fabric can be placed at the base of the trench to prevent upward piping of underlying soils. The sides of the trench can be lined with a geotextile fabric to prevent migration of soil into the rock media. Filter fabric can extend to cover the top of the trench if (porous) topsoil is used, to minimise migration of soil particles.

An observation well can be installed through the media to enable monitoring of water levels in the trench for maintenance purposes.

Alternative Infiltration Trenches

Dry wells

Small infiltration trenches can be designed to drain small areas (e.g. to capture roof runoff), and are known as dry wells. They are generally applicable to either small individual commercial buildings or single residences. These wells rarely include pre-treatment. A downpipe can direct roof runoff into the upper portion of the stone reservoir. The stone reservoir can be located below the ground surface and can include an observation well for routine inspection. Runoff that exceeds infiltration capacity can be discharged to the surface via an overflow pipe located within the down pipe. Schueler (1987), ARC (1992), MDE (1986) and CIRIA (1992) contain further details.

Pervious pipes

Pervious pipe systems are perforated along their length allowing exfiltration of water through the pipe wall as it is conveyed downstream. Pre-treatment to remove coarse sediment is appropriate. Pervious pipe systems can be implemented with reasonably flat slopes (0.5%) to promote exfiltration. Double pipe systems can be used, involving a conventional stormwater pipe over a perforated pipe. This is more expensive, but provides a contingency conveyance system if the perforated pipe becomes clogged. Additionally, the perforated pipe can be plugged during the construction phase until the site has stabilised.

Construction considerations

Before development of the site, the area proposed for an infiltration trench could be fenced off to prevent heavy equipment compacting underlying soils. The base of the trench could be ripped or tined before placing the rock fill, to increase infiltration. Compaction of the base of the trench should not occur. The rock fill can be covered when stored on site to prevent soil coating the fill, or the rock should be washed before placement. Infiltration tests on the base of the trench can be undertaken before placing the rock fill.

During the construction phase, sediment and runoff should be diverted away from the trench area. To minimise the potential for blockage of the trench, operation of the trench should generally not start until the catchment has stabilised.

INSPECTION/MONITORING

Inspections on a regular basis and after large storm events can be undertaken to assess the following:

- any surface ponding (which would indicate clogging)
- water remaining in the trench after the design infiltration period, which may indicate clogging of the rock fill or the base of the trench
- sediment in the upper layer of the rock fill.

MAINTENANCE

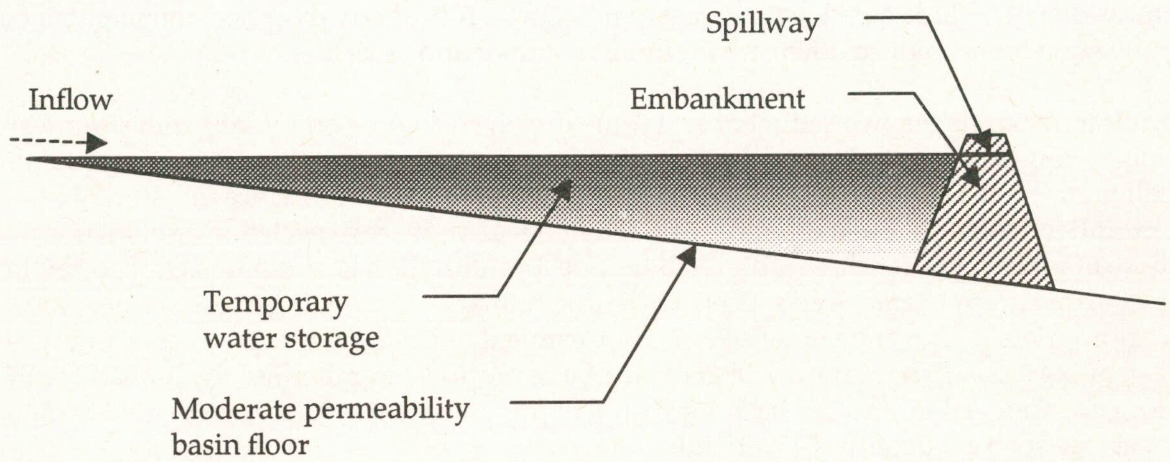
The following maintenance requirements are generally be appropriate:

- removal and washing of any clogged media, particularly the upper layers
- replacement of the top filter fabric layer to relieve clogging
- replacement of the entire trench if the base of the trench become clogged.

REFERENCES FOR FURTHER INFORMATION

Schueler (1987), Schueler et al (1992), OMEE (1994), Whelans et al (1994), ARC (1992), Galli (1992), MDE (1984), CIRIA (1992), Bettess (1996), Ferguson (1994), Mikkelson et al (1996)

5.6 Infiltration Basins



DESCRIPTION

Stormwater infiltration basins are open excavated basins that are designed to infiltrate runoff through the floor of the basin.

SELECTION CRITERIA/ADVANTAGES

- Principal objective is the removal of particulates and some dissolved pollutants.
- Reduces peak runoff rates and volumes, and recharges groundwater.
- Appropriate for areas with moderate permeability soils.
- Generally applicable for stabilised urban residential catchments < 5 ha.

POLLUTANT TRAPPING EFFICIENCY

Litter	L	Sediment	H	Nutrients	M
Oxygen demanding material	M	Oil and grease	L	Bacteria	H

LIMITATIONS/DISADVANTAGES

- Risk of sediment clogging the infiltration surface (pre-treatment is required).
- Should not be used until catchment has been stabilised or when sediment yields are high.
- Risk of groundwater contamination and low dissolved pollutant removal in coarse soils.
- Potential for metals accumulation in the base of the basin.
- Cannot be placed on steep slopes, fill or unstable areas.
- Large land consumption.
- Inadequate maintenance has been a cause of the high failure rate for these basins.

ADDITIONAL INFORMATION

Stormwater infiltration basins are designed to temporarily store and subsequently infiltrate the runoff from a design storm event through the basin floor. These basins are intended for overflow for storms larger than the design event. Unlike infiltration trenches, infiltration basins do not include a rock (stone) reservoir storage. If properly designed and maintained, infiltration basins reduce downstream runoff volumes and velocities.

Infiltration basins remove sediment and some dissolved soluble pollutants from stormwater runoff. Pollutant removal occurs principally through filtration and the adsorption of soluble pollutants on to soil particles during the infiltration process. Performance monitoring of infiltration basins has been limited, although removal rates are expected to be similar to those of infiltration trenches. A high failure rate for infiltration basins has been reported in the north-eastern United States (Galli 1992), due primarily to clogging of the surface, and inappropriate design and site selection. Pre-treatment of runoff to remove coarse sediment (e.g. by a gross pollutant trap) will generally be appropriate to minimise the probability of clogging. Clogged infiltration basins are difficult to restore, but could be converted to other measures such as constructed wetlands.

SITE SELECTION

The approach described for selecting sites for infiltration trenches (Section 5.5) could be used for selecting infiltration basin sites.

KEY PERFORMANCE FACTORS

- Pre-treatment to remove coarse sediment (to reduce the likelihood of blockage).
- Moderate underlying soil permeability.
- Deep water table.
- Appropriate infiltration period.
- Infiltration of a significant portion of the annual runoff.
- Uniform flow distribution.
- Low flow velocities (maximise sedimentation and minimise re-suspension).
- Safety considerations (including appropriate side slopes and inlet/outlet structures).

DESIGN CONSIDERATIONS

Sizing

The sizing approach described for infiltration basins (Section 6.5) could be adopted for sizing infiltration basins. When using this technique, the basin volume would need to be substituted for the effective trench volume.

Geometry

The following additional design considerations are commonly applicable for infiltration basins (CDM 1993):
- Avoid locating the basin on fill areas or on or near steep slopes.
- Provide energy dissipaters at the inlet to minimise erosion and distribute flows across the basin floor.

- Grass the floor and sides of the basin to minimise erosion and reduce the tendency for the floor to clog with fines.
- Construct the basin sides slopes to meet maintenance and safety requirements (e.g. flatter than 4:1 to 6:1). Warning signs might be appropriate if ponding depths are significant.
- The basin floor should be flat.
- Provide a spillway or bypass for events in excess of the design event.
- Provide access for maintenance vehicles to the floor of the basin.
- A subsoil drainage system could be installed beneath the basin to help infiltration.

Construction considerations

It is important that the pre-development infiltration rate of the soils at the basin site be protected during the construction phase. This can be achieved by the use of relatively light construction plant and construction practices that minimise compaction. Before development of the site, the area proposed for the basin could be fenced off to prevent heavy equipment compacting underlying soils. As some compaction of the underlying soils is probably inevitable, the floor of the basin could be tilled and levelled. Infiltration basins should generally not be used as a sedimentation basin during the construction phase of a development. Runoff and sediment should be diverted away from the basin site. If the basin is to be used as a sediment basin during the construction phase, the floor of the sediment basin could be located above the floor level for the infiltration basin. The sediment that accumulates during the construction phase, and the additional soil layer, can be removed for the earthworks for the infiltration basin. To minimise the risk of clogging, the operation of the infiltration basin could be delayed until after the site has been completely stabilised.

INSPECTION/MONITORING

Monitoring of infiltration basins can include:
- monitoring of the duration of ponding in the basin compared to the design infiltration period. This will provide an indication of any potential clogging of the basin floor. This can be particularly appropriate during the first few months of operation.
- inspection of the basin floor for erosion, sediment deposition (e.g. high deposition areas can indicate poor flow distribution in the basin) and grass growth.
- inspection of the condition of the spillway (e.g. cracking, erosion)
- field monitoring of infiltration rates to confirm rates against design rates. This could be a long term program to monitor any reduction in infiltration rates over time.

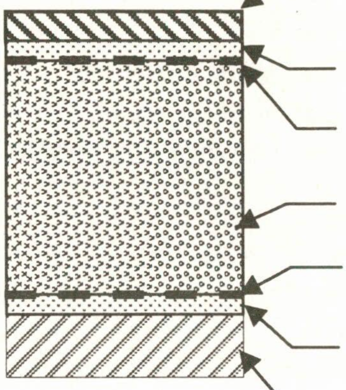
MAINTENANCE

Maintenance of infiltration basins can include:
- removal of deposited sediment
- tilling to enhance infiltration if infiltration rates have dropped to an unacceptable level
- grass mowing and maintenance.

REFERENCES FOR FURTHER INFORMATION

Schueler (1987), Schueler et al (1992), OMEE (1994), Whelans et al (1994), ARC (1992), Galli (1992), MDE (1984), CDM (1993).

5.7 Porous Pavements



(Source: after CIRIA 1992)

DESCRIPTION

Porous pavements allow runoff to drain through a coarse (open) graded concrete/asphalt pavement or open concrete blocks subsequently to infiltrate to the underlying soil.

SELECTION CRITERIA/ADVANTAGES

- Applicable for pavements subject to low traffic volume and light vehicle weight (e.g. parking areas, pedestrian paths).
- Slope of porous pavement should generally not exceed 5%.
- Reduces site runoff and increases groundwater flow rates.
- Most practical and cost-effective for small catchments generally between 0.1–4 ha.

POLLUTANT TRAPPING EFFICIENCY

Litter	N	Sediment	M	Nutrients	M
Oxygen demanding material	L–M	Oil and grease	N	Bacteria	L–M

LIMITATIONS/DISADVANTAGES

- High reported failure rate due to limited infiltration (attributed to partial or total clogging of the pavement surface).
- Inappropriate where catchment or wind erosion generates significant sediment loads.
- Possible risk of groundwater contamination.
- Possible pavement deflection, particularly if traffic loads are significant.
- Inadequate maintenance has been a cause of the high failure rate for these devices.

ADDITIONAL INFORMATION

Porous pavements can be deep, open-graded asphalt/concrete pavements with a large proportion of the normal fine aggregate material excluded. An alternative surface to open-graded asphalt/concrete pavements is modular paving. This surface coarse can be located above a deep gravel layer (or reservoir) bedded on a sand filter layer. Runoff percolates through the asphalt/concrete layer into a gravel storage reservoir. The runoff infiltrates through the subgrade or is collected by subsoil drains connected to the stormwater system.

Removal of particulates and some dissolved pollutants is achieved by filtration and adsorption on to soil particles. Moderate soil infiltration rates are required, as low rates will result in long exfiltration periods and there is a risk of groundwater pollution when infiltration rates are high. Table 5.6 indicates the reported retention rates for porous pavements.

Table 5.6 - Pollutant Retention Rates for Porous Pavements

Pollutant	Retention (%)	Pollutant	Retention (%)
Suspended solids	50 – 95	Total phosphorus	50 – 71
Total nitrogen	<0 – 85	Chemical oxygen demand	≈ 82
Lead	50 – 98	Biochemical oxygen demand	≈ 80
Zinc	62 – 99		

Source: Mudgway et al (1997)

The use of porous pavements should be approached with caution, due to their high reported failure rates (Galli 1992, Schueler et al 1992). This is due to clogging of the surface of the pavement by sediment. Pre-treatment for sediment removal is not possible for runoff from the pavement, although any overland flow could be pre-treated by grass filter strips.

SITE SELECTION

The relevant portions of the site selection techniques for infiltration trenches (Section 5.5) could be adopted for selecting porous pavement sites.

The factors that will maximise the likely success of a porous pavement include:
- low traffic volumes and light vehicle weights
- low sediment loads
- moderate soil infiltration rates
- regular and appropriate maintenance of the pavement's surface.

KEY PERFORMANCE FACTORS

- Pre-treatment to remove coarse sediment, if practical (to reduce the likelihood of blockage).
- Moderate underlying soil permeability.
- Deep water table
- Appropriate infiltration period
- Infiltration of a significant portion of the annual runoff.

DESIGN CONSIDERATIONS

A modification of the approach for designing infiltration trenches (Section 5.5) could be used for porous pavement design. Additional considerations include the need for the subgrade soil to be able to support design load under saturated conditions. The geometry of the rock reservoir could be based on the geometry for infiltration trenches. Other sizing techniques are described in Schueler (1987) and MDE (1984).

To prevent premature clogging, porous pavement should generally not be placed until all of the surface drainage areas contributing to the pavement have been stabilised. During construction, heavy equipment should not be used on the porous pavement area, to prevent compaction of soils and subsequent reduction of infiltration rates.

INSPECTION/MONITORING

Inspections can include checking for:
- areas of sediment buildup and clogging
- potholes and cracking
- areas of significant pavement deflection.

MAINTENANCE

The following maintenance activities might be required:
- high suction vacuum sweeping and/or high-pressure jet hosing to maintain porosity
- repair of potholes and cracks
- replacement of clogged areas
- rectification of any differences in pavement levels.

REFERENCES FOR FURTHER INFORMATION

Schueler (1987), Schueler et al (1992), OMEE (1994), Whelans et al (1994), Galli (1992), CIRIA (1992).

6 TERTIARY STORMWATER TREATMENT MEASURES

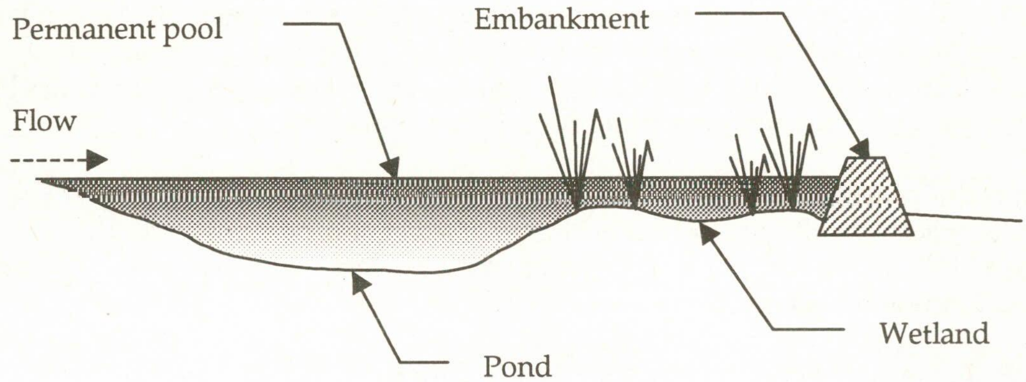
Until recently there has been only one tertiary treatment technique, namely the constructed wetland system. This comprises:

- a pond (or deep water zone): open water that might have submerged plants , but with emergent macrophytes occurring around the fringe (littoral macrophytes), and
- a wetland: an area vegetated with emergent plants, with various vegetation zones being distinguished by depth, frequency and duration of inundation.

A pond can also be termed a water pollution control pond, a wet basin or a deep water zone. Ponds are commonly constructed upstream of a wetland, and can be contiguous within a wetland system. The pond and wetland may, however, be constructed separately to perform different functions.

Sand filters that include a media layer with an adsorption capacity (e.g. peat or humus) could also be classified as tertiary treatment measures. This is a relatively recent innovation in sand filter design (until recently, sand filters have been classified as secondary treatment measures). Further details are contained in section 5.4.

6.1 Constructed Wetland Systems



DESCRIPTION

A constructed wetland system (or a water quality control pond or wet basin) commonly has two components: an upstream pond with relatively deep water and littoral macrophytes, and a downstream wetland with extensive macrophyte vegetation.

SELECTION CRITERIA/ADVANTAGES

- Principal water quality objective is the retention of fine sediment and nutrients.
- Comparatively high retention efficiency for a range of runoff event sizes.
- Potential for multi-objective designs to provide habitat, recreational and visual amenity.
- A flood storage component can be included to attenuate downstream flows.
- Generally applicable for catchments larger than 5–10 ha.

POLLUTANT TRAPPING EFFICIENCY

Litter	L	Sediment	M–H	Nutrients	M–H
Oxygen demanding material	M-H	Oil and grease	M	Bacteria	M

LIMITATIONS/DISADVANTAGES

- Require pre-treatment to remove coarse sediment or incorporate coarse sediment removal in the design.
- Reliable inflow needed to remain 'wet', unless designed as an ephemeral wetland.
- Potential impact on public health and safety from a physical, chemical or biological (e.g. mosquito-borne disease) perspective.
- Could have an impact on groundwater, or groundwater could have an impact on the wetland.
- Relatively large land requirement.

ADDITIONAL INFORMATION

There are two principal components of a constructed wetland system designed for stormwater treatment:

- **the pond (also known as a deep water zone or wet basin)**—a relatively deep, open water body with littoral emergent macrophyte plantings (possibly with submergent macrophytes)
- **the wetland (also known as a macrophyte zone or reed bed)**—a comparatively shallow water body, vegetated with emergent plants, with various vegetation zones being distinguished by the depth, frequency and duration of inundation (e.g. deep marsh, shallow marsh, ephemeral swamp).

These can be considered as part of a 'treatment train', which may include a GPT—see Figure 6.1.

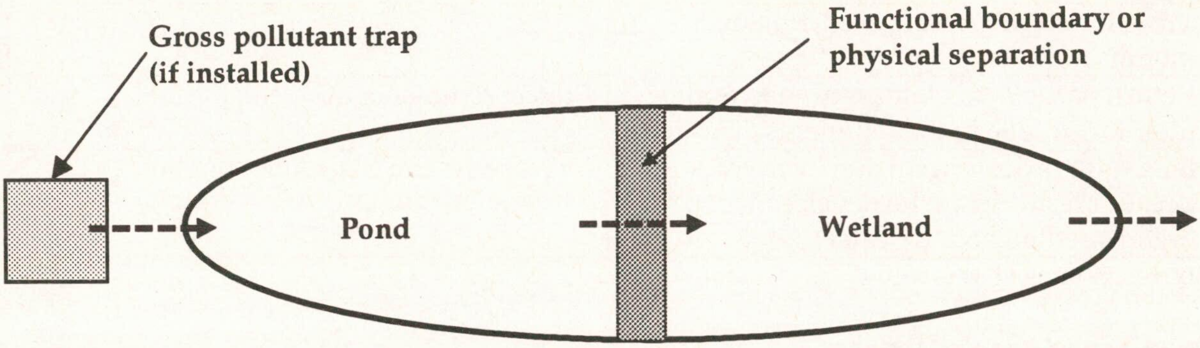


Figure 6.1 - Functional schematic of a constructed wetland system

The principal functions of these two components are indicated in Table 6.1

Table 6.1 - Principal Functions of Ponds and Wetlands

Pond	Wetland
• traps 'readily settleable' solids (generally down to coarse–medium silt) • traps pollutants adsorbed to these particles • provides an overall pollutant sink • regulates the oxidation/reduction process of the sediments • manages the hydrology and hydraulics of the wetland • provides an area for UV exposure (to encourage bacterial die-off) • the primary functions of the littoral macrophytes are to aerate the sediments, to manage flow characteristics to enhance sedimentation, and to minimise bank erosion.	• traps dissolved pollutants by adsorption and bio-film growth on macrophytes • traps fine suspended solids by continued sedimentation • regulates the oxidation/reduction process of the sediments • transforms organic components (from labile to refractory forms) • provides a substrate for bio-film growth • provides a zone for the accumulation of plant litter from the macrophytes • provides an area for UV exposure (to encourage bacterial die-off) • provides an area for predation by aquatic fauna.

Treatment Function of Macrophytes

The treatment functions of macrophytes and other aquatic vegetation in constructed wetland systems under baseflow and eventflow conditions are noted in table 6.2.

Table 6.2 – Treatment Functions of Vegetation in a Constructed Wetland System

During baseflow	During eventflow
Act as substrata for epiphytes (Epiphytes	Promote even distribution of flows
convert soluble nutrients into particulate	
biomass that can settle out and enter the	Promote sedimentation of larger particles
sediments - this is a short term process occurring over days to weeks)	
Consolidate nutrients trapped in the sediments into macrophyte biomass (This is	Provide surface area for adhesion of smaller particles
a medium-term process occurring over months to years)	
Return particulate biomass as macrophyte litter for storage in the sediments (This is a	Protect sediments from erosion
long term process occurring over years to decades resulting in the development of organic sediment and peats)	Increase system hydraulic roughness

Source: Somes et al (1995)

Loading of Organic Matter

Minimising the loading of organic matter to a constructed wetland is important to reduce the desorption of pollutants from anaerobic sediments within the wetland (Lawrence & Baldwin 1996).

Controlling organic matter inputs to the wetland is best achieved by source control. Examples include minimising the use of grass-lined channels (which require grass cutting) and appropriate selection and planting of street trees. 'Natural' channels also trap litter in pools, reducing loads to wetlands. If the control of organic matter loads cannot be achieved, it might be necessary to increase the surface area of the wetland to lower the organic matter loading rate.

Managing the organic matter buildup within the wetland can also be achieved by ensuring that the macrophyte system has a drying cycle. Drying the sediments increases the rate of organic degradation and progressively renders the phosphorus less available.

Multiple Uses

Constructed wetland systems can be designed for multiple uses, including habitat and aesthetic considerations. Systems that incorporate multiple uses can increase the value of adjacent land and hence the benefit–cost ratio of the stormwater system. Community and stakeholder involvement in the design process can be beneficial to obtain multiple use objectives. However, satisfying these other beneficial uses can conflict with the primary objective of a stormwater treatment wetland (i.e., enhanced stormwater quality). As noted in Section 3.2, these multiple objectives can be evaluated during the design development process.

Further information

The design and management of constructed wetlands is a complex and evolving field, and a relatively brief summary is contained in this section. Further details can be found in the Department of Land and Water Conservation's *Constructed Wetlands Manual* (in press).

SITE SELECTION

Location on Watercourses

The preferred location for constructed wetland systems is:

- on-line (i.e. located on the watercourse) in the upper reaches of a watercourse, or
- off-line (i.e. adjacent to the watercourse), in the middle/lower reaches.

This is illustrated schematically in Figure 6.2. In the latter situation, the stormwater system conveying frequent flows (i.e. a minor system) can be designed to direct flows from the local catchment into the wetland, rather than directly to the watercourse. Infrequent flows (i.e. the major system) can flow directly to the watercourse, bypassing the constructed wetland system. In comparison with locating the wetland system on-line on a major watercourse, these arrangements:

- enable the movement of aquatic fauna and sediment along the watercourse
- minimise the impact on any significant aquatic habitats and riparian vegetation
- decrease the potential for damage to macrophytes and re-suspension of sediments due to high flows in the off-line system
- reduce the likelihood of weed invasion of the wetland (due to the limited wetland catchment areas)
- lower the cost of the wetland's spillway.

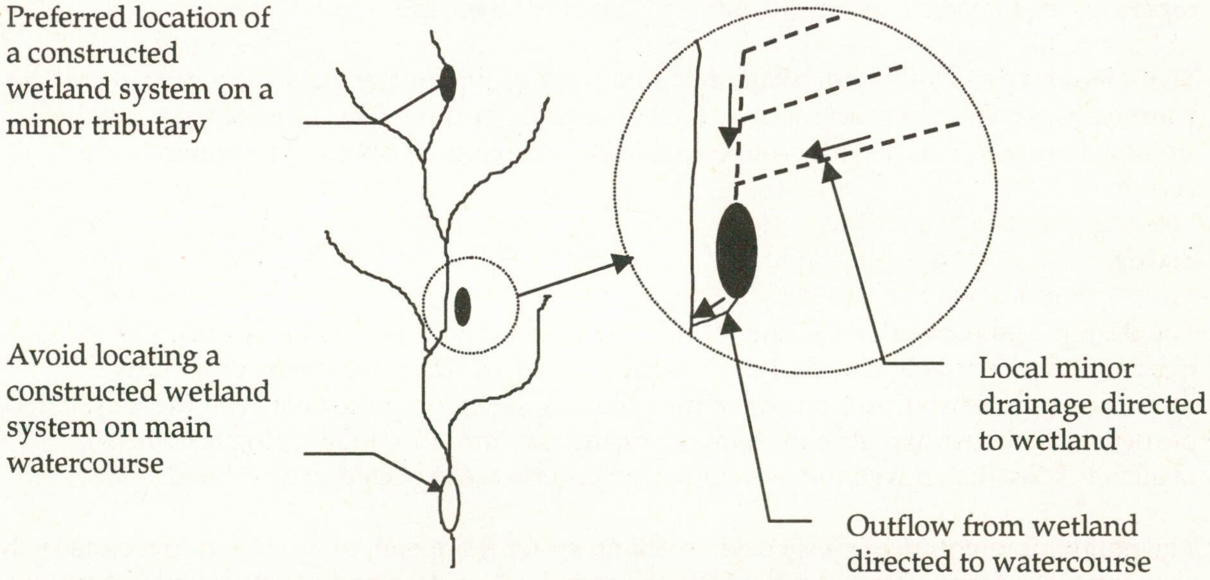


Figure 6.2 - Location of constructed wetlands on watercourses

Constructed wetland systems located in the upper reaches of watercourses will generally be on-line, unless topographic constraints enable the off-line construction of the wetland (and off-line construction is warranted). Under these circumstances, the wetland could incorporate hydrological controls to minimise velocities through the wetland.

Minimum Catchment Areas

It is difficult to provide general recommendations on the minimum catchment area for constructed wetlands. This area will depend on local stream flow and groundwater conditions, the design of the wetland and the types of macrophytes (e.g. tolerance of ephemeral conditions). Make-up water might be required for small catchment areas.

KEY PERFORMANCE FACTORS

- Uniform flow distribution through the wetland (minimise short-circuiting).
- Maximise contact time with macrophytes in macrophyte system (low flow velocities).
- Coarse sediment removal upstream of the macrophyte system.
- Minimise loading of organic matter to the wetland.
- Design for operations and maintenance, particularly sediment removal and weed management.

DESIGN CONSIDERATIONS

Coarse Sediment Removal (Pre-treatment)

Coarse sediment removal upstream of the wetland will minimise damage to the macrophytes. This can be achieved by either:

- installing a sediment trap upstream of the pond, or
- using the pond as a coarse sediment trap.

The former is likely to result in lower relative maintenance costs. Maintenance of the pond might have an impact on aquatic fauna and flora, and might generate community concerns regarding maintenance of a more 'natural' treatment measure.

The removal of coarse sediment might result in the sediment starvation of downstream waterways, potentially exacerbating channel erosion. This is expected to be a potentially significant problem only with on-line wetlands in the middle reaches of a watercourse system.

Sizing

The design, and particularly sizing, of constructed wetland systems for stormwater treatment is a relatively new and evolving field. Further, there has been comparatively limited comprehensive monitoring of the pollutant retention performance of these systems, particularly in Australia. As the results of further performance monitoring become available, constructed wetland system design criteria are expected to be refined.

The pollutant retention of constructed wetland systems appears to be related to factors such as the nature of the inflows (particularly the sediment grading and geochemistry), the ionic composition of the wetland waters, and the geometry and macrophyte planting scheme of the wetland. There are currently insufficient data available to quantify these influences.

The components to be sized for a constructed wetland system relate to the temporary and permanent storage volumes in the pond and wetland, as indicated in Figure 6.3. If the constructed wetland system is constructed as a single water body, the distribution of these volumes between the macrophyte and wetland systems will need to be considered.

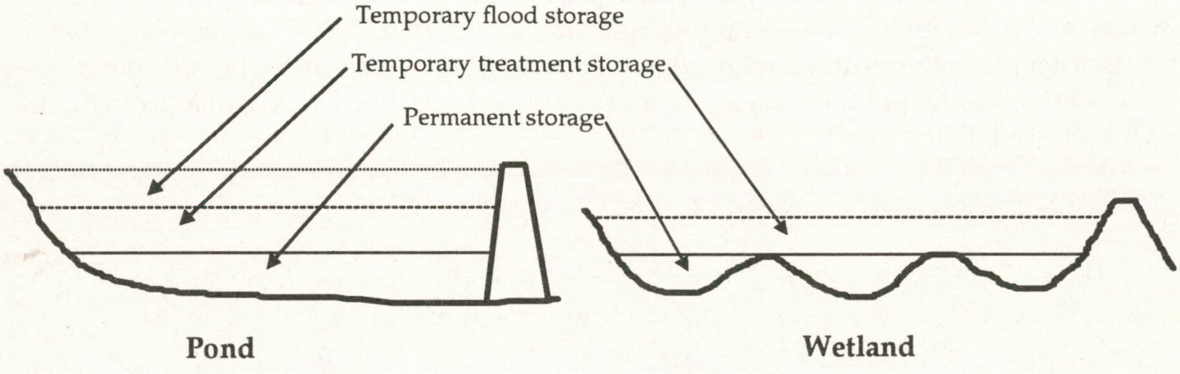


Figure 6.3 – Definition sketch for wetland volumes

The principal purposes and potential sizing criteria for these volumes are noted below:

Ponds

- **temporary flood storage**—can be used for attenuating peak flows up to the 100 year ARI event, and is applicable for on-line ponds (e.g. ponds located in the base of a retarding basin). This volume, if provided, can be sized using a rainfall–runoff model to meet flood mitigation criteria
- **temporary treatment storage (or extended detention storage)**—can be used to enhance the hydraulic residence time of the permanent storage for sedimentation of coarse particulates. It can also be used to attenuate flows to protect the downstream wetland. The actual pollutant removal capacity is difficult to predict, particularly for nutrients.
- **permanent storage**—intended principally for water quality control by sedimentation. Ideally, this volume depends on the particle size characteristics of the inflow and the mineralogy of the particles (particularly their adsorption capacity). Other factors affecting the volume include the organic carbon loading and the resulting influence on re-mobilisation and subsequent leakage of pollutants attached to the sediments. The sizing of this volume is discussed below.

Wetlands

- **temporary storage**—the principal function is to provide a variable wetting–drying cycle to encourage macrophyte diversity. A depth range of 0.5–1.0 m and a hydraulic residence time of less than 3–5 days for a design storm could be reasonable design parameters. This depth range can generally be tolerated by emergent aquatic macrophytes. Further sedimentation can also occur in this volume, which can also be used to attenuate downstream flows. The use of a riser-type outlet or syphon can more readily control water level fluctuations than a weir outlet. The sizing approach for this volume could also be used for the equivalent volume of a pond.
- **permanent pool**—an essentially permanent volume to encourage biofilm growth on the macrophytes and the continuation of the sedimentation processes. There is currently no technique available to accurately relate residence time or surface area to pollutant

retention in these systems dominated by macrophyte-based processes. The sizing of the permanent pool is discussed below.

A sizing technique for estimating the permanent pool component of a constructed wetland system has been developed, and is described in the Appendix. This technique relates pollutant retention to the hydraulic loading rate of the wetland. This rate is the average annual runoff volume divided by the wetland system's surface area, and is discussed further in the Appendix. These relationships are presented in Figure 6.4 for suspended solids, total phosphorus and total nitrogen. The resulting surface area of the wetland system for a desired pollutant retention and resulting hydraulic loading rate (from Figure 6.4) can be calculated from:

$$A = R/L$$

where A = surface area of the constructed wetland system (m^2)
R = annual runoff volume (m^3/yr)
L = hydraulic loading rate ($m^3/m^2/yr$)

Alternatively, this can be expressed in terms of the area ratio of the wetland system (ie the ratio of the wetland's surface are to its catchment area) to the runoff depth:

$$a = r/(10\ L)$$

where a = area ratio of the constructed wetland system (%)
r = annual runoff depth (mm)
L = hydraulic loading rate ($m^3/m^2/yr$)

These relationships were derived on the basis of reduction in inflow concentration. As there is generally no statistically significant relationship between event runoff volume and EMC (Duncan 1997a, US EPA 1983), this approach can also be used to estimate the long-term pollutant load retention.

The total surface area of the constructed wetland system needs to be distributed between the pond and wetland. An approximately equal distribution might be appropriate for average conditions, with an increased wetland area being appropriate if the inflows have a large dissolved fraction, and the converse applying when the inflows are largely particulate. Due to the current uncertainty regarding the performance of constructed wetland systems, some caution should be taken when estimating the performance these systems. Alternative sizing techniques are discussed in the Appendix.

Morphology

Appropriate morphologies of ponds and wetlands are summarised below. The morphology to be adopted for a particular site will often depend on the site-specific topographic conditions.

Ponds

Principal objective—optimise effective hydraulic residence time:

- energy dissipation is required at the upstream end of the system to reduce inflow velocities and distribute flows across the cross section

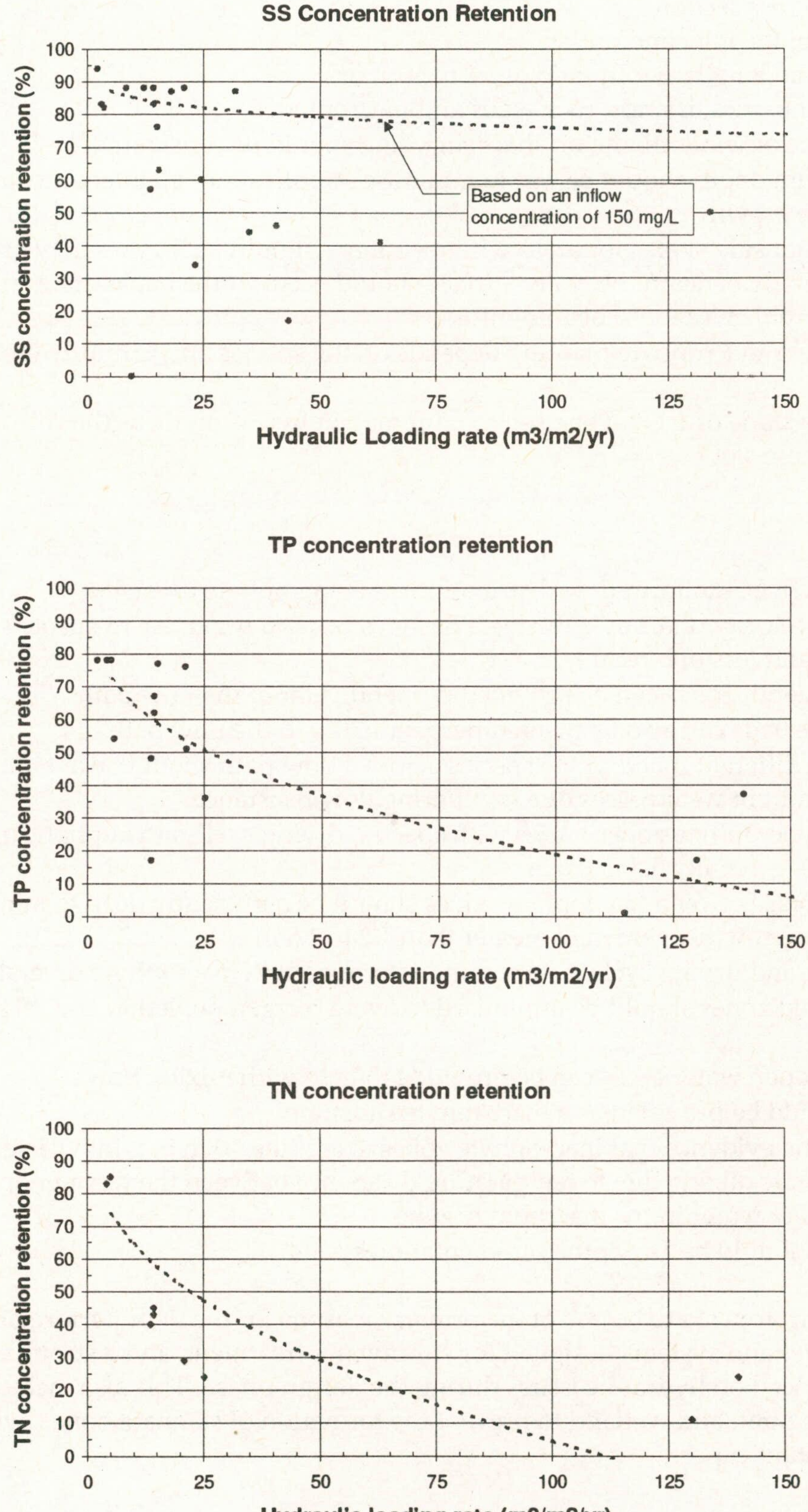


Figure 6.4 – Pollutant retention curves

- short circuiting should be minimised to achieve a uniform flow distribution
- a range of techniques is available to meet the objective:
  —uniform cross section
  —location of the inlet and outlet
  —a minimum length: width ratio of 3:1 to 10:1, or
  —the use of baffles, islands, rock walls and macrophytes
- the influence of wind mixing on short circuiting should be considered
- the maximum depth should be 2 m to minimise stratification, maintenance and macrophyte growth
- the maximum side slopes for areas without a fence should be 8:1 on safety grounds. No change in grade beneath the water surface should occur. If the banks are evenly graded, mosquito problems should be minimised.
- the depths for macrophyte planting depends on the species and turbidity (generally < 0.6 m)
- a minimum slope of 1:1 can be used to limit macrophyte growth, as the rhizomes are not stable on these slopes.

Wetlands

Principal objective—uniform flow distribution and macrophyte diversity:
- maximum velocity of 0.2 m/s during a design storm (to minimise re-suspension of sediments and loss of biofilm)
- a range of depth zones can be provided perpendicular to the flow path
- macrophyte beds can also be planted perpendicular to the flow path
- planting of different macrophyte species across a flow path should be avoided to minimise in-built weaknesses due to differing flow resistance.
- depths of macrophyte zones varies with species, drying cycle and euphotic depth (turbidity)—generally 0.0 to 0.6 m
- the open areas between macrophyte zones should be sufficiently deep to minimise macrophyte growth—generally greater than 1.2 to 1.5 m
- the wetting and drying cycle is important determinant in macrophyte diversity
- poorly mixed zones should be minimised to avoid oxygen depletion and related problems
- sequential open water areas can be provided to help with mixing flows
- topsoil should be provided as a macrophyte substrate
- there is some evidence that macrophyte zones exceeding 10 m in width can create hydrodynamic mixing due to temperature difference between the warmer open water and the cooler water in the macrophyte zone
- side slopes should be as per the sedimentation system.

Short-circuiting appears to be one of the primary reasons for the poor performance of constructed wetland systems designed for stormwater treatment, and a strong emphasis needs to be placed on hydraulic issues during the design phase. This also includes the shaping of the pond and wetland to prevent the formation of stagnant zones, which may result in pollutant export.

Macrophyte Planting

Macrophyte planting in pond primarily occurs around the fringe of the system and at the inlet. The macrophytes at the inlet enhance sedimentation, provided flow velocities are sufficiently low. The fringing macrophytes help with the aeration of sediments in this area, rather than facilitating pollutant removal. Macrophytes can also be used to help with creating uniform flow distribution within a pond.

Within a wetland, four zones can be created, namely an open water zone (covering approximately 25% of the area) and three bands of macrophytes with different species according to the depth. These three bands can be:

- 0.0–0.2 m: shallow marsh
- 0.2–0.4 m: marsh
- 0.4–0.6 m: deep marsh

The principal purposes of these zones are to achieve uniform flow across the wetland and encourage macrophyte diversity. Other techniques could be used to achieve this function (DLWC, in press). The principal purposes of the open water zone for UV disinfection and oxygenation. The macrophyte planting schemes are indicated in Figure 6.5.

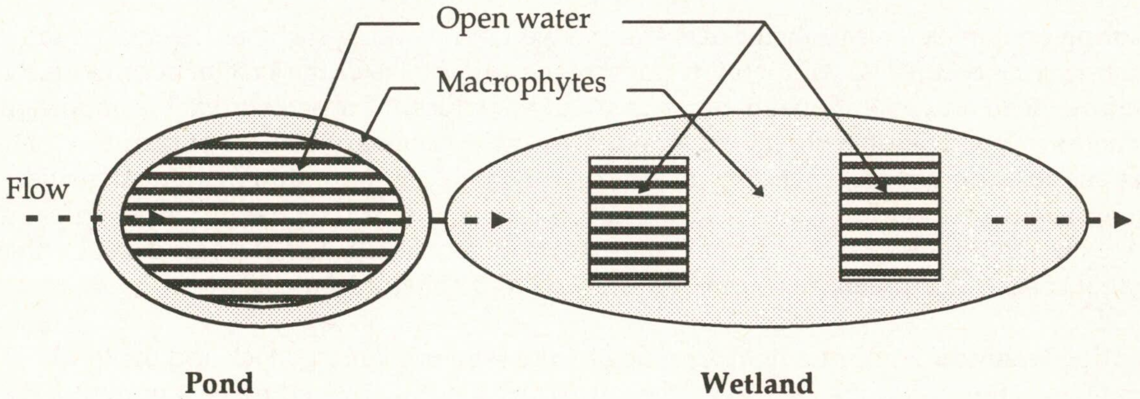


Figure 6.5 – Schematic layout of macrophyte planting zones

Successful planting of wetland habitats depends on six main factors:

1. Planting Design
2. Site Preparation
3. Supply and Quality of Planting Stock
4. Planting
5. Water Level Control
6. Wetland/Vegetation Maintenance

Planting design for water quality control wetlands should be based on an understanding of the water treatment processes that occur in wetlands and how these can be enhanced by the characteristics of individual species and the characteristics of particular wetland vegetation zones (see Table 6.2). In wetland zones where the trapping of fine particles is an objective, plant surface area should be maximised. The above-normal water level part of emergent aquatic macrophytes can also be used for filtration, if the wetland basin is designed to allow for a water level increase during event flows.

The appropriate choice of plant species is a balance between selecting species for particular depth ranges or hydrological conditions and selecting plants to enhance particular treatment processes. Observations of the natural distribution of species can usually help identify the optimal inundation depth, frequency and duration. However, most constructed wetlands are developed for water quality improvement, and the poor quality of inflow water commonly limits the distribution of many species. Consequently, the expected distribution in constructed systems, based on experience of natural distributions, should be conservative.

Planting density is a major factor determining wetland planting success. The greater the planting density, the less competition from weeds and the faster the system becomes fully commissioned and operational maintenance can be reduced.

Site preparation. The major elements of site preparation for planting are the provision of a suitable substratum for growth and the control of weeds and non-target plants. The successful propagation of wetland plants requires an adequate covering and depth of top soil (about 0.2 m).

Planting density is a major factor determining wetland planting success. The greater the planting density, the less competition from weeds and the faster the system becomes fully commissioned and operational maintenance can be reduced. Consequently, it is important to choose a propagation technique that will deliver the necessary volume of plants in the best possible condition. Depending on the particular species, optimum planting densities can vary between 9 (0.5 m centres) to 25 (0.25 m centres) per square metre. Larger spreading species that normally occur in deeper water can typically be planted at lower densities than smaller species that are more common in shallow and ephemeral areas.

Planting technique is normally determined by the type of planting stock and the local terrain and site conditions. However, the key to any planting procedure is to minimise the damage to the stock during planting. As a result, sensitive planting procedures typically rely considerably on manual labour.

Water level control. The establishment conditions and typical growing conditions for many wetland plant species are often very different. Consequently, to establish plantings by either direct seeding, planting of nursery seedlings or even transplanting of clonal material it is usually necessary to reduce water levels and have good control over water level fluctuations during the establishment phase. A water depth of less than 0.2 m is typically required for the establishment of even the largest of emergent macrophytes.

Wetland/vegetation maintenance needs to occur on two scales:

- establishment maintenance
- ongoing maintenance.

During the establishment phase plant growth and condition should be monitored very regularly. It is during this establishment period that plantings are most vulnerable to impacts and damage. Regular monitoring during the establishment allows a rapid response to any problems and helps minimise the extent of any adverse effects. Factors that need particularly close attention include:

- water level
- weed invasion
- animal damage (insect infestation, grazing by water birds).

Ongoing maintenance requires attention to all the issues discussed for the establishment phase, albeit at a reduced frequency, plus a range of issues associated with the long-term stable functioning of the system. Long-term maintenance issues include monitoring of:

- vegetation composition and structure
- accumulation of above-ground senescent biomass
- accumulation of sediment
- variation in the hydraulic behaviour of any wetland cells
- development of potential pest habitat.

Many weed species are transported during flood events. As a result, monitoring should occur after each major event. This also allows for inspection of any physical damage to the system caused by high flows.

Outlet Structures

The design of outlet structures can facilitate variable water level control within the constructed wetland system. Water level control is useful for:

- planting macrophytes
- optimising macrophyte growth during commissioning
- achieving a diversity of macrophytes
- providing weed and mosquito control
- facilitating wetland operation to optimise water quality improvement.

The types of outlet that can be used include:

- weirs—rectangular, v-notch or proportional discharge
- perforated riser
- reverse slope pipe
- syphon.

These are discussed further in Section 5.3 (Extended Detention Basins)

Management of High Flows

High flows (e.g. 10–100 year ARI) need to be managed for both on-line and off-line wetland systems, although different management techniques are appropriate. This management is necessary to minimise permanent damage to the macrophytes and re-suspension of sediment. It might be appropriate to limit velocities during a design high flow event to less than approximately 2 m/s, although the biofilms attached to the macrophytes will be lost under these conditions. The macrophytes will provide a degree of armouring to the sediments contained in a wetland.

For on-line constructed wetland systems, flow attenuation can be undertaken by providing temporary flood storage above the permanent pool of the system, as illustrated in Figure 6.6. This is similar to constructing a wetland in the floor of a retarding basin.

For off-line constructed wetland systems, a high flow bypass could be provided upstream of the pond. Alternatively, the bypass could be provided upstream of the wetland, when the pond is not contiguous with the wetland. Under these conditions, the pond could also incorporate flood storage.

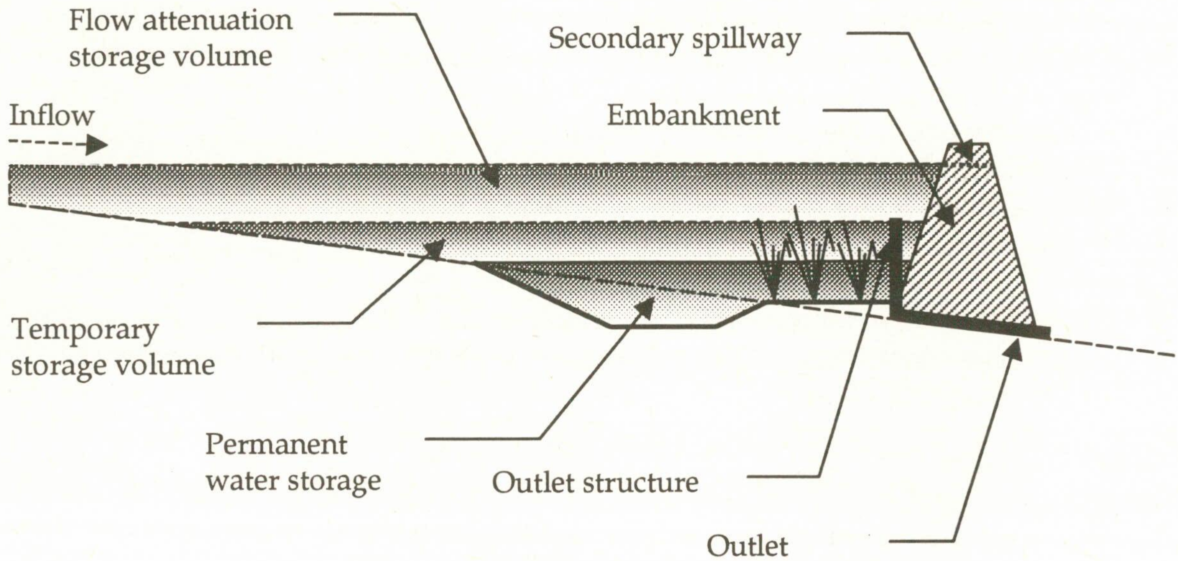


Figure 6.6 – Schematic of an on-line constructed wetland

Safety Issues

Techniques for addressing safety issues associated with constructed wetland systems include:
- installation of hand rails around inlet and outlet structures and areas with steep side slopes
- signs warning of potential dangers
- avoiding high velocities at the inlet or outlet
- safety booms installed at inlets and outlets (on large systems).

Inlets and outlets can also be designed to avoid trapping fauna (e.g. turtles and water birds). Occupational health and safety considerations for maintenance staff should also be considered during the design stage.

Groundwater Considerations

Groundwater inflows and outflows can have a significant effect on a constructed wetland system. These influences can include the effect of groundwater chemistry on processes, including sedimentation and loss of permanent pool volume. This particularly occurs if the wetland is constructed in porous strata below the water table. The wetland may need to be designed to be isolated from groundwater or accommodate this influence.

Mosquito Control

Techniques for minimising potential mosquito problems associated with constructed wetlands include:
- even grading of the side slopes, avoiding localised depressions
- management of water depth (mosquitoes prefer depths less than 0.4 m), including water level manipulation (particularly during summer)
- avoiding zones with poor water movement

- minimising litter input to the wetland, as mosquitoes can breed in litter
- encouraging predation of mosquito larvae by aquatic fauna; this is encouraged by designing the wetland as a viable ecosystem.

Artificial control techniques can also be used including aerators, sprinklers and sprays. A mosquito risk analysis can also be undertaken (DLWC (in press), Russell and Kuginis 1995).

Design Issues for Maintenance

Due to their size and morphology, it is important that operations and maintenance considerations be addressed during the design of the wetland (White, 1995) to minimise subsequent maintenance costs.

The pond and wetland geometry can be designed to facilitate sediment removal by equipment that is likely to be readily available to the maintenance authority. This requirement can limit the maximum width of the pond or wetland, or alternatively submerged berms or similar structures could be installed to facilitate equipment access.

The ability to draw down the pond and wetland for sediment removal and weed management also needs to be considered during the design phase to minimise subsequent operation and maintenance costs.

Designing the wetland for weed management is important, particularly in the warmer regions of Australia. Infestation by weed species (e.g. *Salvinia*) can be managed by a range of techniques (DLWC (in press)), including designing the wetland as separate cells that can be individually drained.

MAINTENANCE

An operation and maintenance plan should be prepared for all constructed wetlands. Maintenance considerations include the following.

- A higher level of maintenance is expected during wetland commissioning.
- Sediment and litter removal will be required on a regular basis. If litter accumulates for more than two weeks following an event, breakdown products of some litter might re-pollute the water.
- Weed maintenance will be required, especially during commissioning.
- A major refit or decommissioning could be required when the wetland reaches its design life.

Macrophyte harvesting is not considered necessary to maintain the long-term nutrient retention capacity of the constructed wetland system.

INSPECTION/MONITORING

Inspection can include checking the following:

- performance of the outlet structure, including blockage or downstream erosion
- integrity of the embankment
- weed infestation
- mosquito breeding
- litter and sediment levels
- health and diversity of macrophytes.

Water quality monitoring of the wetland system's performance can also be undertaken.

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APPENDIX – CONSTRUCTED WETLAND SIZING TECHNIQUES

A.1 Derivation of Constructed Wetland Sizing Technique

The sizing technique presented in Section 6.1 of this document is an extension of the work undertaken by Duncan (1997a) at the CRC for Catchment Hydrology (CRCCH) on the pollutant removal efficiency of ponds and wetlands. The original work by Duncan (1997a) investigated the relationship between a range of factors and pollutant output percentage, being the ratio of the outflow to inflow event mean concentration of a pond/wetland (expressed as a percentage). The factors included in the analysis were:

- area ratio—the ratio of the surface area of the wetland to the wetland's catchment area
- inflow concentration (concentration of the pollutant in the inflow to the wetland), and
- storage depth—the ratio of the wetland's volume to its catchment area.

A total of 88 Australian and overseas studies were collected by the CRCCH for use in this analysis, described in Duncan (1997b).

A review was undertaken of all studies before their inclusion in the analysis, to assess the design of the pond or wetland against 'good practice' and to assess the quality of the monitoring data. Consequently, a weighted scoring system was developed for evaluating the quality of the pond/wetland design, and the quality of the sampling data used to evaluate the performance of the pond/wetland. This approach was adopted to avoid the results of poorly monitored studies or poorly designed ponds/wetlands biasing the results. Each study was rated according to:

- a design index—a measure of the design of the pond/wetland against factors considered to represent 'good design practice', and
- a data index—a measure of the quality of the monitoring data.

The design index is presented in Table A.1, and is based on allocating high scores to factors likely to enhance performance. Further, the use of the design index was considered to provide a reasonable basis for combining the pond and wetland data to increase the number of studies available for analysis, by effectively eliminating poorly designed ponds, which are unlikely to reflect the expected performance from constructed wetlands.

Table A.1 – Design Index

Factor	Description	Score
Shape	Length: width ratio > 3	1
	Intermediate or unknown	0
	Length: width ratio < 2	-1
Multiple cells	Single cell	-1
	Unknown	0
	Multiple cells	1
Permanent pool	Dry basin	-1
	Extended detention basin or unknown	0
	Permanent pool	1
Mixed use	Flow passes through a pond then a wetland	1
	Single pond/wetland or unknown	0
Depth	Mean depth < 1 m	1
	Intermediate depth or unknown	0
	Mean depth > 2 m	-1
Potential score range		*-4 to 5*

The data index is presented in Table A.2, and is weighted towards the factors that are most important in achieving an accurate determination of the pond or wetland's long-term performance.

Table A.2 – Data Index

Factor	Description	Score
Event-based monitoring	Yes	5
	No or unknown	1
Flow weighted monitoring	Yes	5
	No or unknown	1
Monitoring duration	Duration > 6 months	3
	Duration between 2 and 6 months	2
	Duration < 2 months	1
	Unknown	*
Number of events	Number > 10	6
	Number between 6 and 10	4
	Number < 6	2
	Unknown	**
Land use	Urban > 75%	2
	Urban between 50 and 75%	1
	Urban < 50%	0
Potential score range		*4 to 21*

* same rank score as that given for event-based monitoring
** same rank score as that given for monitoring period

Only studies with a design index greater than 1 and a data index greater than 16 were included in the subsequent analysis. Further, ponds/wetlands with considerably smaller area ratios than those expected in constructed wetland design (less than 0.1%) were also excluded from the analysis to avoid biasing the resulting regression. The design index was not found to be a statistically significant explanatory variable for output of SS, and sites with low index scores were therefore included in the analysis for this variable. The resulting studies used in the analysis are noted in Table A.3.

Regression analyses were undertaken between the output percentage of each pollutant and the following explanatory variables:

- area ratio (pond/wetland as a percentage of catchment area)
- storage (ratio of pond's or wetland's volume to its catchment area)
- average annual hydraulic residence time—the ratio of the pond/wetland volume and the estimated annual runoff volume
- hydraulic loading rate (also known as the upflow or overflow rate)—the ratio of the estimated average annual runoff volume and the surface area of the pond/wetland
- inflow concentration.

Table A.3 – Studies Used in Regression Analysis

Site	Location	Parameters available			Reference
		SS	TP	TN	
Crookes Wetland	Albury		•	•	Raisin and Mitchell (1995)
Lake Ridge	Minnesota, USA		•	•	Oberts et al (1989)
DUST Marsh (3)	California, USA		•		Meiorin (1989)
Whispering Heights	Seattle, USA	•			Dally (1984)
Carver Ravine	Minnesota, USA		•	•	Oberts et al (1989)
Orlando pond	Florida, USA	•			Martin and Miller (1987)
Montgomery basin	Maryland, USA	•			Grizzard et al (1986)
McCarrons (3)	Minnesota, USA		•	•	Wotzka and Obert (1988)
Bellevue 31	Seattle, USA	•			Reinhelt and Horner (1985)
Lake Annan	Campbelltown	•			SKM (1996)
The Paddocks	Adelaide	•	•	•	Tomlinson et al (1993)
Greenview (2)	Florida, USA	•			Yousef et al (1990)
Waverly Hills	Michigan, USA	•			Athayde et al (1983)
Lake Ellyn	Illinois, USA	•			Athayde et al (1983)
Unqua pond	New York	•			Athayde et al (1983)
Orlando wetland	Florida, USA	•			Martin and miller (1987)
Stedwick	Washington DC, USA	•			Athayde et al (1983)
Hidden Lake	Florida, USA	•			Harper et al (1986)
Hayman Park	Auckland, NZ		•		Leersnyder (1993)
Frisco Lake	Missouri, USA	•			Oliver & Grigoropoulos (1981)
Springhill	Florida, USA	•			Holler (1989)
Orlando Ponds	Florida, USA	•	•		Harper (1988)
Pacific Steel	Auckland, NZ	•	•		Leersnyder (1993)
Westleigh	Washington DC, USA	•			Athayde et al (1983)
Orlando Highway	Florida, USA	•	•	•	Harper (1988)
Wayzata	Minnesota, USA	•	•		Hickock et al (1977)
Orlando Pond	Florida, USA	•			Harper (1988)

The original investigation by Duncan (1997) showed that the relationships were strongest when the analysis was undertaken on log-transformed data (i.e. a log-domain analysis) for both axes, and this approach was therefore adopted for this analysis.

The analysis results indicated that the inflow concentration was a statistically significant variable (at the 5% level) only for suspended solids. The correlation coefficients (R^2) from this $\log_{10}$-domain analysis are indicated in Table A.4. Inflow concentration did not significantly affect the output percentage of total nitrogen or total phosphorus.
The regressions are strongest for suspended solids and weakest for total nitrogen. The high variability (i.e. low predictability) in nutrient (particularly nitrogen) retention is consistent with a number of previous studies of pond and wetland performance (e.g. Schueler et al 1992).

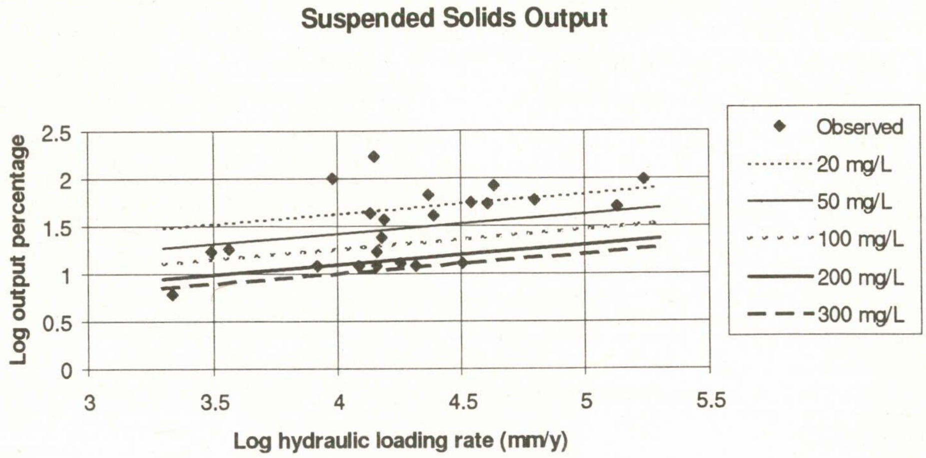
Table A.4 – Correlations between pond/wetland performance and explanatory variables)

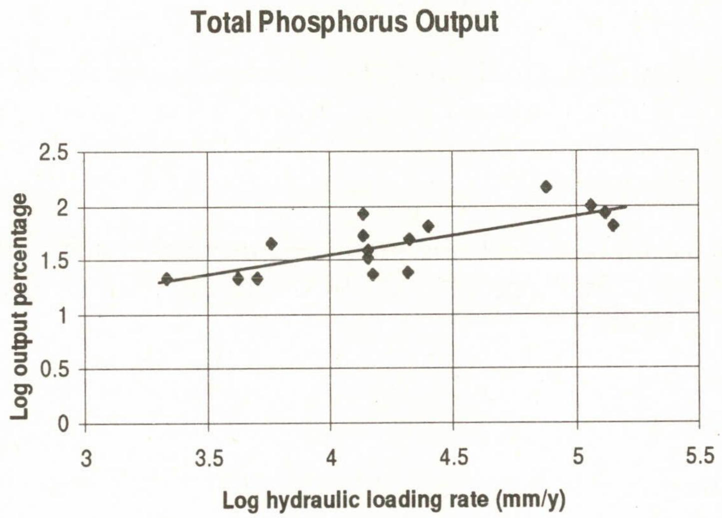
Parameter	R^2 for variable :			
	Area ratio	**Storage depth**	**Residence time**	**Loading rate**
Suspended Solids**	0.78	0.79	0.79	0.78
Total phosphorus	0.52	0.43	0.38	0.56
Total nitrogen	0.35*	0.10*	0.24*	0.69

variables $\log_{10}$ transformed
* correlation was not statistically significant at the 5% level
** inflow concentration was also a significant explanatory variable

The best explanatory variable for TP and TN was hydraulic loading rate, with the correlation for this variable being only marginally lower than the best regressions for SS. For nutrient retention, hydraulic residence time was a considerably poorer explanatory variable than hydraulic loading rate.

The resulting regressions between log hydraulic loading rate and log output percentage are presented in Figure A.1. These regressions were used to derive the pollutant retention curves presented in Figure 6.6.





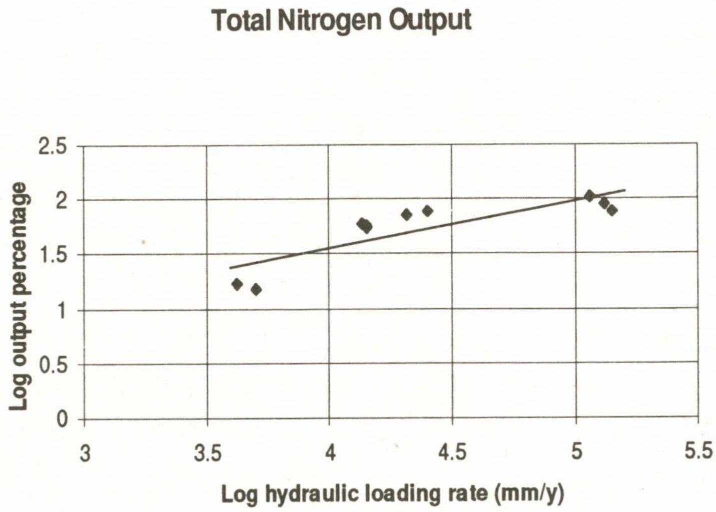


Figure A.1 – Pond/wetland pollutant output relationships

A.2 Alternative Constructed Wetland Sizing Techniques

A.2.1 Hydraulic Residence Time Relationships

Relationships between hydraulic residence time and pollutant removal were first derived by Lawrence (1986) in Canberra. Additional curves have been derived by Tomlinson (1996) in Adelaide, SKM (1996) in Sydney (Campbelltown) and Hunter and Constandopoulos (1997) from Sydney (Blacktown). The combined curves are presented in Figure A.2. These curves were all derived on the basis of different monitoring and analysis techniques, and are consequently not directly comparable.

There is, however, a reasonable degree of variability among curves for different sites, and a significant degree of data scatter at each site. This is expected to be partly due to the influence of factors other than residence time, such as inflow characteristics and wetland morphology and macrophyte planting schemes.

A.2.2 Settling Relationships

The US EPA (1986) describes a technique for sizing wet detention devices that has formed the basis of many sizing techniques used in the United States. The technique is based on an analysis of rainfall patterns to determine settling periods, and the use of settling velocity data. This technique was based on column settling tests of stormwater runoff, and a probability distribution of suspended solids settling velocities was calculated. Relationships were then determined between basin surface area, rainfall characteristics and basin depth for particular sites. A generic relationship was derived, whereby a ratio of basin volume to mean storm volume of 2.5 yielded a 75% retention of SS.

This technique accounts for only pollutant removal by sedimentation. Further, it is based on laboratory settling velocity data, which can be expected to be higher than settling within the turbulent conditions that occur in a constructed wetland. Further, Leersnyder (1993) found that the settling velocities from three sites in Auckland, New Zealand, were considerably slower than those from the US EPA (1986) study. This was attributed to different inflow particle size distributions and specific gravities,

A.2.3 Algal Growth Relationship

A technique was proposed by Hartigan (1986) for sizing basins for phosphorus removal, based on achieving an average annual hydraulic residence time of 14 days. This was found by Rast et al (1983) to correspond to the minimum period between phosphorus loading and algal growth (phosphorus consumption) in US lakes and reservoirs. This technique results in a larger volume requirement than that derived by the settling relationship technique.

A.2.4 Decay Rate Relationship

Kadlec and Knight (1996) proposed a first-order decay relationship for the design of wastewater treatment wetlands. This relationship estimates the outflow concentration of a pollutant as a function of the inflow concentration, hydraulic loading rate and rate constants. Wong and Geiger (in press) have modified this approach to derive a design technique for constructed wetlands for stormwater treatment.

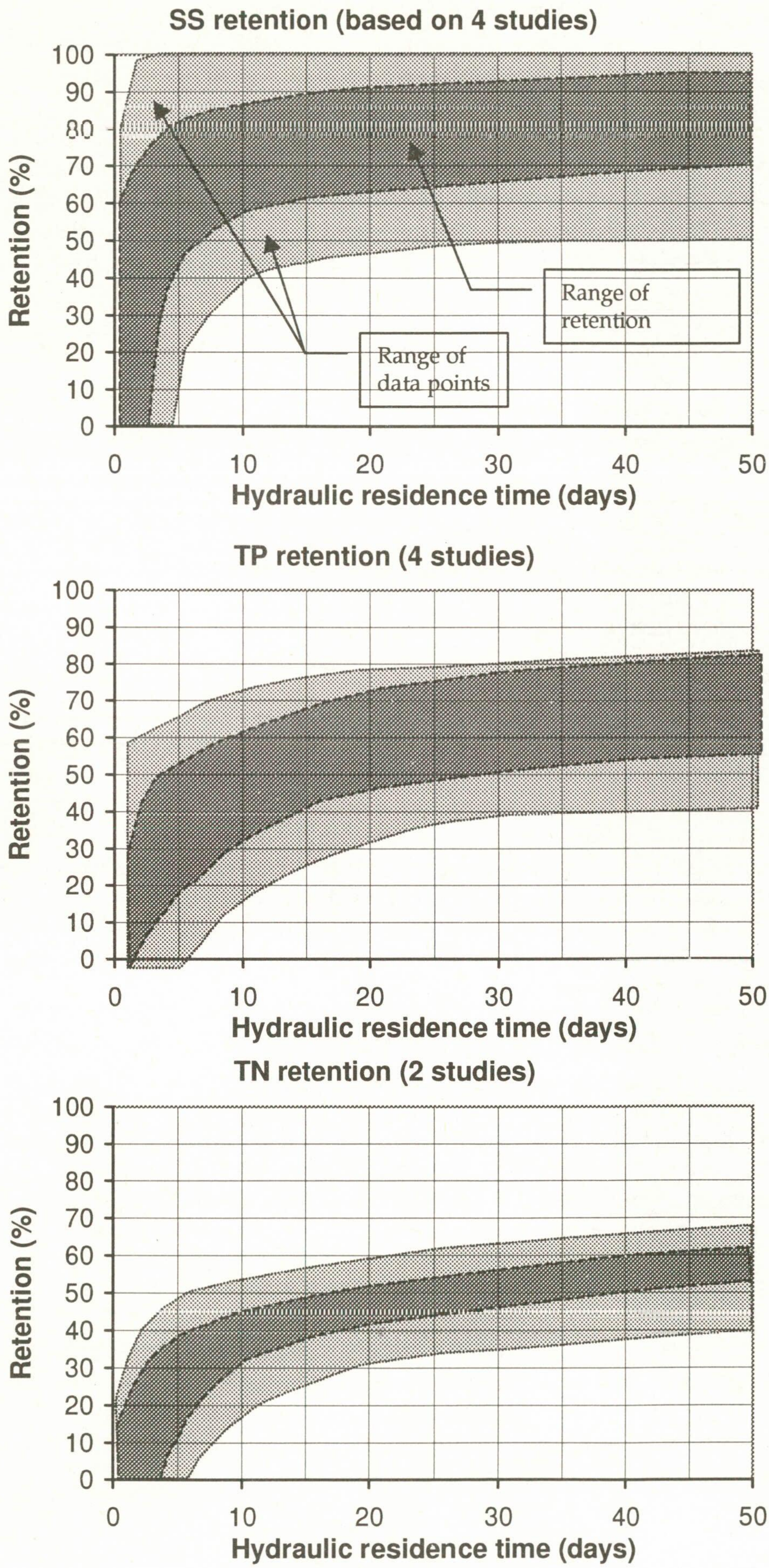


Figure A.2 – Hydraulic residence time relationships