Appendices

Appendix A: Example of a litter trap action plan

Adapted from 'Moreland City Council Litter Trap Action Plan', a report by Allison Partners for Moreland City Council, 1998.

1 Introduction

This case study provides an example of how to develop a Litter Trapping Action Plan based on work undertaken in the city of Moreland.

Moreland City Council is approximately 5 kilometres north of central Melbourne and is typical of inner city suburbs. The council undertakes regular street sweeping, has widespread usage of litter bins and employs litter officers for particular problem areas. Despite these efforts litter is recognised as a problem pollutant for stormwater and the receiving creeks and rivers.

The approach is based on targeting high litter generation areas.

Structural stormwater treatments such as litter traps can be expensive to install and maintain. Therefore, litter traps should generally be used where litter generation rates are high, such as commercial areas. Litter traps in residential areas with low litter generation rates will commonly trap much more vegetation than litter. Maintenance costs can therefore be significant for relatively low levels of litter trapped. In areas with low litter generation rates (e.g. typical suburban low density residential), source controls are likely to be the most effective measure. The approach taken here is to target the areas that produce the highest litter loads.

2 Methodology

The Litter Trap Action Plan was developed through a five step process. The steps were:

1 identifying high litter generation areas from examination of land-use maps, consultation with council officers and field inspections;

2 determining drainage pathways for each high generation area from examination of the drainage plans;

3 determining the suitability of each area for different types of litter trapping systems (i.e. either source, in-transit or end-of-pipe traps);

4 identifying suitable locations for installing litter traps (including field inspections); and

5 recommending a list of potential litter trap locations, based on achieving maximum litter trapped per dollar spent.

3 Litter generation areas

The areas within council that are likely to generate large amounts of litter should be determined by examining land-use maps and identifying commercial, industrial and other areas likely to produce litter. Research in Melbourne has found that commercial areas can contribute twice as much stormwater litter as residential areas, and light-industrial areas also produce more than residential areas. This suggests that it is these areas that should be targeted for a stormwater litter trapping program.

The areas can be grouped into categories such as: major commercial, light industrial, medium-sized commercial, and local commercial litter generation areas—as was done in the Moreland City Council case study.

Major commercial areas are large retail outlets likely to produce the most litter (e.g. large shopping centres). Light industrial areas generally have automotive industries as well as any other light industry, with associated fast food outlets. There can also be high traffic loads in these areas. Medium sized commercial areas are typically a strip shopping centre (e.g. group of twenty shops or more), often with a supermarket and car parks. Local commercial areas are generally small strip shopping areas with between three and ten retail shops.

In the City of Moreland there were five major commercial, six light industrial, eleven medium sized commercial and twenty-nine local commercial areas identified for the study.

Example: Glenroy Shopping Centre (major commercial area)

The Glenroy shopping area includes regional strip shops as well as several supermarkets and other retail outlets. The majority of the area is on the west of the railway line, although there are retail outlets along Wheatsheaf and Glenroy Roads to the east of the railway line.

The area is typically busy with retail activity, pedestrians and car parking along either side of Pascoe Vale Road, and larger car parks present to the east of the railway line.

4 Drainage pathways from litter source areas

The next task is to identify the drainage pathways from the largest litter generation areas and discuss the suitability of various litter trapping alternatives for the individual drainage networks.

It is particularly important to identify drainage paths that convey run-off from several of the litter generation areas or where 'hot-spots' are serviced by more than one drainage system.

Major and medium-sized high litter generation areas

In-line litter traps are generally preferred in major and medium sized litter generation areas where the majority of the source area drains through one outlet, and that outlet doesn't capture significant run-off from many other areas. In-line litter traps have the advantage of requiring that only one location be visited for maintenance. Should the inlets from the source areas lead into different drainage networks or be diluted with run-off from other locations (with lower litter generation potential), entrance litter baskets are likely to be more appropriate.

Local high litter generation areas

The small size of many local commercial areas means that in-line traps may not be cost effective. In these cases, entrance type baskets may be more appropriate. They can target the pits that are likely to contribute the most litter without treating water from areas that do not produce much stormwater litter.

Example: Glenroy shops

Drainage pathways

The main commercial area of Glenroy is along Pascoe Vale Road and east along Glenroy and Wheatsheaf Roads. The area drains in two directions: south-west and south-east from either side of the railway line.

All the commercial area on the west side of the railway line is drained through one outlet that is directed under Prospect Street (Melways 16 G3) in an 825 mm diameter drain. Approximately half the catchment is residential in Prospect Street, however, the drain runs north along Pascoe Vale Road until Gervase Avenue, therefore collecting many road pollutants in addition to all the busy retail areas west of the railway.

Litter trapping options

Prospect Street is a quiet residential area with access to the drain from the roadway and is a potential location for an in-line litter trap. The most appropriate treatment for this area would be an in-line litter trap installed on an 825 mm drain (Prospect Street) to capture run-off from the commercial area west of the railway line.

East of the railway line, the commercial and light industrial areas drain into three different pipes draining down Glenroy, Wheatsheaf and Waterloo Roads. There are approximately twenty-five drain inlet pits that service the strip shopping areas (although draining into different catchments). These inlets could be retrofitted with entrance baskets to prevent a larger in-transit trap being required on three drains downstream of the commercial area.

5 Litter trap locations

Potential locations for traps can be assessed according to their suitability for installation with particular types of litter traps (e.g. at source or in-line).

The suitability of a location for installing a litter trap depends on:

- the presence of any existing stormwater controls;
- the location in the drainage network relative to the high litter generation area;
- pipe system details (e.g. sizes, gradients, etc.);
- proximity to underground services or any other space constraints;
- access to the site for maintenance; and
- potential disturbance to the local community during construction and maintenance.

After identifying any existing litter traps in the area, the next step is to identify the most suitable type of litter traps (either entrance or in-line traps) for each high litter generation area. The following is a guide to the suitability of various trapping systems.

Entrance traps: installation suited to smaller areas.

Small in-line traps: suited to small to medium sized areas, except where inlets from the source area lead into different drainage networks or where flow is diluted with run-off from other low generation areas (consider entrance traps in these situations).

Large in-line end-of-pipe traps: suited to medium to major generation areas or where a number of smaller areas are serviced by the same drainage pathway.

Based on field inspections of the drainage pathways, a final recommendation can be made on litter trap locations. However, in the case of in-line litter traps, detailed investigations would be required before construction in order to size the trap and take account of more specific issues such as proximity to other underground services.

6 Priorities and recommendations

Along with the potential location for installations, estimates of the gross pollutant loads and cost ranges for installation and maintenance for the recommended works should also be made.

Example: Prospect Street (Glenroy shops)

The main Glenroy commercial area west of the railway drains down the southern side of Prospect Street. It is a quiet residential street with a wide easement. There is potentially a location for installing an in-line litter trap. There appears to be a considerable slope down Prospect Street, which may aid the performance of some in-line traps.

Prospect Street looking west along drainage path.

Litter loads for each trap location can be estimated using a decision support system derived from data collected from a study in Melbourne (Allison et al., 1998b). The computer program estimates litter loads from catchment size, land-use and rainfall information.

Costs for traps can be estimated using the size of the catchment areas and cost ranges presented in Chapter 7 of these Guidelines. The values in the Guidelines estimate trap costs in Melbourne using the size of the catchment as the determining factor.

For the Glenroy shopping area, a combination of in-line and entrance traps is recommended. These potential works need to be ranked against other potential litter trap locations in the City of Moreland.

Example: Moreland City Council

A combination of in-line and entrance type litter basket traps were proposed for Moreland City Council. Twelve medium sized commercial and light industrial areas and all twenty-nine local commercial areas were recommended to be installed with entrance type baskets. Ten locations were recommended for in-line traps.

Table A1 presents the ranking of the ten suggested locations for in-line litter traps in the City of Moreland. It includes some details relevant to design and construction issues, estimates the litter and gross pollutant load (i.e. including vegetation) from each catchment and gives a range of likely construction costs for each trapping location.

Example: Moreland City Council in-line trap recommendations

Potential in-line litter traps: prioritised locations

Location	Catchment areas	Installation issues	Estimated annual litter loads* (kg)	Estimated annual gross pollutant loads* (kg)	Estimated installation cost range ($'000s)**	Receiving waterway
Prospect Street	Glenroy shops	quiet residential street, street slope, drain under easement, captures Glenroy shops west of railway, 825 mm diameter pipe	1100	3400	35–105	Moonee Ponds
Poplar Road	south Sydney Road, Phoenix Street industrial area, Grantham Street shops	outlet into natural stream reach, large catchment area, 1450 mm pipe diameter, adequate pipe slope and space for most installations, in Royal Park—not in Moreland City	2900	8100	100–300	Moonee Ponds
Union Street	south Sydney Road	isolates a large section of Sydney Road shops, pipe under road, mild slopes, traffic disturbance during construction and maintenance, 825 mm pipe diameter	1300	3000	28–84	Moonee Ponds
Montifore Street	Gaffney Street light industrial area	light industrial and residential catchment, pipe under roadway, quiet residential area, mild slope ?, 1050 mm diameter pipe	700	2100	40–120	Moonee Ponds
Williams Street	Gaffney Street light industrial area	busy light industrial area and pipe under busy through road, pipe under east gutter, 900 mm diameter, steep slope	600	1000	22–66	Merri
Dawson Street	Gaffney Street light industrial area	busy light industrial area, drain under roadway (towards edge as it goes south), steep slope, busy through road, 600 mm diameter pipe	400	700	15–45	Merri
Fallon Street	Phoenix Street light industrial area	residential/light industrial catchment, pipe under roadway, large trees on median strip, 900 mm pipe with mild slope, traffic diversions during construction and maintenance	300	700	15–45	Moonee Ponds
Lygon Street	south Lygon Street, Barkley Square, south Brunswick light industrial activity	large commercial/light industrial catchment, pipe under very busy roadway but located under west gutter, open space adjacent to drain at Park Street, 1275 mm diameter, traffic diversions may be required	1900	5400	65–195	Merri
Albion Street	Albion Street shops, Holmes Street shops and light industrial area	commercial/light industrial and residential catchment, located under dead-end road, deep pipe at outfall, 1000 mm diameter, potentially space further upstream along reach	900	3000	40–120	Merri
Harding Street Laneway	Sydney Road north shops	825 mm pipe under laneway, run-off is mainly from Sydney Road, width of laneway may limit construction	500	1500	17–51	Merri
Total cost					377–1100	

* Loads were estimated using a decision support system model for gross pollutant trapping systems (Allison et al. 1998b).
** Costs vary depending on the type of in-line trap selected.

Table A1 Example of recommended priorities for installing in-line litter traps (Moreland City Council 1998).

Appendix B: Sizing of sediment settling basins

In addition to length:width ratios, there are two parameters that need to be sized for a sediment basin;

- the basin's surface area; and
- the basin's depth.

Settling basin surface area

The surface area is commonly sized on the basis of settling theory. The settling velocity of discrete particles under ideal settling conditions (Class I settling) is presented in Table B1.

Classification of particle size range	Particle diameter (micrometres)	Settling velocity (millimetres per second)
Very coarse sand	2000	200
Coarse sand	1000	100
Medium sand	500	53
Fine sand	250	26
Very fine sand	125	11
Coarse silt	62	2.6
Medium silt	31	0.66
Fine silt	16	0.18
Very fine silt	8	0.04
Clay	4	0.011

Table B1 Settling velocities under ideal conditions (after Maryland Department of the Environment 1987).

In practice however, ideal settling conditions rarely occur. This is because of many factors including:

- **sediment concentration variability**: particles interfere with the settling of others;
- **sediment shape variability**: non-spherical particles settle slower than others;
- **sediment size variability**: the settling of larger particles can cause currents, which can inhibit the settling of finer particles. Conversely, smaller particles effectively increase the fluid density, inhibiting larger particle settling;
- **particle densities**: these vary according to geology and organic matter content;
- **turbulence**: turbulence and non-uniform flow distribution can resuspend sediments; and
- **flocculation and coagulation**: this can occur during inter-event periods and increase the removal of finer particles.

Consequently, the settling velocities that can be expected within sediment basins will be lower than those predicted by ideal settling models. Barnes et al. (1981) noted that the design of sediment settling tanks for wastewater treatment could incorporate a factor of safety, based on an assumption that the settling velocities will be 40 to 60 per cent of theoretical values. However, Ontario Ministry of Environment and Energy (OMEE) (1994), using data collected in the Nationwide Urban Run-off Program (US EPA 1983, 1986), estimated that settling velocities in stormwater can be as low as 2 per cent of theoretical values.

Because of the difficulties in estimating site specific settling velocities, designs could be based on ideal settling characteristics, with recognition of the lower velocities that will occur in reality. These can be expected to result in lower trapping efficiencies for the design particle during design storms. However, during flows that are lower than the design storm, finer particles would be expected to be trapped (Whytecross et al. 1989).

Under constant flow conditions settling theory suggests that (Barnes et al. 1981):

$$v_s = \frac{Q}{A}$$

where:

v_s = settling velocity of target sediment (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:

$$L = \left(\frac{rQ}{v_s}\right)^{0.5}$$

where:

L = basin length (metres)

r = length:width ratio of the basin

To minimise short circuiting, length:width ratios should exceed 2:1 to 3:1 for sediment traps (OMEE 1994; Willing and Partners 1992b). This sizing technique assumes that the flow rate remains constant during the settling period and that water discharges from the sediment basin across the full downstream cross-section. This is often not the case, with water leaving the sedimentation basin via overflow of a weir structure at the downstream end.

A modification to the conventional design equation was proposed by Wong et al. (1999) to allow sizing of sedimentation basins which may have a permanent pool and a weir outlet structure. The equation also incorporates provision for turbulence and short circuiting of flow paths, i.e.

$$R = 1 - \left(1 + \frac{V_s[S_p + S_e]}{Qd_e}\right)^{-n}$$

where:

R = the proportion of the target particle size retained

n = a factor to account for general hydraulic inefficiency of the sedimentation basin; n = 1 for poor hydraulic efficiency; n = 3 for good hydraulic efficiency, n = 5 for very good hydraulic efficiency; and n = infinity for ideal conditions

S_p = the storage volume of the permanent pool

S_e = extended detention storage respectively

d_e = the depth range of the extended detention storage

Overall, there are many limitations with current design techniques for sediment basin sizing, mainly because of non-ideal settling characteristics and dynamic flow conditions.

Settling basin depth

The risk of sediment resuspension and the desired maintenance frequency for cleaning are the two main factors which determine the depth of a sediment basin.

The most common technique for estimating scouring velocities for particular particle sizes is based on channel erosion studies (Camp 1946, cited in Metcalf and Eddy 1991). Table A2 presents estimates of the velocities that will initiate scour for various particle sizes (derived using averaged values for the erosion equation constants suggested by Metcalf and Eddy 1991).

Particle diameter (micrometres)	Scouring velocity (metres per second)
2000	0.72
1000	0.51
500	0.36
250	0.25
125	0.18
62	0.13
31	0.09
16	0.06

Table B2 Estimated scouring velocities (after Metcalf and Eddy 1991).

These scouring velocities can be used to estimate the basin depth required to avoid re-suspension of the design particle size during the design storm. Some scouring is probably inevitable at the inlet to the basin, due to the jet action of the inflows, although this can be minimised by incorporating appropriate energy dissipation in the design.

Assessment of the scouring velocity could be based on the cross-section averaged velocity corresponding to the outflow discharge. The depth for this calculation could be based on the depth of water in the basin and the flow depth above this level. An estimate of the flow depth can 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 submergent effects included. Submergent effects only become significant when the downstream water depth is greater than about 90 per cent of the upstream depth (Bradley 1978).

The depth of the sediment retained in the basin will be related to 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) for urban catchments and sediment trap performance. Furthermore, sediment deposition within traps tends to be non-uniform.

An allowance for sediment storage of at least an additional 30 to 50 per cent of the basin depth estimated by the scouring velocity technique should be provided.

Construction considerations

Durable, strong, rust-free materials should be used. These are usually concrete, but may be other materials such as masonry brick walls, gabions and reno mattresses. Reinforced concrete is durable, strong and low maintenance, but has a high construction cost and unattractive appearance. Masonry brick walls are cheaper than reinforced concrete, but are also unattractive. Gabions and reno mattresses are cheaper than reinforced concrete and provide a slightly more pleasing structure. 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.

Maintenance considerations

The following issues can be considered for maintenance:

- access: vehicular access to sediment trap for sediment removal;
- sediment removal: concrete or hard stand base to allow ease of removal of debris and sediment;
- silt drying: silt drying area may be required to drain removed material;
- trap de-watering: the design can provide for de-watering of the trap, with discharge to an adjacent sewerage system. If this is not feasible, a tanker may be required to transport the sediment-laden water from the site for approved disposal. If the drainage of a trap is to the sewer, then the liquid contents only of a trap can be discharged; and

- filter medium replacement: traps capable of gravity drainage to a dry condition can have filtering medium replaced if ponding occurs.

Safety considerations

These include:

- safety barriers: railings or vegetation can be placed on either side of a major trap to discourage the public from approaching the sides of the trap. If railings or low fences are to be constructed, they should be aligned parallel to the direction of flow during floods, to reduce their impact on flood levels;
- slope: the slope of any side batter to the side walls of a major trap should not exceed 1(V):4(H) and should preferably be 1(V):6(H) to 1(V):8(H);
- signage: signs to discourage entry into the trap; and
- other considerations.

The visual impact of a trap can be improved by landscaping, mounding and selection of construction materials. Vegetation will also prevent easy entry to the structure. If the trap location prevents its disguise by vegetative means, the use of local rock may be appropriate. The structure may also be made with coloured concrete and shaped to resemble the local rock.

Appendix C: Sizing of filter strips and grass swales

Filter strip design

Horner et al. (1994) outline a technique for sizing filter strips and grass swales based on results of studies conducted in Seattle, USA. These studies indicate that optimum pollutant retention occurs when the hydraulic residence time along the filter strip is approximately nine minutes. Performance was found to noticeably deteriorate when the residence time fell below five minutes. In addition to a hydraulic residence time of nine minutes, other basic design criteria suggested by Horner et al. (1994) for filter strips include:

- design flow velocity of less than 0.3 m/s
- design flow depth not to exceed 12 mm

Horner et al. (1994) suggest the following seven steps to design a filter strip:

1 Estimate the design flow for the design storm event.

2 Determine the slope of the filter strip.

3 Set the design flow depth (to be less than 12 mm).

4 Solve Manning's equation to determine the required width of flow. Manning's 'n' values suggested are as follows:

–0.20 for mowed filter strips; and

–0.24 for natural grasses or infrequently mowed strips.

5 Determine the flow area, based on the calculated flow width and established depth.

6 Calculate the resulting velocity. Reduce the flow, increase the flow width or reduce the depth of flow if the velocity exceeds 0.3 m/s.

7 Using the resulting velocity, calculate the flow length to achieve a residence time in the filter strip of nine minutes. An absolute minimum residence time should be five minutes.

One critical design element of filter strips is to maintain sheet flow over the length of the filter strip. Grass filter strip performance has been found to reduce if they are located on grades exceeding 5 per cent, particularly if the slope exceeds approximately 15 per cent with the formation of rills and high erosion potential. In these steep conditions check dams, and note that benches are needed at regular intervals to spread the flow.

Filter strips that are flatter than 2 per cent can be susceptible to water ponding and a sub-soil drainage system should be considered to ensure effective infiltration.

The slope of the filter strip should be uniform and the cross-section, level. Particular attention should be paid to this during construction.

The integrity of the filter strip may be impaired if flows greater than the design event enter the strip. Velocities exceeding the design velocity can be expected to result in reduced strip pollutant removal efficiency until the grass has recovered. Such flows may also result in scouring of the strip. A by-pass 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, a shallow soil depth for pollutant retention presents a possible risk of pollution entering the groundwater.

Grass swale design

Basic design guidelines for grass swales include:

- geometry: minimise sharp corners using parabolic or trapezoidal shapes and side slopes no steeper than 3:1 (h:v);

- longitudinal slope: keep slopes between 2 per cent and 4 per cent to promote uniform flow conditions across the cross section of the channel. Check dams should be installed if slopes exceed 4 per cent and underdrains installed if slopes are less than 2 per cent;
- swale width: should not exceed 2.5 m, unless structural measures are used to ensure uniform spread of flow and should be level;
- maximum flow velocity: keep below 0.5 m/s for the 1 year ARI event and a maximum velocity of 1.0 m/s for the 100 year ARI event; and
- Mannings 'n' value: adopt a Manning's 'n' value of between 0.15 and 0.2 for flow conditions where the depth of flow is below the height of the vegetation. For the 100 year event, the Manning's n value will be lower and can be assumed to be 0.03.

Horner et al. (1994) recommended a maximum depth of flow to be one third of the grass height in infrequently mowed swales and half the grass height, to a maximum of 75 millimetres, in regularly mowed swales. Greater flow depths would be appropriate in grassed waterways designed to convey floodwaters (for example 100 year ARI).

The integrity of the swale may be impaired if flows greater than the design event enter the swale. Velocities exceeding the design velocity can be expected to result in reduced swale pollutant removal efficiency until the grass has recovered. Such flows may also result in scouring of the swale. A by-pass for high flows could be installed to prevent large concentrated flows eroding the swale.

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, a shallow soil depth for pollutant retention presents a possible risk of pollution entering the groundwater.

Appendix D: Design guide for infiltration trenches

Trench site selection

Site selection criteria have been developed in the USA for choosing potentially suitable sites for infiltration trenches. Horner et al. (1994) present the following site selection criteria, which aim to reduce the potential for failure, minimise groundwater pollution and achieve water quality improvement:

- the bed should be at least 1.0 to 1.5 metres above the seasonal high water table, bedrock or a relatively impermeable layer;
- the percolation rate should be at least 0.8 to 1.3 millimetres per hour;

- the soil should have not more than 30 per cent clay or 40 per cent clay and silt combined;
- when the facility drains to groundwater, the maximum infiltration rate should not exceed 60 millimetres per hour;
- generally, only loams, sandy loams and loamy sands are suitable for infiltration trenches;
- trenches should not be constructed on fill material or on a slope exceeding 15 per cent; and
- baseflows should not enter the trench.

Camp Dresser and McKee (CDM) (1993) suggests a point system for evaluating potential infiltration sites, which is presented in Table D1. A site that obtains fewer than 20 points is considered unsuitable, while a site earning more than 30 points is considered excellent. Argue (1995) has developed site selection criteria for Australian conditions.

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

Table D1 Point system for evaluating infiltration sites (Source: CDM 1993).

Trench sizing

The pollutant retention achieved by an infiltration trench is a function of the amount of run-off captured and infiltrated to the soil. The greater the percentage of the annual run-off captured, the higher the long term removal rates.

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

The sizing of an infiltration trench could be based on infiltrating an expected percentage of the mean annual run-off from the catchment or a design storm event, with high flows exceeding the design storm designed to by-pass the trench. Continuous simulations of infiltration systems using rainfall data for all capital cities in Australia were undertaken by Argue (1999) to develop design curves for infiltration systems. These curves represent the relationship between the soil hydraulic conductivity, the size of the infiltration system and the percentage of mean annual stormwater run-off infiltrated into the system.

There are two simplified techniques commonly used for sizing an infiltration trench and they involve the use of maximum allowable drain time and flow rate through porous media design criteria.

The first and relatively simple technique (CDM 1993; Auckland Regional Council 1992; OMEE 1994) estimates the base area of the infiltration trench according to:

$$A = \frac{V}{d}$$

where:

A = area of infiltration surface (square metres)

V = effective volume of infiltration trench (cubic metres)

d = depth of trench (metres)

The effective volume of the trench (V) is the design storm run-off volume less the volume of rock within the trench (this commonly occupies 30 to 40 per cent of the trench volume).

The depth of the trench can be estimated from:

$$d = \frac{It}{S}$$

where

I = infiltration rate (metres per hour)

t = infiltration time (hour)

S = factor of safety

Due to the difficulty in obtaining reliable percolation rate estimates, the Washington State Department of Ecology (1993) recommends making several site measurements then adopting the lower value, in addition to adopting a factor of safety of two.

More accurate and comprehensive field measurements could result in a lowering of this factor of safety. Estimates of the infiltration rate can be obtained from soils texts, based on soil textural classes. If this approach is taken, a higher factor of safety would be appropriate.

The choice of an infiltration period is related to the inter-event period and the need to minimise anaerobic conditions in the underlying soil (during warmer periods, these encourage the growth of algae which may clog the soil). Reducing the infiltration time results in a smaller volume but higher surface area for a given soil type. Pollutant removal is enhanced by increasing the surface area of the bottom of the trench, which also reduces the risk of clogging.

Infiltration periods of 24 to 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. Auckland Regional Council (ARC) (1992) adopts an infiltration period for the mean storm of at least 50 per cent of the mean inter-storm period. As the majority of the infiltration occurs during the inter-event period, an approximate infiltration period could be determined from an analysis of the site's rainfall data history. Similar criteria to those of ARC (1992) could then be applied. These analyses have been carried out for major cities in Australia by Wong et al. (1998).

The second sizing technique is that described by Horner et al. (1994). This method calculates 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 (see Appendix F). Argue et al. (1998) have outlined a number of design procedures for sizing of infiltration trenches in Australian catchments.

Trench configuration

Pre-treatment of run-off entering an infiltration trench is necessary for removing coarse particulates that can cause clogging. This pre-treatment may consist of a grass filter strip, grass swales, a sand filter or primary treatment measure.

The length and width of the trench will be determined by the site characteristics. If stormwater is conveyed to the trench as uniform sheet flow, the length of the trench perpendicular to the flow direction can be maximised. If run-off is conveyed as channel flow, the length of the trench parallel to the direction of flow can be maximised. The base of the trench should be level, to evenly distribute exfiltration.

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

Clean, washed stone aggregate, typically 25 to 75 millimetres diameter, can be used as fill. Increasing the diameter of the stone will increase the effective volume of the trench. 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. To minimise migration of soil particles, filter fabric can extend to cover the top of the trench if (porous) topsoil is used.

An observation well can be installed through the media to permit trench water level monitoring.

Alternative infiltration trenches

Dry wells

Known as dry wells, small infiltration trenches can be designed to drain small areas (e.g. to capture roof run-off). These wells are suitable for use either with small individual commercial buildings or single family residences and rarely include pre-treatment. Roof run-off is delivered via a downpipe into the upper portion of the stone reservoir. The stone reservoir may be located about 300 millimetres below the ground surface and can include an observation well for routine inspection. Run-off exceeding infiltration capacity is discharged to the surface via an overflow pipe located within the downpipe.

Pervious pipes

These systems comprise a pipe network that is perforated along its length, allowing exfiltration of water through the pipe wall as it is conveyed downstream. Pre-treatment to remove coarse sediment is appropriate to prevent blocking of the perforations. To promote exfiltration, pervious pipe systems can be implemented with reasonably flat slopes (0.5 per cent). Double pipe systems can be used, comprising a regular drain pipe over a perforated pipe. This is more expensive, but provides a contingency conveyance system if the perforated pipe becomes clogged. In addition, the perforated pipe can be plugged during the construction phase until the site has stabilised.

Trench construction considerations

Prior to development of the site, the proposed infiltration trench area should be fenced off to prevent heavy equipment compacting underlying soils. To increase infiltration, the base of the trench should be ripped prior to placing the rock fill. Compaction of the base of the trench should not be performed. Clean rock fill only should be used. Infiltration tests on the base of the trench should be carried out prior to the placement of the rock fill.

During the construction phase, sediment and run-off should be diverted away from the trench area to minimise the potential for blocking the trench. Operation of the trench should not commence until the catchment has stabilised.

Appendix E: Design guide for extended detention basins

Basin outlet design

The proper design of an extended detention basin outlet is critical to its performance. Fundamental to its optimum operation are the hydraulic characteristics of the outlet structure.

Potential outlet types include:

- weir: this is a common outlet structure but often does not provide adequate hydraulic control over the full depth range of the extended detention storage, owing to its efficient discharge characteristics. Extended detention systems controlled by a weir outlet often have very low depth fluctuation for most of the storm events, with the desired detention period only provide for the design storm event. Difficulties can be experienced achieving low release rates at low heads, even with the use of a V-notch weir. An alternative is a proportional discharge weir.

- perforated riser: this outlet structure consists of vertical riser pipe with small orifices located along its length. Gravel can be placed around the riser pipe to act as a filter but special care will need to be taken to prevent blockage of the orifices. Inverted pipe elbows are commonly used to alleviate this problem. Risers promote the narrowest range of detention period for a wide range of stormwater inflow characteristics. Risers can be designed to match the stage-storage relationship of the extended detention basin, so a near-linear storage-discharge relationship is achieved. Such characteristics would ensure that there is a consistent period of stormwater detention in the system regardless of the size of the inflow event.

- culverts and reverse slope pipe: culvert outlets are common in extended detention systems such as flood retarding basins and water quality detention ponds. Reverse slope pipes are applicable where there is a deep pool located at the outlet of the basin and have similar hydraulic characteristics (especially under design conditions) to conventional culverts. The reverse sloped pipe can be used to draw water from below the surface of the pool, thereby avoiding floating materials in the basin. This is particularly useful where blockages from floating material may be a problem. A gate valve may also be installed in the reverse slope pipe to allow the drawdown time to be modified to improve pollutant removal.

Further details on outlets are provided in Schueler (1987), OMEE (1994), and Somes and Wong (1998).

The sizing of the basin for a single design event does not guarantee that adequate residence time will be achieved for all events—smaller events may pass through the basin in a period too short for effective treatment. Stage or depth to discharge relationships of the outlet structure define the distribution of detention period over the operating life of the detention basin. The stage-discharge relationships for the outlet structures can be established using conventional hydraulic equations for weir flow and orifice/culvert flow.

If sizing the extended detention basin is to be based on the adoption of a design storm, it will be necessary to examine the operation of the storm for a number of storm durations to define the critical storm duration corresponding to the design average recurrence interval. The equation for the adopted outlet type can be specified in a rainfall run-off model of the basin, which also includes a stage-storage volume relationship. The design storm events are then routed through the storage to determine the storage requirement and residence time.

The use of a culvert is common practice in extended detention systems. An approximate technique for sizing the outlet (cited in CDM 1993) can be applied when side slopes of the basin 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 (square metres)

A = average surface area of pond (square metres)

Δh = difference between full and empty levels (metres)

c = discharge coefficient (normally between 0.67 and 0.80)

t = draw down time (hours)

g = gravitational acceleration constant (9.81 metres per second^2)

If the residence time for small events is found to be inadequate, the hydraulic characteristics of the outlet structure can be modified (e.g. adoption of a riser outlet structure) or a two-stage outlet used. This can involve incorporating a second culvert at the half-basin height. The lower culvert can be designed to drain half the basin in 24 hours, while the combined capacity of the two culverts is designed to drain the entire basin in 40 hours.

The design of risers uses the same hydraulic equations as that for culverts. The establishment of the stage-discharge relationship for the riser, however, will need to integrate the respective discharges of the orifices along the vertical length of the riser pipe at every stage increment. Wong et al. (1999) provide an empirical relationship for selecting the orifice area for a riser to promote near linear storage-discharge relationship. This was based on fitting a curve of best fit to a height-discharge relationship derived theoretically for a riser with orifices placed at 250 mm interval and is:

$$Q = c_d A_{0.25} \sqrt{2g}(1.07y^2 + 2.05y + 0.036)$$

where

c_d is the orifice discharge coefficient (~0.67)

$A_{0.25}$ is the area of the orifice to be placed at 0.25 m interval along the riser

g acceleration due to gravity (i.e. 9.81 m/s^2)

Basin geometry and layout

Sedimentation is the primary pollutant removal process in extended detention basins. There are a number of features that should be considered during the design:

- effective residence time: it is important that the inflow volume is uniformly distributed within the basin volume and short circuiting is minimised. This can be achieved by a basin 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 overcome problems with the inlet being too close to the outlet, berming may be included to redirect flows.
- velocity distribution: sedimentation will be enhanced by low flow velocities and avoiding strong flow jets at the inlet to the basin. These can be minimised by installing energy dissipators 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 metres may be appropriate.
- off-line basin: by-passing high flows above the design storm flow can reduce the possibility of scouring. Alternatively, flood storage could be provided above the permanent water storage, which will reduce flow velocities through the basin.
- stabilised low flow path: to minimise scouring during frequent events.
- grassing of the basin floor: this helps filter the sediment and assists in binding sediment to the basin floor.

Other aspects of the basin geometry include:

- side slopes: for grassed basins, the basin side slopes should be designed to meet safety and maintenance requirements. Maximum slopes of 8:1 may be appropriate. Steeper side slopes may be used in ungrassed areas, for example where retaining walls or shrub beds are used and are provided with safety fences.
- basin floor: the basin floor should be designed to drain freely. A slope steeper than 1 to 2 per cent is suggested. Flat slopes can result in difficulties with grass mowing and

mosquito breeding. Grass species planted on the basin floor should be tolerant of frequent inundation. Subsoil drains may be provided to address this problem.

- groundwater: depth to groundwater should be considered during the design. If the water table is high, problems may be experienced with grass mowing—the grass species may need to be tolerant of the groundwater chemistry.
- access: vehicular access for maintenance should be provided.
- downstream outlet: an energy dissipator should be considered at the downstream end of the outlet pipe from the basin, to minimise erosion of downstream waterways.

Appendix F: Design guide for sand filters

Large sand filters

Sizing

There are two key 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 or using design curves based on continuous simulations techniques such as that developed for infiltration systems by Argue (1999). High flows in excess of the design storm can be designed to by-pass 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 achieving suspended solid retention of 60 to 75 per cent during a design storm and a drawdown time of 24 hours. A perforated riser pipe can be used as the outlet for the basin, as described in Section 7.8 (Secondary treatment Type No. 6: Extended detention basins).

ARC (1992) recommends a settling basin with a permanent pool. This pool can be sized to achieve the same retention as the extended detention settling basin, using settling velocity theory or the retention curves contained in Section 7.9 (Constructed wetlands).

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

$$A = \frac{Vw}{Kt(h + D)}$$

where

A = area of filter (square metres)

V = volume to be infiltrated (cubic metres)

K = hydraulic conductivity (metres per hour)

t = drainage time (hours)

h = average head above filter (half the storage depth—metres)

w = thickness of the filter layer (metres)

D = depth of filter (metres)

ARC (1992) recommends a hydraulic conductivity of 0.033 metres per hour, which corresponds to a system with partial and full pre-treatment (City of Austin 1988). This value is less than the typical conductivity of new sand media and hence allows for some clogging.

A minimum media depth of 0.4 metres is recommended by ARC (1992). A filtration time of 24 hours is recommended by CDM (1993) and City of Austin (1988). ARC (1992) recommends a filtration period determined according to rainfall patterns at the proposed site—a 16 hour period was used, corresponding to one third of the mean inter-event period. This approach was adopted so that the filter has a dry period between events, to maintain aerobic conditions and hence long term infiltration capability.

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;
- minimise flow velocities to prevent resuspension (e.g. less than 0.3 metres per second);
- effective use of storage volume, that is, minimising short circuiting. A length to width ratio of at least 5:1 could be adopted;
- access for maintenance; and
- trash rack at the settling basin outlet.

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 may be used for this purpose; and
- a geotextile fabric over a coarse gravel layer above the under-drain.

CDM (1993) adopts a sand size of between 0.5 and 1.0 millimetres, while the City of Austin (1988) uses sand sized at 0.25 to 0.5 millimetres. ARC (1992) recommends 10 per cent of sand should pass a 63 micrometre sieve and 90 per cent should pass a 500 micrometre sieve.

Small sand filters

Sizing

Two key components need to be sized for small sand filters; the upstream sedimentation chamber and the filter.

These components can be designed on the basis of a design storm event or using design curves based on continuous simulation techniques, such as that developed for infiltration systems by Argue (1998). High flows in excess of the design storm can be designed to by-pass the filter. The sedimentation chamber can be designed using sediment basin sizing techniques described in Appendix B and the filter designed using the technique described in Appendix C.

Geometry

Shaver (1996) recommends a number of points for small sand filter geometry. These include:

- run-off should enter the sand filter by overland flow or gutter flow;
- a grated cover should be provided over the sedimentation chamber;
- a weir should be fitted between the sedimentation chamber and the filter to obtain sheet flow over the filter (the sediment chamber should have no outlets other than the overflow to the filter);
- a minimum depth of sand in the filter of 450 millimetres;
- a minimum distance of 150 millimetres between the top of the outflow pipe and the top of the sand filter. For a 450 millimetre deep filter, this equates to a maximum outlet pipe diameter of 300 millimetres. Multiple outlet pipes may be used if required;
- a screen covered with filter fabric should be installed at the outlet pipe to prevent loss of sand from the filter; and
- a maximum sand particle size of two millimetres.

Appendix G: Derivation of constructed wetland sizing technique

The wetland system sizing technique presented in Section 6.9 of this document is an extension of the work undertaken by Duncan (1997b) at the Cooperative Research Centre for Catchment Hydrology (CRCCH) on the pollutant removal efficiency of ponds and wetlands. The original work by Duncan (1997b) investigated the relationship between a range of factors and the pollutant output percentage. This is 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 eighty-eight Australian and overseas studies were collected by the CRCCH for use in this analysis (Duncan 1997a).

A review was undertaken of all studies before their inclusion in the analysis, to assess both the design of the pond or wetland against 'good practice' and the quality of the monitoring data. Consequently, a weighted scoring system was developed for evaluating the quality of both the pond/wetland design and the sampling data used to evaluate the performance of the pond/wetland. This approach was adopted to avoid biasing the overall results with the incorporation of data from poorly monitored studies or poorly designed ponds/wetlands. 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 G1 and is based on allocating high scores to factors likely to enhance performance. The use of the design index was considered to provide a reasonable basis for combining the pond and wetland data. This increased the number of studies available for analysis, by effectively eliminating poorly designed ponds, which are unlikely to reflect the expected performance from constructed wetlands.

The data index is presented in Table G2 and is weighted towards the factors that are most important in achieving an accurate determination of the pond or wetland's long-term performance.

Only those 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 per cent) 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 suspended solids (SS), so sites with low design index scores were therefore included in the analysis for this variable. The resulting studies used in the analysis are noted in Table G3.

Factor	Condition	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 metre	1
	▪ Intermediate depth or unknown	0
	▪ Mean depth > 2 metres	–1
Potential score range		–4 to 5

Table G1 Design index.

Factor	Condition	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 2–6 months	2
	▪ Duration < 2 months	1
	▪ Unknown	*
		6
Number of events	▪ Number > 10	
	▪ Number 6–10	
	▪ Number < 6	
	▪ Unknown	
Land-use	▪ Urban > 75 per cent	2
	▪ Urban 50–75 per cent	1
	▪ Urban < 50 per cent	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

Table G2 Data index.

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

- area ratio: pond/wetland area 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 run-off volume;
- hydraulic loading rate: the ratio of the estimated average annual run-off volume and the surface area of the pond/wetland (also known as the upflow or overflow rate); and
- inflow concentration.

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 and 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)

SS* = suspended solids   TP† = total phosphorous   TN‡ = total nitrogen

Table G3 Studies used in regression analysis.

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

The analysis results indicated that the inflow concentration was a statistically significant variable (at the 5 per cent level) only for suspended solids. The correlation coefficients (R2) from this log10-domain analysis are indicated in Table G4. 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 retention—particularly nitrogen—is consistent with a number of previous pond and wetland performance studies (e.g. Schueler et al. 1992).

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

Parameter	R2 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 per cent level
** inflow concentration was also a significant explanatory variable

Table G4 Correlations between pond/wetland performance and explanatory variables).

The resulting regressions between log hydraulic loading rate and log output percentage are presented in Figure G1. These regressions were used to derive the pollutant retention curves presented in Figure 7.34.

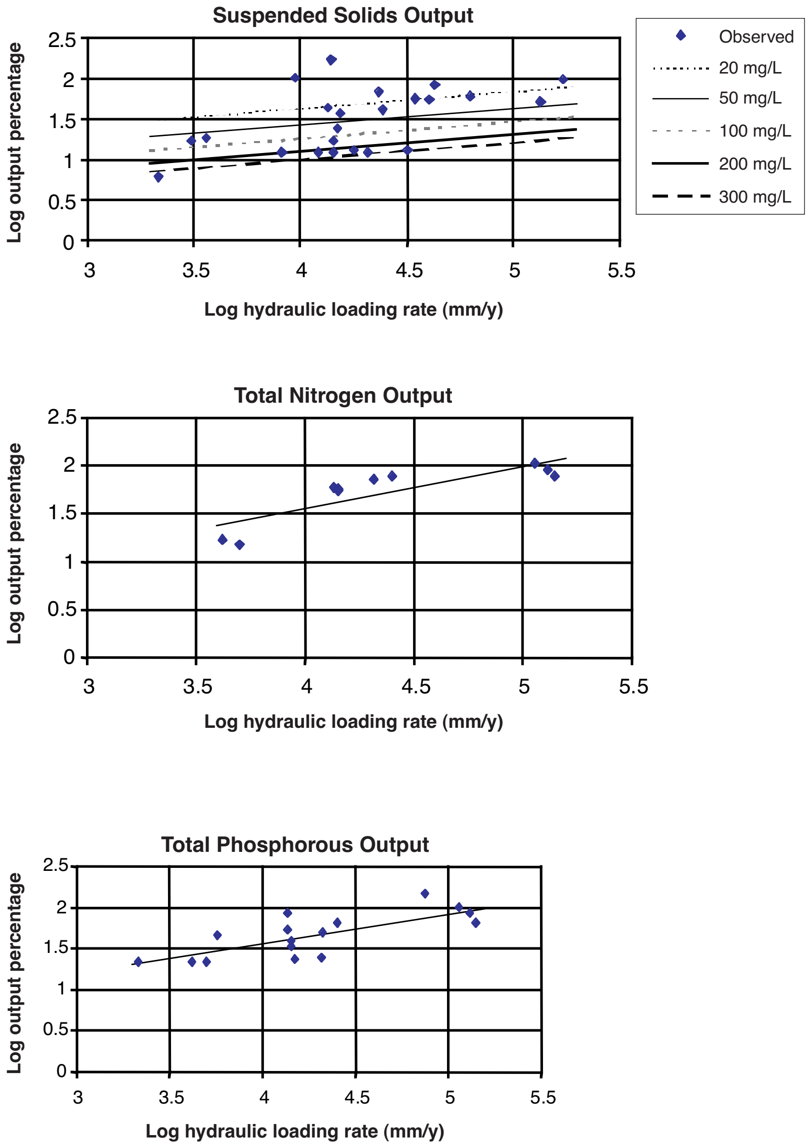

Figure G1 Pond/wetland pollutant output relationships.

Appendix H: Vegetation bands for wetlands

Ephemeral swamp

Typical ecological characteristics

Dominant species: e.g. Eucalyptus, Melaleuca, Poa, Juncus

Vegetation: 2 metre woodland overstorey, low-high density open-closed canopy, approximately 0.5 metres low-high density grassland-rushland groundcover

Typical physical characteristics

Surface area:volume ratio: high (when inundated)

Water depth: ~0.1 to 0.2 metres

Natural water regime: ephemeral (mostly dry, occasional irregular inundation cycle)

Potential treatment processes and mechanisms

Solids removal: sedimentation and filtration (particularly of fine particles)

Mineralisation: microbial growth, enhanced by wetting and drying

Nutrient uptake and transformation: microbial and macrophyte growth

Nutrient storage: sediment adsorption

Shallow marsh

Typical ecological characteristics

Dominant species: e.g. Eleocharis acuta (Common Spike-rush)

Vegetation: 0.3 to 0.7 metres, low-medium density open canopy, typically supports epiphytic algae on submerged culms

Typical physical characteristics

Surface area:volume ratio: high

Water depth: approximately 0.1 to 0.2 metres

Natural water regime: ephemeral (regular seasonal dry cycle)

Potential treatment processes and mechanisms

Aeration: surface exchange and epiphytic photosynthesis

Solids removal: filtration (surface adhesion)

Mineralisation: microbial growth, enhanced by wetting and drying

Nutrient uptake and transformation: microbial, epiphyte and macrophyte growth

Nutrient storage: sediment adsorption

Marsh

Typical ecological characteristics

Dominant species: e.g. Bolboschoenus medianus (Marsh Club-rush)

Vegetation: 0.5 to 1.5 metres high, high density closed canopy, high litter production.

Typical physical characteristics

Surface area:volume ratio: medium-high

Water depth: approximately 0.3 metres

Natural water regime: ephemeral (occasional-regular dry cycle)

Potential treatment processes and mechanisms

Solids removal: sedimentation and filtration

Mineralisation: microbial growth

Nutrient uptake and transformation: microbial and macrophyte growth

Nutrient storage: sediment adsorption and litter accumulation

Deep marsh
Typical ecological characteristics Dominant species: e.g. Schoenoplectus validus (River Club-rush) Vegetation: 1–2 metres, medium-dense, semi-closed canopy, supporting some epiphytic algae, moderate litter production **Typical physical characteristics** Surface area:volume ratio: medium Water depth: approximately 0.4 to 0.6 metres Natural water regime: permanent (occasional irregular dry cycle) **Potential treatment processes and mechanisms** Solids removal: sedimentation and filtration Mineralisation: microbial growth Nutrient uptake and transformation: microbial, epiphyte and macrophyte growth Nutrient storage: sediment adsorption and litter accumulation
Open water
Typical ecological characteristics Dominant species: algae (or submerged macrophytes in low nutrient conditions) Vegetation: phytoplankton growth resulting in secondary solids production (macrophyte growth inhibiting mixing and removing solids by sedimentation and filtration) **Typical physical characteristics** Surface area:volume ratio: low Water depth: 1.5 to 2.0 metres Natural water regime: permanent, generally well mixed, but may stratify during still conditions, particularly in the warmer months **Potential treatment processes and mechanisms** Solids removal: sedimentation (and filtration) Aeration: wind mixing, algal photosynthesis Sterilisation: UV exposure Nutrient uptake and transformation: phytoplankton and submerged macrophyte growth Nutrient storage: sediment adsorption and accumulation
After Somes et al. (1996).

Appendix I: State Planning Policy Framework

General implementation

Catchment planning and management

Planning authorities must have regard to relevant aspects of:

- any regional catchment strategies approved under the Catchment and Land Protection Act 1994 and any associated implementation plan or strategy, including regional vegetation plans, regional drainage plans, regional development plans, catchment action plans, landcare plans and management plans for roadsides, soil, salinity, water quality and nutrients, floodplains, heritage rivers, river frontages and waterways.
- any special area plans approved under the Catchment and Land Protection Act 1994.

Planning and responsible authorities should coordinate their activities with those of the Boards of catchment management authorities appointed under the Catchment and Land Protection Act 1994 and consider any relevant management plan or works program approved by a catchment authority.

Planning and responsible authorities should coordinate their activities with those of the Boards of catchment management authorities on downstream water quality and freshwater, coastal and marine environments, and where possible, should encourage:

- the retention of natural drainage corridors with vegetated buffer zones at least 30 metres wide along waterways to maintain the natural drainage function, stream habitat and wildlife corridors and landscape values; to minimise erosion of stream banks and verges; and to reduce polluted run-off from adjacent land-uses.
- measures to minimise the quantity and retard the flow of stormwater run-off from developed areas.
- measures, including the preservation of floodplain or other land for wetlands and detention basins, to filter sediment and wastes from stormwater prior to its discharge into waterways.

Responsible authorities should ensure that works at or near waterways provide for the protection and enhancement of the environmental qualities of waterways, and that their instream uses and are consistent with Guidelines for Stabilising Waterways (Rural Water Commission 1991) and Environmental Guidelines for River Management Works (Department of Conservation and Environment 1990), and that they have regard to any relevant river restoration plans or waterway management works programs approved by a catchment authority.

Water quality protection

Planning and responsible authorities should ensure that land-use activities potentially discharging contaminated run-off or waste to waterways are sited and managed to protect the quality of surface water and groundwater resources, rivers, streams, wetlands, estuaries and marine environments.

Incompatible land-use activities should be discouraged in areas subject to flooding, severe soil degradation, groundwater salinity or geotechnical hazards where the land cannot be sustainably managed to ensure minimum impact on downstream water quality or flow volumes.

Planning and responsible authorities should ensure land-use and development proposals minimise nutrient contributions to waterways and water bodies and the potential for the development of algal blooms, consistent with the Preliminary Nutrient Guidelines for Victorian Inland Streams (EPA 1995), the Victorian Nutrient Management Strategy

(Government of Victoria 1995) and any nutrient or water quality management plans approved by the government.

Responsible authorities should use appropriate measures to restrict sediment discharges from construction sites in accordance with construction Techniques for Sediment Pollution Control (EPA 1991) and Environmental Guidelines for Major Construction Sites (EPA 1995).

Planning and responsible authorities should utilise mapped information available from the Department of Natural Resources and Environment to identify the beneficial uses of groundwater resources and have regard to potential impacts on these resources of proposed land-use or development.

General implementation

Planning and responsible authorities should ensure that water quality in water supply catchments is protected from possible contamination by urban, industrial and agricultural land-uses.

Urban development must be provided with sewerage at the time of subdivision, or lots created by the subdivision must be capable of adequately treating and retaining all domestic wastewater within the boundaries of each log consistent with the Code of Practice: Septic Tanks (EPA 1996) and relevant State environment protection policies.

Planning and responsible authorities should ensure that:

- planning for urban stormwater drainage systems considers the catchment context and is coordinated with adjacent municipalities;
- best environmental management practice is used where practicable in the design and management of urban stormwater drainage systems, including measures to reduce peak flows and assist screening, filtering and treatment of stormwater, to enhance flood protection and minimise impacts on water quality in receiving waters; and
- drainage systems are protected where practicable from the intrusion of litter, in accordance with strategies set out in Victoria's Litter Reduction Strategy (EPA 1995).

The re-use of wastewater including urban run-off, treated sewage effluent, and run-off from irrigated farmland should be encouraged where appropriate, consistent with the Guidelines for Wastewater Re-use (EPA 1996).