Chapter 12 **Rainwater tanks**

Slim-line rainwater tank

12.1 Introduction

The core sustainability objective of using **rainwater tanks** is to conserve mains water. In addition to conserving mains water, rainwater tanks help to protect urban streams by reducing **stormwater** runoff volumes, particularly from small storms, and associated stormwater pollutants from reaching downstream waterways. Rainwater and stormwater harvesting on individual allotments are some of the initiatives that can be implemented to deliver such a potable water conservation objective.

Another important household initiative to conserve water is the use of AAA plumbing fittings and AAA and AAAA appliances. These are often adopted as a first priority in water conservation initiatives as they are easy to adopt, have high cost effectiveness and broader environmental benefits such as reduced wastewater **discharges**. Recent research (Melbourne Water 2001) has found that the adoption of AAA rated showerheads and dual flush toilets can reduce indoor water use by 15%–20% (11%–15% of total internal and external water use). Following improving the efficiency of water use within a household, finding supplementary sources for water is fundamental to further reducing demand on mains water. The use of rainwater tanks to collect roof runoff is an accepted means of supplementing mains water supplies which is simpler to implement than other potential alternative water sources such as greywater or surface stormwater.

There are no quantitative performance targets (e.g. size of tank, targeted reductions in potable demand) in any existing local government and state authority policies and guidelines regarding the use of rainwater tanks. However, it can be inferred from the various policies and guidelines that do exist that a performance target for rainwater tanks (or any other form of rainwater and stormwater harvesting, storage and reuse scheme) is to provide a 'reliable' supply of suitable quality water to meet the demand requirements of a stipulated preferred 'end-use' (e.g. toilet flushing).

This design procedure focuses on factors associated with selecting and using a rainwater tank, including sizing rainwater tanks such that they will provide a reliable source of water to supplement mains water supply. Variables that need to be considered in sizing a rainwater tank include the size or area of roof directed to a tank, the quantity and nature of the demand and the rainfall pattern of a particular area.

12.2 Rainwater tank considerations

The use of rainwater tanks to reduce demand on reticulated potable water supplies and stormwater runoff volume needs to consider several issues as follows.

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

These issues are further discussed below.

12.2.1 Supply and demand considerations

Supply and demand considerations should be examined during the concept investigation phase of a project. Nevertheless, several key considerations are discussed below to ensure that they are sufficiently addressed before implementing a rainwater harvesting scheme.

Low roof area to occupancy ratio

An obvious limitation of rainwater tanks as an alternative water source is where a roof area is too small to yield sufficient runoff for a cost-effective supply of water. This situation is most likely to arise on projects with medium and high density residential dwellings (i.e. where the ratio of roof area to the number of occupants in the dwelling is low). In these situations, it is probably most important to maximise the use of water efficient fittings and appliances to reduce the demand on the reticulated water supply so that the additional supply opportunities that are presented by a rainwater tank are maximised.

A smaller ratio of roof area to number of occupants (i.e. increasing density) has the effect of increasing the size of rainwater tank required to deliver a given ***reliability*** of supply (the percentage of water demand that is met by that supply) to the connected end uses. With high density, multistorey developments (>4.5 people/100 m^2 of roof), there is a diminishing opportunity for the effective use of roof water for all households as a means of supplementary supply.

Increasing the number of end uses connected to a tank (e.g. laundry and garden in addition to toilets) will reduce the reliability of the supply. While the reliability decreases with increasing end uses, the total use of available rainwater increases because there is a greater frequency of drawdown and reduced frequency of overflow.

The reliability of supply, thus, may not necessarily be a concern if potable water is available to supplement a supply (e.g. as s mains water top-up). The cost of connection/plumbing to a greater number of end uses, and the additional complexity of the in-house water reticulation system, may increase and this could reduce the beneficial effect of the total potable water savings resulting from additional end uses. As a general rule, it is recommended that the reliability should be at least 50% for a viable reticulation system.

Low rainfall regions

The effectiveness of rainwater tanks as a supplementary water source is reduced in low rainfall regions such as Northern Victoria (e.g. Mildura's Mean Annual Rainfall (MAR) = 306 mm compared to Melbourne = 660 mm). The reduced rainfall and the higher seasonality effect in these areas can often lead to significant increases in rainwater tank size to achieve a similar level of supply reliability (and, hence, the cost-effectiveness reduces).

The use of rainwater tanks on projects in the north-western region of Victoria (and other similarly low annual rainfall regions) will need to consider carefully the viability of tanks as a cost-effective alternative water source. Other potential water sources such as reclaimed water and/or greywater reuse may need to be given greater consideration in these regions as these water sources are independent of local climatic conditions and can provide a higher reliability of supply.

12.2.2 Water quality

Water quality is an important consideration with all roof water systems, especially in urban and industrial areas. Possible pathways for contamination of roof water are:

- atmospheric pollution settling onto roof surfaces
- bird and other animal droppings with bacteria and gastrointestinal parasites
- insects, lizards and other small animals becoming trapped and dieing in a tank
- Roofing materials and paints – lead based paints in particular should never be used on roofs where water is collected for potable water uses; tar-based coatings are also not recommended, as they may affect the water's taste; zinc can be a significant pollutant in some paints and galvanised iron or zincalume roofs (particularly when new) should not be collected for potable use
- detergents and other chemicals from roofs painted with acrylic paints can dissolve in the runoff; runoff from roofs made of fibrous cement should be discarded for an entire winter due to the leaching of lime
- chemically treated timbers or lead flashing should not be used in roof **catchments** and rainwater should not be collected from parts of the roof incorporating flues from wood burners
- overflows or discharge pipes from roof mounted appliances, such as evaporative air-conditioners or hot water systems, should not discharge onto a roof catchment or associated gutters feeding a rainwater tank.

The presence of these contamination pathways will vary between projects and will largely depend on:

- proximity of the project to areas of heavy traffic, incinerators, smelters or heavy industry, and users of herbicide and pesticides (e.g. golf course, market gardens)
- roofing materials and roof-mounted appliances
- provision of a well-sealed rainwater tank with a first flush device and with inlet and overflow points provided with mesh covers to keep out materials such as leaves and to prevent the access of mosquitos and other insects.

The quality of roof water collated from relevant Australian studies is summarised and further discussed in Engineers Australia (2003).

Water quality requirements of an end use connected to a rainwater tank will determine whether or not additional water quality treatment needs to be provided between the tank and the end use. For all non-potable uses (e.g. toilet flushing, washing machines, garden watering) available monitoring data indicates that typically there are low levels of risk to consumers if additional water quality treatment (e.g. disinfection) is not provided (Coombes 2002). One exception in this regard is where a rainwater tank is connected to the hot water system where there is a heightened potential of human ingestion of rainwater (e.g. when showering, children in the bath). If connected to the hot water system, some disinfection is required which may include providing hot water at a certain temperature (to allow for complete pasteurisation) or other disinfection methods (e.g. chlorination).

12.2.3 Stormwater quality benefits

Using collected rainwater reduces the total volume of stormwater runoff from a site and therefore reduces pollutant discharges. The percentage reduction of stormwater from a site can be estimated based on the reuse demand, reuse reliability and MAR.

Percentage reduction = *reliability* × *reuse demand* (kL)/*Rainfall volume* (m^3)

Where, reuse demand = average annual toilet flushing demand (assumed to be 8 kL/person per year) × no. of household occupants

Rainfall volume = MAR (m) × contributing roof area (m^2)

For example, the percentage reduction in stormwater from a rainwater tank that provides 70% reliability for a house in Bendigo (MAR 570 mm) with three occupants and a roof area of 120 m^2 is calculated as follows:

Stormwater reduction = 70% × (8 kL/person per year × 3 people)/(0.57 m × 120 m^2) = 25%.

Therefore, the reduction in stormwater runoff and hence Total Soluble Solids, Total Phosphorus and Total Nitrogen loads from the roof due to reuse from the rainwater tank is 25%.

Additionally, rainwater tanks provide some treatment of water that is not removed from the tank for reuse (i.e. water that is stored for some period and then spills when the tank overflows). The dominant process is the settlement of suspended solid loads. The reduction in pollutant loads in water that is spilt from rainwater tanks is likely to be small compared with the reduction due to the removal from the system.

12.2.4 Cost considerations

Typically the cost of a rainwater tank installation for supplementary water source ranges from \$1200 to \$2000 for residential detached or semi-detached dwellings. Three cost components are normally involved: the tank, installation and plumbing, and a pump. Costs may increase with higher density development as space constraints could require more specialised tanks to be fitted (see Section 12.2.5) unless communal use of a centralised rainwater tank can be facilitated.

The typical payback period of a rainwater supply system purely through a reduction in domestic water charges is about 35 years under current water pricing and will often not be able to justify the use of rainwater as an alternative source of water to mains water. This is mostly because the present pricing of mains water does not reflect the true environmental and social cost of the water resource. Terms such as 'total resource cost' and 'total community cost', in addition to the more commonly used terms of 'life cycle cost' and 'whole of life cost', have emerged in recent analysis of the value of water. These terms are meant to more holistically reflect the beneficial outcomes associated with water conservation practices through the adoption of alternative water sources and associated matching of their respective water quality with fit-for-purpose usage. When such 'total resource cost' issues, and the potential benefits of rainwater capture/reuse in regard to reduced stormwater flows are considered, more positive economic benefits can apply (Coombes 2002).

12.2.5 Available space considerations

Small allotments with large building envelopes are becoming more common as dwindling land stocks require the provision of smaller lots to meet increasing demand. However, the public's desire for 'traditional'-sized houses remains strong and as a consequence front and back yards are being reduced to allow large houses to be built onto progressively smaller allotments. This phenomenon imposes a potential constraint on the use of rainwater tanks where tanks are installed external to a building and above ground (as is conventional practice). Competing demand for the use of external areas raises the potential for resistance to the imposition of rainwater tanks on small allotments with large building envelopes. This can be overcome by

Figure 12.1 A slim-line tank (left) and modular rainwater tank system (right).

burying tanks or placing them underneath houses but these techniques have associated cost implications for construction and maintenance.

Rainwater tank designs have advanced recently with slim-line rainwater tank designs reducing tank footprints. Modular rainwater tank systems are also now being developed. These systems can be interconnected to form boundary fences or potentially walls for a garden shed or carport. Eventually rainwater tanks will be developed that can be designed into the building floor or walls, thus removing any impost on the use of external areas. Rainwater tanks in buildings can also provide energy benefits through thermal inertia of the stored water moderating temperature variations within households. Examples of slim-line and modular rainwater tanks are shown in Figure 12.1.

The final decision on the acceptability of using rainwater tanks on small lots is likely to be influenced by the size of tank required (which is influenced by the available roof area and the water conservation outcome to be attained from a rainwater tank), the compatibility of commercially available tank systems with the built form and the available area for a tank. This decision needs to be made case-by-case.

12.2.6 Competing uses for stormwater runoff

There may be situations, especially on larger precinct-wide projects, where there may be one or more competing uses for stormwater runoff generated from roof areas and ground-level impervious surfaces. Rainwater tanks may not provide the optimal strategy from a sustainability perspective, especially when comparing the life cycle cost and resource use outcomes of a centralised stormwater harvesting scheme with a decentralised rainwater harvesting scheme. These issues need to be investigated thoroughly during the concept design stage of a project.

A common example of competing uses is associated with residential development adjoining public open spaces and golf courses. In development scenarios such as this, it is often more cost effective (from both a capital and asset maintenance perspective) to implement a precinct-wide stormwater harvesting scheme and supply the water for public open-space watering.

12.2.7 Maintenance considerations

Although the maintenance of a rainwater tank-based system to augment the mains supply is not particularly arduous for a property owner, it is nevertheless an additional requirement for households that normally would have their water supply sourced from a reticulated system. This may have possible long-term effects on the sustainability of a rainwater tank supply scheme, especially if homeownership changes. With more realistic water pricing policies and appropriate education practices, the impacts of this consideration should be minimal, however (see Section 12.4.4).

12.3 Australian standards for installation of rainwater tank systems

Rainwater tanks need to be installed in accordance with the Plumbing and Drainage Standards (AS/NZS 3500 2003).

Although not strictly a standard, rainwater should be sourced only from roof sources, and flows from roads, footpaths, and other common areas at ground level, are addressed through separate stormwater treatment processes. If supply is supplemented by an interconnection with a reticulated water supply, backflow prevention via either an air gap or proprietary device is required in accordance with Australian Standard ASNZS 3500.1.2 (1998) and the requirements of the local water supply authority. For treatment and usage it is suggested (Donovan 2003) that:

- the collection system should incorporate a first flush device or 'filter sock' to divert or filter initial runoff from a roof
- the tank system should be connected to the toilet, hot water, laundry and garden irrigation fixtures, and there should be no direct supply from the mains water to these services
- there should be no connection to other indoor fixtures from the rainwater tank unless measures are undertaken to make the supply fit for consumption
- the tank is enclosed and inlets screened, in order to prevent the entry of foreign matter and to prevent mosquito breeding
- overflow from a rainwater tank should be directed to a detention device, **swale** or stormwater drain.

12.4 Design procedure: rainwater tanks

Design considerations when evaluating a rainwater tank system include the following:

1. selection of end uses
2. determination of size and associated reliability relationship
3. hydraulic fixtures, such as
   - water filter or first flush diversion
   - mains water top-up supply
   - onsite detention provisions
4. maintenance provisions.

12.4.1 Selection of end-uses

Water consumption in a household varies depending on the type and location of the house. Typical water consumption figures for residential areas expressed on per capita are summarised (Table 12.1).

The effect of using water-efficient appliances on reducing the water demand when sizing rainwater tanks should be considered. Consumption of water for toilet flushing has reduced significantly since the mandatory introduction of dual flush toilets over a decade ago. Table 12.2 lists the likely reduction in indoor household water demands resulting from the adoption of such water efficient appliances.

The most obvious water uses for rainwater are toilet and garden supply as they avoid the requirement for treatment to potable standards. Replacement of mains potable water for toilet flushing is considered to be the more effective of the two because of its consistent demand

Table 12.1 Typical household water consumption in Melbourne (after Melbourne Water 2001)

Water uses	Per person usage (kL/person per year)	Percentage of total usage (%)
Garden	32	35
Kitchen	5	5
Laundry	14	15
Toilet	18	19
Bathroom	24	26
Total	92	–
Hot water	24	26

Table 12.2 Estimation of reduction in water demand by water efficient appliances (after New South Wales Department of Infrastructure Planning and Natural Resources 2004)

Water uses	Conventional demand (kL/person per year)	Reduced demand with water efficient appliances and fittings (kL/person per year)
Shower	20.8	13.5
Bath	3.2	3.2
Hand basin	2.2	1.2
Toilet	12.8	7.3
Washing machine	17.0	11.9
Kitchen sink	4.4	2.3
Dishwashing	1.1	0.6
Total	61.5	40.0

pattern and, thus, a higher reliability of water supply can be achieved for a given size of rainwater tank. While having a higher water demand, water usage for garden watering is seasonal and the demand pattern is 'out-of-phase' with the supply pattern (i.e. high garden watering demand occurs during low rainfall periods) and thus a larger rainwater tank storage may be required to achieve comparable reductions in potable water usage compared with toilet flushing.

The next appropriate use of rainwater, after the use of rainwater for toilet flushing and garden watering, is in the laundry (e.g. washing cold tap). Supplementing the supply for hot water is also an effective option. Hot water usage constitutes about 30% of household indoor usage. The quality of water delivered from a rainwater tank via a hot water system is improved by the combined effects of high temperature pasteurisation, pressure in the pump and the instantaneous heat differentials between the rainwater tank and a hot water service.

12.4.2 Tank size and supply reliability

The supply reliability of a rainwater tank is directly influenced by three factors:

1. Supply characteristics – as defined by the size of the catchment (i.e. roof area connected to the rainwater tank) and the rainfall pattern of a region (MAR and seasonal pattern).
2. Demand characteristics – as defined by the type of uses. If indoor use, this depends on household occupancy and if for garden watering, demand depends on garden design and climatic conditions of the region.
3. Storage size.

Because rainwater is intermittent, the most appropriate analytical approach for assessing the reliability of supplies is a continuous simulation (modelling) approach using long records of rainfall data. Engineers Australia (2003) provide a detailed discussion on appropriate modelling techniques for determining a relationship between tank size and rainwater supply reliability.

A simple generic procedure that covers all regions of Victoria is presented here for selecting rainwater tanks for toilet use. The procedure is based on continuous simulations of the performance of rainwater tanks of varying sizes to meet toilet flushing demands (assumed to be 20 L/person per day) for the 45 **pluviographic** stations used in determining the treatment measure performance described in Chapter 2. Household occupancies equivalent to 1.5 persons, 2.5 persons, 3.5 persons and 4.5 persons per 100 m^2 of roof area catchment (i.e. the roof area directed to a tank) were used to represent the scenario of increasing development density. Melbourne was selected as the reference site. This procedure is only applicable for roof water used for toilet flushing (or any indoor water usage that is highly correlated to household occupancy).

For any assessments evaluating more widespread usage of rainwater, rigorous assessments using models such as PURRS, AQUACYCLE and UVQ are recommended (see Engineers Australia 2003, chapters 5 and 13). Assessments for more widespread use have already been conducted for Melbourne (Coombes and Kuczera 2003). These assessments demonstrate the significant benefits that can be gained from rainwater tank systems.

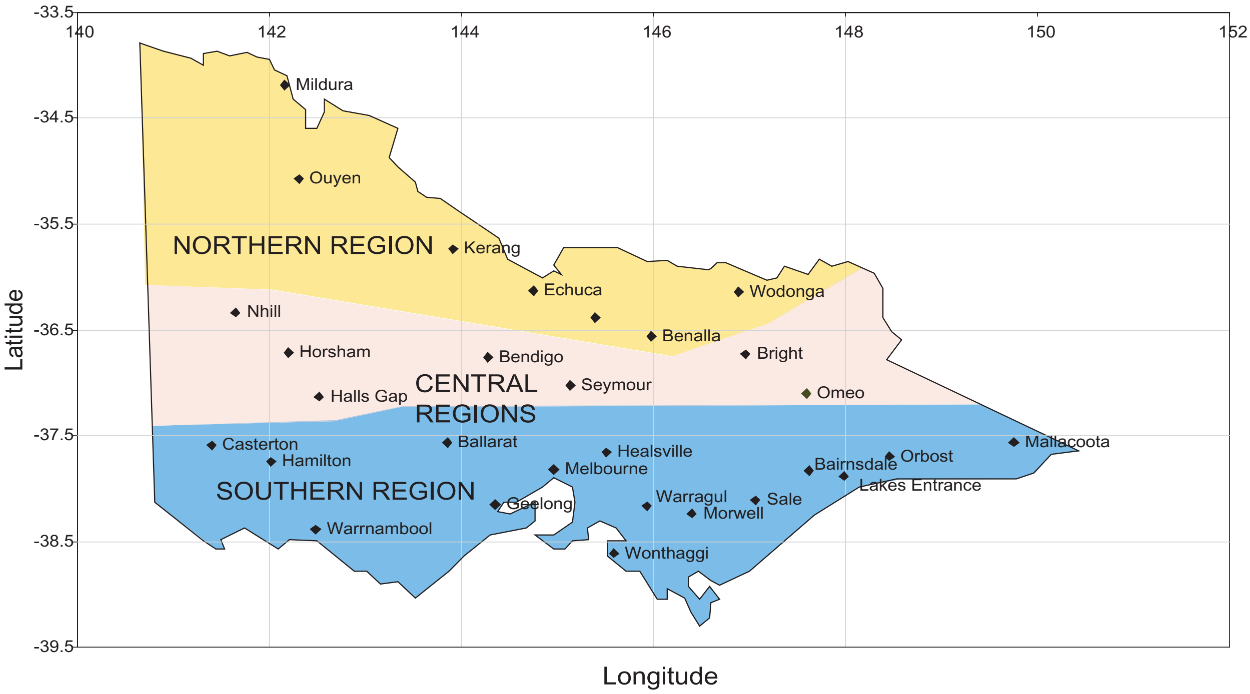

Figure 12.2 Rainwater tank design hydrologic regions.

The results of the analysis (Appendix C) using the **Model for Urban Stormwater Improvement Conceptualisation (MUSIC)** (Cooperative Research Centre for Catchment Hydrology 2003) led to the delineation of three rainwater tank design regions that cover Victoria (Figure 12.2). It was not possible to include Mildura in the Northern Region as the low rainfall in the area proved to significantly increase tank sizes compared with those required for the remaining reference pluviographic stations to achieve comparable performances.

The procedure proposed for determining an appropriate size of rainwater tanks for use in toilet flushing is as follows.

1. Using the reference site curves (Melbourne – see Figure 12.3)
   a. Select the appropriate supply and demand characteristic (represented by the occupancy to roof area ratio).
   b. Either
   i. Select a desired water supply reliability and read from the curves provided (interpolate between curves where appropriate) a required tank size;
   or
   ii. Select a tank size and read from the curves provided (interpolate between curves where appropriate) the resulting reliability of supply.
2. Relate the tank size or reliability to a location in Victoria (using design curves derived for the appropriate design region – see Figure 12.2)
   a. Select an appropriate design chart for the location in question by locating the appropriate rainwater tank **hydrologic design region** (see Figure 12.2)
   b. Determine an equivalent tank size for the location to achieve an equivalent level of supply reliability derived from the reference site (Melbourne). Interpolation between curves may be required.

Tanks sizes for Melbourne

Figure 12.3 shows relationships between water supply reliability and rainwater tank volume for a range of toilet demands and supply catchment areas (as represented by occupancy to roof area ratios). The plots are based on assuming 20 L/day per occupant in the modelling (representing water-efficient dual-flush toilets). Increasing residential density (i.e. higher occupancy to roof area ratio) results in decreasing water supply reliability. Similarly, a larger rainwater tank is required to maintain the same reliability of water supply for a higher occupancy to roof area ratio.

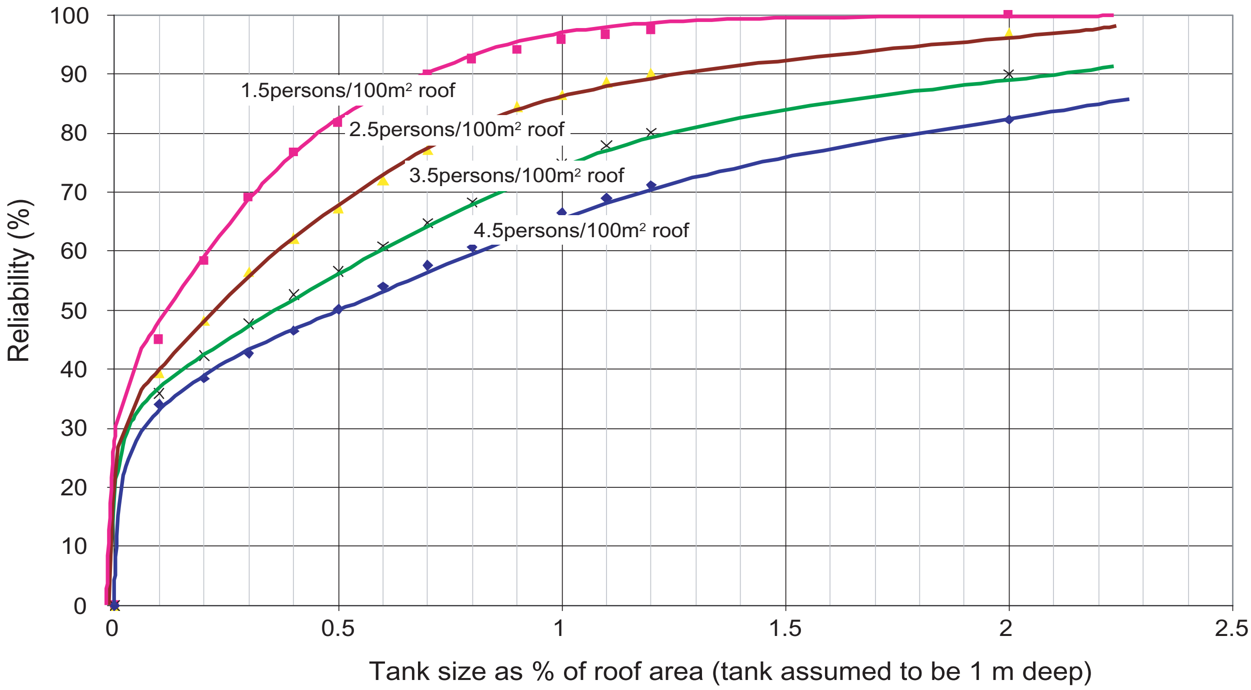

Figure 12.3 Relationship between toilet flushing water supply reliability and rainwater tank size for Melbourne.

Example: to achieve a 70% reliability of water supply for toilet flushing in a household with three people where the roof area connected to a rainwater tank is 120 m^2 will require a tank size equivalent to 0.6% of the roof area or 720 L.

Determining tanks sizes for all locations in Victoria

Figures 12.4, 12.5 and 12.6 show relationships of tank sizes and MAR for the three rainwater tank hydrologic regions in Victoria for varying reliability of supply. The tank size required is expressed as a percentage of the roof area (and assuming a 1 m deep tank) and each of the curves in the plots represents the reliability of an equivalent tank size in the Melbourne region (derived from Figure 12.3).

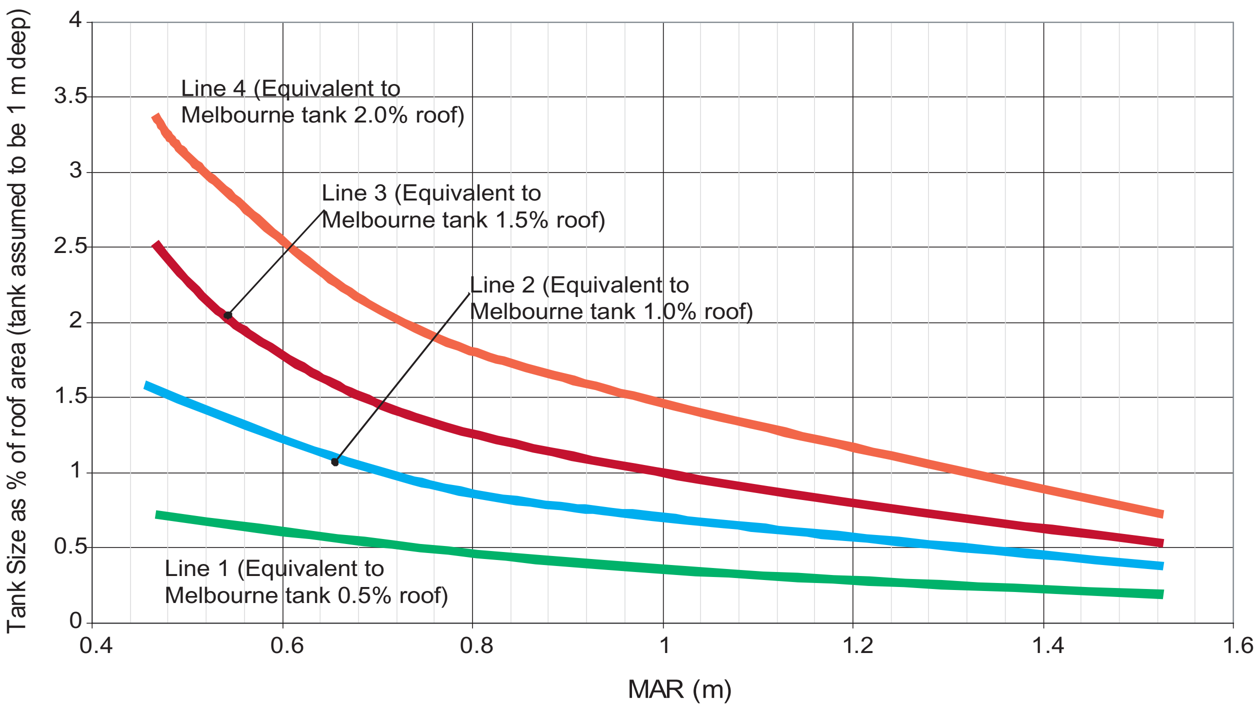

Figure 12.4 Tank size versus Mean Annual Rainfall (MAR) – Southern Region.

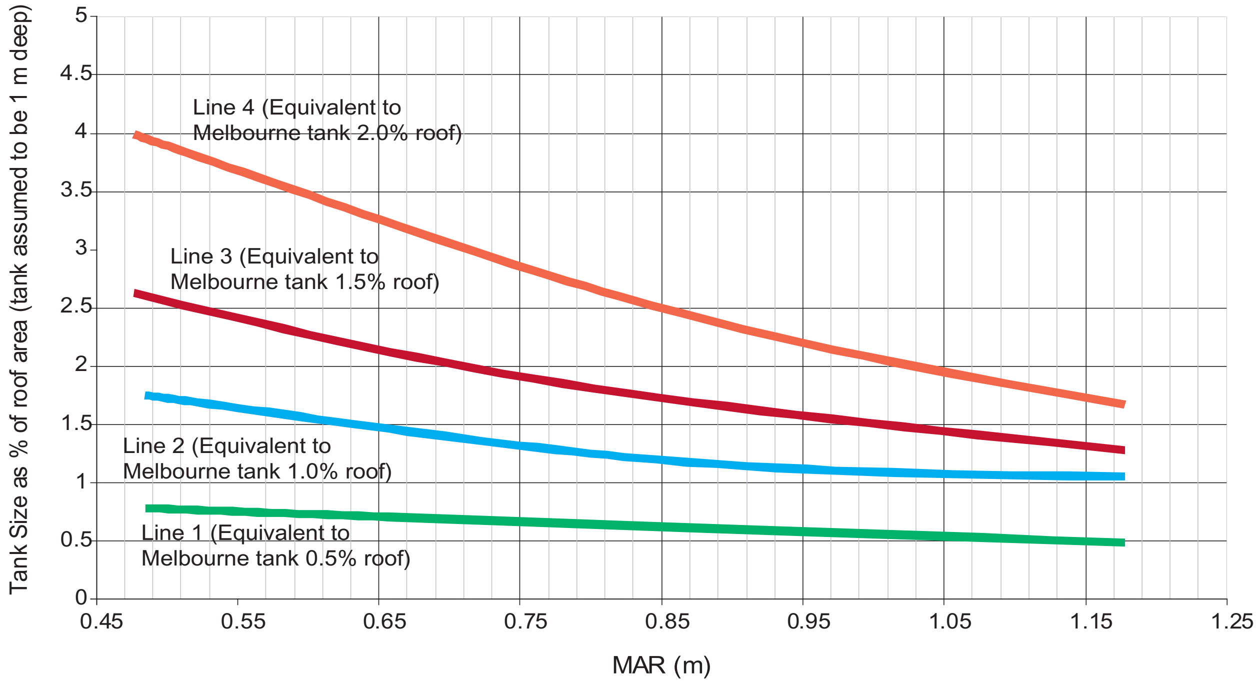

Figure 12.5 Tank size versus Mean Annual Rainfall (MAR) – Central Region.

Four reference curves were used to represent the design characteristics of rainwater tanks each in the Southern and Central regions (Figures 12.4 and 12.5).

It is not possible to represent rainwater tank performance with the same four reference performance curves for the Northern Region owing to insufficient rainfall in this region to attain water supply reliabilities equivalent to tank sizes of 1.5% and 2% of the roof area in Melbourne. Thus, curves for required tank sizes to attain water supply reliabilities equivalent to tank sizes of 0.4%, 0.5%, 0.75% and 1.0% were used.

Example: As discussed in the previous section, a 720 L rainwater tank will provide 70% reliability of toilet flushing supply to a household of 3 people with 120 m^2 roof area connected to a rainwater tank. If the same scenario occurs in Bendigo (Central Region) (MAR of 570 mm), the required rainwater tank size to achieve a 70% water supply reliability can be determined by interpolating between Line 1 and Line 2 in Figure 12.5.

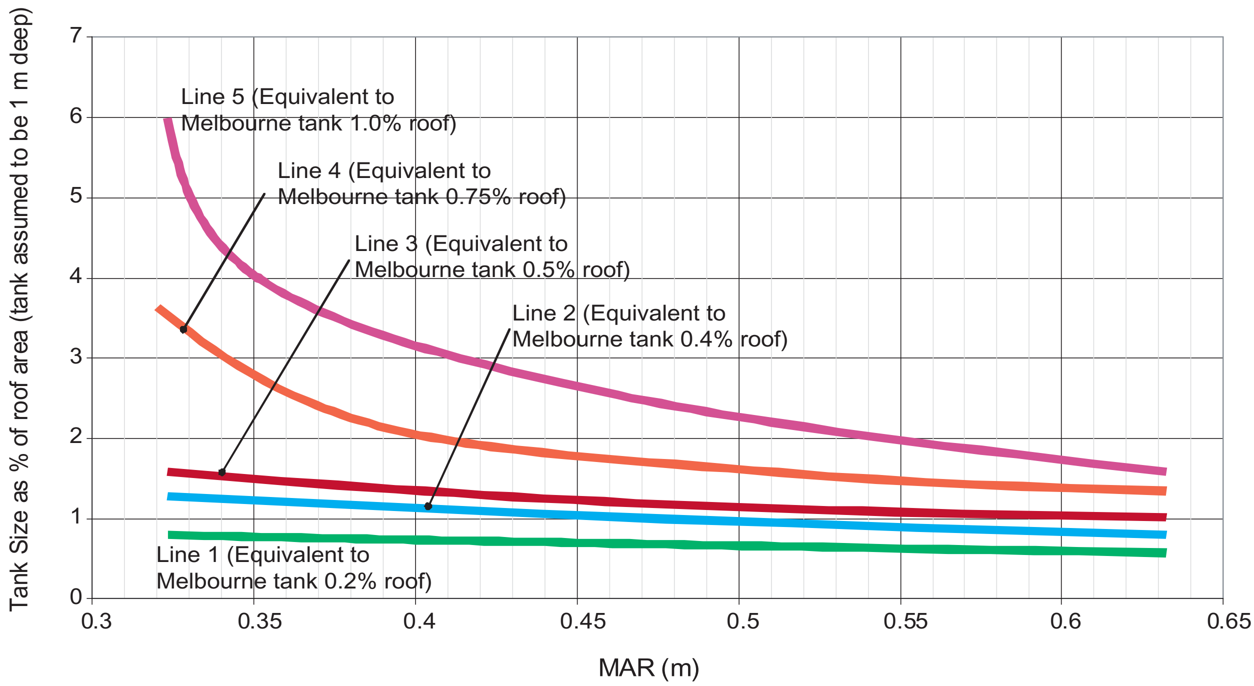

Figure 12.6 Tank size versus Mean Annual Rainfall (MAR) – Northern Region.

From Figure 12.4:
For MAR of 570 mm → Line 1 (0.5%) reading = 0.75%;
Line 2 (1%) reading = 1.6%
Interpolating between 0.75% and 1.6% gives a required tank area of
0.9% of roof area = 1.1 kL tank.

12.4.3 Tank configuration

There are many guidelines on the suitable configuration and installation of a rainwater tank system available from such water authorities as Melbourne Water and Sydney Water. The following are some websites from which these guidelines can be accessed (Table 12.3).

Table 12.3 Guidelines on the suitable configuration and installation of a rainwater tank system

Source	Web address
Gold Coast City Council	http://www.goldcoast.qld.gov.au/attachment/goldcoastwater/GuidelinesTankInstall.pdf
Lower Hunter & Central Coast Regional Environmental Management Strategy	http://www.lhccrems.nsw.gov.au/pdf_xls_zip/pdf_wsud/4_Rainwatertanks.pdf
Sydney Water	http://www.sydneywater.com.au/everydropcounts/garden/rainwater_tanks_installation.cfm
Your Home Consumers Guide (A joint initiative of the Australian government and the design and construction industries)	http://www.greenhouse.gov.au/yourhome/technical/fs22_2.htm

Inlet filter

Some form of filter is strongly recommended on all flows being directed to a rainwater tank. This filter will provide a primary treatment role in regard to removing leaf litter and some sediment that would otherwise enter the tank, and possibly contribute to water quality degradation. Such a filter can also serve to isolate the tank from access by vermin and mosquitos.

First flush diverter

Diversion of the 'first flush' from a roof is also a recommended practice, as this can minimise the ingress to the tank of fine particulates, bird/animal faeces and other potential contaminants. Current research does not enable the specification of a definitive First Flush, with values between 0.25 and 1.0 mm of runoff typically being quoted.

Proprietary devices are available that often provide a joint 'filter/first flush' diversion role.

Maintenance drain

Periodic removal of sludge and organic sediments that accumulate in the base of a rainwater tank may be necessary if buildup is excessive, and as such a suitable outlet should be provided. This sludge layer, and **biofilms** that develop on the walls of a tank, may be important in the natural purification processes occurring in the tank; therefore, removing a sludge layer should only occur when buildup impedes the tank operation.

Mains top-up

Most rainwater tanks will require an automatic top-up system to ensure uninterrupted supply to the household. This top-up should occur as a slow 'trickle' such that there are benefits in regard to reducing peak flow rates in the mains supply system (which, if properly planned, can enable smaller mains infrastructure to be installed in a 'greenfields' situation).

The volume/rate of top-up should be such that there is always at least one day's supply contained within the tank. Top-up should also occur when tank levels are drawn down to a depth of 0.3 m, or one day's capacity, whichever is the greater, to both guarantee supply and to minimise sludge/sediment resuspension.

A final consideration with any top-up system is that there is a requirement for an 'air gap' between the entry point of the top-up supply and the full supply level in the tank in order to ensure there is no potential for backflow of water from the tank into the potable supply system. A suitable air gap is about 100 mm.

Overflow

Rainwater tank overflows should be directed to the stormwater collection system. Given the clean nature of such overflows, smaller diameter pipe systems may be acceptable. In areas with suitable soils and slopes, discharge to a lot-scale infiltration trench may also be possible (see Procedure 8, Chapter 11, for more detail in this regard).

Overflows should also be located below the mains top-up supply point in order to prevent the potential for backflow.

Pump

The supply to the household from the rainwater tank can occur via a pressure pump system, or alternatively a solar panel, pump or header tank system may be implemented, if low heads are acceptable. Careful selection of a suitable pump system is recommended to minimise operational costs and noise issues.

On-site detention

In some situations, rainwater tanks can be configured with an active 'detention' zone located above the 'capture and reuse' zone. This system reduces the effective yield from a tank, but may deliver greater downstream stormwater conveyance benefits through the delivery of lower peak flows for low to moderate ARI events. In such applications, ensure that the potable supply top-up is located above the 'detention' zone, not just above the 'capture and reuse' zone.

12.4.4 Maintenance provision

Rainwater tanks are low maintenance, not 'no maintenance' systems. Good maintenance practice is necessary and should include the following.

- Routine inspection (every six months) of roof areas to ensure that they are kept relatively free of debris and leaves. Roof gutters should be inspected regularly and cleaned if necessary. There are special gutter designs available for limiting the amount of debris and litter that can accumulate in the gutter to be subsequently transported to the rainwater tank. These special gutters cost about twice normal guttering but require little maintenance.
- Pruning of surrounding vegetation and overhanging trees which may otherwise increase the deposition of debris on the roof.
- Cleaning of first flush devices once every three to six months, or as required.
- Regular inspection of all screens at inlet and overflow points from the tank to check for fouling, say, every six months.
- Tank examination for the accumulation of sludge at least every two to three years. If sludge is covering the base of the tank and affecting its operation (i.e. periodically resuspending, or reducing, storage capacity), it should be removed by siphon, flushed from the tank or by completely emptying of the tank. Professional tank cleaners can be used.
- Covering of the rainwater tank..

Any pumping system should be maintained in accordance with the manufacturer's specifications.

12.5 Worked example

Refer to the example in Section 12.4.2 for a worked example.

12.6 Inspection and maintenance schedule

The following inspection schedule is recommended for a rainwater tank system and maintenance of the pump:

- roof/gutters – six monthly, possible more frequently for gutters if required
- first flush device – three to six monthly, cleaning if required

- inlet/overflow screens – six monthly
- sludge accumulation – every two to three years, and desludge if required
- pump system – as required/specified by pump manufacturer.

12.7 References

Coombes, P.J. (2002). 'Rainwater tanks revisited: new opportunities for urban water cycle management', PhD thesis, University of Newcastle, Callaghan, NSW.

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Chapter 13 **Aquifer storage and recovery**

Morphettville Racecourse, Adelaide

13.1 Introduction

Aquifer storage and recovery (ASR) is a means of enhancing water recharge to underground aquifers through either pumping or gravity feed. Stored water can then be pumped from below ground during dry periods for subsequent reuse and can, therefore, be a low cost alternative to large surface storages. In the **stormwater** context, it may also be used as a method to store excess water produced from urbanisation during wet periods (e.g. winter) which can then harvested during long dry periods to reduce reliance on mains supply for uses such as irrigation.

Both stormwater and treated wastewater are potential sources for an ASR system. This chapter focuses on stormwater ASR systems, although many of the concepts are the same for both systems. Stormwater ASR systems are designed to harvest increased flows attributed to urbanisation. Harvesting urban runoff and diverting it into underground groundwater systems also requires that the quality of the injected water is sufficient not to degrade the existing and potential future beneficial uses of the groundwater supplies. The level of treatment depends on the quality of the groundwater. In most instances, the range of management measures described in this Manual will provide sufficient treatment prior to injection.

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

Broad requirements of ASR systems include:

- protecting or improving groundwater quality where ASR is practiced
- ensure that the quality of recovered water is fit for its intended use
- protecting aquifers and aquitards (fractured rock) from being damaged by depletion or excessive pressure (from overinjection)
- avoiding problems such as clogging or excessive extraction of aquifer sediments
- ensuring reduced volumes of surface water downstream of the harvesting point are acceptable and consistent with a **catchment** management strategy.

In addition to the physical requirements of an ASR system, they also require permits to divert water, to install treatment measures, to inject into groundwater as well as extraction for the intended use. A thorough investigation of the required permits should be undertaken. The Victorian Smart Water Fund plans to develop Best Practice Guidelines for ASR in Victoria. Further information on this project can be found at http://www.smartwater.com.au.

The following material has been reproduced from the *Code of Practice for Aquifer Storage and Recovery* (SA EPA 2004) with the permission of the author, to provide an overview of the main components of an ASR system.

13.2 Components of an ASR system

An ASR scheme that harvests stormwater typically contains the following structural elements (Figure 13.1):

- a diversion structure from a stream or drain
- a control unit to stop diversions when flows are outside an acceptable range of flows or quality
- some form of treatment for stormwater prior to injection
- a **wetland**, detention **pond**, dam or tank, part or all of which acts as a temporary storage measure (and which may also be used as a **buffer** storage during recovery and reuse)
- a spill or overflow structure incorporated in wetland or detention storage
- well(s) into which the water is injected (may require extraction equipment for periodic purging)
- a well equipped to recover water from the aquifer (injection and recovery may occur in the same well)
- a treatment system for recovered water (depending on its intended use)
- systems to monitor water levels, and volumes injected and extracted
- systems to monitor the quality of injectant, groundwater and recovered water
- sampling ports on injection and recovery lines
- a control system to shut down recharge in the event of unfavourable conditions.

13.3 Treatment and pollution control

For stormwater ASR systems, water quality treatment will be required prior to injection into groundwater. The level of treatment depends on the quality of the groundwater (beneficial uses) and local regulation should be checked. Many of the treatments described earlier in this Manual will provide sufficient treatment for an ASR system. These systems also have the added benefit of reducing the risk of 'clogging' the ASR injection well because of efficient fine sediment removal.

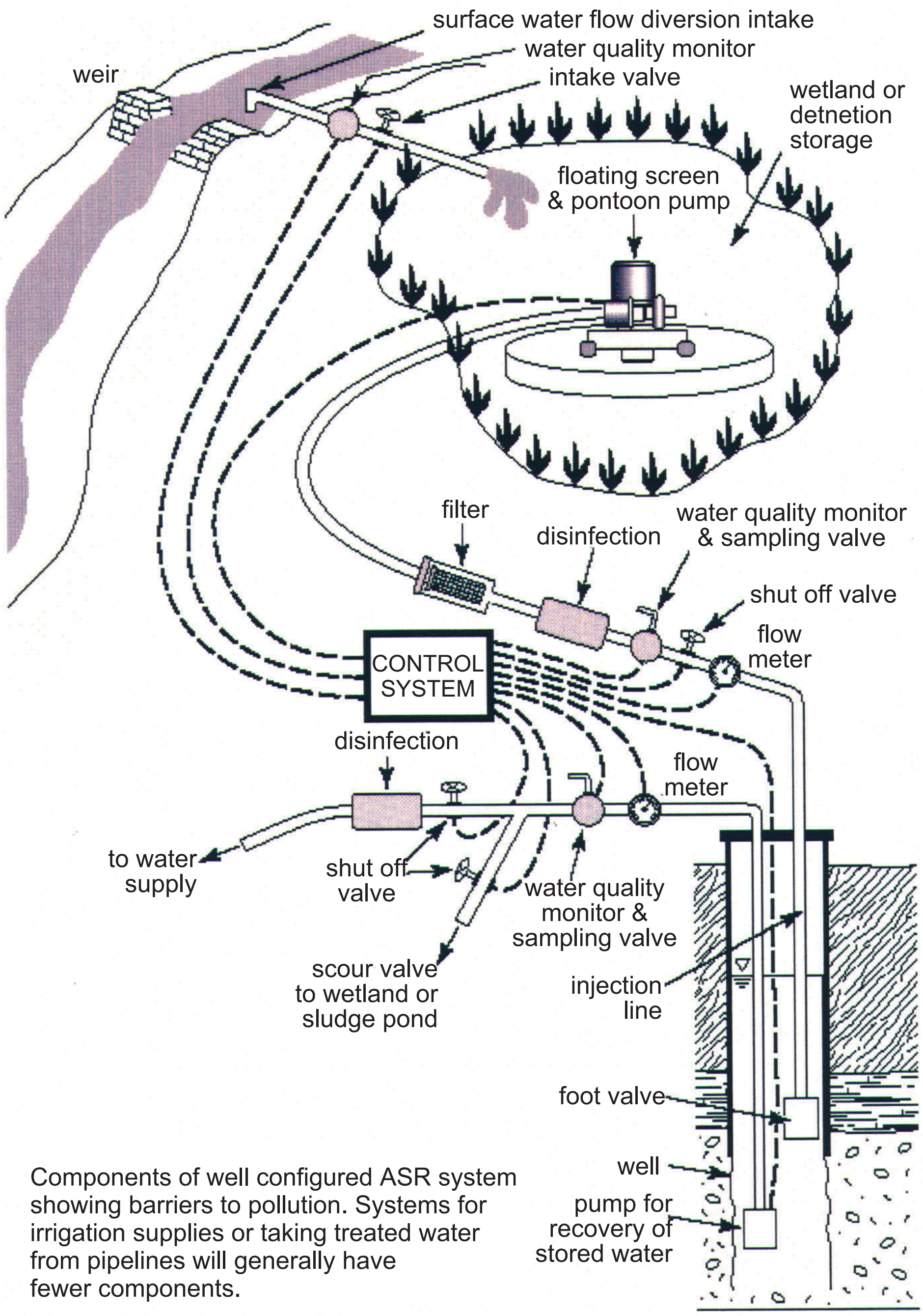

Figure 13.1 Components of a well-configured Aquifer Storage and Recovery (ASR) system (diagram CSIRO Land and Water).

13.3.1 Knowledge of pollutant sources in the catchment upstream

Each ASR scheme must identify potential pollution sources within a catchment and plan risk management strategies, including pollution contingency plans. For urban stormwater harvesting, treatment measures described in this Manual are considered a minimum requirement.

Comparisons with native groundwater quality and its environmental values will indicate the requirements for treatment of water detained for injection. An evaluation of the pollutants that may be present within the injectant water needs to be carried out on a catchment basis. Pollutants will vary according to whether the catchment drains urban residential, urban industrial, rural or a combination of any of these catchment types.

The concentrations of pollutants typically have seasonal or within-event patterns, and heavy pollutant loadings can be avoided by being selective in the timing of diversions. Knowledge of the potential pollutant profile helps to define water quality sampling and analysis costs when determining the viability of the ASR project.

13.3.2 Aquifer selection

The quality of water to be injected must be no worse than the quality of water already in the aquifer, and better if possible. As discussed earlier, the aquifer may already be providing beneficial uses to others and the quality and flow requirements of these users needs to be considered in the aquifer selection. This may exclude using aquifers for ASR schemes that contain high-quality groundwater.

Factors to consider when choosing a suitable aquifer include:

- environmental values of the aquifer (beneficial uses)
- sufficient permeability of the receiving aquifer
- salinity of aquifer water greater than injection water
- possible damage to confining layers due to pressure increases
- adverse effects of reduced pressure on other groundwater users
- higher recovery efficiencies of porous media aquifers
- effects on other aquifer users
- aquifer mineral dissolution, if any, and potential for well aquitard collapse.

13.3.3 Pretreatment prior to injection

Many of the treatment measures describe in earlier chapters of this Manual are suitable as pretreatments for ASR schemes. In general, methods that have long **detention times** are advantageous to reduce pathogenic microorganisms in addition to other pollutants.

An advantage of using a treatment with large storages (e.g. wetlands) is the dilution effect should an isolated pollution event occur, thus reducing the risk of aquifer contamination.

13.3.4 Injection shutdown system

Controls should be incorporated to shut down an injection pump or valve if any of the following exceed the criteria for the environmental values of the aquifer:

- standing water level in the well
- injection pressure
- electrical conductivity (salinity)
- turbidity
- temperature
- pH
- dissolved oxygen concentrations
- volatile organics
- other pollutants likely to be present in injectant water that can be monitored in real time.

13.3.5 Maintenance and contingency plans

Protection of the treatment and detention system from contamination is a necessary part of the design in ASR systems. This includes constructing treatment systems away from flood-prone land, taking care with or avoiding the use of herbicides and pesticides within the surrounding catchment, planting non-deciduous vegetation, and preventing mosquitos and other pests from breeding in the storage pond.

Contingency plans should be developed to cater for the possibility of contaminated water being inadvertently injected into the aquifer. These include how to determine the duration of recovery pumping (to extract contaminated water), what sampling intervals are needed and how to manage recovered water.

13.3.6 Recovered water post-treatment

For supplies of drinking water, recovered water may need to be treated (e.g. using ultraviolet disinfection). For some other forms of supply, such as irrigation via drippers, it may be necessary to insert a cartridge filter.

13.3.7 Discharge of well development/redevelopment water

In the development of wells for use in an ASR system, the well needs to be 'developed', that is, it needs to be purged for some time to remove poor quality water that may have been created as part of the construction of the well. Usually this water is high in fine sediment and as such must not be disposed of to a waterbody or a watercourse unless it is of suitable quality. It may be used on site, possibly for irrigation, discharged to the sewer (with the approval of the relevant authority), or returned to a treatment system.

13.3.8 Groundwater attenuation zones

In some cases the effect of certain groundwater pollutants can be diminished over time due to natural processes within the aquifer. Chemical, physical and microbiological processes can occur to ameliorate the harm or potential harm caused by these pollutants.

13.4 Quality of water for injection and recovery

The selection of a storage aquifer and the quality of water that can be injected will be determined by a Water Quality Policy of the relevant agency (e.g. EPA, water authorities).

Designated environmental values of the recovered water, such as raw water for drinking, stock water, irrigation, ecosystem support and groundwater ecology are determined from:

- ambient groundwater quality, with reference to the National Water Quality Management Strategy (NHMRC & ARMCANZ 1996; ANZECC & ARMCANZ 2000)
- local historical and continuing uses of those aquifers

Artificial recharge should improve or at least maintain groundwater quality.

13.5 Domestic scale aquifer storage and recovery

It is also possible to install an ASR scheme at the domestic scale. Generally these schemes are subject to the same considerations as larger scale design; however, being smaller systems they are likely to be shallower and therefore additional considerations are required.

Domestic scale ASR in shallow aquifers should not be undertaken in locations where water tables are already shallow (< 5 m) or in areas where:

- saline groundwater ingress to sewers occurs
- water tables could rise to within 5 m of the soil surface as a result of ASR in areas of expansive clay soils
- other structures such as cellars or basements could be adversely affected by rising water tables
- dryland salinity is an issue in the local catchment.

The water recharged must be of the highest possible quality, equivalent to roof runoff after first flush bypass, such as overflow from a rainwater tank, and must be filtered to prevent entry of leaves, pine needles and other gross pollutants into a well.

Runoff from paved areas must not be admitted, unless this has first passed through a treatment measure (as described in previous chapters) to reach the required quality for injection.

An inventory should be made of other potential pollutants in the well's catchment and strategies devised to ensure these are excluded from the well, or are treated and removed before water enters the well.

The aquifer pressure must be below ground level at all times. To achieve this, injection should be by gravity drainage into the well, rather than by using a pressurised injection system, and there should be an overflow facility (e.g. to a garden area) where excess water **discharges** to or to the urban stormwater drainage system.

13.6 Additional information

This chapter provides a brief introduction into ASR and the considerations required to assess feasibility. Considerably more investigations and consultation are required to determine the functional details of a possible ASR system.

There are some Australian guidelines available for ASR systems (particularly from South Australia where there is considerable experience with these systems) as well a Victorian Guideline for ASR being developed as part of the Smart Water Fund. Some relevant websites are presented in Table 13.1 and further information is presented in the reference list.

Table 13.1 Sources of information on aquifer storage and recovery systems

Source	Web address
Aquifer Storage Recovery	www.asrforum.com
International Association of Hydrogeologists – Managing Aquifer Recharge (IAH–MAR)	www.iah.org/recharge/
CSIRO water reclamation project in Australia	www.clw.csiro.au/research/catchment/reclamation/
Smart Water Fund	www.smartwater.com.au
Environment Protection and Heritage Council (EPHC)	www.ephc.gov.au/index.html
Department of Water, Land and Biodiversity Conservation (regarding licencing requirements)	www.dwlbc.sa.gov.au

13.7 References

ANZECC & ARMCANZ (2000). *Australia and New Zealand Guidelines for Fresh and Marine Water Quality*, Australia and New Zealand Environment Conservation Council and Agriculture and Resource Management Council of Australia and New Zealand, Canberra, ACT.

Dillon, P.J. and Pavelic, P. (1996). 'Guidelines on the quality of stormwater and treated wastewater for injection into aquifers for storage and reuse', *Research Report No. 109*, Urban Water Research Association of Australia,

Environment Protection Authority (South Australia) (2004). *Code of Practice for Aquifer Storage and Recovery* (www.environment.sa.gov.au/epa/pdfs/cop_aquifer.pdf)

NHMRC & ARMCANZ (1996). *Australian Drinking Water Quality Guidelines*, National Health and Medical Research Council, and Agriculture and Resource Management Council of Australia and New Zealand, Canberra, ACT.

Chapter 14 **Other measures**

14.1 Introduction

There are a range of 'other' **stormwater** management and treatment measures that can be considered as part of the available toolkit for the WSUD practitioner. These 'other' measures are either proprietary devices or may have a differing scope of application to the more mainstream techniques discussed earlier in this Manual and, as such, no detailed design procedures have been prepared for them. The following sections of this chapter provide general guidance on the characteristics of these additional techniques for review and further consideration by interested designers of a WSUD-oriented project.

The techniques that are discussed include the following:

- subsurface **wetlands**
- proprietary products
- porous pavements
- use of natural areas including reforestation and revegetation.

14.2 Subsurface wetlands

Figure 14.1 provides an example of indicative cross-sections of both free surface and subsurface wetlands (source Queensland Department of Natural Resources and Mines (DNRM) 2000). The 'free surface' wetland illustrated in Figure 9.1 is addressed in Chapter 9 of this Manual. The discussion here relates to the subsurface flow wetland in which the flow to be treated passes through a porous media such as sand or gravel which underlies the wetland.

Subsurface wetlands are typically applied in a wastewater treatment system where there is a relatively consistent influent flow rate. To date in Australia, there have been few, if any, applications of these techniques in the stormwater field, though there are obvious overlaps between a porous media, a planted bioretention system and a vertical subsurface flow wetland.

One of the major issues associated with the use of subsurface flow wetlands in a stormwater treatment context relates to the highly episodic nature of stormwater events. A subsurface wetland would require considerable volumes of balancing/detention storage above it to attenuate stormwater inflows. There may also be problems with the subsurface wetlands excessively drying under prolonged low rainfall conditions with associated losses of algal and microbial slime layers.

The following general guidance on subsurface flow wetlands has been broadly sourced from DNRM (2000).

- Subsurface wetlands, commonly referred to as reed beds, consist of channels or basins that contain gravel or sand media which support emergent type vegetation. The purpose of the vegetation is to provide some oxygen to the root zone.
- The environment with a subsurface wetland is mostly anoxic or anaerobic. Some oxygen is supplied to the roots, which is likely to be used up in the biomass growing there rather than penetrate too far into the water column and, for this reason, subsurface wetlands are effective in denitrification.

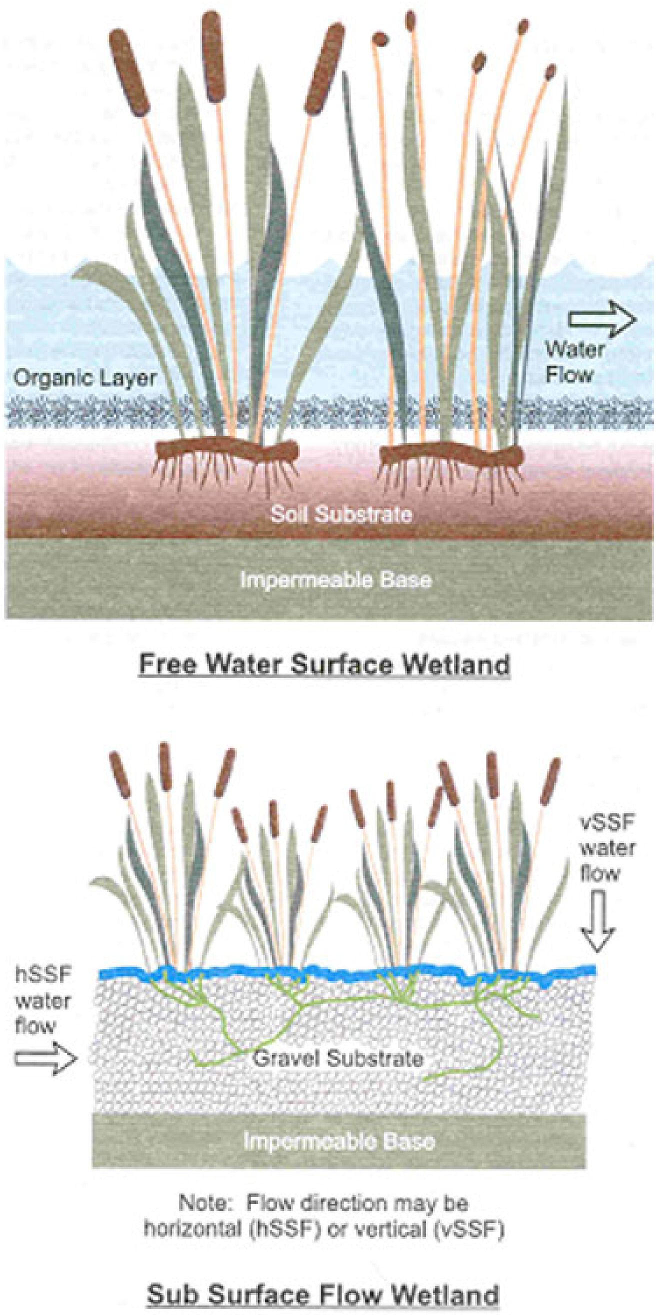

Figure 14.1 Major types of **constructed wetlands** (from Queensland Department of Natural Resources and Mines 2000)

Reported advantages of subsurface wetlands are as follows:

- significant ability to treat high organic loads
- high cold weather tolerance
- greater treatment per unit area when compared to free surface wetlands
- mosquitos and odours are generally not a problem
- there are no public safety issues as the wetland is not a body of open water
- resuspension of sediment (e.g. due to wind, birdlife) is eliminated (unlike surface wetlands)
- horizontal flow paths through porous media require only mild hydraulic gradients (hence long **detention times**)
- there are minimal harvesting needs.

Reported disadvantages of subsurface wetlands are as follows:

- intermittent stormwater flows may adversely affect treatment
- higher capital cost, associated with media supply

Table 14.1 Typical design criteria and expected effluent quality for subsurface wetlands (after Crites and Tchobanoglous 1998)

Item	Value
Detention time	3–4 days
Biological Oxygen Demand (BOD)loading	0.01 kg/m^2 per day
Suspended solids loading[A] (see note)	0.04 kg/m^2 per day
Water depth	up to 0.6 m
Media depth	up to 0.75 m
Harvesting	Limited
Expected effluent quality:	
BOD	< 20 mg/L
Suspended solids	< 20 mg/L
Total nitrogen	< 10 mg/L
Total phosphorus	< 5 mg/L
Note [A]For wetland length to width ratio of greater than 4:1, the influent suspended solids loading may be a concern. To avoid entry zone blockages, suspended solid loadings should not exceed 0.08 kg/m2 per day (Bavor et al. 1989).	

- they can be prone to blockage, particularly at the **inlet zones**
- they are limited to smaller pollutant loadings.

A frequently reported problem with subsurface wetlands is blockage of the inlet zones which then leads to short circuiting and surface flow. Attention needs to be given to good inflow distribution and the placement of larger aggregate within this inlet zone. Inlet apertures need to be large enough to avoid being blocked by algal growth and designs should aim to facilitate regular inspections for maintenance purposes.

Primary design criteria for subsurface flow wetlands are:

- detention time
- organic loading rate
- hydraulic loading rate
- media size
- bed depth
- aspect ratio.

Typical design criteria from other countries for wastewater subsurface wetlands are provided in Table 14.1.

14.3 Proprietary stormwater treatment devices

In the development of a WSUD **treatment train** for a site, there is an extensive array of proprietary products available for consideration. Such proprietary products usually take a primary treatment role, removing gross pollutants and litter before other devices (as described in the earlier design procedures in this Manual) address the fine sediment, nutrient and pathogen content of urban stormwater. However, there are also products available for **sedimentation**, spill controls, oil separation and fine filtration.

Given the diversity of forms and configurations of these proprietary devices and, in some cases, the confidential nature of their design and performance data, this Manual provides general guidance as to the issues that should be considered when selecting such devices. We also provide some guidance as to those factors which should be considered when reviewing performance values often ascribed to such devices by suppliers.

14.3.1 Selection issues

Engineers Australia (2003) provide guidance as to those issues which should be considered when selecting a **gross pollutant trap, GPT**. Such devices constitute most proprietary products. The following summary of key selection issues has been developed on the basis of the Engineers Australia (2003) advice.

A decision of which type (and brand) of proprietary device to select is a trade-off between the life cycle costs of the device (i.e. by combining capital and ongoing costs), expected pollutant removal performance in regard to the values of the downstream waterbody and social considerations.

A life cycle cost approach is recommended. This approach allows the ongoing cost of operation to be considered and the benefits of different devices to be assessed over a longer period. The overall cost of a proprietary device is often determined more by the maintenance costs rather than the initial capital costs.

The expected pollutant removal rate is a function of the amount of runoff treated (i.e. the quantity of flow diverted into a proprietary device compared to that which bypasses) and the pollutant removal rate for flows that go through a proprietary device.

This section highlights some issues that should be considered as part of the decision-making process. The issues raised are primarily based on experience with existing proprietary device installations.

14.3.1.1 Life cycle costs

Life cycle costs are a combination of the installation and maintenance costs and provide an indication of the true long-term cost of the infrastructure. It is particularly important to consider life cycle costs for proprietary devices as maintenance costs can be significant compared to the capital costs of installation.

To determine life cycle costs, an estimated duration of the project (i.e. lifespan of the treatment device) needs to be assumed (e.g. 20 or 25 years). If the device is to control pollutants during the development phase only (e.g. a sediment trap) its life cycle may be only three to ten years.

Life cycle costs can be estimated for all devices and then, with consideration to the other influences (e.g. expected pollutant removal, social), the most appropriate device can be selected.

14.3.1.2 Installation costs and considerations

Installation costs include the cost of supply and installation of a proprietary device. These prices should be evident on proposals for proprietary device installations but it should be checked that all installation costs are included. Variables in terms of ground conditions (e.g. rock or groundwater conditions) or access issues may vary construction costs significantly and cost implications of these should be assessed. The likely occurrence of these issues should be weighed up when estimating an overall installation cost.

Issues that should be checked as being addressed by tenderers include:

- price includes supply and installation (not just supply)
- provision for rock or difficult ground conditions
- proximity to services (and relocation costs)
- required access and traffic management systems for construction.

A true installation cost should then be used when estimating life cycle costs.

As important as obtaining a true installation cost is ensuring that the device will suit local conditions. Issues that should be assessed to ensure a proprietary device will suit an area include:

1 the size of the unit
2 hydraulic impedance caused by the device
3 particular construction issues.

Size of the unit (footprint, depth)

The sizes of proprietary devices vary considerably and this will need to be accommodated by the potential location for the device. Things to consider when assessing the size of a device include:

- required footprint (plan size of device and any required flow diversion)
- depth of excavation (to the bottom of the sump in some cases) – rock can substantially increase installation costs
- sump volume required (where applicable)

- proximity to groundwater
- location of any services that affect construction and likely cost for relocation (e.g. power, water sewer).

Hydraulic impedance/requirements

Some proprietary devices require particular hydraulic conditions in order to operate effectively; for example, some devices require a drop in a channel bed for operation. Requirements such as these can affect which devices may or may not be suitable in a particular area.

Other considerations are possible upstream effects on flow and a hydraulic gradeline because of the installation of the device. This can increase flooding risks and all devices should be designed to not increase the flooding risk during high flows. Therefore, if a device increases the flooding risk above acceptable limits, it may not be considered further.

Other construction issues

For each specific location there will be several other considerations and points of clarification that may sway a decision on which device may be the most suitable. These include the following.

- Does the cost include any diversion structures that will be required?
- Is specialist equipment required for installation (e.g. special formwork, cranes or excavators) and what cost implications do these have?
- Is particular below-ground access required, will ventilation and other safety equipment be needed – at what cost?
- Will the device affect the aesthetics of an area – will landscape costs be incurred after the device installation – if so how much?
- Will the device be safe from interloper or misadventure access?
- Do the lids/covers have sufficient loading capability (particularly when located within roads) – what is the cost of any increase in load capacity and will it increase maintenance costs?
- Will the device be decommissioned (e.g. after the development phase) and what will this cost be – what will remain in the drainage system?
- Are there tidal influences on the structure and how will they potentially affect performance or construction techniques?
- Will protection from erosion be required at the outlet of the device (particularly in soft bed channels), and what cost implications are there?

14.3.1.3 Maintenance costs and considerations

Maintenance costs can be more difficult to estimate than the installation costs (but are sometimes the most critical variable). Variation in the techniques used, the amount of material removed and the unknown nature of the pollutants exported from a **catchment** (thus disposal costs) all influence maintenance costs. It is, therefore, imperative to carefully consider the maintenance requirements and estimate costs when selecting a proprietary device. As part of a tender process, tenderers should be asked to quote annual maintenance prices, based on the relevant site conditions (not just generic estimates).

One important step is to check with previous installations by contacting the owners and asking their frequency of cleaning and annual operation costs (vendors can usually supply contact information).

All maintenance activities should be developed that require no manual handling of collected pollutants because of safety concerns with hazardous material.

Below is a list of maintenance considerations that should be applied to all proprietary devices.

- Is special maintenance equipment required (e.g. large cranes, vacuum trucks or truck-mounted cranes)? Does this equipment need to be bought or hired – at what cost?
- Is special inspection equipment needed (e.g. access pits)?
- Are any services required (e.g. washdown water, sewer access)?
- Are there overhead restrictions such as power lines or trees?
- Does the water need to be emptied before the pollutants – if so how will it be done, where will it be put and what will it cost?
- Can the device be isolated for cleaning (especially relevant in tidal areas)?

- Are road closures required and how much disturbance will this cause?
- Are special access routes required for maintenance (e.g. access roads or concrete pads to lift from) – and what are these likely to cost?
- Is there a need for dewatering areas (e.g. for draining sump baskets) and what implications will this have?

Disposal costs

Disposal costs will vary depending on whether the collected material is retained in wet or free draining conditions in the proprietary device. Handling of wet material is more expensive and will require sealed handling vehicles.

- Is the material in a wet or dry condition and what cost implications are there?
- Are there particular hazardous materials that may be collected and will they require special disposal requirements (e.g. contaminated waste –what cost implications are there?
- What is the expected load of material and what are likely disposal costs?

Occupational health and safety

- Is there any manual handling of pollutants and what will safety equipment cost?
- Is entering the device required for maintenance and operating purposes – will this require confined space entry? What cost implications does this have on the maintenance cycle (e.g. minimum of three people on site, safety equipment such as gas detectors, harnesses, ventilation fans and emergency oxygen)?
- Are adequate safety features built into the design (e.g. adequate step irons and inspection ports) or will these be an additional cost?

14.3.1.4 Miscellaneous considerations

Social considerations can be an important component of the selection of a proprietary device. Consultation with key stakeholders is fundamental to selecting an appropriate proprietary device. Influences on the decision process may include:

- potential odour concerns at a location
- likelihood of pests and vermin such as mosquitos or rats
- suitability of the proprietary device materials, particularly in adverse environments (e.g. marine)
- effect on the aesthetics of an area
- education and awareness opportunities
- potential trapping of fauna (e.g. turtles, eels and fish).

These issues should be considered early in the selection process and taken into account when finalising a proprietary device type.

14.3.1.5 Checklist for selecting proprietary products

The following checklist is reproduced from Engineers Australia (2003) and provides guidance on issue to consider when selecting proprietary stormwater treatment products.

1. GENERAL	YES	NO
• Is there available space for the device (i.e. required footprint, access routes, services)?	☐	☐
• Does the location suit catchment treatment objectives (e.g. position in a ‘treatment train’)?	☐	☐
• Is the pollutant holding chamber suitable (wet or dry retention)?	☐	☐
• Are there sufficient safety precautions (i.e. preventing entry, access for cleaning)?	☐	☐
• Is the visual impact satisfactory (and odour potential)?	☐	☐
• Is the treatment flow sufficient to meet treatment objectives?	☐	☐
• Has the flooding impact being demonstrated to be satisfactory?	☐	☐
• Has sufficient consultation taken place with operation staff and affected locals?	☐	☐
• Is the expected pollutant removal rate sufficient to meet treatment objectives (consult with owners of existing installations if required)?	☐	☐

2. INSTALLATION		
• Does the price include installation?	☐	☐
• Are there sufficient contingencies for ground conditions (e.g. rock, shallow water table, soft soils etc.)?	☐	☐
• Have relocation of services being included?	☐	☐
• Are there sufficient access or traffic management systems proposed as part of construction?	☐	☐
• What are the cost implications of the above points? $______________	☐	☐
3. MAINTENANCE		
• Is the method of cleaning applicable to local conditions (e.g. OH&S issues, isolation of the unit from inflows etc.)?	☐	☐
• Are the maintenance (cleaning) techniques suitable for the responsible organisation (ie. required equipment, space requirements, access, pollutant draining facilities etc.)?	☐	☐
• Is a maintenance contract included in the proposal?	☐	☐
• Is the size of the holding chamber sufficient?	☐	☐
• Have disposals cost being accounted for?	☐	☐
• What are the cost implications of the above points? $______________	☐	☐

14.3.2 Performance issues

When considering the adoption of a proprietary device for a particular site, as well as the selection issues addressed above, it is recommended that consideration be given to how the device will perform, especially in respect to the levels of performance which are often attributed to such devices by their suppliers.

In this regard, it is recommended that consideration be given to the following key issues (Auckland Regional Council 2003).

- Whether the operating parameters of the system have been verified.
- Existing or proposed monitoring data.
- Documentation of processes by which pollutants will be reduced (physical, chemical, biological).
- Documentation and/or discussion of potential causes of poor performance or failure of the device.
- Key design specifications or considerations.
- Specific installation requirements.
- Specific maintenance requirements.
- Data to support claimed pollutant removal efficiencies. If the device is new or the existing data is not considered reliable, such data should be viewed with caution.

14.4 Porous pavements

Fletcher et al. (2003) provides an Australian review of available data on porous pavements, combined with advice on maintenance and operational issues, is contained in the following material has been reproduced from this publication (with the permission of the lead author).

14.4.1 Description

Porous pavements, as their name implies, are a pavement type that promote infiltration, either to the soil below, or to a dedicated water storage reservoir below it. Porous pavements come in several forms (Figure 14.2), and are either monolithic or modular. Monolithic structures include porous concrete and porous pavement (asphalt). Modular structures includes porous pavers (which may be either made of porous material, or constructed so that there is a gap in between each paver), modular lattice structures (made either of concrete or plastic). Porous pavements are usually laid on sand or fine gravel, underlain by a layer of geotextile, with a layer of coarse aggregate below. Design should ensure that the required traffic load can be carried.


Figure 14.2 Examples of porous pavement: porous car park, Washington, DC, USA (Photo: Ecological Engineering); porous car park, road gutter, Manly, NSW (Photo: Tim Fletcher).

An advantage of modular pavers is their ability to be lifted, backwashed and replaced when blockage occurs. Pavers that are porous from the use of gaps between individual pavers should be carefully chosen with reference to likely catchment inputs, such as leaves and debris that can quickly block the gaps.

Porous pavements should generally be located in areas without heavy traffic loads. In high traffic areas the loads of pollutants can significantly decrease the ability to remain porous. Consideration of the maintenance advantages of modular pavers should also be considered, given that the consequence of blockage with monolithic material

Porous pavement has two main advantages over impervious pavement, in terms of stormwater management:

1 improvement to water quality, through filtering, interception and biological treatment
2 flow attenuation, through infiltration and storage.

14.4.2 Studies of performance

Investigations into the performance of porous pavements have investigated (a) water quality and (b) flow effects.

14.4.2.1 Flow behaviour

Porous pavements can potentially reduce peak flow rate, and total flow volume, the individual or combined effect of initial loss, infiltration, storage and evaporation. The level of flow attenuation is dependent in part on (where appropriate) the amount of storage, and the infiltration capacity of the porous pavements, its underlying base material (including any underlying geotextile), and the soil below.

14.4.2.2 Water quality behaviour

Porous pavements act to improve water quality through a number of mechanisms:

- filtering through the pavement media, and underlying material
- potential biological activity within the pavement and base material
- reduction of pollutant loads, as a result of reduced runoff volumes.

Observed behaviour is likely to be a function of the particular storm event (its magnitude and intensity), the input concentration, and the characteristics of the pavement media and underlying filter material.

Importantly, since contaminants such as heavy metals and hydrocarbons are often attached to sediment, the filtering behaviour acts not only to reduce sediment loads, but also those of associated contaminants. Because of the ability of porous pavement to provide an initial rainfall loss, runoff from porous pavement is less likely to have the oft-observed 'first-flush' effect, where greatly elevated pollutant concentrations are observed in the first part of a storm.

Table 14.2 Summary of expected porous pavement performance
(after Fletcher et al. 2003)

Pollutant	Expected concentration reduction (%) (+ range)	Comments
Total Suspended Solids (TSS)	80 (70–100)	
Total Nitrogen (TN)	65 (60–80)	Will decrease with proportion dissolved
Total Phosphorus (TP)	60 (40–80)	Will decrease with proportion dissolved
Hydrocarbons/Oils/Grease	85 (80–99)	Depends on level of microbial activity.
Biological Oxygen Demand (BOD)	–	Inadequate data
Lead, copper, cadmium, zinc and nickel	75 (40–90)	Will decrease with proportion dissolved
Litter	–	Litter will simply 'wash off'
Pathogens	–	Inadequate data

14.4.2.3 Summary of expected performance

Based on the studies of flow performance reviewed by Fletcher et al. (2003), and contingent upon the properties and condition of the porous pavement and its subsoil, a reduction in runoff coefficient from around 0.95 for traditional pavements, to around 0.40 can be expected. However, the expected hydraulic performance of any porous pavement can be easily modelled, either for a single rainfall event (using a spreadsheet approach), or using a rainfall–runoff model, such as that provided in the **Model for Urban Stormwater Improvement Conceptualisation (MUSIC)** (Cooperative Research Centre for Catchment Hydrology 2003), for a real (or synthetic) rainfall series.

Based on the studies of water quality performance reviewed Fletcher et al. (2003), the pollutant removal by porous pavement appears to be relatively consistent. However, this finding should be viewed with some caution, because it may reflect at least, in part, the lack of studies which have specifically reported on performance relative to input variables, such as inflow concentration, hydraulic loading, and properties of the pavement.

Table 14.2 provides a summary of expected performance of porous pavements, based on the studies reviewed by Fletcher et al. (2003).

14.4.3 Maintenance

Porous pavements are permeable pavement with an underlying storage reservoir filled with aggregate material. Modular block pavements (including lattice block pavements) or permeable pavements overlie a shallow storage layer (typically 300 mm–500 mm deep) of aggregate material that provides temporary storage of water prior to infiltration into the underlying soils. Maintenance activities vary depending on the type of porous pavement (Table 14.3). In general, porous pavement should be inspected for cracks and holes, and removal of accumulated debris and sediment should be undertaken every three to six months. Depending on the design of lattice pavements, weeding or grass mowing may need to be undertaken. If properly maintained, and protected from 'shock' sediment loads, porous pavements should have an effective life of at least 20 years (Bond et al. 1999, Pratt 1999, Schluter et al. 2002 as cited in Fletcher et al. 2003).

14.4.4 Capital costs and maintenance costs

The capital cost of porous pavements is disputed, with conflicting estimates given, but consensus is that its cost is similar to that traditional pavement, when the total drainage infrastructure cost is taken into account Landphair et al. (2000). This conclusion is supported by a trial of several types of porous pavements, based on real case studies in the Puget Sound. The long-term maintenance costs remain relatively unknown, with no reliable Australian data available.

Some estimates of porous pavement costs were provided at a recent workshop run by 'Water Sensitive Urban Design (WSUD) in the Sydney Region' (www.wsud.org) in March 2003 (no maintenance costs were provided):

- permeable paving allowing infiltration: A\$111/m^2
- permeable paving over sealed subgrade, allowing water collection: A\$119/m^2

Table 14.3 Porous pavement maintenance issues

Design category	Maintenance activities and frequency	Equipment	Design attributes that facilitate maintenance activities
Modular block, lattice pavements or permeable pavements	Maintenance activities for porous pavements should be undertaken every 3 to 6 months and may include: • Inspection of pavement for holes, cracks and excessive amounts of accumulated materials • Removal of accumulated debris and sediment on surface of pavements • Hand weeding largely for aesthetic purposes • Mowing of grass if used between lattice pavements • Periodical removal of infiltration medium (about every 20 years) and replacement of geotextile fabric to ensure permeability is maintained to the underlying soils	• High suction vacuum sweeper and high pressure jet hoses • Gloves, spade, hoe • Lawn mower and waste removal vehicle • Bobcat or excavator and waste removal vehicle (e.g. tipper truck)	• Separate the upper 300 mm using geotextile fabric for easy removal and replacement of upper component • Recommended for **low traffic volume** areas only • Recommended for use in low sediment loading areas • Invert of system should be at least 1 m above impermeable soil layer and seasonal high watertable • Allowance should be made for a 50% reduction in design capacity over a 20 year lifespan

- permeable paving with concrete block paving: A$98/m^2 with infiltration, A$122/m^2 with water collection
- permeable paving with asphalt: A$67/m^2 with infiltration or A$80/m^2 with water collection
- permeable paving with concrete block: A$90/m^2 with infiltration, A$116/m^2 with water collection.

The Californian Stormwater Quality Association (www.cabmphandbooks.com) have produced a handbook for best practice stormwater management in new development and re-development (http://www.cabmphandbooks.com/Development.asp). The report draws on research undertaken by Landphair et al. (2000), who reported annual maintenance costs of about A$9700 per hectare per year. Little information was given on what basis this was calculated. Based on amortised construction and maintenance costs over 20 years, this equated to around A$9 per kilogram of TSS removed, inclusive. Landphair et al. also lament the lack of life cycle cost data for stormwater treatment measures, including porous pavements, and point out that both construction and maintenance costs are very site specific. Although some local data may be available, there are not the cost relationships which allow maintenance costs to be predicted for any given site.

14.4.5 Protection and maintenance of porous pavements

Along with evidence of many successful implementations of porous pavements, there are many instances of failure, because of clogging. *It is absolutely critical that porous pavements are protected from large sediment loads during and shortly after the construction phase.* Failure to do so could see the effective lifespan of the pavement reduced to less than 10% of the predicted lifespan.

14.4.6 Design and supply of porous pavements

There are several suppliers of both monolithic and modular porous pavement systems within Australia (although for commercial equality reasons they are not listed here). When seeking information from suppliers on their products, the following information should be sought:

- cost/m^2, including supply and installation (taking into account site conditions)
- required depth of installation and details of the subbase, geotextile and associated components
- maintenance requirements and pollutant collection processes for particular pavement
- independent performance data (infiltration capacity and pollutant removal)
- potential for application of porous pavement for only part of the paved surface (and effects on infiltration and pollutant removal performance).

14.5 Use of natural areas including reforestation and revegetation

Another technique considered worthy of consideration is the use of reforestation and revegetation measures. The following text, largely based on material contained in Auckland Regional Council (2003), should provide initial guidance to practitioners in this regard.

This technique involves the utilisation of existing areas of vegetation, from forested areas to scrub vegetation to pasture areas. The scale of this approach can be made to vary. In a micro sense, redirecting pathway and driveway stormwater runoff onto adjacent grassed or otherwise vegetated areas (also referred to as the minimisation of directly connected impervious areas), illustrates this concept of natural area use. All such opportunities should be considered where redirection can be done without causing problems, such as concentrated flow increasing slope erosion.

For those situations where vegetation already exists, use of that vegetation or enhancement of the vegetation is a good approach. Significant benefits can be gained also by reforesting or revegetating portions of sites that would improve an existing situation or restore a degraded resource.

Reforestation/revegetation includes the planting of appropriate tree and shrub species, coupled with the establishment of an appropriate ground cover around trees and shrubs in order to stabilise soil and prevent an influx of invasive plants and weeds. The practice is highly desirable because, in contrast to many other management approaches, reforestation actually improves in its stormwater performance over time.

Reforestation benefits relate closely to benefits cited in the literature on riparian stream **buffer** protection, although reforestation is not linear in configuration.

Plant species should be selected carefully to match indigenous species that exist in the area and care should be taken to use species that reflect the combination of environmental factors which characterise the area.

Reforestation areas need periodic management, at least for the first five years. This will ensure good survival rates for the newly planted stock. The level of management decreases as the plantings mature. During the first two to three years, annual spot applications of herbicide may be necessary around the planted vegetation to keep weeds from outcompeting the new trees and shrubs for water and nutrients.

To the extent that vegetation of different types is already established, the stabilised natural area offers various physical, chemical and biological mechanisms which should further maximise contaminant removal as well as attaining water quantity objectives.

14.6 References

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Appendix A **Suggested plant species for WSUD treatment elements**

A.1 Introduction

Appendix A provides a list of plants that are suitable for different Water Sensitive Urban Design treatment elements, including:

1. **sediment basins**
2. **bioretention swales**
3. **bioretention basins**
4. **swales** and **buffer** strips
5. **wetlands**
6. ponds.

Tables A.1 and A.2 (located at the end of this Appendix) are to be used as a guide to select appropriate species to perform a water quality function. Once species are selected from these tables they should be checked for consistency with local recommended species. Indigenous nurseries and/or other relevant agencies (Councils, Catchment Management Authority and Melbourne Water) should be consulted as part of the plant selection process.

Table A.1 includes plants suitable for bioretention swales, bioretention basins, buffer strips and swales. Table A.2 includes plants suitable for sediment basins, wetlands and ponds. These plant species are principally categorised according to their water depth. Littoral vegetation can be planted around all of the systems. Ponds will have submerged vegetation. Wetlands that have a full depth range will include plants recommended for all of the six zones [littoral, **ephemeral** marsh, shallow marsh, marsh, deep marsh and pool (submerged marsh species)].

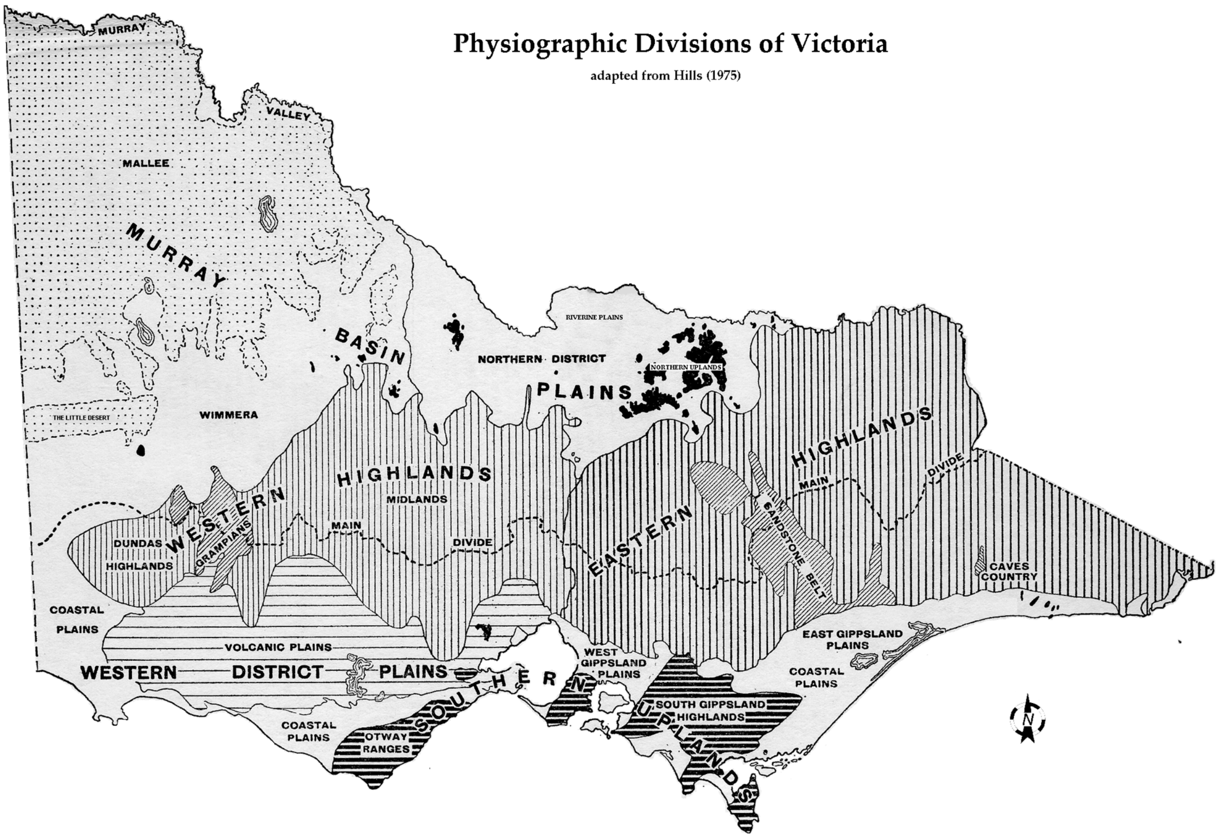

Figure A.1 Physiographic divisions of Victoria (from Hills 1975).

Most of the species listed in Tables A.1 and A.2 are widespread and occur throughout Victoria. Many species that will also be suitable for planting in WSUD elements will occur on a regional basis. Hills and Sherbon (1975) classified the physiography of Victoria into 11 regions (Figure A.1). The physiogeographic regions, or bioregions, are influenced by topography and elevation, climate, geology and edaphic (soil) characteristics (Figure A.1). The physiographic regions roughly correspond with the hydrologic regions outlined in this Manual. The hydrologic regions (Figure A.2) can be used to guide the selection of appropriate species (i.e. regionally endemic) throughout Victoria. Figure A.3 can be used as an initial guide to the soil types likely to be found in Melbourne suburbs; however, to select the most suitable plants (those from the local area) a thorough understanding of local soils is required (possibly involving laboratory testing).

Rather than solely using plants with a wide distribution, plants that are local to a particular bioregion can be used. Plants that occur in a particular bioregion will be well adapted to the local conditions and will add and enhance regional biodiversity. Use of locally occurring plants, some of which might be endemic, will encourage regional fauna.

A.2 Bioretention systems, swales and buffer strips

These WSUD elements typically treat **stormwater** close to its source (surfaces that water runs off). These elements include bioretention swales, bioretention basins, swales and buffer strips. Swales and buffer strips are typically constructed using local soils whereas soils in bioretention systems require specific hydraulic characteristics and local soils may require amendment. In some cases imported soils will be required.

Bioretention soils must meet filter media specifications (primarily a prescribed hydraulic conductivity) in addition to supporting plant growth (see Chapters 5 and 6).

Sandy loam soils are commonly used in bioretention systems because they typically have particle size distributions similar to suspended solids in urban stormwater runoff and therefore provide good retention of suspended particles. While sandy loams are typically used, other soil types can be used that suit the local vegetation, if they will support plant growth and are amended to meet the system requirements.

A.2.1 Constructing suitable soil/filter media

To ensure the soil/filter media provides for a design hydraulic conductivity and is able to support plant growth the following approach is suggested.

1 Identify if local topsoil is capable of supporting vegetation growth and if there is enough topsoil (some topsoils are very shallow) to be used as a base for the filter media (may require active collection of topsoil during the construction process). Any topsoil found to contain high levels of salt, extremely low levels of organic carbon (<<5%), or any other extremes which may be considered as a retardant to plant growth should be rejected. If the topsoil is not suitable, a sandy loam soil can be purchased from a supplier for use as a base soil.
2 Conduct laboratory tests to estimate the saturated hydraulic conductivity of the topsoil/base soil using standard testing procedures (see Appendix H in AS 4419).
3 If the soil needs to be amended to achieve the desired design saturated hydraulic conductivity, either mix in a loose non-angular sand (to increase saturated hydraulic conductivity) or a loose soft clay (to reduce saturated hydraulic conductivity).
4 The required content of sand or clay (by weight) to be mixed to the base soil will need to be established in the laboratory by incrementally increasing the content of sand or clay until the desired saturated hydraulic conductivity is achieved (within reasonable bounds). The sand or clay content (by weight) that achieves the desired hydraulic conductivity should then be adopted on-site.
5 The base soil should have sufficient organic content to establish vegetation on the surface of the bioretention system. If the proportion of base soil in the final mix is less than 50%, then it may be necessary to add in additional organic material to the mix but should not result in more than 10% organic content (measured in accordance with AS1289 4.1.1).
6 The pH of the soil mixture for the filtration layer is to be amended to between 6 and 7, before delivery to the site.

A.2.2 Importance of vegetation

Vegetation is an integral component of the treatment systems. The vegetation needs to fulfil several functions such as the following.

1 Provide a surface area to trap suspended solids and other pollutants as the water flows horizontally through the treatment systems.
2 Produce a biologically active root zone to help the removal of pollutants as water infiltrates vertically. This function is crucial for bioretention systems.
3 Reduce soil compaction and maintain infiltration rate.
4 Decrease flow velocities and bind and stabilise the substrate, thereby limiting erosion.
5 Create a prominent and diverse landscape element in the development and enhance local biodiversity.

A.2.3 Required plant characteristics

The species outlined in Table A.1 have been specifically selected, based on their life histories, physiological and structural characteristics to meet the functional requirements of swale/ bioretention systems. Other species can be used as long as they can fulfil the functional roles described below.

In general, plant species that satisfy these functional roles have the following general features:

1 are able to tolerate short periods of inundation punctuated by longer dry periods – these dry periods may be reasonably severe due to the free-draining nature (relatively low water-holding capacity) of bioretention filter media
2 have either a prostrate or erect habit.
3 if prostrate, would be typically low mat-forming stoloniferous or rhizomatous plants (e.g. Couch Grass, *Cynodon dactylon*; *Phyla nodiflora, Dichondra repens*)

4 if erect, would be typically rhizomatous with simple vertical leaves (e.g. Rush, *Juncus* spp.; *Carex* spp.)
5 preferably would have spreading rather than clumped growth forms
6 would be perennial rather than annual
7 would have deep, fibrous root systems
8 would form the understorey if also grown with shrubs and trees..

Well-established uniform vegetation is crucial to the successful operation of drainage swale and bioretention systems. As a result, both the aesthetic and functional requirements of the systems need to be considered when the species are selected.

Swale/bioretention system vegetation can be either single or mixed species designs. Herbaceous groundcover species (e.g. *Phyla nodiflora, Brachyscome multifida;* Kidney Weed, *Dichondra repens*) are nearly always best planted as mixtures. Grasses, rushes, sedges and lilies can typically be planted as single (e.g. Tall Sedge, *Carex appressa*) or mixed species (e.g. *Pennisetum alopecurioides, Dichelachne crinata*; and Weeping Grass, *Microlaena stipoides*) stands depending on the landscaping requirements. Some of the prostrate shrubs that form scrambling thickets may be better suited to single species planting (e.g. *Hibbertia scandens,* and *Hardenbergia violacea*) in isolated areas for aesthetic impact. These species may also require pruning to ensure even plant cover and to maintain an even root distribution below ground.

Planting density generally varies depending on the species and the type of stock specified. Some lawn and turf species could be established from seed, hydroseeding or established as rolled on turf. Native grasses, rushes, sedges and lilies are typically supplied in small tubes (35–60 mm). In drainage swale/bioretention systems this stock should be planted at high densities (12–16 plants/m^2). Dicotyledon species (e.g. *Goodenia hederacea,* and *Hibbertia scandens*) are typically supplied in pots (50 mm). In drainage swale/bioretention systems this stock should also be planted at high densities (8–10 plants/m^2). These high densities are required to ensure runoff does not establish preferential flow paths around the plants and erode the swale surface. High density planting is also required to ensure a uniform root zone in the bioretention systems.

A.2.4 Plant species selection

Plant species suitable for use in bioretention systems, buffer strips and swales are listed in Table A.1.1. The suggested species occur in Victoria. Most of the species are widespread but some only occur in specific regions or in certain conditions (e.g. substrate type or salinity). Species' ranges should therefore be checked before they are recommended for a particular site.

The plant list in Table A.1 is not exhaustive. A diverse and wide-range of plants can be used for WSUD elements (subject to the characteristics described in Section A.2.3). Table A.1 includes only plants indigenous to Victoria. Non-indigenous natives and exotics should only be considered when there is a specific landscape need and the species has the appropriate growth form, habit and patterns of wetting and drying.

If non-indigenous natives and exotics are chosen, careful consideration should be given to the potential effects on downstream drainage systems. For example, Japanese Sacred Bamboo (*Nandina domestica*) and Carpet Weed (*Phyla nodiflora*) are both suitable for use in onsite WSUD elements. Similarly, species that are endemic to particular regions within Victoria (i.e. indigenous but not widespread) can be used.

Plant species should be selected based on several factors:

1 objectives, besides treatment function, of the WSUD element (e.g. landscape, aesthetics, biodiversity, conservation and ecological value)
2 region, climate, soil type and other abiotic factors
3 roughness of the channel (if a conveyance system)
4 extended detention depth.

Species that have the potential to become invasive weeds should be avoided.

The typical heights of the plant species (listed in Table A.1) will help with the selection process. Low-growing and lawn species are suitable for conveyance systems that require low roughness coefficients. The treatment performance of bioretention systems, in particular, requires dense vegetation to a height equal to that of the extended detention depth. Therefore, a system with a 300 mm extended detention should have vegetation at least 300 mm high. All of the selected plant species are able to tolerate periods of both wetting and drying.

Included in Table A.1 is the recommended planting density for each of the species. The planting densities recommended should ensure that 70–80% cover is achieved after two growing seasons (two years).

Although low-growing plants (e.g. grasses, sedges and rushes) are usually used, trees and shrubs can be incorporated into WSUD elements. If using trees and shrubs in bioretention systems, they should be planted in the local soil adjacent to the filter medium, so that the roots do not interfere with the perforated pipes. Shrubs listed provide a wide range of sizes from small to large. Geotechnical advice may be required if using trees in some systems.

A.2.5 Vegetation establishment and maintenance

Conventional surface mulching of swale/bioretention systems with organic material such as tanbark should not be undertaken. Most organic mulch floats and runoff typically causes this material to be washed away with a risk of causing drain blockage.

New plantings need to be maintained for a minimum of 26 weeks. Maintenance includes regular watering, weed control, replacement of dead plants, pest monitoring and control, and rubbish removal. Once established, lawn, grass and groundcover plantings will need to be mown to maintain the design vegetation height.

A.3 Sediment basins, wetlands and ponds

The WSUD elements sediment basins, wetlands and ponds typically treat stormwater away from its source. Stormwater may be transported through a conventional drainage system or it may be transported via WSUD elements, so would receive some pretreatment.

A.3.1 Importance of vegetation

Sediment basins are designed to trap coarse particles (>125 μm) before the stormwater enters a wetland. Aquatic vegetation is therefore not specified for the sediment basins except in the littoral zone around the edge of the basin. The littoral vegetation is not part of the water quality treatment process in sediment basins so it is not essential. However, plants can stabilise banks, so vegetation should be prescribed if erosion is a potential problem. Dense planting of the littoral berm zone also inhibits public access to the treatment elements, minimising the safety risks posed by water bodies. It can also improve the aesthetic appeal of the landscape and screen basins, which are typically turbid.

Ponds are principally designed to be open water features providing landscape value. Unless the ponds have hard edges, littoral vegetation should be planted along the edges. These plants will provide habitat for local fauna, will help to stabilise the banks against erosion, and will inhibit weed invasion. Littoral vegetation also plays a treatment role when the water is above normal water level. Dense planting of the littoral zone will also inhibit public access to ponds, minimising the safety risks posed by water bodies.

Submerged plants should be planted in the deep areas of ponds. Submerged plants will be seen occasionally, such as after a long dry period, when they surface to flower and seed, or when birds rip up plant fragments. However, they will mostly be totally submerged and will provide an open water perspective. Submerged plants are crucial for maintaining high water quality and minimising the chance of an algal bloom. They also inhibit weed invasion.

Wetlands are dominated by emergent **macrophytes** (aquatic plants). **Constructed wetlands** are designed to trap the fine polluted particles (<125 μm) where they can be safely stored for long periods (15–20 years). Wetland plants extract nutrients and other dissolved substances, and provide a framework for microbial **biofilms**. Wetlands, therefore, clean water through biotic absorption, ingestion and decomposition of pollutants, as well as other chemical transformations resulting from the range of oxidation states.

Vegetation should also be planted along the edges of wetlands. Littoral vegetation will help to filter and treat water during times when the water is above normal water level. Dense planting of the littoral zone will also inhibit public access to the treatment elements, minimising potential damage to the plants and the safety risks posed by water bodies.

A.3.2 Required plant characteristics

The species outlined in Table A.2 have been specifically selected, based on their life histories, physiological and structural characteristics, to meet the functional requirements of wetland systems. This includes consideration of the wetland zone/depth range and the typical extended **detention time** (48–72 h) and depth (0.5 m). Other species may be used to supplement these core species, although they must be selected to suit the particular depth range of the wetland zone and have the structural characteristics to perform particular treatment processes (e.g. distribute flows, enhance **sedimentation**, maximise surface area for the adhesion of particles and/or provide a substratum for algal epiphytes and biofilms). In general, species that perform these functions have the following general features:

1. grow in water as emergent macrophytes (e.g. marsh species) or tolerate periods of inundation (e.g. ephemeral marsh species), typically sedges, rushes or reeds.
2. generally have rhizomatous growth forms
3. should be perennial rather than annual
4. are generally erect with simple vertical leaves (e.g. Twig-rushes, *Baumea* spp.; and Rushes, *Juncus* spp.)
5. have spreading rather than clumped growth forms
6. should have a fibrous root system
7. would form an understorey if grown with shrubs and trees (which are generally only planted in the littoral or ephemeral zones).

The locations within a wetland that are best suited to specific wetland plants are determined by the interaction between basin **bathymetry**, outlet hydraulics and **catchment** hydrology – the hydrologic regime (Wong et al., 1998). Individual species have evolved preferences for particular conditions for the length of water depth-inundation periods and this must be checked (e.g. with wetland plant suppliers/nurseries) prior to recommending them for a particular wetland zone planting.

A.3.3 Plant species selection

The plant species listed in Table A.2 have suitable life histories, physiological and structural characteristics for sediment basins, wetlands and ponds. The distribution of the species within the wetland relates to their structure and function. Plants recommended for shallow marsh should be used in shallow marsh and not deep marsh, for example. The planting densities recommended should ensure that 70–80 % cover is achieved after two growing seasons (two years).

Suitable plant species have also been recommended for the littoral zone that will surround the wetlands, ponds and sediment basins. The littoral zone (berms or batters) refers to the perimeter of the treatment elements and extends over a depth range of 0–0.5 m. Plants that have a drier habit should be planted towards the top of batters, whereas those that are adapted to more moist conditions should be planted closer to the water line.

When selecting plants for wetlands, wetlands should be divided into a series of zones based on their water depth [pools (or submerged marsh), deep marsh, marsh, shallow marsh, ephemeral marsh and littoral zones]. The relative size of the zones is determined by the wetland bathymetry. Table A.3 shows the typical permanent depth ranges of the six zones commonly found in wetlands. The zones referred to in Table A.2 correspond with the depth ranges shown in Table A.3. Some plant species can be used in more than one zone, but plant species are generally categorised into one zone based on their preferred water range.

Like the plants in Table A.1, Table A.2 provides examples of the plants that can be used in Victorian wetlands. The plant species listed in Table A.2 are recommended as the core species for the zones, but several other plants could be used. The species recommended are all thought to satisfy the functional treatment requirements of the zone, and are adapted to the hydrologic conditions of the zone. Indigenous species are generally recommended as they provide habitat for native wetland fauna.

Table A.3 Depth ranges of wetland macrophyte zones
Depths refer to the mean water depth at Normal Water Level (NWL) for the summer permanent pool. Natural variation below the NWL is expected to regularly expose the shallow marsh section and much of the marsh section. During events water will temporarily be stored above the NWL and inundate the ephemeral section.

Zone	Macrophyte zone type	Depth (m)
P	Pool – submerged marsh	0.5– ~ 1
DM	Deep marsh	0.35–0.5
M	Marsh	0.2–0.35
SM	Shallow marsh	0–0.2
EM	Ephemeral marsh	+0.2–0
L	Littoral	+0.5–0

A.3.4 Wetland vegetation establishment

To maximise the success of plant establishment in wetland macrophyte zones the following vegetation establishment program is recommended. The program outlines procedures involved in site preparation, vegetation preparation, planting and maintenance.

Plant growth medium

After establishing a bathymetry of the wetland, a layer of topsoil is required as a substrate for aquatic vegetation. Although there are a few plants that can grow in subsoils such as heavy clays (e.g. Phragmites), growth is slow and the system would have low species richness, which is deemed undesirable. Wetlands should therefore have a layer of topsoil not less than 200 mm deep (deeper if possible). Topsoil removed from a site during excavation should be stockpiled for subsequent use as a growth medium for wetland macrophytes. If the topsoil is unsuitable (i.e. will not support plant growth, wetland plants typically prefer silty to sandy loams), it is advisable to purchase appropriate soils from a supplier. If stockpiled topsoil is to be used, it is recommended that it be screened to remove any coarse organic mater prior to placement in a wetland. Other topsoil treatment requirements are listed below.

Soil treatment

The topsoil covering the bed of a system (macrophyte zone and open water zones) should be treated with gypsum or lime (standard on most construction sites). By facilitating flocculation, gypsum will reduce the turbidity of the water column, which will be particularly valuable in the early stages of establishment of the wetland system. With lower turbidity, higher levels of light will be able to reach the plants, thereby facilitating their growth and establishment. It is important that the gypsum not be added too far in advance of the vegetation planting; with clear water and no aquatic plants competing for resources, conditions will be favourable for algal growth, thus increasing the threat of an algal bloom. The gypsum should be applied about one week prior to planting at a rate of 0.4 kg/m^2. Subsequent application may be required at intervals depending on **pond** condition and the amount of exchangeable sodium. Fertilisers should not be applied to the topsoil or to terrestrial areas in or around the wetland system, particularly in the early stages of plant establishment, due to the threat posed by algal blooms, particularly cyanobacteria (blue-green algae). The inadvertent addition of nutrients to the wetland system could facilitate the growth of cyanobacteria, particularly when the competing macrophytes and submerged plants are in their early developmental stages, thus raising the likelihood of algal blooms.

Plant propagation

Plants should be ordered from a vegetation supplier prior to the time of planting to enable the supplier sufficient time to grow the required number of plants and species types and for the plants to grow to a suitable size (maturity) to ensure low mortality rates. The supplier should be made aware of the planned planting layout and schedule.

To ensure successful establishment of wetland plants, particularly in deeper marsh zones, it is strongly recommended more mature tube stock be used (i.e. at least 0.5 m in height). For shallower zones of a wetland, younger tube stock or seedlings may suffice.

As a minimum a nursery should provide the following plant stock for deep marsh and marsh zone planting:

1 50 mm tube stock
2 3–4 shoots or leaves
3 500–600 mm height.

As a minimum a nursery should provide the following plant stock for shallow marsh and ephemeral marsh zone planting:

1 preferably 50 mm tube stock but 25 mm container stock should suffice
2 4–5 shoots or leaves
3 300–400 mm height.

Smaller (20 mm) seedling pots should be avoided as these seedlings are considered to be relatively immature and will result in high loss rates and patchy growth.

Planting water level manipulation

To maximise the chances of successful establishment of the vegetation, water levels within a wetland system should be manipulated in the early stages of vegetation growth. When first planted, vegetation in the deep marsh and pool zones may be too small to exist in their prescribed water depths (depending on the maturity of the plant stock provided). Seedlings intended for the deep marsh sections will need to have at least one-third of their form above the water level. This may not be possible if initially planted at their intended depth. Similarly, if planted too deeply, the young submerged plants will not be able to access sufficient light in the open water zones. Without adequate competition from submerged plants, phytoplankton (algae) may proliferate.

Water depth should, therefore, be controlled in the early establishment phase. Deep marsh zones should have a water depth of about 0.2 m for the first 6–8 weeks. This will ensure that deep marsh and marsh zones are inundated at shallow depths and the shallow marsh zone remains moist (muddy) which is suitable for plant establishment. After this period, water levels can be raised to normal operating levels.

Planting

Planting in all zones of a wetland should occur at the same time. With water levels controlled as described in the previous section, deep marsh and marsh zones will be inundated with water and the shallow marsh zone will be moist to allow appropriate growth (however, some water over shallow marsh zones may be required). Planting of ephemeral zones will require irrigation at a similar rate as terrestrial landscaping surrounding the wetland.

Establish operating wetland water level

After six to eight weeks of growth at a controlled water level, wetland plants should be of sufficient stature to endure deeper conditions so the wetland can be filled to its normal operating water level. Therefore, after eight weeks, the connection between the inlet pond and the macrophyte zone should be temporarily opened to allow slow filling of the wetland to normal operating water level. Once filled to normal water level, the connection between inlet pond and macrophyte zone should again be closed to allow further plant establishment without exposure to significant water level variations.

Connecting the inlet pond to the macrophyte zone

The temporary blockage located on the connection between the inlet pond and the macrophyte zone can be removed once the building construction within the wetland catchment has been completed. At this time it will be necessary to desilt the inlet pond which will have been operating as a sediment basin during the building phase. Planting of the zones disturbed during desilting will be required.

Vegetation assessment

Ensure the wetland is operating at the end of the construction landscape period and the planted macrophytes are established and healthy at the operating water level. If successful, the wetland should have a 70–80% even macrophyte cover after two growing seasons (two years).

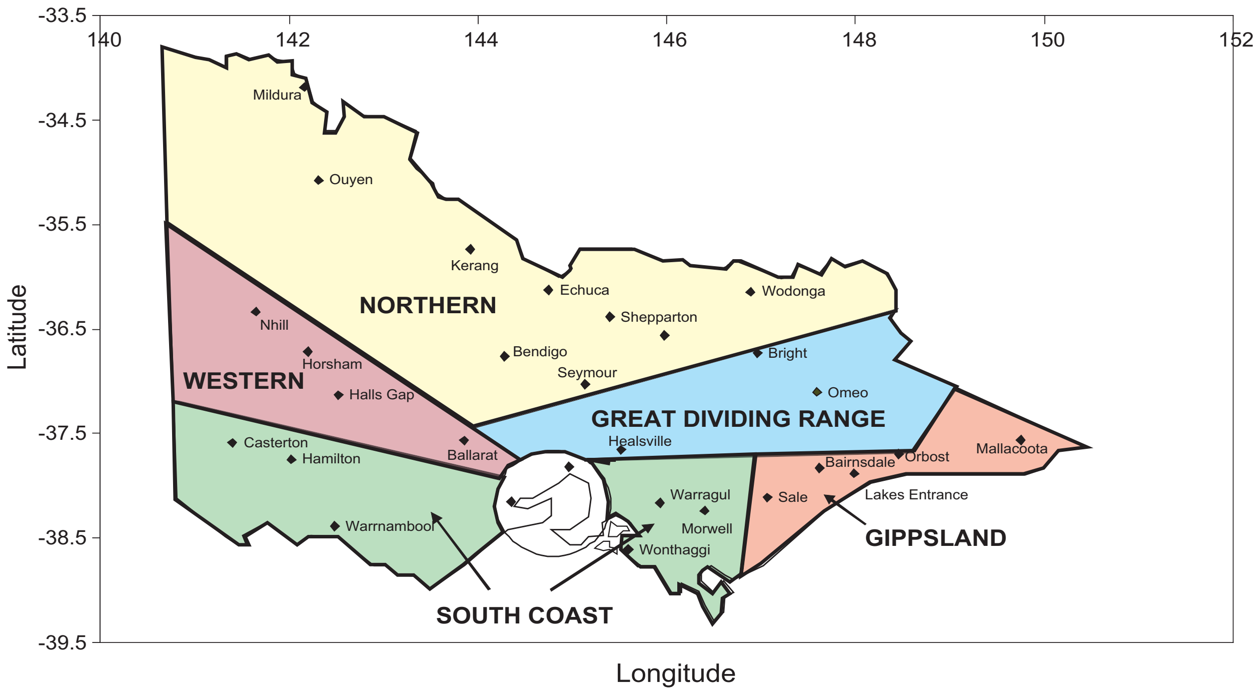

Figure A.2 Map of Victoria indicating statewide recommended vegetation regions.

A.4 Steps to choosing appropriate vegetation

The following steps should be followed when selecting vegetation for WSUD treatment elements.

1. Determine what soil type is in the local area and if it requires amendment to meet the prescribed hydraulic conductivity (for bioretention systems) and/or amendment to support plant establishment.
2. Refer to Tables to select appropriate species for each macrophyte zone (Table A.2) or swale/bioretention system (Table A.1).
3. Ensure species selection is consistent with the local hydrologic regions (listed in Tables A.1 and A.2) (see Figure A.1).
4. Consult local indigenous nurseries and/or other relevant agencies (e.g. councils, CMA and Melbourne Water) to ensure consistency with local vegetation strategies, avoiding locally invasive or exotic species and selecting for locally indigenous species.
5. Where species listed in the Tables do not comply with local vegetation strategies seek advice from relevant agencies regarding alternative species with similar characteristics.

A.4.1 Additional notes on the tables

1. The **planting stock** of the different species recommended will require differing degrees of maturity at planting. For example, even though water level management is recommended at planting times, deep marsh species will need to be more advanced stock suitable for planting in deeper water than the species recommended for the shallow marsh zone.
2. **Planting density** indicates the mean number of plants per square metre for the species spatial distribution within the zone. The planting densities recommended are suggested minimums. While planting density can be either increased or decreased depending on budget, any reduction in planting density has the potential to reduce the rate of vegetation establishment, increase the risk of weed invasion and increase maintenance costs.
3. The **total number of plants** required for each zone can be calculated as follows.

Number of plants = (recommended planting density × section area × proportion of section planted × cover density),

where 'the proportion of the section' planted refers to the proportion of the section area that will be planted with the identified species; and where 'cover density' refers to the proportional

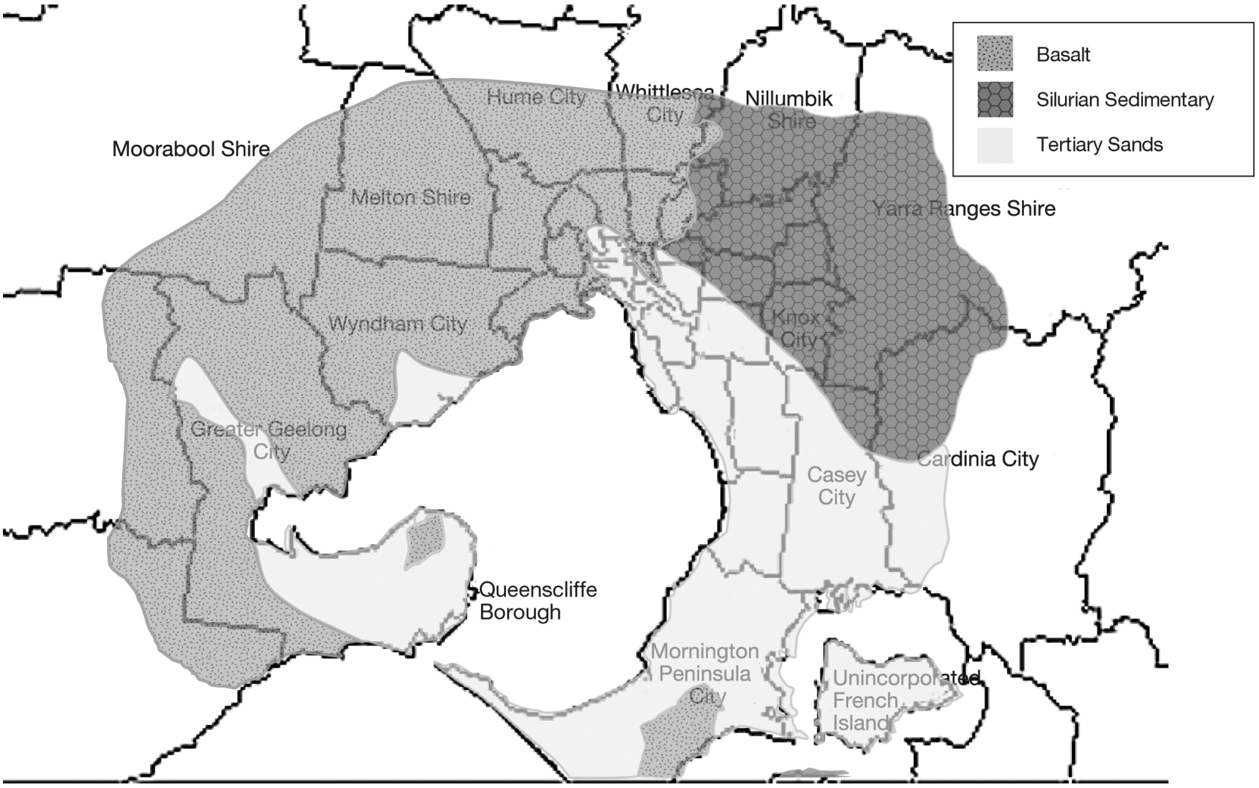

Figure A.3 Map of metropolitan recommended vegetation regions based on broad soil types (after Australian Plants Society Maroondah 2001; and Land Conservation Council 1973).

cover of that particular plant species in the designated location. The cover density of all of the plant species in a given area typically sums to 1.0.

A.4.2 Key to plant species (Tables A.1 and A.2)

Tables A.1 and A.2 outline suggested plant species for various WSUD treatment elements. The key to these tables is given below.

Type/zone		Form	
DM	Deep marsh	T	Shrubs and trees
EM	Ephemeral marsh	G	Groundcover
F	Forest		
L	Littoral	E	Erect herbs
M	Marsh	S	Submerged macrophytes
P	Pool (submerged marsh)		
SM	Shallow marsh	M	Emergent macrophytes

Recommended vegetation regions (see Figures A.2 and A.3)	
Statewide (after Walsh and Entwistle 1999)	
BA	Basalt
GDR	Great Dividing Range
GIP	Gippsland
Metro	Metropolitan (after Australian Plants Society Maroondah 2001)
N	Northern
SC	South Coast
SS	Silurian Sedimentary
TS	Tertiary Sands
W	Western

A.4.3 References

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Table A.1 Plant species for bioretention systems, swales and buffer strips (indicative)

Scientific name	Common name	Form	Height (m)	Description	Planting density (plants/m^2)	Requirements	Comments	Region
Blechnum cartilagineum	Gristle Fern	E	0.5–1.5	Upright tufting fern with short creeping stoloniferous rhizomes, forming spreading patches	2–4	Moist, well drained soils; tolerates drier conditions once established	Aesthetic; readily available	SC, GDR, GIP Metro: SS
Chrysocephalum apiculatum	Common Everlasting	E	Prostrate – 0.3	Variable, dense spreading perennial herb	2-4	Well drained soils	Aesthetic; widespread	Statewide Metro: All
Dianella longifolia var. longifolia	Pale Flax-lily	E	0.3–0.8	Tufted perennial clump with short rhizomes	8	Moist, well drained soils	Aesthetic; easy maintenance; ideal under trees	Statewide Metro: All
Epacris impressa	Common Heath	E	0.5–1.5	Open wiry shrub	2–4	Moist, well drained soils; tolerates limited dry or wet periods once established	Victoria's floral emblem	W, SC, GDR, GIP Metro: All
Hibbertia prostrata	Stalked Guinea-flower	E	0.3–0.6	Low erect subshrub	4–6	Moist, well drained sandy soils; not clay	Difficult in clay soils	W, SC, GIP Metro: TS
Pimelea linifolia ssp. linifolia	Slender Rice-flower	E	Prostrate – 1.2	Variable prostrate erect or clump-forming, depending on habitat	4–6	Well drained soils	Pruning encourages branching	Statewide Metro: BA, SS
Stypandra glauca	Nodding Blue Lily	E	0.5–1.5	Dark green leafy foliage; bright blue drooping flowers	2–4	Moist, well drained soils	Aesthetic; benefits from occasional pruning	Statewide Metro: All
Viola hederacea	Native Violet	E	Prostrate – 0.15	Stoloniferous herb forming a dense mat	2–6	Moist to wet soils	Aesthetic; rapid growth, prolific once established	W, SC, GDR, GIP Metro: All
Dichondra repens	Kidney Weed	G	Prostrate	Dense spreading herb, forms mats	6–8	Moist, well drained soils; tolerates drying once established	Alternative to grass where foot traffic is light; more vigorous when cultivated; widespread	Statewide Metro: All
Myoporum parvifolium	Creeping Boobialla	G	Prostrate	Dense matting groundcover	4–6	Well drained soils, tolerates dry periods once established	Adaptable groundcover; layering habit useful for soil binding; rare within Melbourne region	N, W, SC Metro: BA
Microlaena stipoides	Weeping Grass	G, E	0.3–0.6, stems 0.6–1.0	Highly variable in size	turf or seeds	Moist, well drained soils	Aesthetic; suitable as a lawn grass; widespread	Statewide (except far N) Metro: All

Table A.1 Plant species for bioretention systems, swales and buffer strips (indicative) (Continued)

Scientific name	Common name	Form	Height (m)	Description	Planting density (plants/m^2)	Requirements	Comments	Region
Carex appressa	Tall Sedge	M	0.5–1.2	Dense, robust and tough; hairy and sticky	4–8	Very moist soils, tolerates periods of inundation and dryness	Stabilises banks against erosion; tough; slow-growing; high surface area; dominates zones	Statewide Metro: All
Carex fasicularis	Tassel Sedge	M	0.5–1.0	Coarse, tufted plant	6-8	Moist soils	Aesthetic	Statewide Metro: SS
Carex inversa	Knob Sedge	M	0.1–0.3	Small tufted or spreading clump	10	Moist, well-drained soils	Variable species; rapid establishment	Statewide Metro: All
Ficnia nodosa	Knobby Club-rush	M	0.5–1.5	Tall, coarse, wiry and densely tufted perennial rush with creeping rhizomes	6–8	Moist soils tolerates dry periods once established	Aesthetic (upright habit); widespread in Melbourne; binds soils in moist areas	N, W, SC, GIP Metro: All
Juncus amabilis	–	M	0.2–1.2	Rhizomatous tufted perennial rush	8–10	Tolerates inundation and dry periods once established	Widespread	Statewide Metro: All
Juncus flavidus	Yellow Rush	M	0.4–1.2	Rhizomatous tufted perennial rush; yellow-green	8–10	More tolerant of dry soils than other *Juncus* spp.	Aesthetic	Statewide Metro: All
Juncus gregiflorus	–	M	0.5–1.4	Rhizomatous tufted perennial rush	8–10	Moist, well drained soils	Widespread in Melbourne and eastern Victoria	W, SC, GDR, GIP Metro: All
Juncus procerus	–	M	1.0–2.0	Rhizomatous tufted perennial rush	8–10	Moist, well-drained soils in a sheltered position	Widespread in southern Victoria	W, SC, GDR, GIP Metro: SS, TS
Lepidosperma gladiatum	Coastal Sword-sedge	M	1.0–1.5	Leaves wide and flat with dark green blades	6	Moist, well drained sandy soils	Sharp-edged leaves - could be used to manage pedestrian traffic	SC, GIP Metro: TS (coastal)
Lepidosperma laterale	Variable Sword-sedge	M	0.5–1.0	Leaves wide and flat with dark green blades	6	Moist to wet soils but tolerates dry periods	Little maintenance once established	Statewide Metro: All
Lepidosperma longitudinale	Common Sword-sedge	M	0.6–1.7	Sedge with long, flat, dark green blades	6	Moist to wet soils	Aesthetic	W, SC, GIP Metro: SS, TS
Lomandra filiformis ssp. filiformis	Wattle Mat-rush	M	0.15–0.5	Small tussock with fine blades	6–8	Moist, well-drained clay or sandy soils; tolerates dry shaded positions once established	Little maintenance	N, SC, GIP Metro: All

Table A.1 Plant species for bioretention systems, swales and buffer strips (indicative) (Continued)

Scientific name	Common name	Form	Height (m)	Description	Planting density (plants/m^2)	Requirements	Comments	Region
Lomandra longifolia var. exilis	–	M	0.5–1.0	Large tussock with broad flat leaves	4–6	Well-drained soils	Tolerates drier conditions than l. Longifolia var. Longifolia	SC, GDR, GIP Metro: All
Lomandra longifolia var. longifolia	Spiny-headed Mat-rush	M	0.5–1.0	Large tussock	4–6	Well-drained soils; tolerates dry shaded positions	Grows well under established trees	W, SC, GDR, GIP Metro: All
Patersonia occidentalis	Long Purple-flag	M	0.2–0.5	Compact clumping perennial herb	6–8	Tolerates inundation and dry periods	Aesthetic; may not persist	W, SC, GIP Metro: SS, TS
Poa labillardieri	Common Tussock Grass	M	0.3–0.8, stems to 1.2	Large, coarse densely tufted tussock	6–8	Adapts to moist or slightly dry soils	Widespread	Statewide Metro: All
Poa morrisii	Velvet Tussock Grass	M	Prostrate – 0.3	Soft, dense	6–8	Moist well drained soils	Aesthetic	Statewide Metro: All
Schoenus melanostachys	–	M	0.5–1.0	Perennial with short stout rhizome; often forms big weeping tussocks	6–8	Moist soils	Tough; spreads to form dense clumps	GIP Metro: (Does not occur)
Callistemon sieberi	River Bottlebrush	T	3–10	Open to dense weeping shrub	1	Very wet to moist conditions in heavy clay soils but tolerates dry periods once established	Aesthetic; very adaptable; widespread	GDR, GIP Metro: BA, SS
Correa alba	White Correa	T	0.5–2.0	Dense, spreading shrub, dwarfed by wind and salt spray	2–4	Well drained soils; tolerates inundation and dry periods once established	Useful for soil binding	SC, GIP Metro: TS (coastal)
Correa reflexa	Common Correa	T	0.3–2.0	Very variable – open upright to spreading shrub	2–4	Well drained soils; dry shaded position	Establishes well under trees	Statewide Metro: SS, TS
Eucalyptus camaldulensis	River Red Gum	T	12–50	Large open spreading tree	< 1	Damp alluvial soils; deep subsoils; tolerates inundation and very dry periods once established	Aesthetic; some forms can be used to combat salinity; widespread	Statewide Metro: All
Eucalyptus ovata	Swamp Gum	T	8–30		< 1	Moist soils; tolerates inundation and dry periods; lake edge	Aesthetic; widespread	W, SC, GDR, GIP Metro: All
Kunzea ericoides	Burgan	T	2–5	Dense to open weeping shrub	< 1	Adaptable, tolerates inundation and dry periods	Aesthetic; rapid growth	Statewide Metro: SS, TS

Table A.1 Plant species for bioretention systems, swales and buffer strips (indicative) (Continued)

Scientific name	Common name	Form	Height (m)	Description	Planting density (plants/m^2)	Requirements	Comments	Region
Leptospermum continentale	Prickly Tea-tree	T	1–4	Rigidly upright, dense or straggling shrub or small tree	< 1	Adaptable; tolerates moisture	Two forms exist in Melbourne; widespread	Statewide Metro: SS, TS
Leptospermum lanigerum	Woolly Tea-tree	T	2–6	Dense shrub to erect small tree	< 1	Moist soils	Aesthetic	W, SC, GDR, GIP Metro: All
Leucopogon australis	Spike Beard Heath	T	1.0–1.5	Upright shrub	2–4	Well drained damp sandy soils	Strongly perfumed flowers	SC Metro: TS
Melaleuca ericifolia	Swamp Paperbark	T	2–9	Erect, open to bushy shrub or small tree	2–4	Moist to wet soils; tolerates dry periods once established	Aesthetic; very adaptable; once established	GP, SC Metro: All
Melaleuca squarrosa	Scented Paperbark	T	2–5 (rarely to 10)	Erect, open to compact large shrub or rarely, a small tree	<1	Moist to wet soils	Aesthetic; salt tolerant; grows well in coastal areas	W, SC, GIP Metro: SS, TS
Pimelea glauca	Smooth Rice-flower	T	0.3–0.6	Erect, many-branched glabrous shrub	2–4	Well drained soils	Aesthetic	Statewide Metro: All
Pultenaea daphnoides	Large-leaf Bush-pea	T	1–3	Erect branching shrub	2–4	Moist, well drained soils; tolerates dry periods once established	Aesthetic	Statewide Metro: SS, TS

Table A.2 Plant species for sediment basins, wetlands and ponds (indicative)

Scientific name	Common name	Zone	Form	Height (m)	Description	Planting density (plants/m^2)	Requirements	Comments	Region
Baumea articulata	Jointed Twig-rush	DM	M	1–2	Tall erect rhizomatous perennial	4	Moist soil to permanent water	Slow growth	Statewide Metro: All
Bolboschoenus fluviatalis	–	DM	M	1–2 (stems)	Semi-aquatic rhizomatous perennial rush	4	Moist soil to permanent water	Plant solo	N, GDR Metro: All
Eleocharis sphacelata	Tall Spike-rush	DM	M	0.5–2	Robust perennial herb with thick woody rhizome; clumps to big dense stands	6	Aquatic; to depth of 2 m; tolerates occasional drying	Plant solo, rhizomes can restrict growth of other plants; slow establishment	Statewide Metro: All
Juncus ingens	Giant Rush	DM	M	1.5–4	Dioecious rhizomatous perennial; in tussocks when grazed but can form uniform stands	6–10	Moist soil to seasonal inundation	Useful for bank stabilisation; slow spreading; dominant – plant solo	N Metro: (Does not occur)
Schoenoplectus tabernaemontani	River Club-rush	DM	M	Stems to 3	Robust, tufted rhizomatous herb	4	Moist soil to permanent water	Rapid establishment	Statewide Metro: All
Blechnum minus	Soft Water Fern	EM	G	0.5–1.2	Dense, erect clump forming spreading patches from underground stolons	4–6	Very moist soils; tolerates wet soils	Adaptable	W, SC, GDR, GIP Metro: SS, TS
Carex appressa	Tall Sedge	EM	M	0.5–1.2	Dense, hairy and sticky; robust; dense and tough	4–8	Very moist soils; tolerates periods of inundation and dryness	Stabilises banks against erosion, tough, slow-growing, high surface area; dominates zones	Statewide Metro: All
Carex inversa	Knob Sedge	EM	M	0.1–0.3	Small tufted or spreading clump	6–8	Moist well-drained soils	Rapid establishment	Statewide Metro: All
Cyperus gunnii	Flecked Flat Sedge	EM	M	0.6–1	Densely tufted perennial herb	6–8	Moist to boggy soils	High surface area	Statewide Metro: All
Juncus amabilis	–	EM	M	0.2–1.2	Rhizomatous tufted perennial rush	8–10	Tolerates inundation and dry periods once established	Widespread	Statewide Metro: All
Juncus flavidus	Yellow Rush	EM	M	0.4–1.2	Rhizomatous tufted perennial rush; yellow-green	8–10	More tolerant of dry soils than other *Juncus* spp.	Aesthetic	Statewide Metro: All
Juncus pallidus	Pale Rush	EM	M	0.5–2.3	Rhizomatous tufted perennial rush	8–10	Grows well with periodic inundation	Rapid growth; adaptable	Statewide Metro: All

Table A.2 Plant species for sediment basins, wetlands and ponds (indicative) (Continued)

Scientific name	Common name	Zone	Form	Height (m)	Description	Planting density (plants/m^2)	Requirements	Comments	Region
Lepidosperma longitudinale	Common Sword-sedge	EM	M	0.6–1.7	Rhizomatous	6	Moist or wet soils	Aesthetic	W, SC, GIP Metro: SS, TS
Melaleuca ericifolia	Swamp Paperbark	EM	T	2–9	Erect, open to bushy shrub or small tree	2–4	Moist to wet fertile soils; tolerates dry periods once established	Very adaptable	SC, GIP Metro: All
Baumea juncea	Bare Twig-rush	L	M	0.3–1	Rush-like clump with creeping rhizomes	8	Moist to boggy soils; tolerates occasional dry periods	Slow establishment	W, SC, GIP Metro: All
Brachyscome cardiocarpa	Swamp Daisy	L	G	0.1–0.3	Tufted perennial herb	2–4	Moist soils	Rapid establishment; aesthetic	W, SC, GIP Metro: SS, TS
Callistemon sieberi	River Bottlebrush	L	T	3–10	Open to dense weeping shrub	1	Very wet to moist, heavy clay soil but tolerates dry periods once established	Very adaptable; widespread; aesthetic	GDR, GIP Metro: BA, SS
Carex bichenoviana	Sedge	L	G	0.25–0.5 (stems)	Tufted grass-like sedge with long creeping rhizome	6–8	Moist depressions on heavy clay	May form dense carpets in shady situations; very rare in Melbourne	N, W, SC, GDR Metro: BA
Carex breviculmis	Short-stem sedge	L	M	0–0.15	Small but densely tufted sedge	6–8	Moist to wet soils; tolerates dry periods	Very adaptable	Statewide Metro: All
Centella cordifolia	Swamp Pennywart	L	G	Prostrate	Creeping perennial herb	2–4	Moist to wet soils	Rapid growth; may become invasive	Statewide Metro: All
Chrysocephalum apiculatum	Common Everlasting	L	G	Prostrate – 0.3	Variable, dense spreading perennial herb	2–6	Well-drained soils	Widespread	Statewide Metro: All
Dianella longifolia var. longifolia	Pale Flax-lily	L	M	0.3–0.8	Tufted perennial clump with short rhizomes	6–8	Moist, well-drained soils	Aesthetic; easy maintenance; ideal under trees	Statewide Metro: All
Dianella tasmanica	Tasman Flax-lily	L	M	0.6–1.5	Robust tufted perennial; may spread vigorously with strong rhizomes	6	Moist soils, prefers shaded position	Tolerant once established; adaptable (including snow cover); aesthetic	W, SC, GDR, GIP Metro: SS, TS
Eleocharis pusilla	Small Spike-rush	L	G	0.002–0.25	Tiny perennial herb with thread-like rhizomes and stems	6–10	Moist to wet soils	Readily grown; easily controlled	Statewide Metro: BA

Table A.2 Plant species for sediment basins, wetlands and ponds (indicative) (Continued)

Scientific name	Common name	Zone	Form	Height (m)	Description	Planting density (plants/m^2)	Requirements	Comments	Region
Gahnia filum	Chaffy Saw-sedge	L	M	1–1.2	Perennial leafy tussock	4–6	Moist sandy soils; salt tolerant	Aesthetic fruits	W, SC, GIP Metro: BA, TS (coastal)
Gahnia siberiana	Red-fruited Sword Sedge	L	M	1.5–3	Clumping perennial sedge	4–6	Moist soils; tolerates dry periods once established	Aesthetic; easily grown from seed	W, SC, GDR, GIP Metro: SS, TS
Goodenia humilis	Swamp Goodenia	L	M	0.05-.1	Suckering, matting plant	2–4	Moist to wet soil	Aesthetic; very adaptable	N, W, SC, GIP Metro: SS, TS
Juncus australis	Austral Rush	L	M	0.6–1.2	Rhizomatous tufted perennial rush	6–10	Moist soils; will tolerate short, dry periods	Common in south-eastern Victoria	SC, GDR, GIP Metro: BA, SS
Juncus pauciflorus	Loose-flower Rush	L	M	0.3–1	Rhizomatous perennial rush	6–10	Moist soils; tolerates dryness once established	Adaptable	W, SC, GDR, GIP Metro: All
Linum marginale	Native Flax	L	G	0.3–0.8	Slender erect perennial	4–6	Moist, well-drained soils	Widespread	Statewide Metro: All
Lomandra filiformis spp. filiformis	Wattle Mat-rush	L	M	0.15–0.5	Small tussock with fine blades	6–8	Moist, well-drained clay or sandy soils; tolerates dry, shaded positions once established	Little maintenance; grows well under trees	N, SC, GIP Metro: All
Lomandra longifolia var. longifolia	Spiny-headed Mat-rush	L	M	0.5–1	Large tussock	4–6	Well-drained soils; tolerates dry, shaded positions	Grows well under established trees	W, SC, GDR, GIP Metro: All
Melaleuca ericifolia	Swamp Paperbark	L	T	2–9	Erect, open to bushy shrub or small tree	<1	Moist or wet soils; tolerates dry periods once established	Very adaptable	SC, GIP Metro: All
Persicaria decipiens	Slender Knotweed	L	M	Prostrate – 0.6	Glabrous, erect to spreading annual herb	2–4	Semi-aquatic to aquatic	Low surface area; aesthetic	Statewide Metro: All
Poa tenera	Slender Tussock Grass	L	G	0.05–0.2	Trailing, sometimes forms open tussocks	6–8	Moist, well-drained soils	Very effective when trailing down embankments	W, SC, GDR, GIP Metro: SS, TS
Phylidrium lanuginosum	Woolly Water Lily	L	M	0.5–1	Erect aquatic perennial herb; low foliage (0.3) and then big spike flower (< 1 m)	6	Semi-aquatic to aquatic	Aesthetic; rare in Victoria	N, W, GIP Metro: TS

Table A.2 Plant species for sediment basins, wetlands and ponds (indicative) (Continued)

Scientific name	Common name	Zone	Form	Height (m)	Description	Planting density (plants/m^2)	Requirements	Comments	Region
Ranunculus inundatus	River Buttercup	L	G	0.05–0.3	Slender, stoloniferous perennial herb; often forms large mats	2–4	Semi-aquatic to aquatic	Rapid establishment	Statewide Metro: SS
Schoenus apogon	Common Bog-rush	L	G	0.05–0.3	Slender perennial tufted herb	8–10	Moist or wet soils	Variable; widespread	Statewide Metro: All
Stypandra glauca	Nodding Blue Lily	L	E	0.5–1.5	Slender tufted; bright blue drooping flowers	4–8	Moist to dry, well drained soil	Aesthetic; other *Stypandra* spp. probably just as suitable	Statewide Metro: All
Lythrum salicaria	Purple Loosestrife	L	E	1–2	Erect, hairy perennial	2–4	Moist soils or shallow water	Dies back after summer	W, SC, GDR, GIP Metro: All
Villarsia reniformis	Running Marsh Flower	L	M	0.4	Tufted; stoloniferous if growing in water	6–8	Moist to wet soils	Aesthetic	Statewide Metro: SS, TS
Marsilia drummondii	Common Nardoo	L	M	0.02–0.3	Rhizomatous aquatic with floating fronds	2–4	Wet soils subject to inundation; shallow water to depth of 0.3 m	Aesthetic; rapidly growth over large areas in ideal conditions without becoming invasive	N, W Metro: BA, SS
Viola hederacea	Native Violet	L	G	Prostrate – 0.15	Stoloniferous herb forming a dense mat	2–4	Moist to wet soil	Rapid growth; aesthetic; prolific growth once established	W, SC, GDR, GIP Metro: All
Baumea rubiginosa	Soft Twig-rush	M	M	0.3–1	Rhizomatous perennial	6–8	Moist soils to prolonged inundation	Slow establishment	SC, GDR, GIP Metro: SS, TS
Baumea tetragonia	Square Twig-rush	M	M	0.3–1	Rhizomatous perennial	6–8	Moist soils to prolonged inundation; 0.2–0.4 m depth	Slow establishment	W, SC, GIP Metro: SS, TS
Bolboschoenus medianus	Marsh Club-rush	M	M	0.7–2	Aquatic to semi-aquatic rhizomatous perennial	4–6	Moist soils to permanent water	Rapid establishment; spreading	N, W, SC, GIP Metro: All
Schoenoplectus pungens	Sharp Club-rush	M	M	0.3–0.6	Robust, tufted rhizomatous herb	4–6	Wet soils to permanent water	Become rare due to urbanisation; rapid establishment	N, W, SC, GIP Metro: BA
Triglochin procera	Water-ribbon	M	M	0.2–0.5	Aquatic or amphibious perennial herb with erect or floating leaves	4	Semi-aquatic to aquatic to depth of 1.5 m	Aesthetic; spreading	Statewide Metro: All

Table A.2 Plant species for sediment basins, wetlands and ponds (indicative) (Continued)

Scientific name	Common name	Zone	Form	Height (m)	Description	Planting density (plants/m^2)	Requirements	Comments	Region
Baumea arthrophylla	–	M	M	0.3–1.3 (stems)	Aquatic perennial with long rhizomes	6–8	Wet soils to permanent water	Spreads quickly	N, W, SC, GIP Metro: All
Bolboschoenus caldwellii	Sea Club-rush	M	M	0.3–0.9	Aquatic to semi-aquatic rhizomatous perennial	4–6	Fresh to brackish water on heavy clay to sandy soils	Coastal/saline; rapid establishment	N, W, SC, GIP Metro: All
Myriophyllum caput-medusae	Coarse Water-milfoil	P	S	0.3–2 (stems)	Perennial herb, sparsely branched below the waterline, profusely above; procumbent	1	Aquatic; depth to 2 m	Heterophyllic	Statewide Metro: All
Myriophyllum verrucosum	Red Water-milfoil	P	S	0.1–1.5 (stems)	Sparsely branched perennial rooting at lower nodes; procumbent	1	Deep fast-flowing water to shallow brackish or calcareous water	Heterophyllic	N, W, SC, GIP Metro: All
Nymphoides geminata	Entire Marshwort	P	S	Stems to 1	Robust native perennial with long petiole leaves and +/– floating stolons up to 2 m long	1	Aquatic; deep permanent water	Aesthetic	GDR, GIP Metro: (Does not occur)
Potamogeton crispus	Curly Pondweed	P	S	To 4.5	Perennial, rhizomatous aquatic herbs	1	Aquatic; deep permanent water	Growth can be dense	N, SC, GDR, GIP Metro: All
Potamogeton ochreatus	Blunt Pondweed	P	S	To 4.5	Annual or perennial, rhizomatous aquatic herbs; submerged floating annuals	1	Aquatic; deep permanent water	Rapid growth; aesthetic; seasonal; salt tolerant (up to 2000 ppm)	Statewide Metro: All
Potamogeton pectinatus	Fennel Pondweed	P	S	Stems to 3	Perennial, rhizomatous aquatic herbs; submerged	1	Aquatic; deep permanent water	Saline (thrive in > 5000 ppm dissolved salt); rarely recommended; not aesthetic; often invasive	N, W, SC Metro: BA
Potamogeton tricarinatus	Floating Pondweed	P	S	stems to 2.7	Perennial rhizomatous aquatic herb; submerged or attached floating	1	Aquatic; shallow semi-permanent water	Seasonal	Statewide Metro: All

Table A.2 Plant species for sediment basins, wetlands and ponds (indicative) (Continued)

Scientific name	Common name	Zone	Form	Height (m)	Description	Planting density (plants/m^2)	Requirements	Comments	Region
Vallisneria gigantea	Ribbonweed	P	S	to 3	Submerged, dioecious tufted stoloniferous perennial with floating flowers	1	Open water; depth of < 0.1–4m	Rapid growth; salt tolerant (1500 ppm)	N, W, SC, GIP Metro: SS
Cyperus lucidus	Leafy Flat-sedge	SM	M	0.6–1.5	Robust, tufted perennial herb with sharply triangular stems; large; dense	6	Wet soils	Can grow as an aquatic plant; slow spreading	W, SC, GDR, GIP Metro: SS
Eleocharis acuta	Common Spike-rush	SM	M	0.3–0.9	Perennial aquatic herb; slender rhizomes	6–8	Heavy damp soils to 0.20 m depth	High surface area; may spread rapidly in shallow water	Statewide Metro: All
Ficnia nodosa	Knobby Club-rush	SM	M	0.5–1.5	Tall; wiry; rhizomatous; densely tufted perennial rush	6–8	Moist soils; tolerates dry periods once established	Widespread in Melbourne; binds soils in moist areas; aesthetic	N, W, SC, GIP Metro: All
Juncus subsecundus	Finger Rush	SM	M	0.5–1	Rhizomatous tufted perennial rush	6–10	Heavy, wet soils	Widespread	Statewide Metro: All
Juncus usitatus		SM	M	0.3–1.2	Rhizomatous tufted perennial rush	6–10	Tolerates up to 0.2 m inundation	Rapid growth	N, GIP Metro: All
Isolepis inundata	Swamp Club-rush	SM	M	0.05–0.3	Tufted perennial rush; small; stoloniferous	6–8	Moist to wet soils; tolerates periodic inundation	Widespread; high surface area; rapid growth	N, W, SC, GDR, GIP Metro: All
Villarsia exaltata	Yellow Marsh Flower	SM	M	0.3, stems to 1.5	Tufted herb; broad basal leaves	6–8	Wet soils to 1 m depth	Leaves float if growing in water	SC, GIP Metro: SS, TS
Juncus krausii	Sea Rush	SM	M	0.6–2.3	Rhizomatous perennial rush	6–10	Brackish to saline conditions	Slow growth; saline; habitat only	W, SC, GIP Metro: All
Carex fasicularis	Tassel Sedge	SM	M	0.5–1	Coarse, tufted plant	6–8	Moist soil; tolerates periods of inundation	Aesthetic	Statewide Metro: SS
Carex gaudichadiana	Tufted sedge	SM	M	0.1–0.6	Coarse, tufted plant	6–8	Gravel or mud at water's edge	Aesthetic; tolerates drawdown	Statewide Metro: SS
Alisma plantago-aquatica	Water Plantain	SM	M	0.5–1	Erect, perennial semi-aquatic herb	6–8	Moist to wet soils; tolerates poorly drained sites	Aesthetic; little surface area; often dies off in winter	Statewide Metro: All

Glossary

Adjustment factor	The required conversion of the size of a treatment device in a given location to achieve the same pollutant reduction as an equivalent treatment device in Melbourne.
Afflux	The rise in water level immediately upstream of, and due to, an obstruction.
Antecent conditions	Pre-existing conditions (e.g. soil wetness).
Aquifer Storage and Recovery (ASR)	The process of recharging water into an aquifer for the purpose of storage and subsequent withdrawal. Injection of recycled water into aquifers for storage, which may be recovered later to meet water demands.
Bathymetry	Topography or the shape of the land below a water surface.
Batter slopes	An edge that slopes backward from perpendicular.
Biofilm	A gelatinous sheath of algae and micro-organisms, including benthic algae and bacteria, formed on gravel and sediment surfaces and surfaces of large plants.
Biological uptake	Take-up of gas or fluid through a cell membrane.
Bioretention basin	A grassed or landscaped basin promoting infiltration into the underlying medium. A perforated pipe collects the infiltrated water and conveys it downstream.
Bioretention swale	A grassed or landscaped swale promoting infiltration into the underlying medium. A perforated pipe collects the infiltrated water and conveys it downstream. Flows are also conveyed along the surface of the swale prior to being infiltrated.
Bollard	Structure designed to prevent vehicular access.
Buffer	A vegetated strip between the edge of a stream or drainage channel and a land use activity, designed to trap the lateral overland flow-borne pollutants.
Catchment	A topographically defined area, drained by a stream such that all outflow is directed to a single point.
Check banks/dams	Flow spreaders constructed across a channel to decrease velocities and promote uniform flows.
Colloidal particles	Particles that remain suspended in a solution (i.e. fail to settle out)
Constructed wetland	An artificially created system containing pond, marsh and swamp features.
Design flow	Calculated flow used to size engineering structures to a defined standard.
Detention time	The time it takes for a 'parcel' of water to flow from the inlet of a wetland system to the outlet. Detention time is never a constant (see also *Notional detention time*).
Discharge	The volume of flow passing a predetermined section in a unit time.
Enhanced sedimentation	Additional sedimentation due to the presence of vegetation and biofilms.

Ephemeral	Temporary or intermittent (e.g.a creek or wetland which dries up periodically).
Extended detention	Volume above wetland normal water level and the overflow weir height in a treatment element (e.g. wetland, bioretention basin, infiltration basin).
Filtration media	Soil media that retain pollutants as stormwater passes through it.
First flush diverter	Device for directing initial roof water collected after a rainfall event away from storage as it is thought to contain a high concentration of pollutants.
Flood retarding basin	A temporary flood storage system used to reduce flood peaks. A basin designed to temporarily detain storm or flood waters, to attenuate peak flows downstream to acceptable levels.
Greenfield site	Broadacre subdivision on land previously used for agriculture or native vegetation.
Gross Pollutant Trap (GPT)	A structure used to trap large pieces of debris (> 5 mm) transported through the stormwater system.
Hydrologic effectiveness	Describes the interaction between runoff capture, detention time and detention volume within a wetland system. Or proportion of runoff from catchment that is treated in treatment element.
Hydrologic design region	A spatial region that has a common rainfall pattern.
Infiltration measure	Trenches filled with permeable material (gravel) and placed to intercept stormwater and direct it to permeable soil or groundwater zones.
Inlet zone	See *Sediment basin*.
Littoral zone	Areas around the shallow margin of wetland characterised by specific vegetation that are alternatively wetted and dried as water level fluctuates.
Macrophyte zone	Vegetated section of wetland.
Macrophyte	A large plant including macroscopic algae, mosses, ferns and flowering plants.
Manning's equation	Commonly used for indirect estimation of discharge in a channel or estimation of channel capacity: $Q = 1/n \times A \times R^{2/3} \times S^{1/2}$ Where: Q = discharge (m^3/s) n = Manning's 'n' A = cross-sectional area of flow (m^2) R = hydraulic radius (m) S = slope.
Manning's *n*	A measure of channel roughness.
MUSIC	The acronym used for the Model for Urban Stormwater Improvement Conceptualisation software developed by the Cooperative Research Centre for Catchment Hydrology to model urban stormwater management schemes.
Notional detention time	The average time it takes for the wetland to return to its normal water level after rainfall event (i.e. the time it takes for the extended detention to drain). Notional detention time is used to provide a point of reference in modelling and determining the design criteria for riser outlet structures.
Permanent pool	The level of water retained within a basin below the invert of the lowest outlet structure.
Pluviograph	An instrument that records rainfall collected as a function of time.

Pond	An artificial open water body.
Porous pavement	Pavements comprising materials which facilitate infiltration of rainwater and transfer to the underlying subsoil.
Rainwater tanks	Tanks used to collect and store rainfall from household roofs for beneficial use.
Rational Method	or Probabilistic Rational Method. Widely used simple method for estimating peak design flow rates: $$Q = C \times I \times A/360$$ Where, Q = design flow rate. C = dimensionless runoff coefficient. I = rainfall intensity (mm/hr). A = catchment area (km^2).
Referral authority	An authority nominated in Section 55 of the *Planning and Environment Act 1987* that has statutory powers to provide conditions or object to a planning permit application.
Reliability (with respect to reuse from tanks)	The percentage of demand met by water from the rainwater tank. The remainder of demand is met from mains water or an alternative water source.
Riser outlet	Hydraulic designed outlet control from a wetland, designed to provide the desired notional detention time.
Rock beaching	Protecting areas of high scour potential by lining them with hard material (rocks).
Sediment basin	Area where velocities are slowed and coarse sediments settle out of stormwater. Typically pools are about 2 m deep.
Sedimentation	Process of particles settling out of a water column.
Stochastic	The random variability in the occurrence and magnitude of a parameter.
Stormwater	All surface water runoff from rainfall, predominantly in urban catchments. Such areas may include rural residential zones.
Swale	A vegetated open channel, designed to intercept and convey surface run-off to a drainage network inlet.
Transition layer	Layer between filtration media and drainage layer in a bioretention system. The purpose of this layer is to prevent filtration media clogging up the drainage layer.
Treatment train	A series of treatment processes designed to collectively meet a prescribed water quality objective (e.g. a gross pollutant trap used in conjunction with a wetland system).
Water Sensitive Urban Design (WSUD)	A philosophical approach to urban planning and design that aims to minimise the hydrological effect of urban development on the surrounding environment.
Weir	A small dam in a stream or basin designed to raise water levels or to divert its flow through a desired channel.
Wetland	An area transitional between land and water systems, which is either permanently or periodically inundated with shallow water.

Appendix B Victorian hydrological regions for sizing stormwater treatment measures

Contents

B.1 Introduction

Achievable treatment objectives for stormwater quality have been defined in Victoria and New South Wales. These objectives are expressed in reductions to be gained in the mean annual pollutant loads discharged from typical urban areas with no stormwater treatments installed (e.g. 80% reduction in Total Suspended Solids, TSS, and 45% reduction in Total Phosphorus, TP, and Total Nitrogen, TN). A range of stormwater treatment measures are capable of treating urban stormwater to meet the treatment objectives stated. The design of stormwater treatment measures often requires a continuous simulation approach to properly consider the influence of antecedent conditions of the treatment measure during a storm event and the wide range of storm characteristics and hydraulic conditions that the individual treatment measures are to operate in. Computer models such as the Model for Urban Stormwater Improvement Conceptualisation (MUSIC) (CRCCH 2002) developed to enable continuous simulations of complex stormwater management treatment trains aid in the development of stormwater management strategies and the design (sizing) of stormwater treatment measures.

This Manual builds upon earlier work (described in *Hydrologic Regions for Sizing of Stormwater Treatment in Victoria*, October 2003) and its purpose is to develop an alternative, simpler design procedure that can be used in small development projects (e.g. single or a small clustered allotment development type) and could serve as a preliminary design procedure. In addition the procedure could be used as a simple design checking tool. An example of a similar tool is the one developed in the Association of Bayside Municipality (ABM) project (where a design chart containing the expected performance of several typical stormwater treatment measures were developed for the Melbourne metropolitan region (Figure B.1).

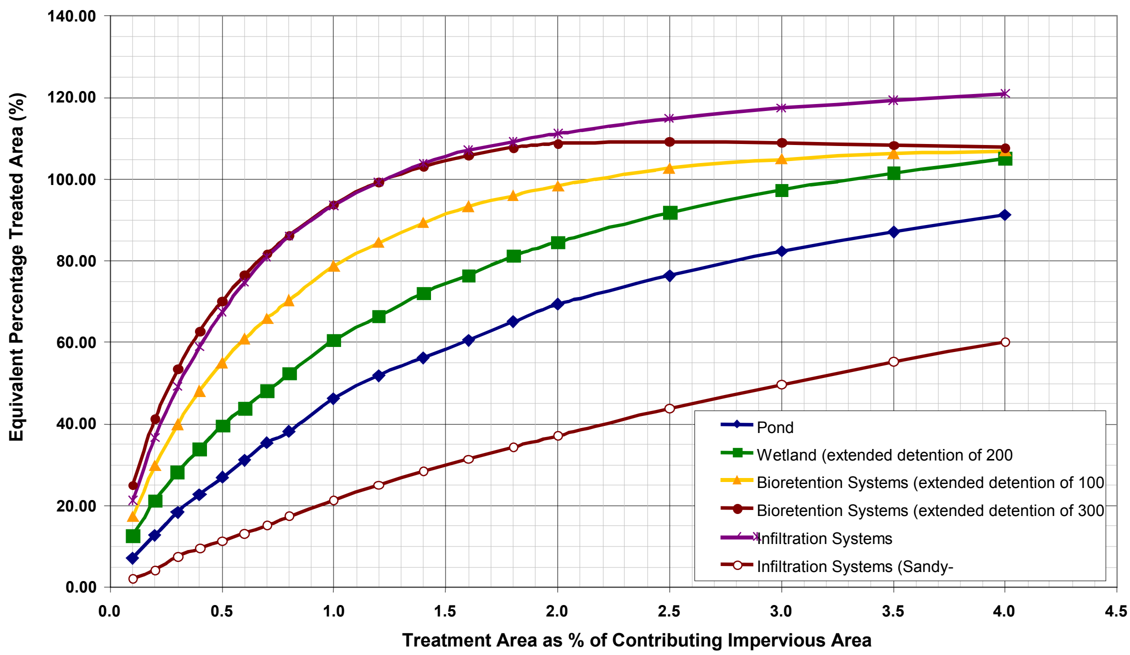

Figure B.1.1 Equivalent Percentage Treated Area (EPTA) chart developed for the Association of Bayside Municipalities. (EPTA is the percentage of impervious area treated to meet Victorian best practice environmental management objectives for urban stormwater.)

It was envisaged that a simple procedure such as the one developed for the ABM project could be developed for all regions in Victoria. Sizing stormwater treatment systems could be based around defining simple empirical design equations that would be applicable in their respective designated hydrologic region within Victoria. This Manual presents results of developing the empirical relationships and the number of regions.

B.2 Methodology

After initial consideration of possible design approaches, the following was used to develop the different regions and adjustment factors for sizing stormwater treatment measures throughout Victoria:

1. A rating was selected to represent the effectiveness of different design configurations of various stormwater treatment measures. Reduction in total nitrogen was a logical choice as it is commonly the limiting parameter in meeting best practice stormwater quality objectives.

2. A reference site was selected for which detailed investigation and design simulations were undertaken to determine the relationship between design configurations (e.g. area, extended detention depth, permanent pool and volume) of a range of stormwater treatment measures and the corresponding improvement in stormwater quality performance. Melbourne was a logical choice and was selected as a reference site.

3. Hydrologic regions were defined within Victoria where practitioners wanting to design stormwater treatments could measure any location in Victoria, refer to the design requirements developed for the reference site (i.e. Melbourne) and apply an adjustment factor to that size to determine the appropriate dimensions of the treatment measure for their particular site.
   For example, in order to meet best practice objectives, a wetland in the Melbourne region must be at least 2% of the contributing impervious area of its catchment. A practitioner designing a wetland of similar configuration in Mildura can simply use an empirical equation to calculate the adjustment factor that is then applied to the size of a wetland sized for the reference site (i.e. Melbourne).

4. It was envisaged that several geographical and meteorological factors could influence the value of the adjustment factor. These include Mean Annual Rainfall (MAR), a measure of seasonal distribution of rainfall and raindays, site elevation and geographical location. Thus, it was expected that Victoria would need to be divided into several hydrologic regions for which empirical equations for determining the adjustment factor need to be derived. In determining the hydrologic regions and the corresponding adjustment factor equations, information on site characteristics need to be readily available from the Bureau of Meteorology (BOM). As such, the set of possible influencing factors that were investigated were limited to those that can be obtained from the BOM website (www.bom.gov.au).

B.3 Determining hydrologic regions

The hydrologic regions for Water Sensitive Urban Design (WSUD) in Victoria were determined by selecting a set of pluviographic stations with a sufficiently long record to enable continuous simulations of the performance of several stormwater treatment measures. A total of 45 stations were selected for analysis, 15 of which are concentrated around the Melbourne/Geelong Metropolitan region. These stations and their BOM rainfall district are shown in the Table B.3.1. Figure 3.1and 3.2 show the respective spatial locations of the selected stations according to their longitude and latitude bearings. The additional stations around Melbourne were considered important because of the expected development activity. There is more available data for this region which enables a finer representation of the climatic factors.

Table B.3.1 Pluviographic stations and Bureau of Meteorology (BOM) districts

BOM district	Stations
Wimmera South	Horsham Tottington Wartook
North Mallee	Mildura
South Mallee	Hopetoun
Lower North	Cobram Kerang
Upper North	Bendigo Tatura Dookie
Lower Northeast	Dartmouth
Upper Northeast	Bright Hume Reservoir Omeo
East Gippsland	Buchan Sarsfield East Combienbar Genoa Wroxham
West Gippsland	East Sale East Tarwin Noojee Yallourn
West Central	Bullengarook
Western Plains	Ararat Ballarat
West Coast	Casterton Weeaproinah Wyelangta Mortlake
West Central	Laverton Melton Werribee
East Central	Melbourne Airport Bundoora Essendon Airport Melbourne Croydon Upwey Narre Warren North Dandenong Carrum Downs Koo Wee Rup

As evident in Figures B.3.1 and B.3.2, the selected pluviographic stations are reasonably well distributed across Victoria to ensure sufficient coverage of the state and the metropolitan region. The MAR for the sites selected ranged from 290 mm to 1900 mm, covering the wide range of rainfall conditions experienced across the state.

Total nitrogen was selected as the measure for representing the effectiveness of various sized treatment devices. In an attempt to define the most suitable hydrologic region and corresponding predictive equations, the influence of the following factors were considered:

- MAR
- the ratio of mean summer raindays to mean winter raindays (as a measure of rainday seasonality)
- the ratio of mean summer rainfall to mean winter rainfall (as a measure of rainfall seasonality)
- site elevation.

Figures B.3.3–B.3.6 are plots of the various meteorological factors and site elevations for the 45 stations.

The MUSIC program was used to simulate the performance of wetlands, bioretention systems, vegetated swales and ponds to size these systems to meet best practice objectives. These sizes were then normalised against the sizes derived for Melbourne and expressed as the ratio of the size of the treatment area for Melbourne. This is thus the adjustment factor described in Step 3 in the methodology (see Section B.2).

Following extensive testing and analysis of the significance of the possible influencing factors described in the above list, it was determined that MAR was the most significant influencing factor with which it was possible to represent Victoria with five hydrologic regions (excluding the Melbourne/Geelong Metropolitan region) (Figure B.3.3). Within the Melbourne/Geelong Metropolitan region a further four regions were used to provide a finer delineation of the influence of climatic conditions on the adjustment factor. Boundaries of the hydrologic region were determined to represent the results of the analysis and be aligned such that they do not dissect major urban areas in Victoria or are aligned with municipal boundaries, as much as possible, in the Melbourne/Geelong Metropolitan area. The exceptions to this are in the Cities of Wyndham and Casey where the hydrologic regions are bounded by Skeleton Creek and Monash Freeway, respectively.

In three of the four hydrologic regions shown in Figure B.3.4, the adjustment factor can be well represented for each treatment device by a single value (i.e. independent of rainfall) with the fourth region (Central and North West Metropolitan) represented as a function of MAR. Inclusion of other factors such as rainday seasonality, rainfall seasonality and elevation did not appear to improve the estimation of the adjustment factors for the 45 pluviographic stations used in the analysis.

The five hydrologic regions for Greater Victoria and the stations used in this analysis are shown in the Table B.3.2.

Table B.3.2—Hydrologic regions for Greater Victoria

Region	Stations
Northern	Mildura Hopetoun Kerang Cobram Hume Reservoir (Wodonda) Tottington Bendigo Tatura Dookie
Western	Horsham Wartook Reservoir Ararat Prison Ballarat
South Coast	Casterton Mortlake Weearproinah Wyelangta Noojee Yallourn East Tarwin
Great Dividing Range	Bullengarook East Darmouth Reservoir Bright Omeo Buchan Post Office
Gippsland	East Sale Sarsfield East Combienbar Wroxham Genoa

The four hydrologic regions for the Melbourne/Geelong Metropolitan area and the stations used in this analysis are shown in Table B.3.3.

Table B.3.3—Hydrologic regions for the Melbourne/Geelong Metropolitan area

Region	Stations
South West	Geelong North Little River Werribee
Central and North West	Melbourne Airport Laverton Melton Bundoora Essendon Airport Melbourne
East	Croydon Upwey Narre Warren North
South East	Dandenong Carrum Downs Koo Wee Rup

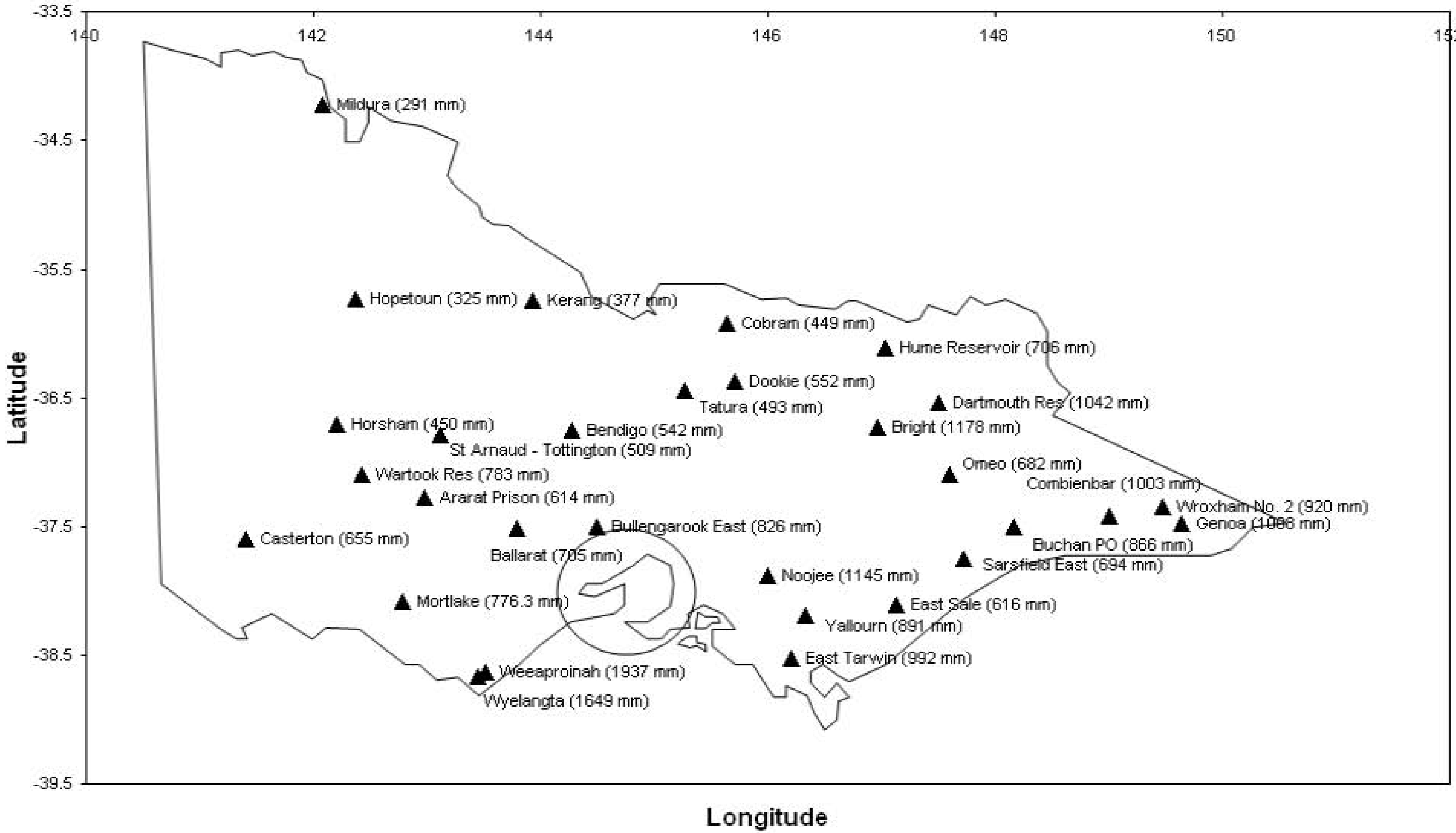

Figure B.3.1 Location of pluviographic stations in Greater Victoria used in defining hydrologic regions.

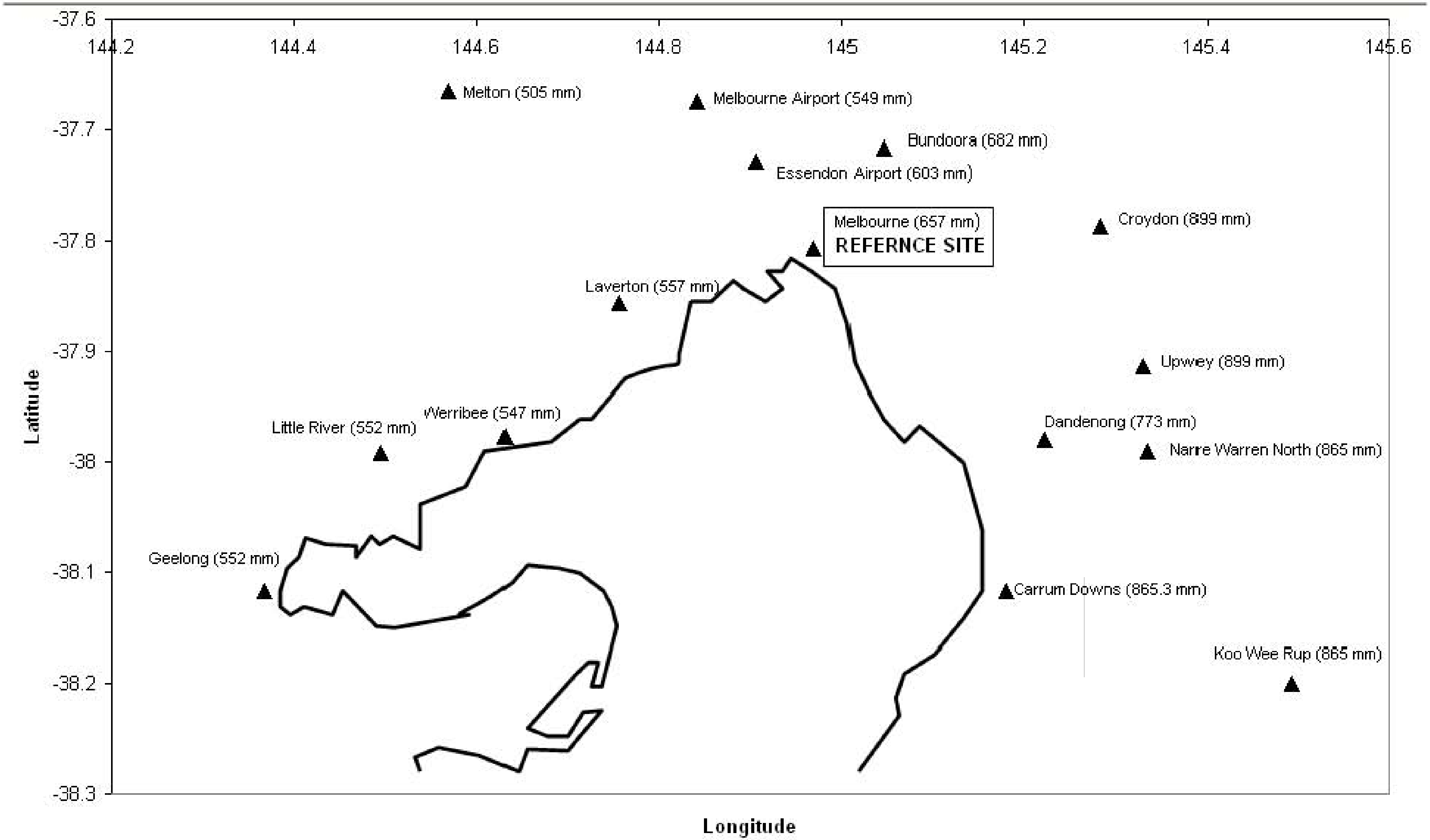

Figure B.3.2 Location of pluviograph stations in Melbourne/Geelong metropolitan region used to determine hydrologic regions.

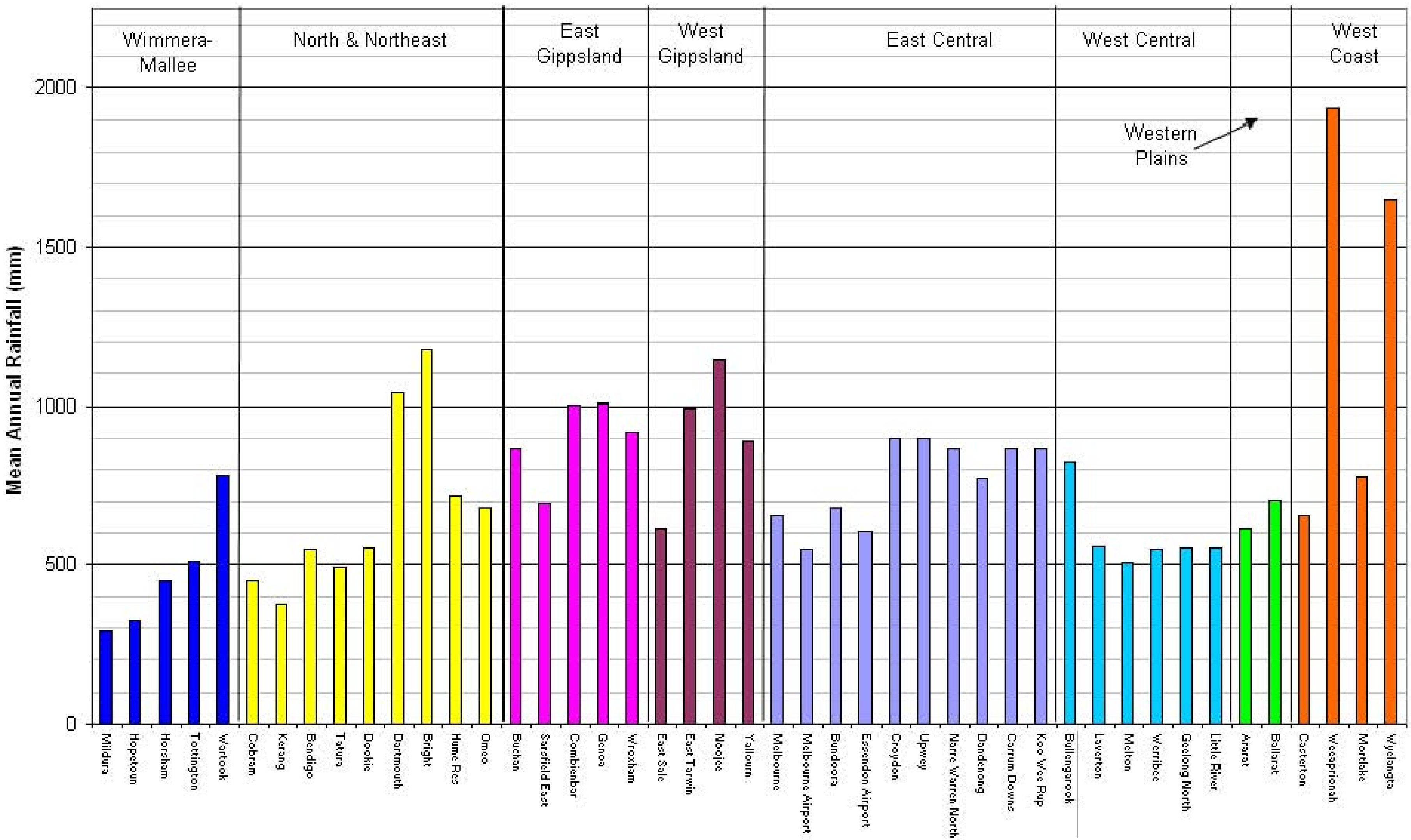

Figure B.3.3 Mean Annual Rainfall (MAR) at pluviograph station sites.

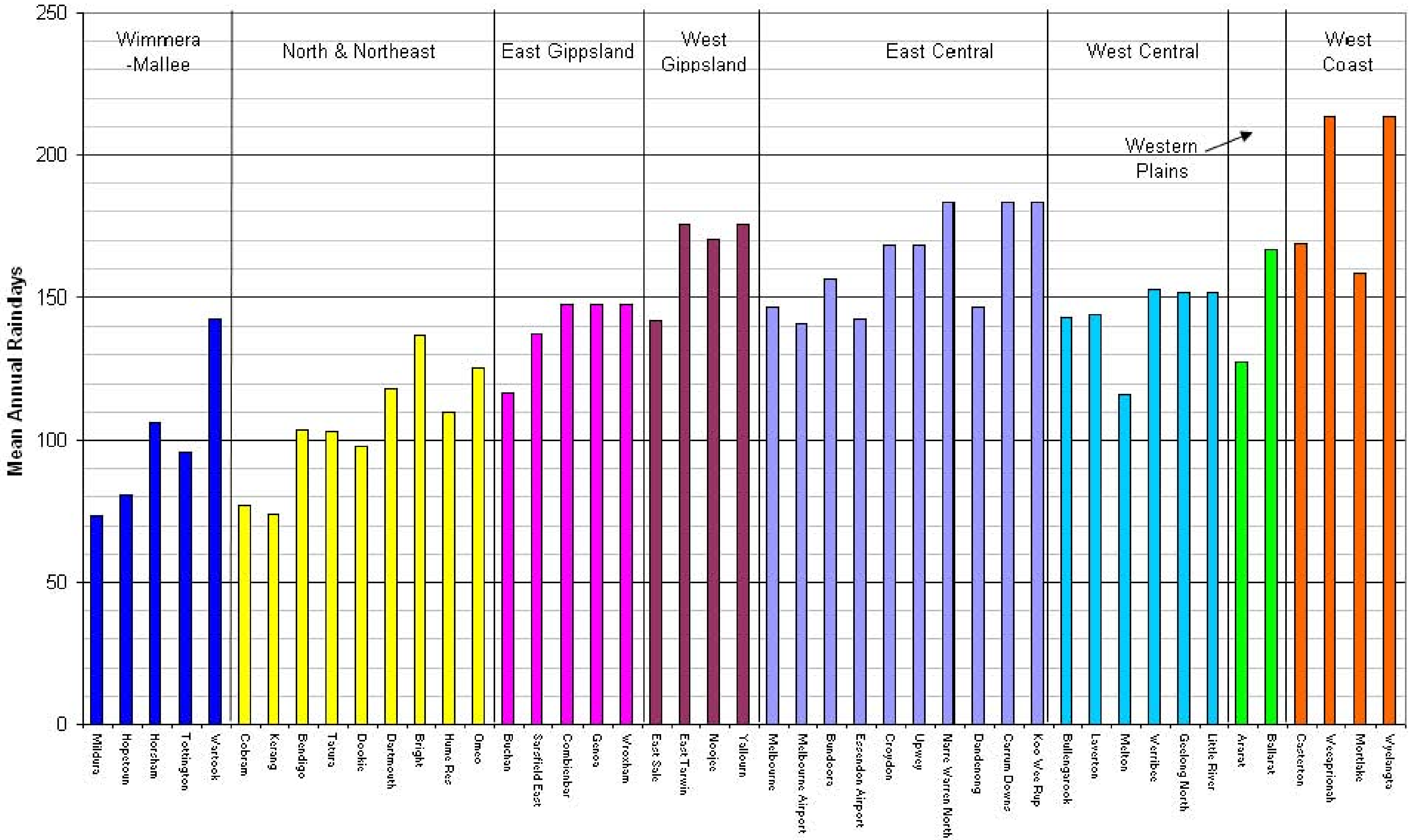

Figure B.3.4 Mean annual raindays at pluviographic station sites.

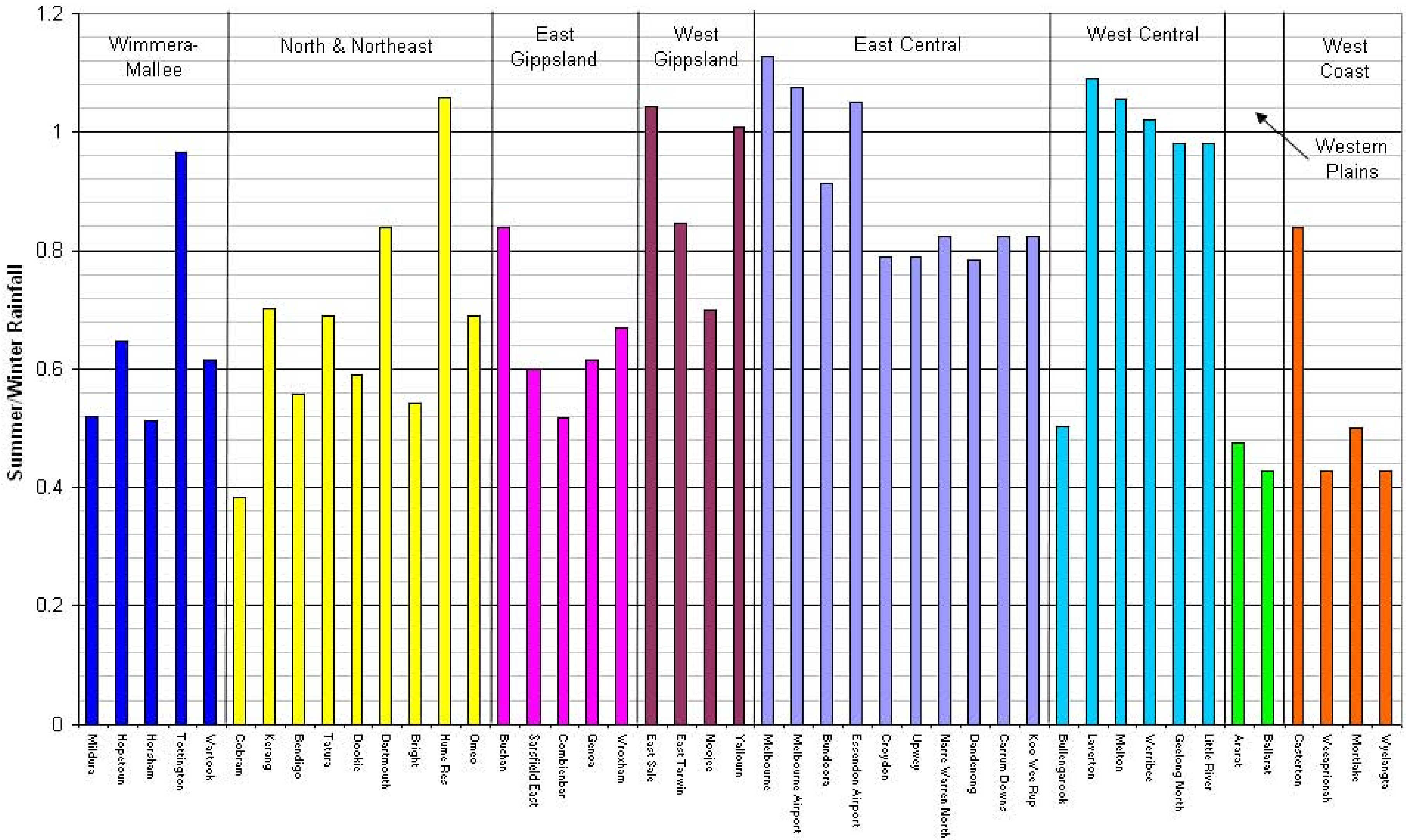

Figure B.3.5 Ratio of mean summer to mean winter rainfall at pluviographic station sites.

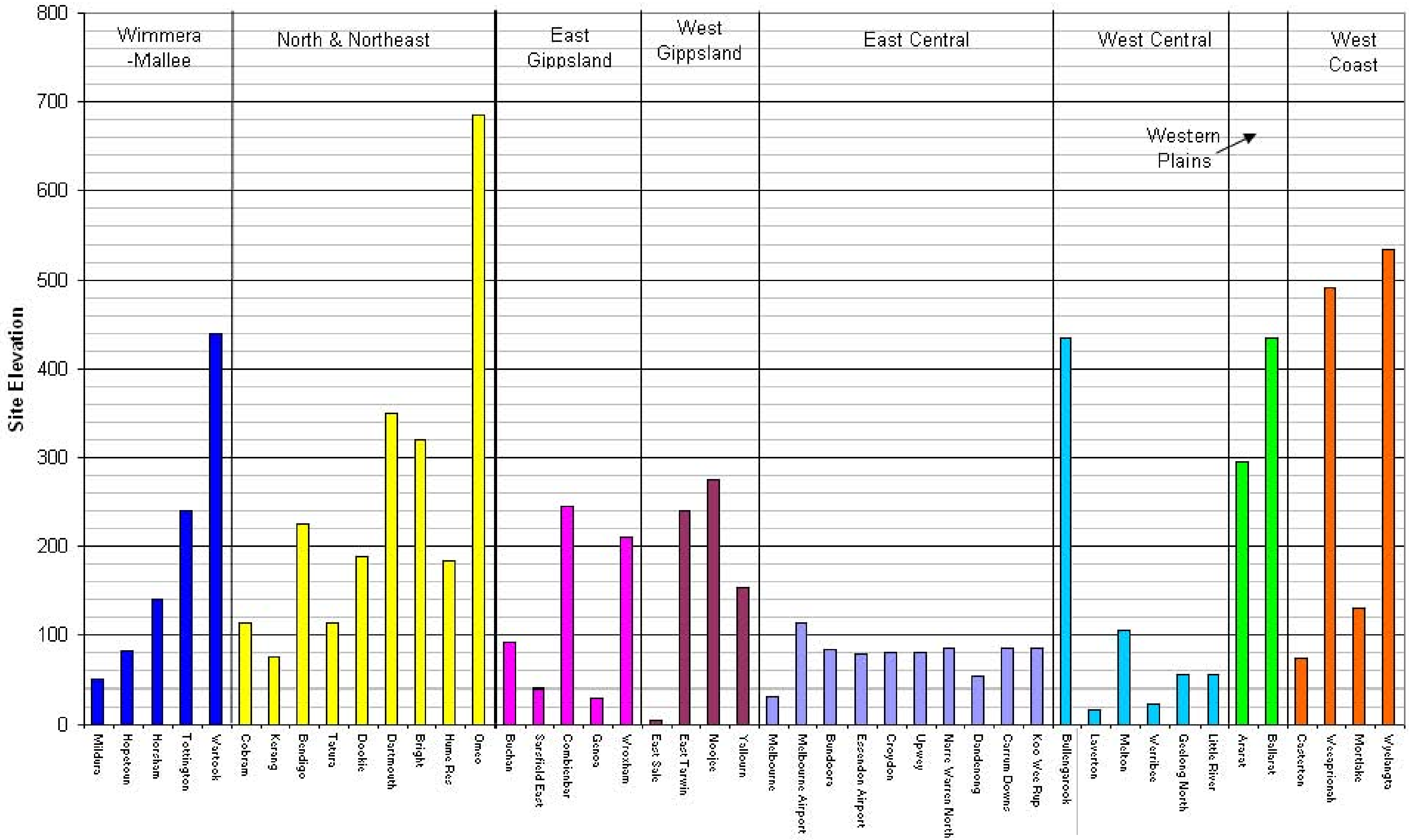

Figure B.3.6 Elevation at pluviographic station sites.

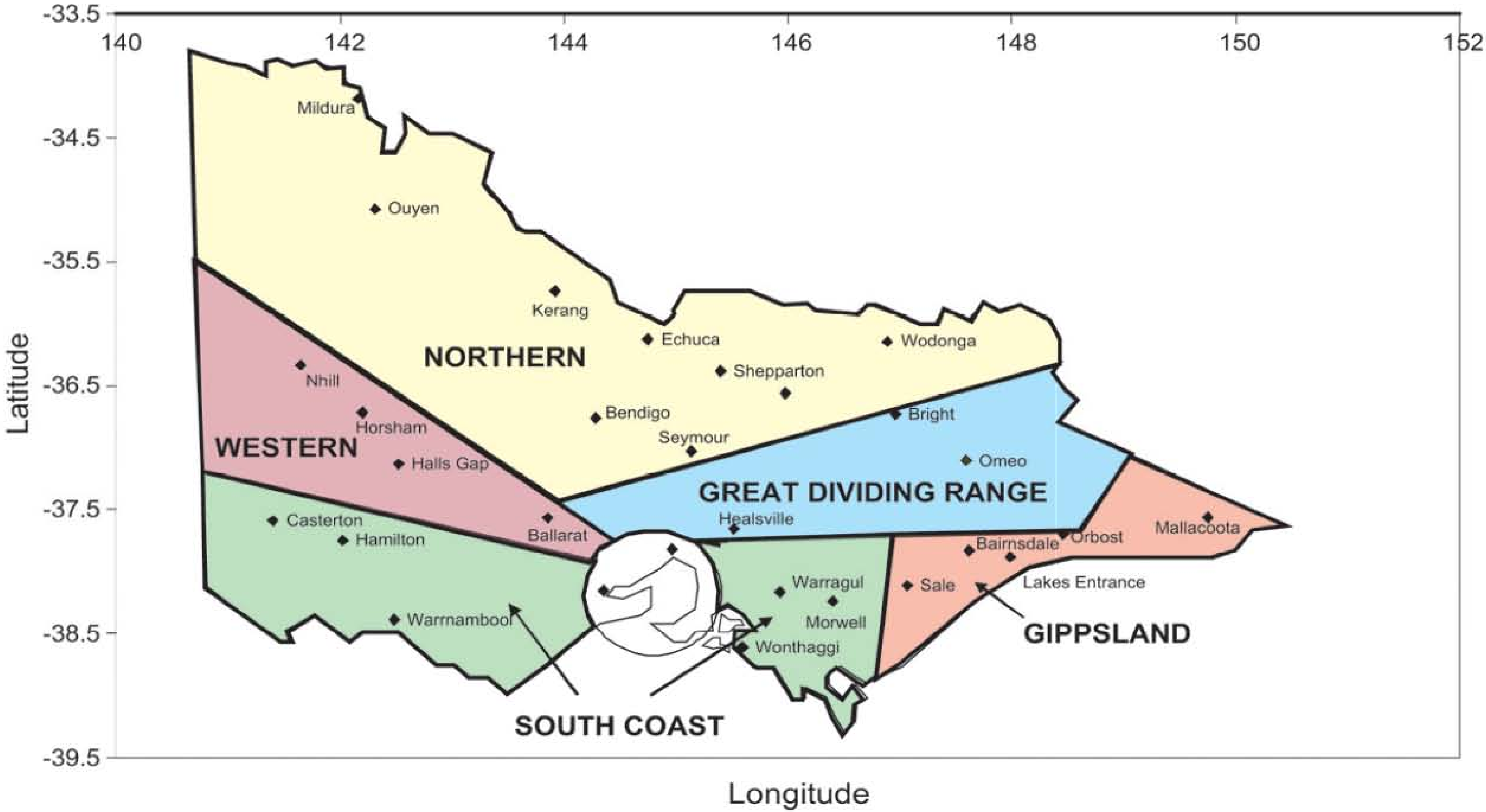

Figure B.3.7 Hydrologic regions for Greater Victoria (Melbourne and Geelong have been considered separately).

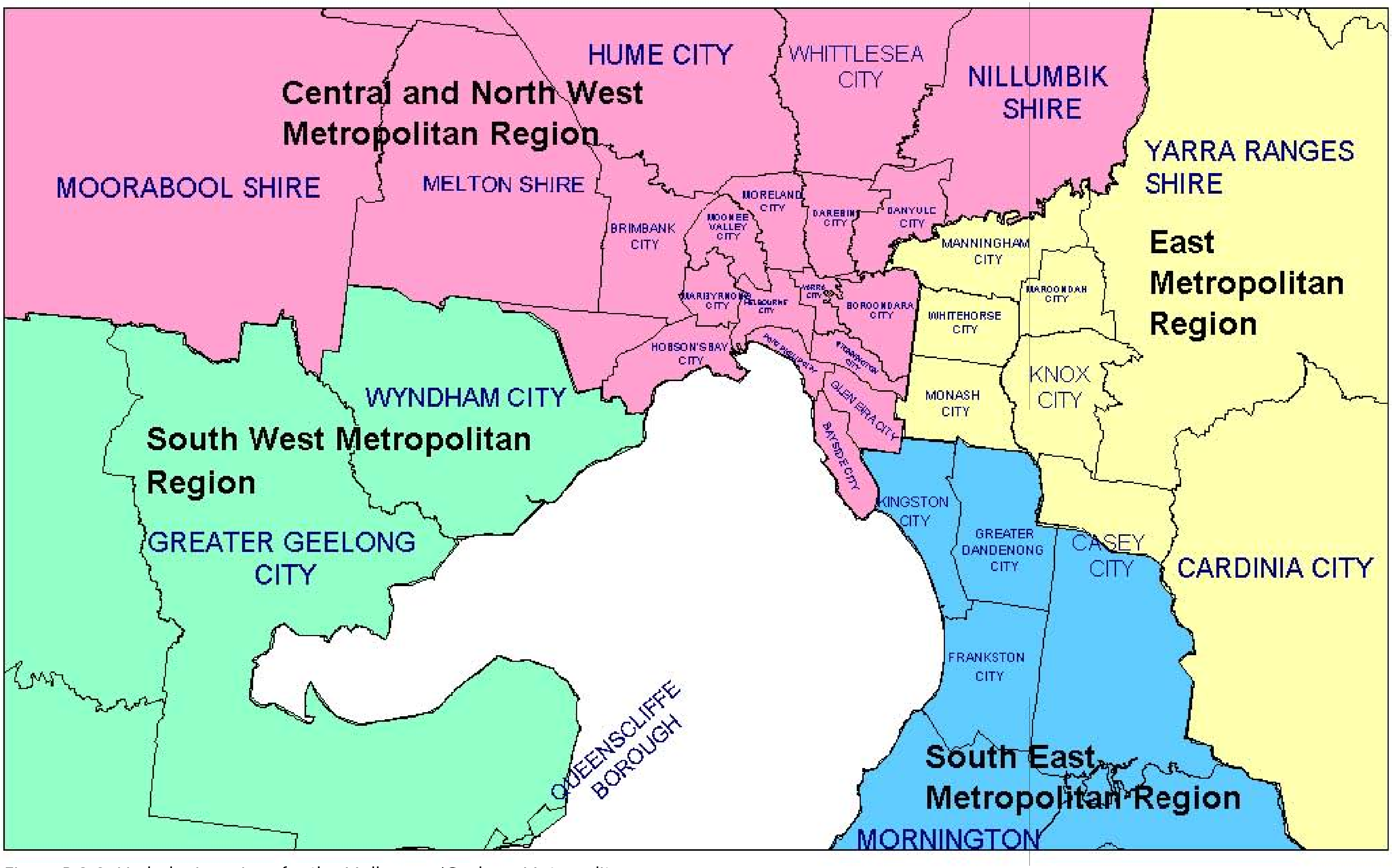

Figure B.3.8 Hydrologic regions for the Melbourne/Geelong Metropolitan area.

B.4 Hydrologic region adjustment factors

B.4.1 Adjustment factors for Greater Victoria

B.4.1.1 Wetlands

Figure B.4.1 shows a plot of the adjustment factors derived against MAR for the 30 stations in Greater Victoria grouped into five hydrologic regions. A trend of increasing adjustment factor with MAR is evident for each of the five regions.

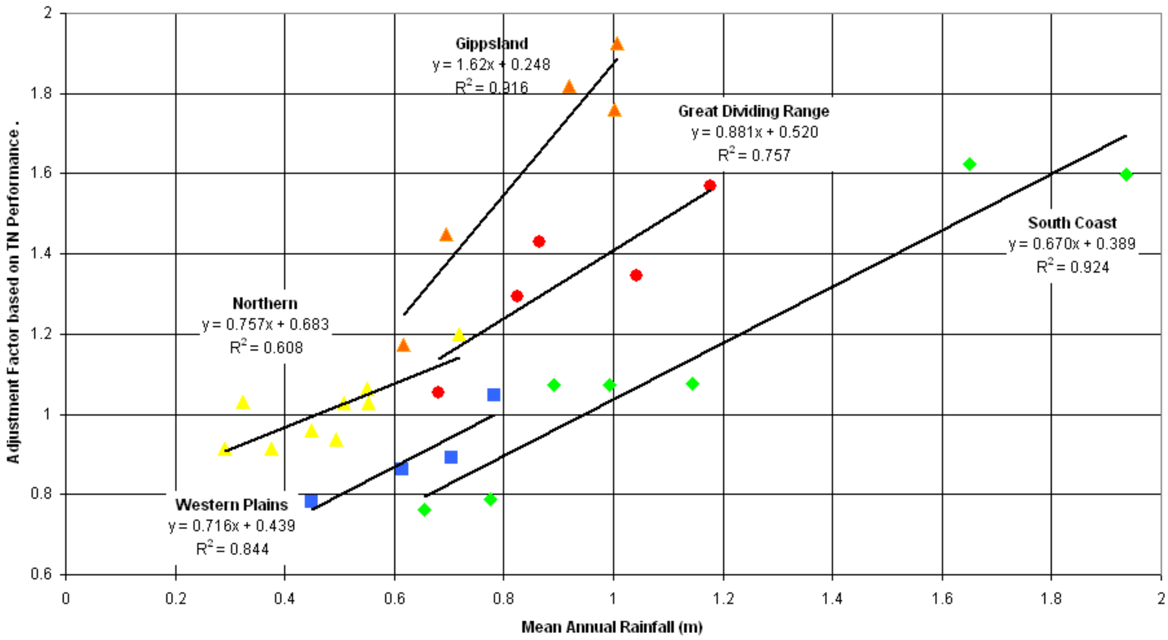

Figure B.4.1 Plot of adjustment factors versus Mean Annual Rainfall (MAR) for wetlands in Greater Victoria.

Equations to compute adjustment factors for each region were obtained by plotting a linear trend (i.e. 'line of best fit') for the points on Figure B.4.1 for each hydrologic region. The wetland adjustment factor equations are shown in Table B.4.1.

Table B.4.1 Wetland adjustment factor equations

Region	Wetland size adjustment factor equation
Northern	Adjustment factor = 0.757(MAR) + 0.683 [R2 = 0.61]
Western Plains	Adjustment factor = 0.716(MAR) + 0.439 [R2 = 0.84]
South Coast	Adjustment factor = 0.670(MAR) + 0.389 [R2 = 0.92]
Great Dividing Range	Adjustment factor = 0.881(MAR) + 0.520 [R2 = 0.76]
Gippsland	Adjustment factor = 1.62(MAR) + 0.248 [R2 = 0.76]

Figure B.4.2 shows a plot of the observed adjustment factor for each station (i.e. determined from the MUSIC modelling) and the predicted adjustment factor (i.e. obtained from the empirical equation determined for each hydrologic region). The dotted lines mark a 10% difference between the predicted and observed adjustment factor. All the predicted adjustment factors are within 10% of the corresponding observed adjustment factors.

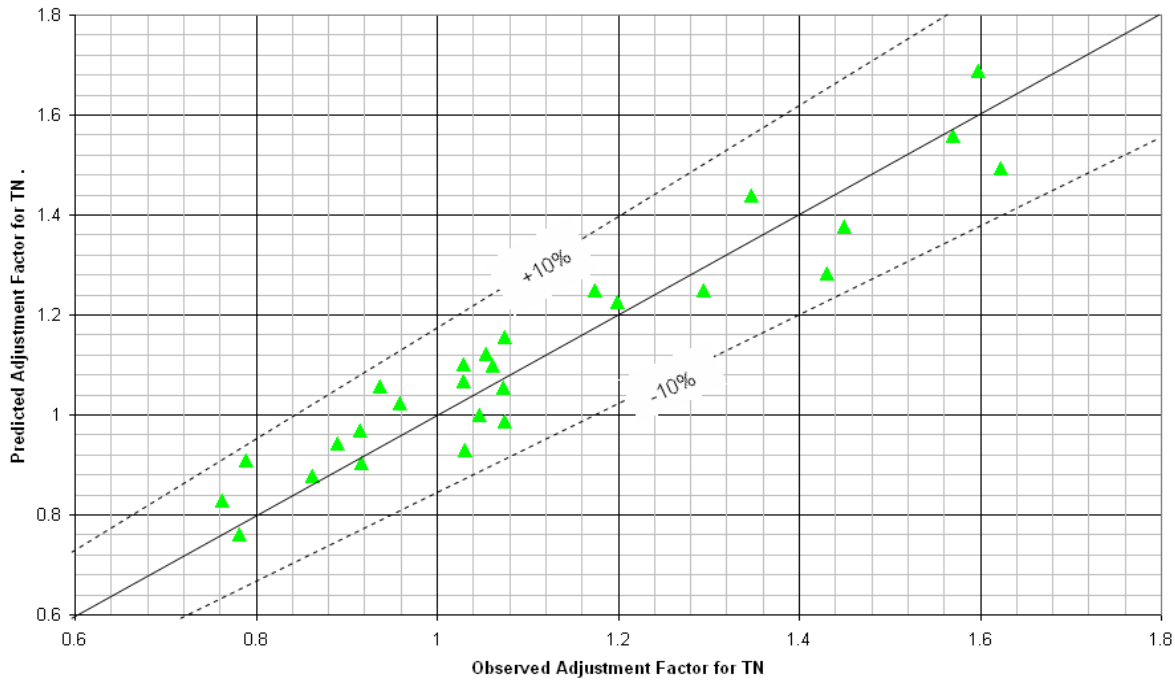

Figure B.4.2 Predicted versus observed adjustment factors for wetlands in Greater Victoria.

B.4.1.2 Bioretention systems

Figure B.4.3 shows a plot of the adjustment factors derived for the 30 stations and the corresponding MAR. Again, a trend of increasing adjustment factor with MAR is evident for each of the hydrologic regions.

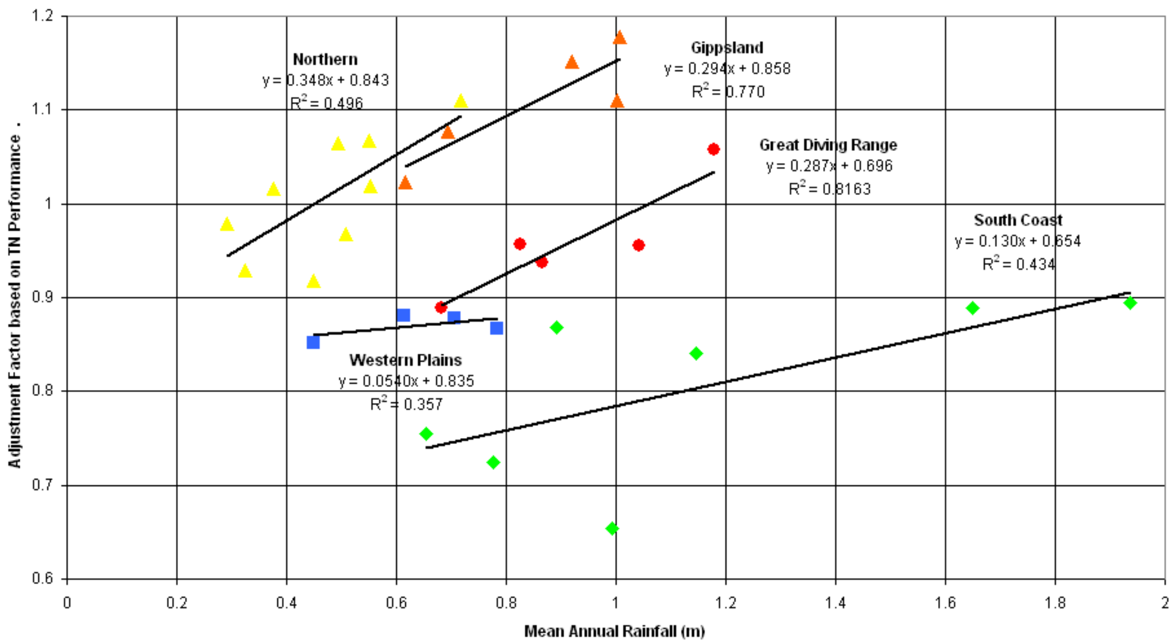

Figure B.4.3 Plot of bioretention system adjustment factor versus Mean Annual Rainfall (MAR) for bioretention systems in Greater Victoria.

The bioretention system size adjustment factor equations are shown in Table B.4.2.

Table B.4.2 Bioretention system size adjustment factor equations

Region	Bioretention system size adjustment factor equation
Northern	Adjustment factor = 0.348(MAR) + 0.843 [R2 = 0.50]
Western Plains	Adjustment factor = 0.054(MAR) + 0.835 [R2 = 0.36]
South Coast	Adjustment factor = 0.130(MAR) + 0.654 [R2 = 0.43]
Great Dividing Range	Adjustment factor = 0.287(MAR) + 0.696 [R2 = 0.82]
Gippsland	Adjustment factor = 0.294(MAR) + 0.858 [R2 = 0.77]

Figure B.4.4 shows a plot of the observed adjustment factor for each station and the predicted adjustment factor. All but two of the predicted adjustment factors are within 10% of the corresponding observed adjustment factors. Predictions for two stations lie outside 10% of the observed values. They are Yallourn (16% difference) and East Tarwin (11% difference), both in the South Coast region.

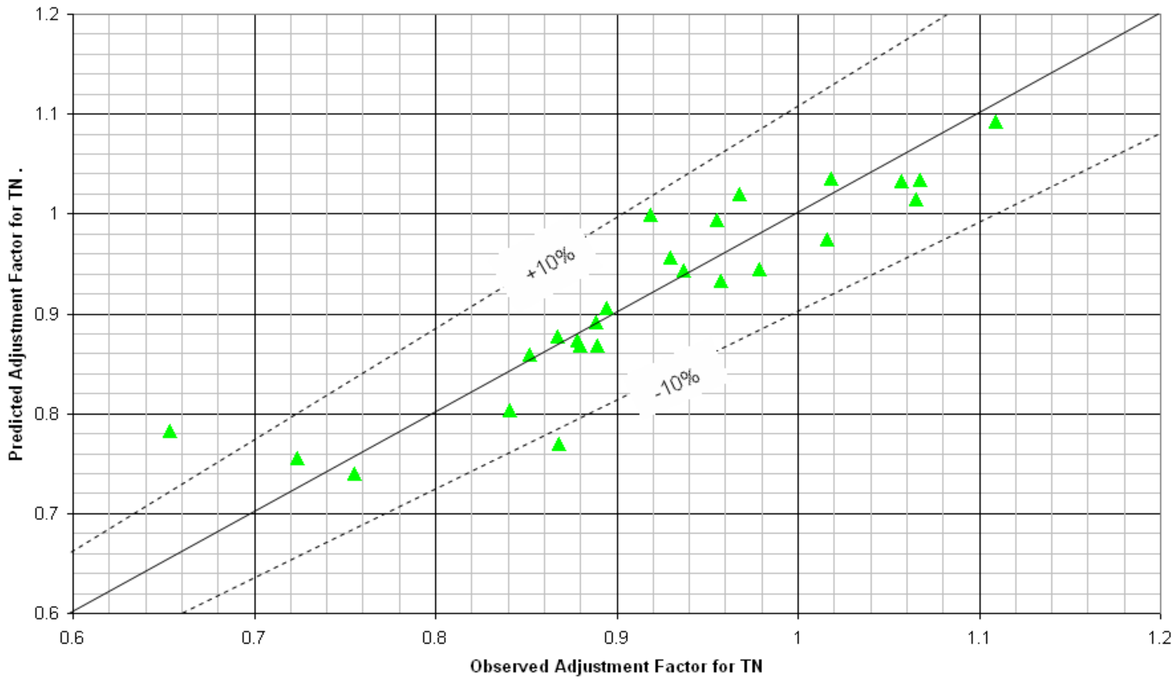

Figure B.4.4 Predicted versus 'observed' adjustment factors for bioretention systems in Greater Victoria.

B.4.2.3 Swales

Figure B.4.5 shows a relationship between the adjustment factors derived and MAR for the 30 stations grouped by their regions. Again, a trend of increasing adjustment factor with MAR is evident for each of the hydrologic regions.

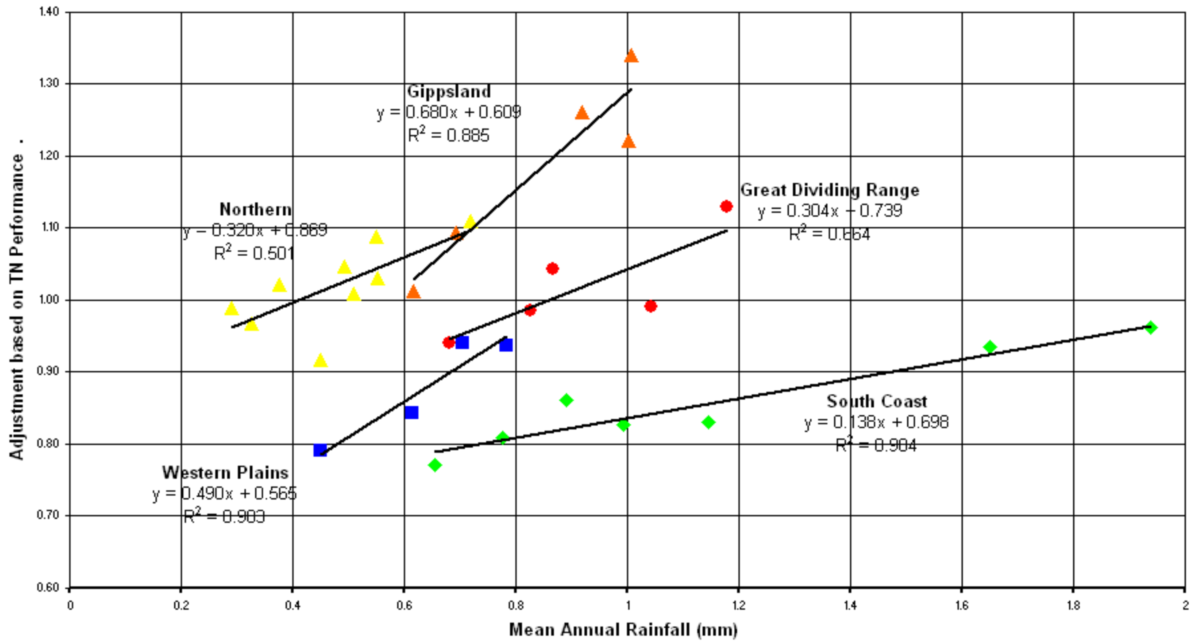

Figure B.4.5 Plot of adjustment factor versus Mean Annual Rainfall (MAR) for swales in Greater Victoria.

The swale size adjustment factor equations are shown in the Table B.4.3

Table B.4.3 Swale size adjustment factor equations

Region	Swale size adjustment factor equation
Northern	Adjustment factor = 0.320(MAR) + 0.869 [R2 = 0.50]
Western Plains	Adjustment factor = 0.490 (MAR) + 0.565 [R2 = 0.90]
South Coast	Adjustment factor = 0.138(MAR) + 0.698 [R2 = 0.90]
Great Dividing Range	Adjustment factor = 0.304(MAR) + 0.739 [R2 = 0.64]
Gippsland	Adjustment factor = 0.680(MAR) + 0.609 [R2 = 0.89]

Figure B.4.6 shows a plot of the observed adjustment factor for each station and the predicted adjustment factor. All the predicted adjustment factors are within 10% of the corresponding observed adjustment factors.

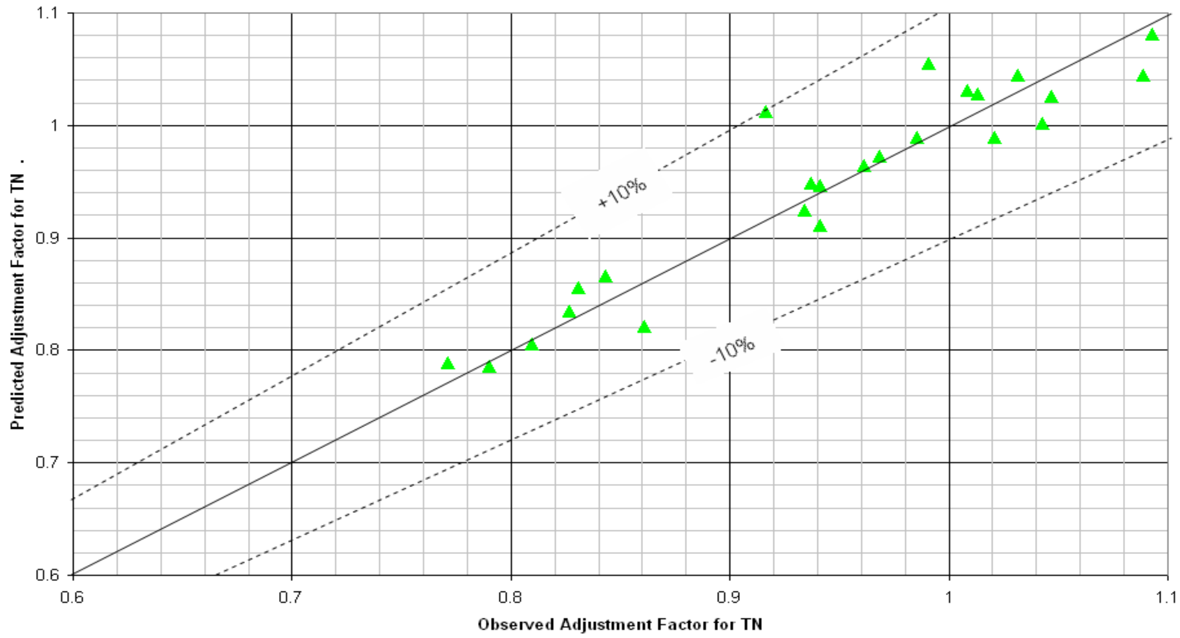

Figure B.4.6 Predicted versus 'observed' adjustment factors for swales in Greater Victoria.

B.4.1.4 Ponds

Figure B.4.7 shows a plot of the relationship between the adjustment factors derived and the MAR for the 30 stations grouped by their regions. Again, a trend of increasing adjustment factor with MAR is evident for each of the hydrologic regions.

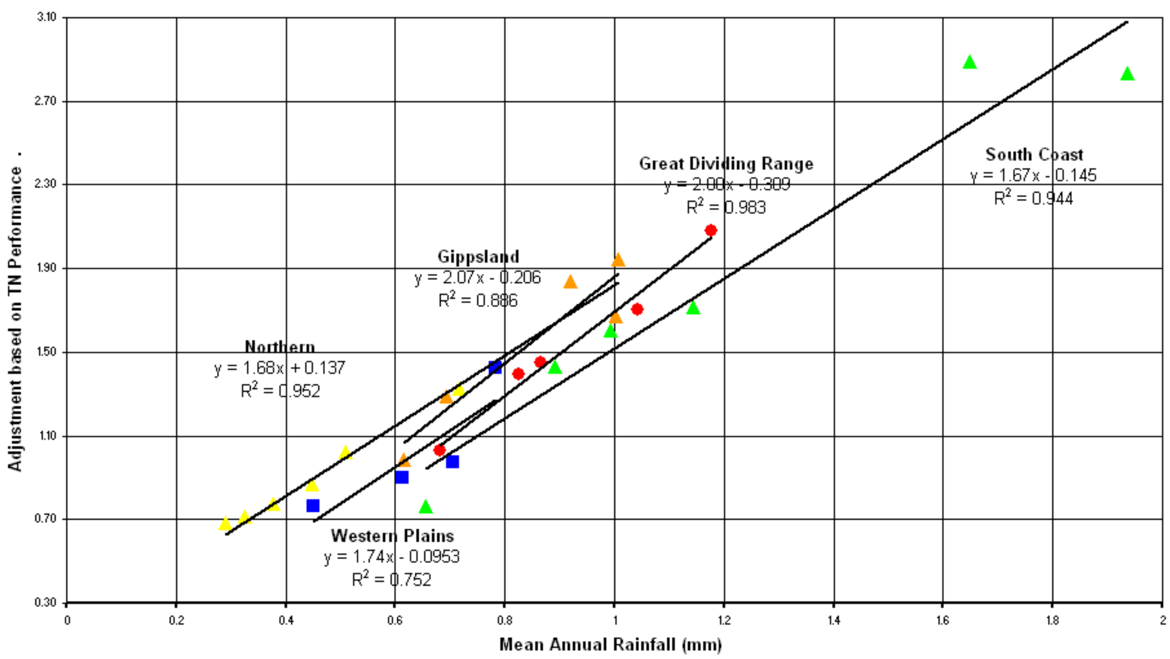

Figure B.4.7 Plot of adjustment factor versus Mean Annual Rainfall (MAR) for ponds in Greater Victoria.

There appears to be a stronger correlation between the size adjustment factor and MAR for ponds than for other treatment measures.

The pond size adjustment factor equations are shown in Table B.4.4.

Table B.4.4 Pond size adjustment factor equations

Region	Pond size adjustment factor equation
Northern	Adjustment factor = 1.68(MAR) + 0.137 [R2 = 0.95]
Western Plains	Adjustment factor = 1.74(MAR) – 0.0953 [R2 = 0.75]
South Coast	Adjustment factor = 1.67(MAR) – 0.145 [R2 = 0.94]
Great Dividing Range	Adjustment factor = 2.00(MAR) – 0.309 [R2 = 0.98]
Gippsland	Adjustment factor = 2.07(MAR) – 0.206 [R2 = 0.89]

Figure B.4.8 shows a plot of the observed adjustment factor for each station and the predicted adjustment factor. All but two of the predicted adjustment factors are within 10% of the corresponding observed adjustment factors. Predictions for the two stations that lie outside 10% of the observed values are Casteron (19% difference) in the South Coast region and Ballarat (14% difference) in the Western Region.

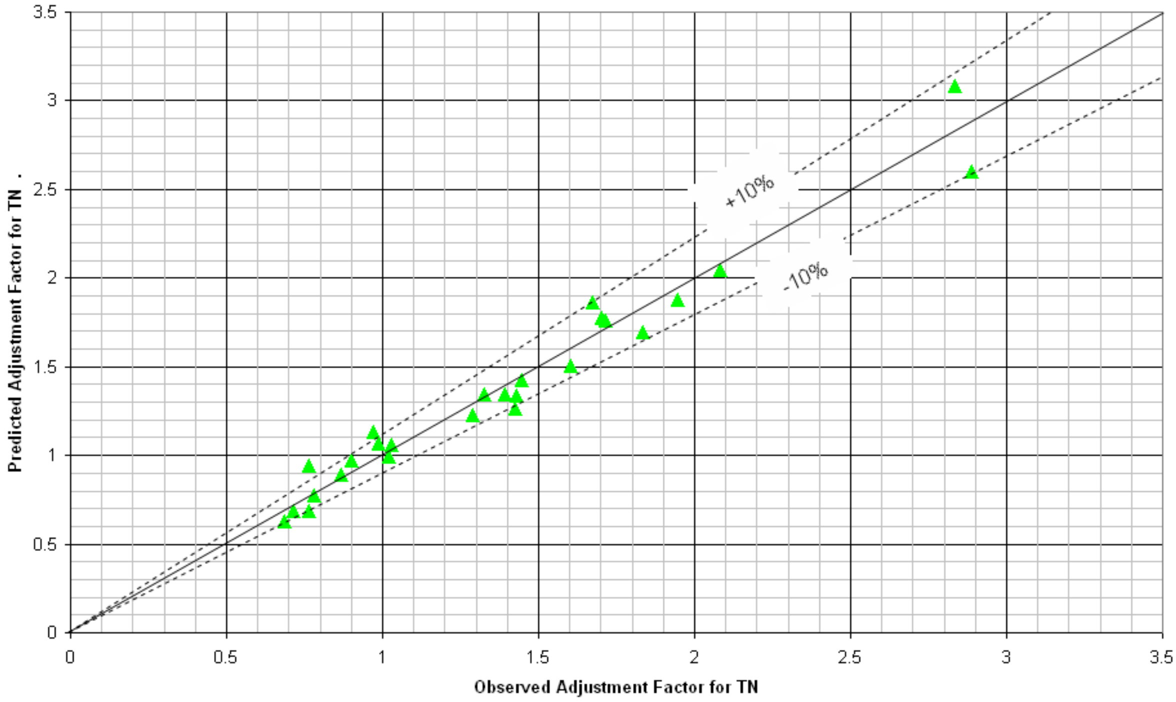

Figure B.4.8 Predicted versus ‘observed’ adjustment factors for ponds in Greater Victoria.

B.4.2 Adjustment factors for the Melbourne/Geelong metropolitan region

B.4.2.1 Wetlands

Figure B.4.9 shows a plot of the wetland size adjustment factor*s* derived against MAR for the 15 stations in the Melbourne/Geelong metropolitan region and grouped into four regions. For the central and north-west region, there appears to be a negative correlation between the adjustment factor and MAR. For the other three regions, the adjustment factor can best be represented by a single value for the region. Rainfall at stations to the east of Melbourne is consistently higher that at stations to the west of Melbourne but this does not necessarily lead to larger required treatment area for wetlands in the eastern metropolitan areas. Seasonal rainfall patterns which are implicitly accounted for in the regionalisation procedure compensate for the influence of MAR in this case. The eastern region of metropolitan Melbourne has a more evenly distributed rainfall over the year compared with the western metropolitan region.

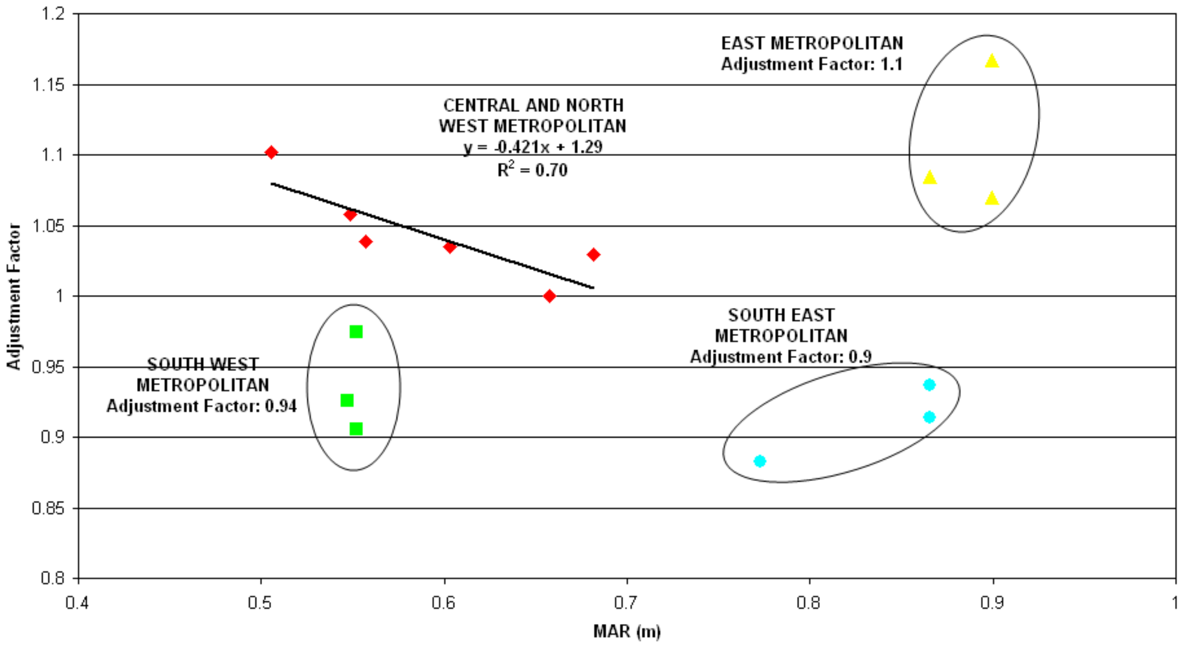

Figure B.4.9 Adjustment factor versus Mean Annual Rainfall (MAR) for wetlands in the Melbourne/Geelong metropolitan region.

The equation to compute the adjustment factor for the central and north-west metropolitan region was obtained by plotting a linear trend (i.e. line of best fit) through the points for this region. This equation and the adjustment factor for the other three regions are shown in Table B.4.5.

Table B.4.5 Wetland adjustment factor equations

Region	Wetland adjustment factor equation
Central and North West Metropolitan	Adjustment factor = -0.421(MAR) + 1.29 [R2 = 0.70]
South West Metropolitan	Adjustment factor = 0.94
East Metropolitan	Adjustment factor = 1.1
South East Metropolitan	Adjustment factor = 0.9

Figure B.4.10 shows a plot of the observed adjustment factor for each station (i.e. determined from the MUSIC model) and the predicted adjustment factor (i.e. that obtained from the equations/values in Table B.4.5. The dotted lines mark a 10% difference between the predicted and observed adjustment factor. All of the predicted adjustment factors are within 10% of the corresponding observed adjustment factors.

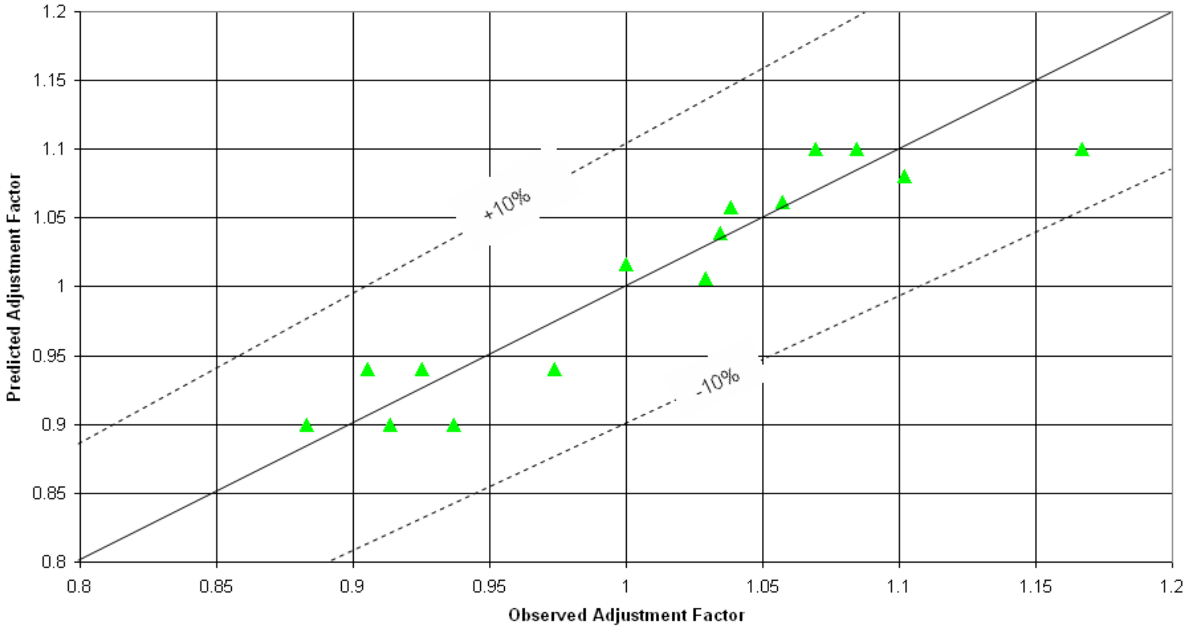

Figure B.4.10 Predicted versus 'observed' adjustment factors for wetlands in the Melbourne/Geelong metropolitan region.

B.4.2.2 Bioretention systems

Figure B.4.11 shows a plot of the bioretention system size adjustment factor*s* derived for the 15 stations in the Melbourne/Geelong metropolitan region and the corresponding MAR. For the central and north-west region, there appears to be a negative correlation between the adjustment factor and MAR. For the other three regions, the adjustment factor can best be represented by a single value for the region.

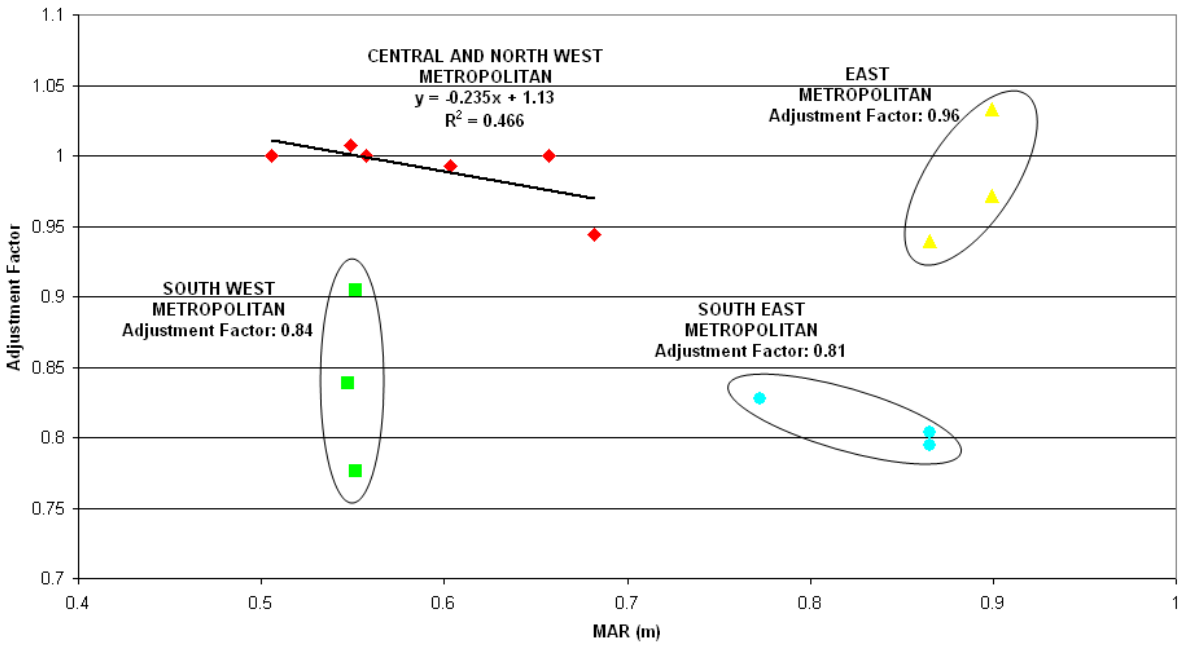

Figure B.4.11 Adjustment factor versus Mean Annual Rainfall (MAR) for bioretention systems in the Melbourne/Geelong metropolitan region.

An equation to compute the adjustment factor for the central and north-west metropolitan region and the adjustment factor for the other three regions are shown in Table B.4.6.

Table B.4.6 Bioretention system size adjustment factor equations

Region	Bioretention system size adjustment factor equation
Central and North West Metropolitan	Adjustment factor = -0.235(MAR) + 1.13 [R2 = 0.47]
South West Metropolitan	Adjustment factor = 0.84
East Metropolitan	Adjustment factor = 0.96
South East Metropolitan	Adjustment factor = 0.81

Figure B.4.12 shows a plot of the observed adjustment factor for each station and the predicted adjustment factor. It can be seen that all of the predicted adjustment factors are within 10% of the corresponding observed adjustment factors.

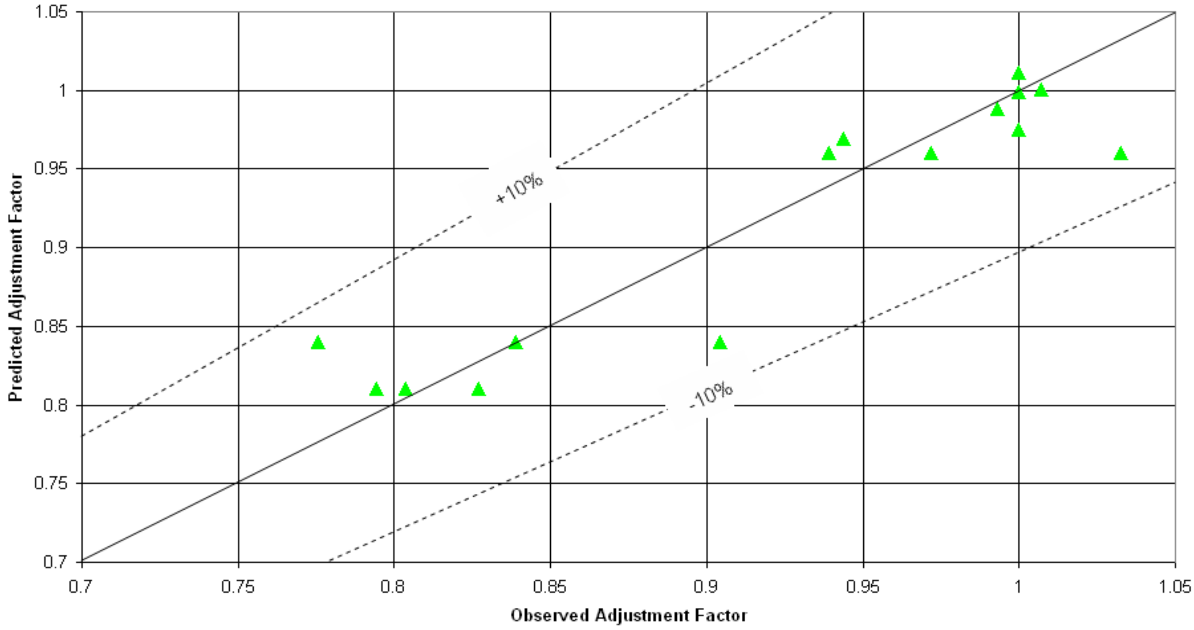

Figure B.4.12 Predicted versus observed adjustment factors for bioretention systems in the Melbourne/Geelong metropolitan region.

B.4.2.3 Swales

Figure B.4.13 shows a plot of the swale size adjustment factors derived for the 15 stations in the Melbourne/Geelong metropolitan region and the corresponding MAR. For the central and north-west metropolitan region, there appears to be a negative correlation between the adjustment factor and MAR. For the other three regions, the adjustment factor can best be represented by a single value for the region.

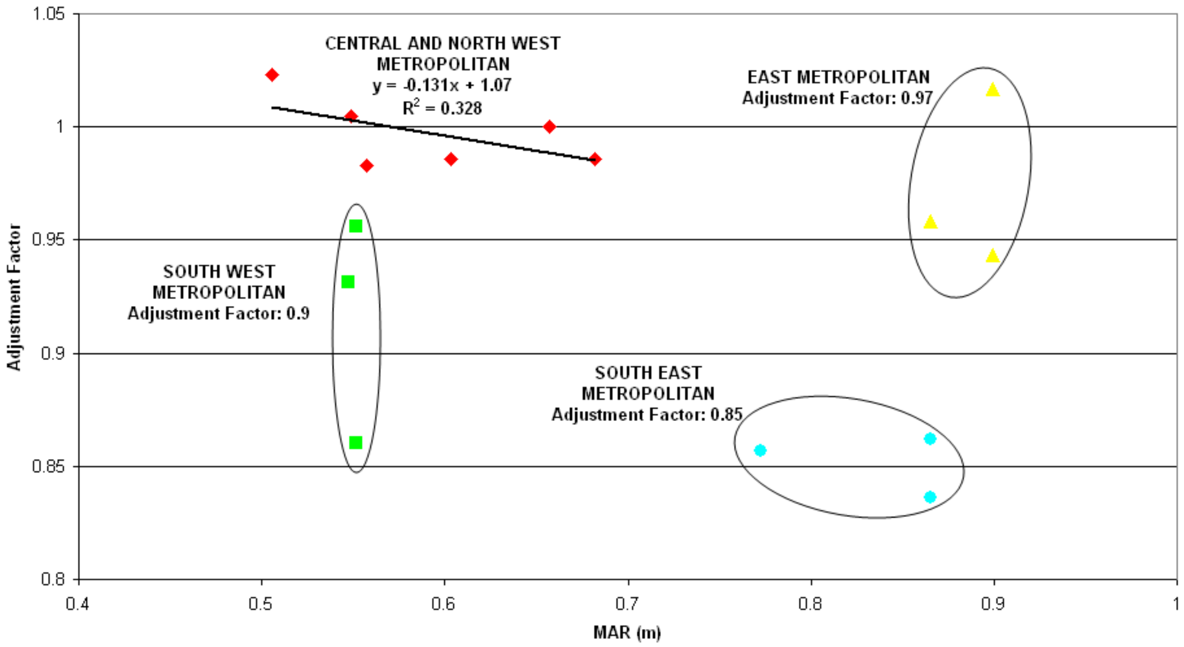

Figure B.4.13 Adjustment factor versus Mean Annual Rainfall (MAR) for swales in the Melbourne/Geelong metropolitan region.

The equation to compute the adjustment factor for the central and north-west region and the adjustment factor for the other three regions are shown in Table B.4.7.

Table B.4.7 Swale size adjustment factor equations

Region	Swale size adjustment factor equation
Central and North West Metropolitan	Adjustment factor = -0.131(MAR) + 1.07 [R2 = 0.33]
South West Metropolitan	Adjustment factor = 0.9
East Metropolitan	Adjustment factor = 0.97
South East Metropolitan	Adjustment factor = 0.85

FigureB.4.14 shows a plot of the observed adjustment factor for each station and the predicted adjustment factor. All of the predicted adjustment factors are within 10% of the corresponding observed adjustment factors.

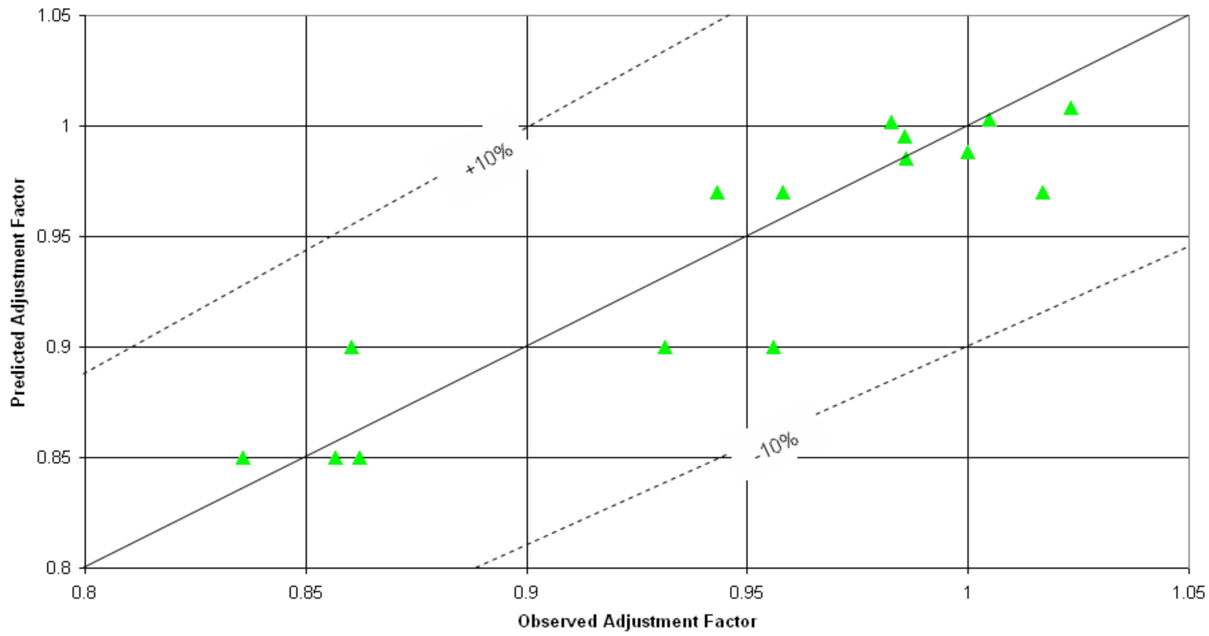

Figure B.4.14 Predicted versus observed adjustment factors for swales in the Melbourne/Geelong metropolitan region.

B.4.2.4 Ponds

Figure B.4.15 shows a plot of the pond size adjustment factors derived for the 15 stations in the Melbourne/Geelong metropolitan region and the corresponding MAR. For the central and north-west metropolitan region, there appears to be a positive correlation between the adjustment factor and MAR. For the other three regions, the adjustment factor can best be represented by a single value for the region.

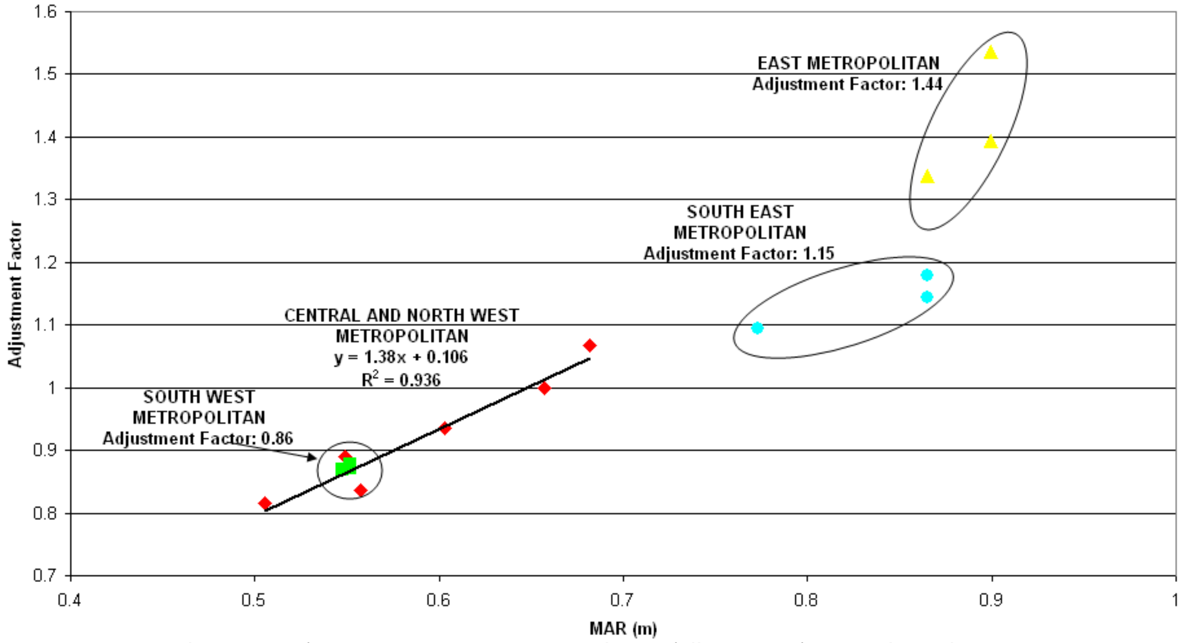

Figure B.4.15 Adjustment factor versus Mean Annual Rainfall (MAR) for ponds in the Melbourne/Geelong metropolitan region.

The impact of MAR appears to be greater for ponds than for the other three treatment measures. The sites to the east of Melbourne with higher MARs have higher adjustment factors than those on the western side.

The equation to compute the adjustment factor for the central and north-west metropolitan region and the adjustment factor for the other three regions are shown in Table B.4.8.

Table B.4.8 Pond adjustment factor equations

Region	Pond adjustment factor equation
Central and North West Metropolitan	Adjustment factor = 1.38(MAR) + 0.106 [R2 = 0.94]
South West Metropolitan	Adjustment factor = 0.86
East Metropolitan	Adjustment factor = 1.44
South East Metropolitan	Adjustment factor = 1.15

Figure B.4.16 shows a plot of the observed adjustment factor for each station and the predicted adjustment factor. All of the predicted adjustment factors are within 10% of the corresponding observed adjustment factors.

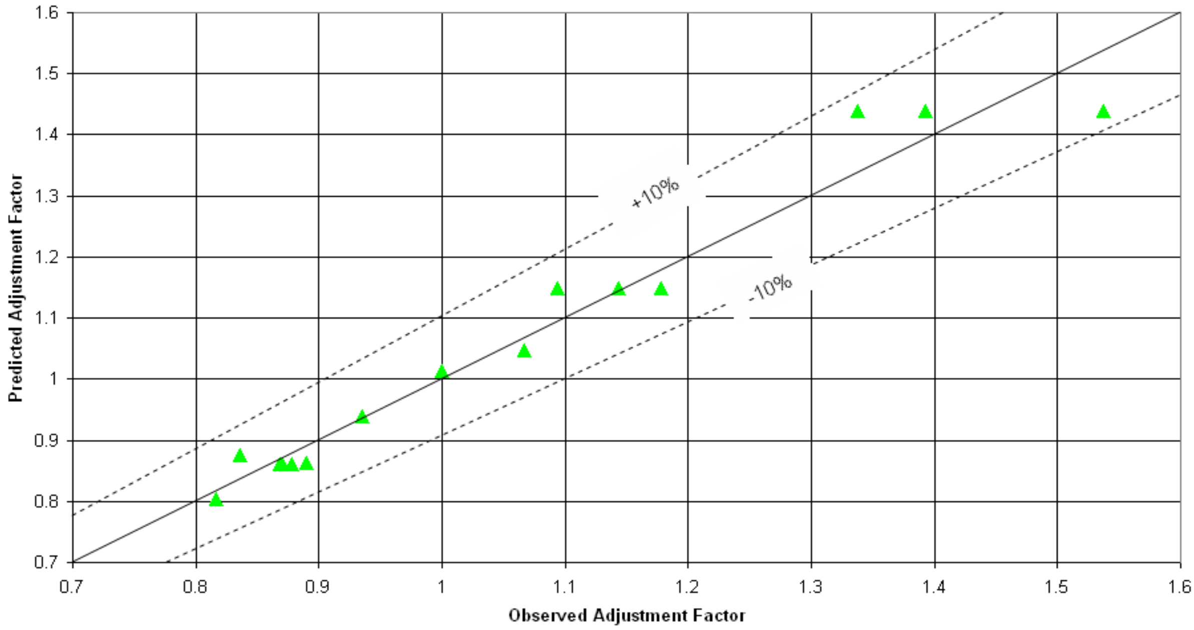

Figure B.4.16 Predicted versus observed adjustment factors for ponds in the Melbourne/Geelong metropolitan region.

B.5 Adjustment factors for reference rainfall stations

The regional equations and constants for computing adjustments factors are the result of pooling modelling results for relevant reference pluviographic stations within each hydrologic region. To ensure a systematic application of the procedure, it is recommended that computation of adjustment factor*s* should exclusively use the regional equations or constants provided instead of individually derived values for adjustment factors, irrespective of the proximity of the site to a reference pluviographic station. This would avoid situations where practitioners get to choose between the adjustment factor computed from the regional approach and that derived for the reference pluviographic station of close proximity to the site in question.

If the option for practitioners to use adjustment factor*s* derived for individual reference pluviographic stations is to be provided, a consistent approach to define the areal extent of applicability of adjustment factors derived for individual pluviographic stations will need to be developed. This areal extent of applicability for individual reference pluviographic station may vary depending on its proximity to other pluviographic stations and will probably be determined in an ad hoc manner. Furthermore, this option could also introduce debate among practitioners about the selection of reference pluviographic stations for the present analysis ahead of others which may be of 'more relevant' to their particular sites.

It is recommended that only regional relationships for adjustment factors be used in this document.

B.6 Recommended adjustment factors

The plots comparing the predicted adjustment factors to those determined from MUSIC modelling indicate that the regional equations and constants derived for the five state-wide hydrologic regions and four regions for the Melbourne/Geelong metropolitan region fall within a 10% band. It is, thus, reasonable to adopt an adjustment factor that is 1.1 times (i.e. within 10%) the amount predicted by these equations and constants to ensure that predicted size of stormwater treatment measures using this method will not be an underestimation of what is required. This preserves the opportunity (and incentive) for practitioners to adopt a more rigorous approach (e.g. MUSIC modelling using local rainfall data) to further refine and reduce the size of treatment measures being considered if they so desire. The recommended equations and constants (including a + 10% adjustment) for computing the appropriate adjustment factors for Victoria, including the Melbourne/Geelong metropolitan region, are summarised in Tables B.6.1 and B.6.2.

Table B.6.1 Greater Victoria adjustment factors

Region	Wetland	Bioretention	Swale	Pond
Northern	0.833(MAR) + 0.751	0.383(MAR) + 0.927	0.352(MAR) + 0.956	1.85(MAR) + 0.151
Western Plains	0.788(MAR) + 0.483	0.059(MAR) + 0.919	0.539(MAR) + 0.622	1.91(MAR) – 0.105
South Coast	0.737(MAR) + 0.428	0.143(MAR) + 0.719	0.152(MAR) + 0.768	1.84(MAR) – 0.160
Great Dividing Range	0.969(MAR) + 0.572	0.316(MAR) + 0.766	0.334(MAR) + 0.813	2.20(MAR) – 0.340
Gippsland	1.78(MAR) + 0.273	0.325(MAR) + 0.944	0.748(MAR) + 0.670	2.28(MAR) – 0.227

Table B.6.2 Melbourne/Geelong metropolitan region adjustment factors

Region	Wetland	Bioretention	Swale	Pond
Central and North West Metropolitan	– 0.463(MAR) + 1.421	– 0.259(MAR) + 1.243	– 0.144(MAR) + 1.18	1.52(MAR) + 0.117
South West Metropolitan	1.03	0.924	0.99	0.946
East Metropolitan	1.21	1.06	1.07	1.58
South-east Metropolitan	0.99	0.891	0.935	1.27

B.7 Example of an application of a Mean Annual Rainfall method

Figure B.7.1 is a plot of the wetland performance of constructed stormwater treatment wetlands based on a series of MUSIC simulations using Melbourne rainfall. This is the reference plot for the sizing of constructed wetlands (with 0.75 extended detention and 72-hour notional detention time) in Victoria.

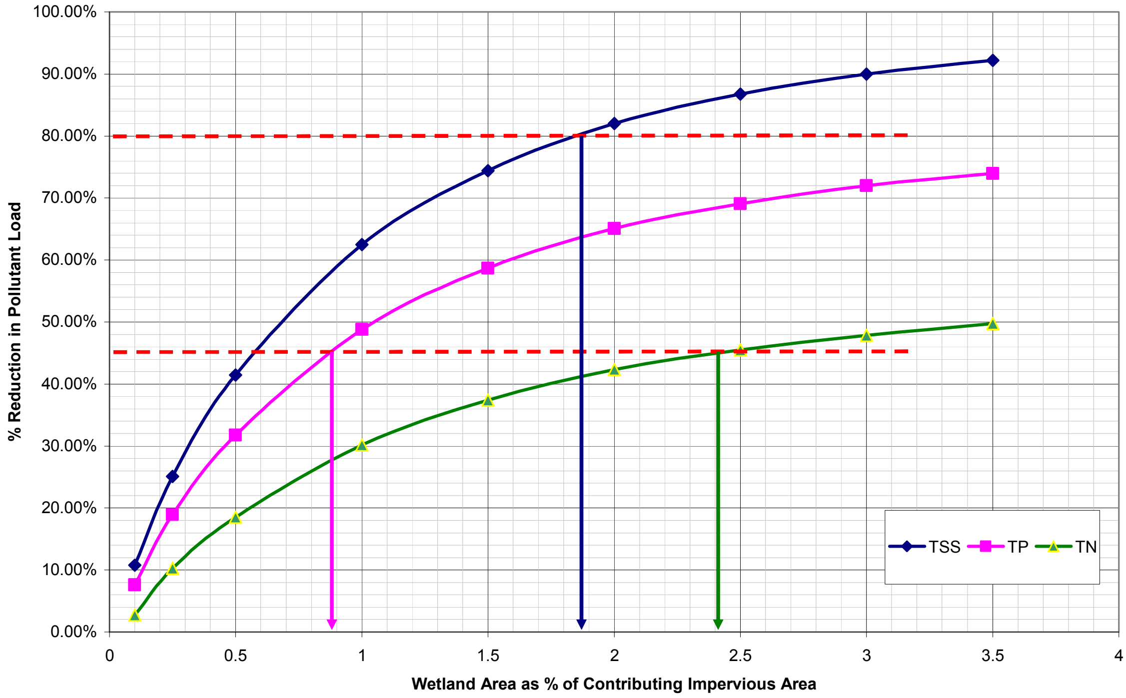

Figure B.7.1 Performance curve for constructed wetlands in Melbourne.

To satisfy the objectives for the performance of stormwater treatment of 80% reduction in TSS and 45% reduction in TP and TN in Melbourne, the required wetland size is to be about 2.4% of the contributing impervious area in the catchment. The required wetland size for reduction of TN was the critical design condition in this case [i.e. a larger wetland is needed to meet the TN objectives than the TSS (1.86% impervious area) and TP (0.88% impervious area)] objectives. The area will then need to be adjusted with the wetland size adjustment factor derived from Table B.6.2.

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

1. From Figure B.7.1, the reference wetland area is 2.4% of the contributing impervious area,

   i.e. contributing impervious area = $0.5 \times 500\,000 = 250\,000$ m^2

   reference wetland area = $0.024 \times 250\,000 = 6000$ m^2.

2. The adjustment factor for Gippsland region is computed using the equation in Table B.6.1:

   adjustment factor = 1.78(MAR) + 0.273

   = 1.78(0.85) + 0.273 = 1.8

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

B.8 Summary

A simple procedure for sizing stormwater treatment measures to meet current best practice environmental management objectives for stormwater is proposed here. This procedure is based on defining nine hydrologic regions within Victoria (four or which are in the Melbourne/Geelong metropolitan area). Empirical methods for determining an adjustment factor for sites within these regions have been derived for the design of constructed wetlands, bioretention systems, swales and ponds.

Melbourne was selected as the reference site in this procedure. Detailed simulations of a wide range of treatment measures with different configurations for this reference site will now be undertaken to provide a comprehensive set of performance curves. These curves can then be adapted for use in different sites across Victoria by use of adjustment factors. The relevant value of an adjustment factor for any particular site can be computed from the relevant equations for the hydrologic region and this is then used to adjust the area of the treatment measure found suitable for Melbourne.

B.9 References

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

Appendix C Victorian hydrological regions for sizing rainwater tanks

Contents

C.1 Introduction

Household rainwater tanks provide an opportunity to reduce mains potable water use by storing collected roof runoff and using it for applications such as toilet flushing and garden watering.

There are no quantitative performance targets in any of Victoria's local government or state authority policies and guidelines controlling the use of rainwater tanks. However, it can be inferred from the various policies and guidelines that a performance target for rainwater tanks (or any other form of rainwater and stormwater harvesting, storage and reuse scheme) is to provide a 'reliable' supply of suitable quality water to meet the demand requirements of a stipulated preferred 'end-use' (e.g. toilet flushing).

Variables include the area of a roof directed into a tank, quantity and nature of the demand, rainfall pattern and the required reliability. Reliability is defined as the percentage of demand that can be met using collected rainwater. Where reliability is less than 100% (i.e. roof runoff cannot meet 100% of demand), an additional water source, such as mains water, will be required to meet a proportion of demand.

This Appendix presents a simple design procedure for sizing rainwater tanks to meet a range of reliabilities for toilet-flushing reuse across Victoria. The procedure is based on developing sizing curves for a reference site (Melbourne) and then adjusting the tank size for other areas in the state depending on location and Mean Annual Rainfall (MAR).

Three tank-sizing regions within Victoria have been defined and show the relationship between MAR and required tank size in each region. These regions are different to those determined to size stormwater treatment measures (see Appendix A) as different aspects of rainfall patterns are important for treatment measures than for reuse applications.

C.2 Methodology

After initial consideration of possible design approaches, the following approach was used to determine hydrologic regions and sizing curves for rainwater tanks throughout Victoria:

1. determine a water reuse application (i.e. toilet flushing) and estimate demand magnitude and distribution (see Section C.3).
2. establish Melbourne tank sizing curves (relating tank size and reuse 'reliability') for a range of reuse demands (see Section C.4).
3. determine the size of tank required at locations around Victoria to achieve an equivalent reliability at certain reference points on the Melbourne tank sizing curves (see Section C.5).
4. define Tank Sizing Regions within Victoria for which the tank size required to achieve the same reliability as a given tank in Melbourne can be predicted based on MAR (see Section C.6).
5. develop rainwater tank sizing curves for each region (see Section C.7).

C.3 Determining reuse application

Water consumption 'per household' varies depending on house type and location although consumption figures 'per person' were found to be less influenced by these factors. Typical water consumption figures for residential areas expressed on a per capita basis are summarised in Figure C.3.1. These consumption rates are for dwellings with water efficient fittings and appliances. Consumption of water for toilet flushing has reduced significantly since the mandatory introduction of dual flush toilets in.

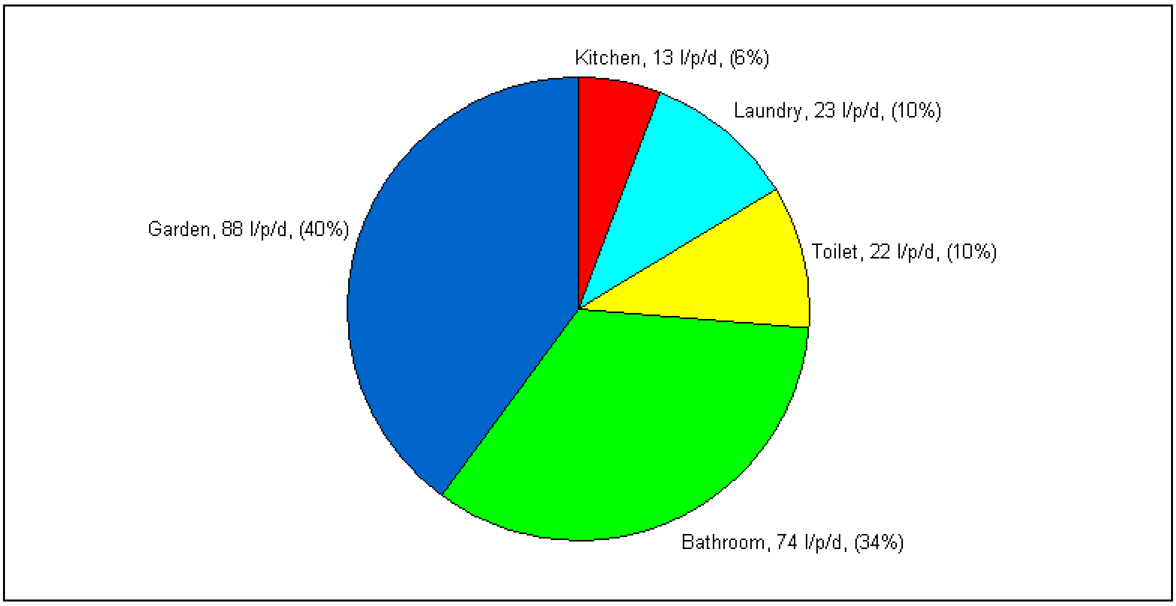

Figure C.3.1 Typical household water use in Victoria (assuming water efficient fittings and appliances) (Coomes Consulting Group 2002, Water Resources Strategy Committee 2001).

Rainwater tanks can be used to supply any one of these uses or combinations of them. The most obvious water uses for rainwater are toilet and garden as they do not require treatment to potable standards. Replacement of mains potable water for toilet flushing is considered to be the more effective of the two because of its consistent demand pattern and thus a higher reliability of water supply can be achieved for a given tank size. Although having a higher water demand, water usage for garden watering is seasonal and the demand pattern is 'out-of-phase' with the supply pattern (i.e. high garden watering demand occurs during low rainfall) and thus requires a larger rainwater tank storage to achieve comparable reductions in potable water usage compared with toilet flushing.

After toilet flushing and garden watering, the next most appropriate use of rainwater is in the laundry (e.g. washing, cold tap). Supplementing the supply for hot water is also an effective option. Hot water usage constitutes about 40% of a household indoor usage. The quality of roof water contained in hot water systems can often be improved through pasteurisation, pressure in the pump and instantaneous heat differentials between the rainwater tank and a hot water service.

Toilet flushing has been selected as the reuse application for this procedure as it is applicable to all types of residential development and the level of use can be predicted with a reasonable degree of certainty. The demand is assumed to be 22 L/person per day (from Figure C.3.1) which represents a 6/3 toilet system.

C.4 Melbourne rainwater tank sizing curves

The Model for Urban Stormwater Improvement Conceptualisation (MUSIC) (Cooperative Research Centre for Catchment Hydrology 2002) was used to establish the curves in Figure C.4.1, which relate reuse reliabilities to tank sizes using Melbourne rainfall data. The simulation period was between January 1980 and December 1989, covering the dry period in 1982/83. Figure C.4.2 shows the MUSIC model set-up.

All units in Figure C.4.1 are dimensionless and so the curves can be used for any sized roof. The four curves relate to a range of occupancy densities that would be expected in Victoria. The curve for a 1.5 person/100 m^2 roof represents low density housing such as a large house in a rural area. In comparison, a 4.5 person/100 m^2 roof represents a much higher density such as an inner city apartment. Figure C.4.1 shows that the lowest occupancy density corresponds to the highest reliability for a given tank size. Reliability increases as tank size increases up to where either 100% of reuse demand is met or 100% of rainwater collected is being used.

Reduction in mains potable water use can be determined by multiplying the reuse demand by the reliability. For example, a 1 kL rainwater tank at a Melbourne house with a 208 m^2 roof and five occupants (i.e. 2.4 people/100 m^2 roof) would provide a reliability of 86% (from Figure C.4.1). The toilet flushing demand is:

22 L/person per day (from Figure C.3.1) $\times$ 5 people $\times$ 365 days = 40.2 kL/year

The reduction in mains potable water is therefore:

40.2 kL/year $\times$ 86% = 35.0 kL/year.

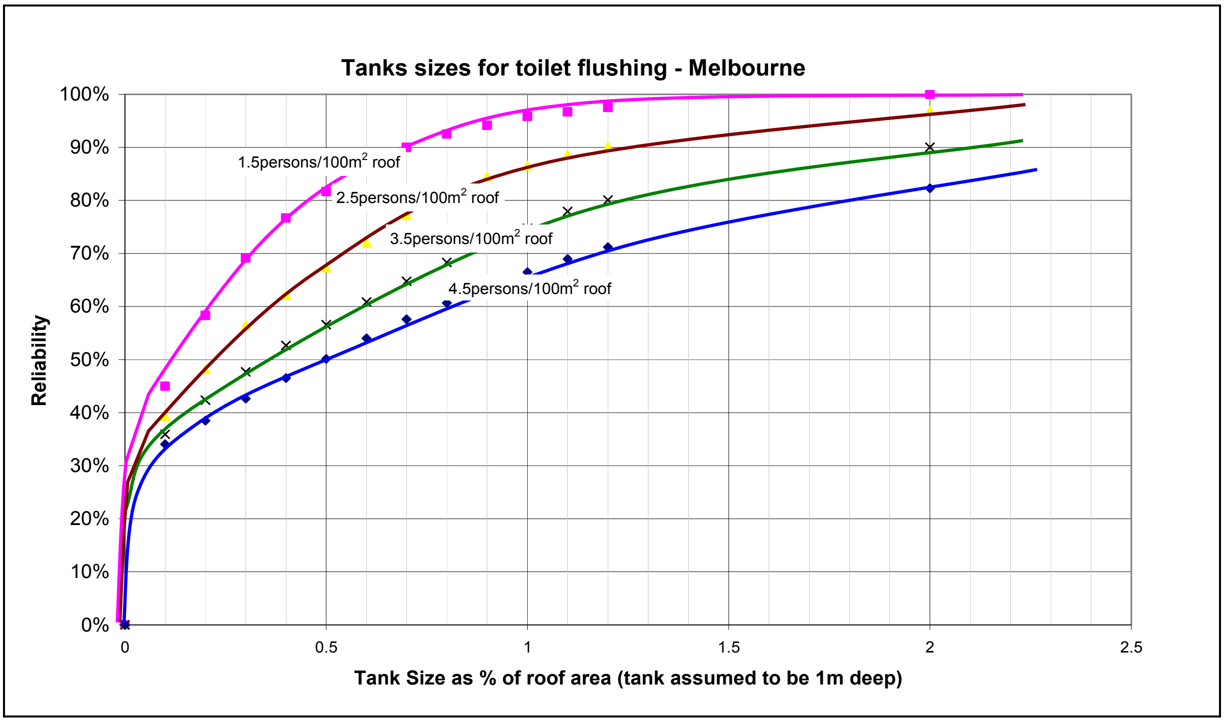

Figure C.4.1 Typical reliabilities for tanks used for toilet flushing reuse in Melbourne.

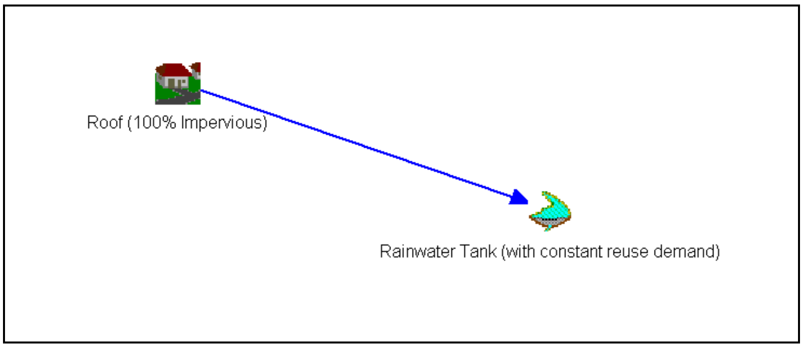

Figure C.4.2 MUSIC Model setup.

C.5 Determining tank sizing regions

Rainwater *tank sizing region*s were determined using data from 45 pluviographic stations throughout Victoria. Fifteen of these are concentrated around the Melbourne/Geelong metropolitan region. The additional stations around Melbourne were considered important because of expected development activity. There are more available data for this region which enables a finer representation of the climatic factors. These stations and their Bureau of Meteorology rainfall district are shown in Table C.5.1 below.

Table C.5.1 Pluviographic stations and Bureau of Meteorology (BOM) districts

BOM district	Stations
Wimmera South	Horsham Tottington Wartook
North Mallee	Mildura
South Mallee	Hopetoun
Lower North	Cobram Kerang
Upper North	Bendigo Tatura Dookie
Lower North-east	Dartmouth
Upper North-east	Bright Hume Reservoir Omeo
East Gippsland	Buchan Sarsfield East Combienbar Genoa Wroxham
West Central	Laverton Melton Werribee Geelong North Little River
East Central	Melbourne Airport Bundoora Essendon Airport Melbourne Croydon Upwey Narre Warren North Dandenong Carrum Downs Koo Wee Rup

Figures C.5.1 and C.5.2 show the distribution of the stations according to their longitude and latitude bearings. The selected pluviographic stations are reasonably well distributed across Victoria and provide sufficient coverage of the state and the metropolitan region. The MAR for the sites selected ranges from 290 mm to 1900 mm, covering the range of rainfall volumes experienced across the state.

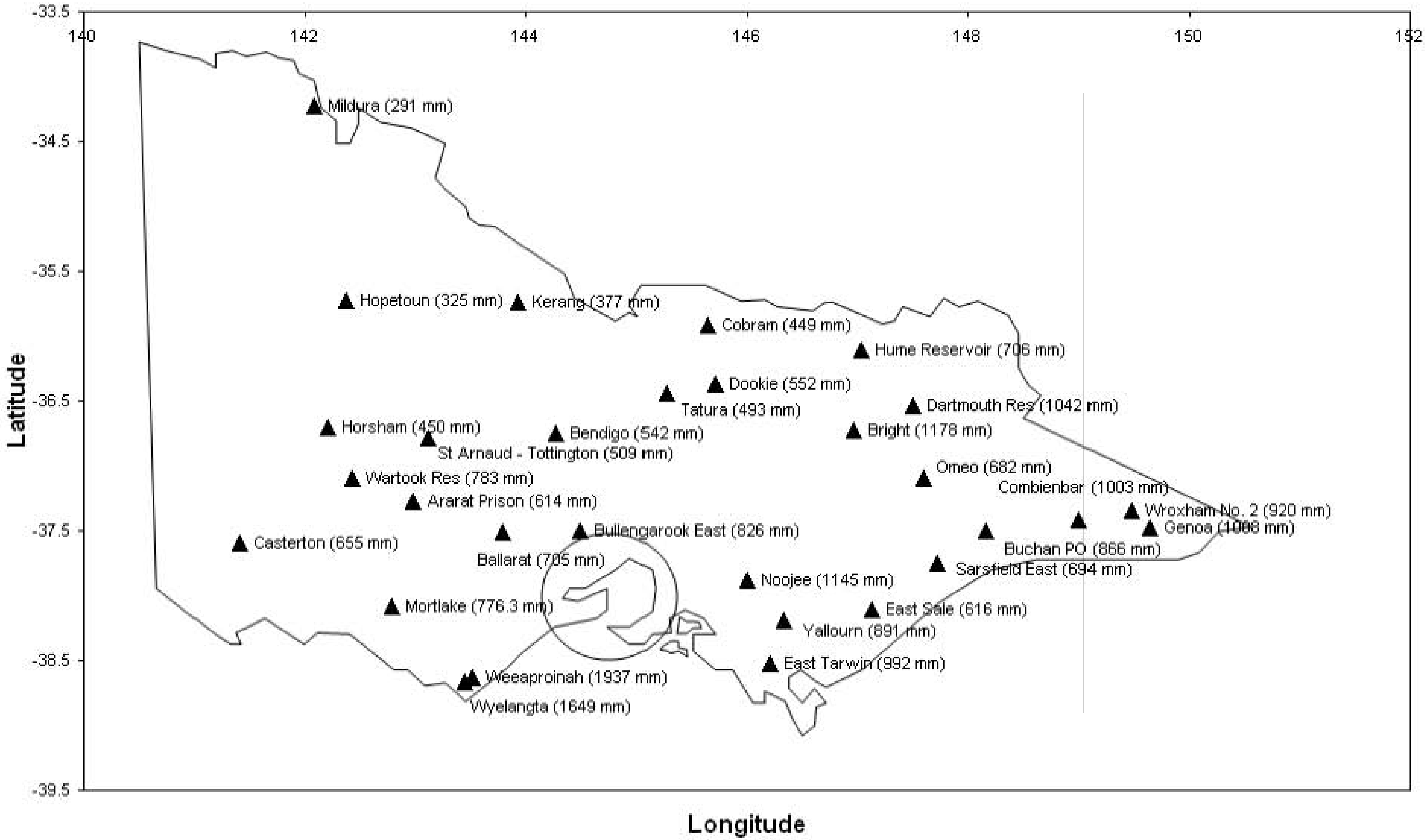

Figure C.5.1 Location of pluviographic stations in Greater Victoria used in defining tank sizing regions.

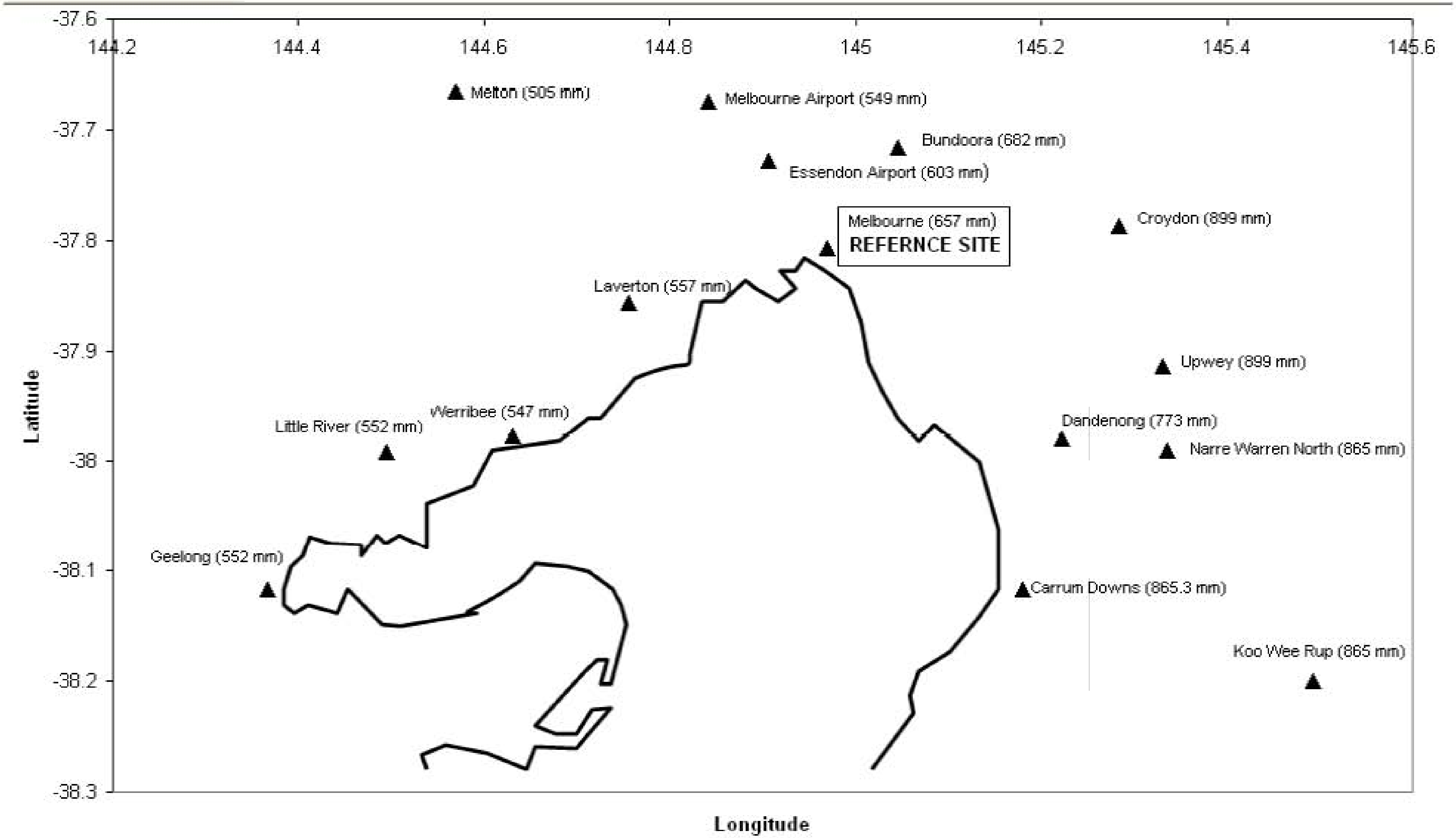

Figure C.5.2 Location of pluviograph stations in Melbourne/Geelong metropolitan region used to determine tank sizing regions.

For each of the 45 pluviographic stations in Victoria, MUSIC modelling was undertaken to determine the tank size required to achieve the same reliability as a series of reference points on the Melbourne tank sizing curves. These reference points are marked on Figure C.5.3 and are listed in Table C.5.2.

Initially, reference points relating to Melbourne 0.5 kL, 1.0 kL, 1.5 kL and 2.0 kL tanks with a 100 m^2 roof area were selected. At several other pluviographic stations in northern Victoria with low MAR, the reliabilities of 1.5 kL and 2.0 kL tanks could not be achieved; therefore, smaller tanks were modelled including 0.2 kL, 0.4 kL and 0.75 kL tanks (with 100 m^2 of roof contributing). For the lowest reuse demand (i.e. 1.5 people/100 m^2 roof), reference points corresponding to 1.5 kL and 2.0 kL were not included as there was a minimal increase in reliability achieved for tanks larger than 1 kL (i.e. it is not thought to be feasible to double tank size to increase reliability by 6%).

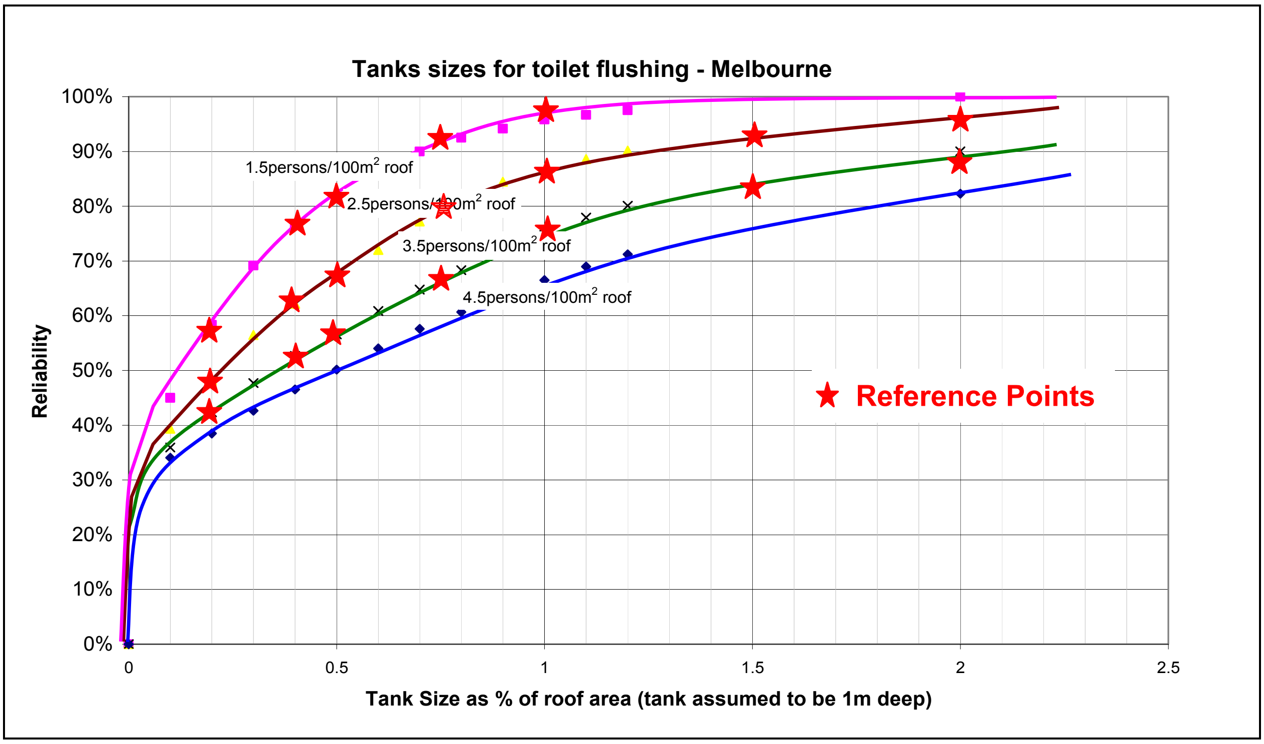

Figure C.5.3 Chart showing ‘reference points’ (stars) on Melbourne rainwater tank sizing curves.

Table C.5.2 List of reference points on the Melbourne tank sizing curves in Figure C.3.6 (stars)

No.	Equivalent Melbourne tank size (kL)	Demand (people/ 100 m^2 roof)	Reliability
1	0.2	1.5	58
2	0.2	2.4	48
3	0.2	3.5	42
4	0.4	1.5	77
5	0.4	2.4	62
6	0.4	3.5	53
7	0.5	1.5	82
8	0.5	2.4	67
9	0.5	3.5	57
10	0.75	1.5	90
11	0.75	2.4	80
12	0.75	3.5	67
13	1	1.5	94
14	1	2.4	87
15	1	3.5	75
16	1.5	2.4	93
17	1.5	3.5	84
18	2	2.4	97
19	2	3.5	90

The required tank size at each pluviographic station for each of these reference points is shown in Table C.5.3. Initially, the required tank size was plotted against MAR using a demand of 2.4 people with a 1 kL tank for 100m^2 of roof area (reference point no. 14) for all 45 pluviographic stations (Figure C.5.4). Although there is a definite negative correlation between required tank size andMAR,there is a ‘large spread’ in the data. The dotted lines in Figure C.5.4 show a range that is about 40% from the line of best fit.

Table C.5.3 Required tank sizes for reference station across Victoria

Melbourne	*Occupancy:* [1]	*3.5*	*2.4*	*1.5*	*3.5*	*2.4*	*1.5*	*3.5*	*2.4*	*1.5*	*3.5*	*2.4*	*1.5*	*3.5*	*2.4*	*1.5*	*3.5*
	Reliability (%):	*42*	*48*	*58*	*53*	*62*	*77*	*57*	*67*	*82*	*67*	*80*	*90*	*75*	*87*	*96*	*85*
	Tank volume: [2]	0.20	0.20	0.20	0.40	0.40	0.40	0.50	0.50	0.50	0.75	0.75	0.75	1.00	1.00	1.00	1.50
		Equivalent Tank Size [2] (% roof area) for same reliability as Melbourne															
Northern Region																	
Mildura		1.03	0.79	0.67	1.68	1.38	1.50	2.02	1.70	1.50	3.25	2.55	2.10	4.90	4.00	2.80	
Hopetoun		0.82	0.64	0.54	1.37	1.23	1.25	1.68	1.65	1.70	3.30	3.70	3.70	6.20	6.65	5.80	
Kerang		0.65	0.57	0.52	1.08	0.98	1.05	1.25	1.21	1.25	2.02	2.10	2.60	3.50	3.70	3.30	
Cobram		0.67	0.58	0.52	1.05	0.96	1.00	1.20	1.15	1.15	1.80	1.85	1.80	2.55	2.60	2.10	
Wodonga		0.58	0.46	0.45	0.75	0.71	0.73	0.84	0.85	0.82	1.23	1.35	1.23	1.60	1.77	1.35	
Tatura		0.76	0.58	0.53	1.04	0.95	1.05	1.15	1.18	1.25	1.90	2.00	2.05	2.90	2.90	2.30	
Dookie		0.57	0.45	0.44	0.78	0.73	0.80	1.18	1.18	1.28	1.29	1.50	1.60	2.90	2.90	2.30	
Southen Region																	
Ararat								0.81	0.62	0.66				1.46	1.35	1.25	1.93
Ballarat								0.76	0.50	0.73				1.73	1.18	1.33	2.87
Weearrionah								0.10	0.20	0.25				0.35	0.50	0.45	0.60
Wyelangta								0.35	0.38	0.38				0.75	0.74	0.65	1.09
Noojee								0.23	0.29	0.36				0.65	0.69	0.85	0.88
Yallourn								0.35	0.38	0.42				0.68	0.78	0.80	0.84
East Tarwin								0.32	0.36	0.41				0.71	0.77	0.82	1.50
Bullengarook								0.43	0.42	0.49				0.87	0.95	1.10	1.40
Buchan								0.43	0.45	0.49				0.86	0.88	0.90	1.22
East Sale								0.65	0.62	0.57				1.28	1.25	1.05	1.88
Sarsfield East								0.63	0.62	0.67				1.16	1.25	4.00	1.82
Wroxham								0.46	0.46	0.46				0.90	0.90	0.90	1.32
Genoa								0.39	0.40	0.43				0.86	0.90	0.85	1.22
Combienbar								0.54	0.53	0.53				1.05	1.02	1.05	1.42
Geelong								0.58	0.57	0.62				1.32	1.41	1.50	2.30
Little River								0.72	0.66	0.66				1.66	1.45	1.30	3.05
Melbourne								0.54	0.53	0.55				1.10	1.08	1.15	1.62
Laverton								0.60	0.56	0.60				1.33	1.25	1.25	2.10
Melton								0.82	0.71	0.72				2.07	1.63	1.65	3.17
Bundoora								0.47	0.47	0.49				1.00	1.05	1.05	1.35
Essendon								0.57	0.56	0.59				1.19	1.24	1.30	1.95
Upwey								0.90	0.33	0.38				0.65	0.78	0.30	1.00
Croydon								0.22	0.40	0.35				0.60	0.94	0.70	0.81
Narre Warren								0.40	0.43	0.48				0.81	0.90	1.20	1.25
Carrum								0.37	0.40	0.44				0.85	0.94	1.10	1.25
Koo Wee Rup								0.40	0.44	0.46				0.77	0.84	0.93	1.15
Central																	
Dartmouth								0.57	0.57	0.65				1.11	1.18	1.32	1.55
Bright								0.57	0.51	0.64				1.11	0.97	1.30	1.28
Omeo								0.75	0.70	0.70				1.68	1.55	2.00	2.53
Wartook								0.49	0.54	0.65				1.07	1.20	1.40	1.58
Horsham								0.78	0.75	0.80				1.60	1.56	1.60	2.55
Tottington								0.88	0.83	0.84				1.88	1.81	1.80	3.00
Bendigo								0.74	0.82	0.76				1.47	1.50	1.55	2.40

[1] people per 100m ^{2} roof

[2] percent of roof area (assuming tank is 1m deep)

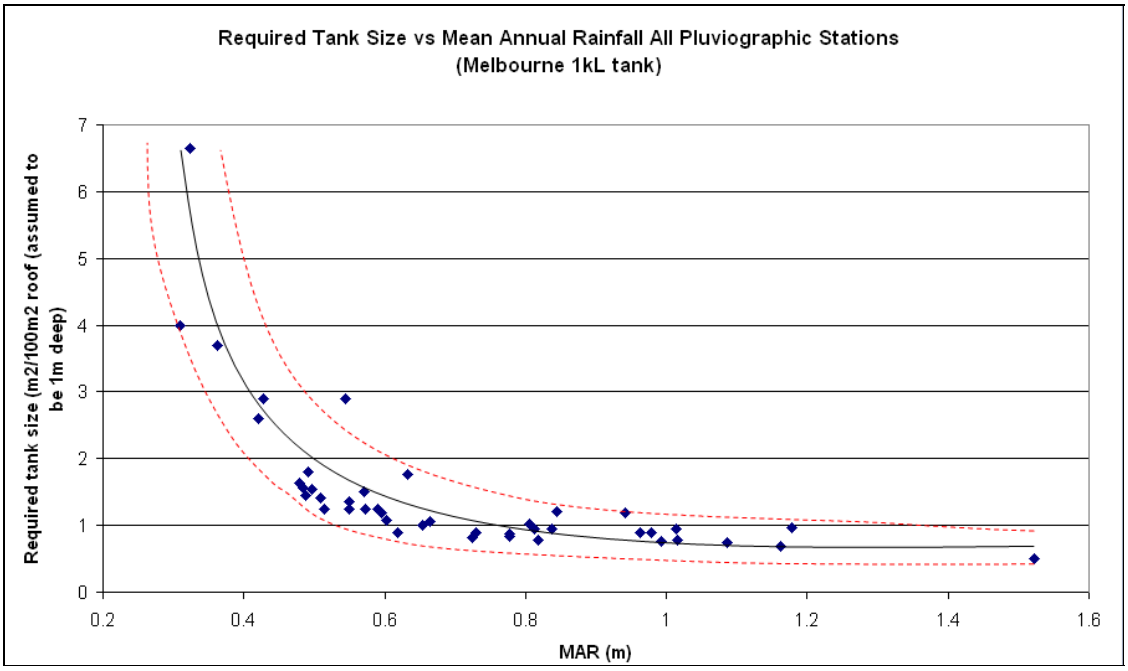

Figure C.5.4 Chart showing required tank size for all pluviographic stations (2.4 people/100 m2 roof; tank 1% catchment area).

An assessment was subsequently undertaken to see if a better correlation could be achieved using the nine regions derived for sizing stormwater treatment devices (see Appendix A).

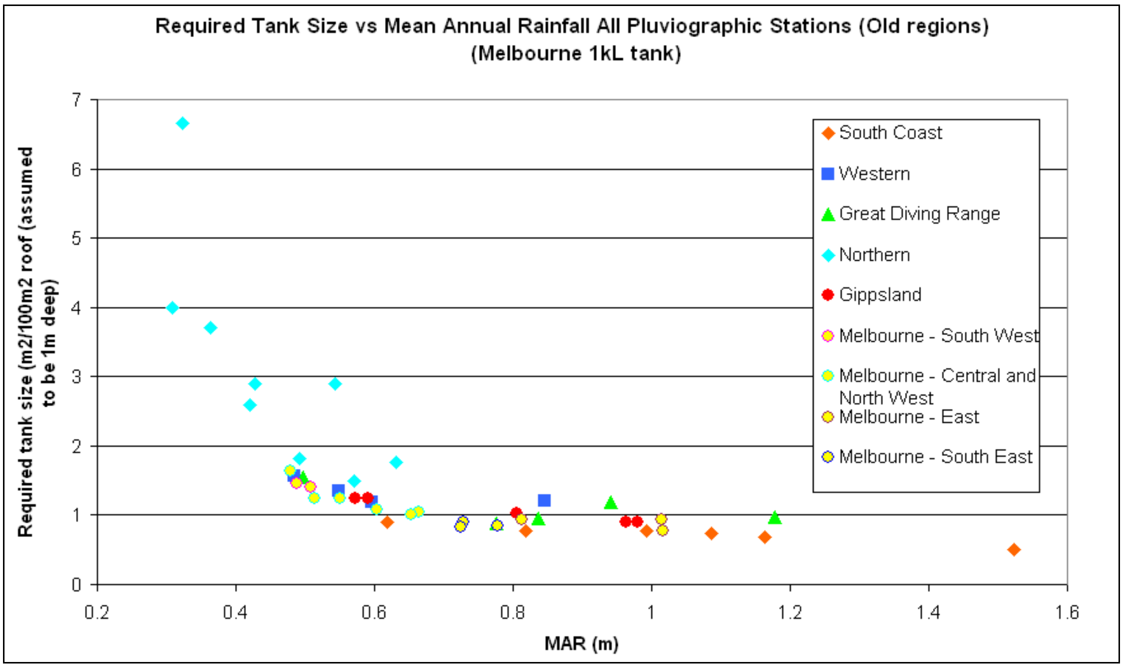

Figure C.5.5 Chart showing required tank size for all stations divided into the hydrologic regions used for sizing stormwater treatment devices.

There appears to be a better correlation within each of the nine regions in Figure C.5.5 than within the entire data set. It was determined, however, that a better correlation can be achieved using different tank sizing regions to those used for treatment device sizing

(Appendix A). Three tank sizing regions were defined: northern, central and southern (Figure C.5.69). Boundaries of the tank sizing regions were determined to represent the results of the analysis and to be aligned so that they do not dissect major urban areas (Figure C.5.7). The pluviographic stations within each hydrologic region are shown in Table C.5.5.4.

The difference between the regions used for sizing rainwater tanks andthose used for sizing stormwater treatment devices may be attributed to the different aspects of rainfall distribution that are relevant to the two applications. The performance of stormwater treatment devices depends on a finer time scale than rainwater tanks.

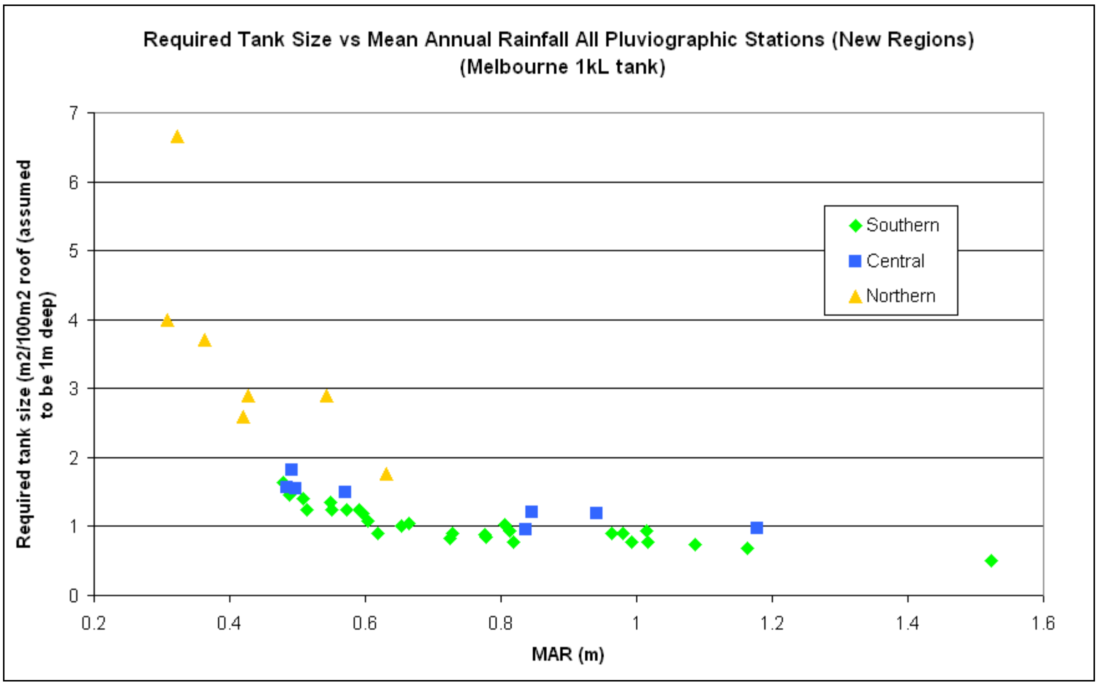

Figure C.5.6 Chart showing required tank size using three hydrologic regions within Victoria.

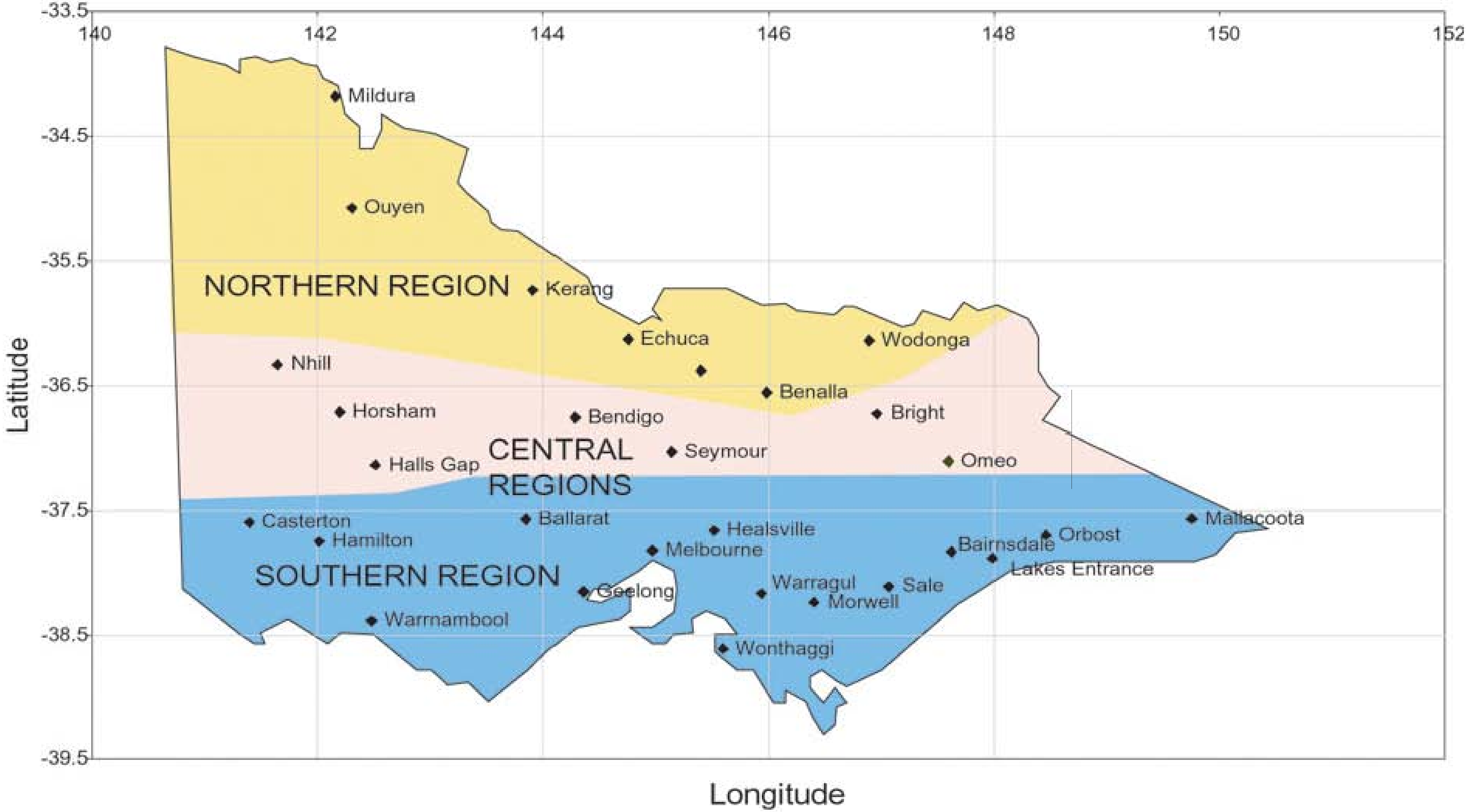

Figure C.5.7 Victorian rainwater tank sizing regions.

Table C.5.4 Pluviographic stations within each hydrologic region

Region	Stations
Northern	Mildura Hopetoun Kerang Cobram Hume Reservoir (Wodonga) Tatura Dookie
Central	Horsham Wartook Reservoir Tottington Darmouth Reservoir Bright Omeo Bendigo

Region	Stations	
Southern	Ararat Ballarat Bullengarook Buchan Casterton Mortlake Weearproinah Wyelangta Noojee Yallourn East Tarwin East Sale Sarsfield East Combienbar Wroxham Genoa	Geelong North Little River Werribee Melbourne Airport Laverton Melton Bundoora Essendon Airport Upwey Croydon Narre Warren North Carrum Dandenong Koo Wee Rup Melbourne

C.6 Determining tank sizing curves

C.6.1 Northern region

For the seven pluviographic stations within the northern region, the tank size required to achieve the same reliability as the reference points described in Section C.5 were plotted against MAR. A curve could be plotted through all the data points relating to a each Melbourne tank size (see Table C.6.1 and Figure C.6.1).

Table C.6.1 Table showing which reference points make up each curve on Figure C.6.1

No.	Equivalent Melbourne tank size (kL)	Demand (people/100 m^2 roof)	Reliability
CURVE 1			
1	0.2	1.5	58
2	0.2	2.4	48
3	0.2	3.5	42
CURVE 2			
4	0.4	1.5	77
5	0.4	2.4	62
6	0.4	3.5	53
CURVE 3			
7	0.5	1.5	82
8	0.5	2.4	67
9	0.5	3.5	57
CURVE 4			
10	0.75	1.5	90
11	0.75	2.4	80
12	0.75	3.5	67
CURVE 5			
13	1	1.5	94
14	1	2.4	87
15	1	3.5	75

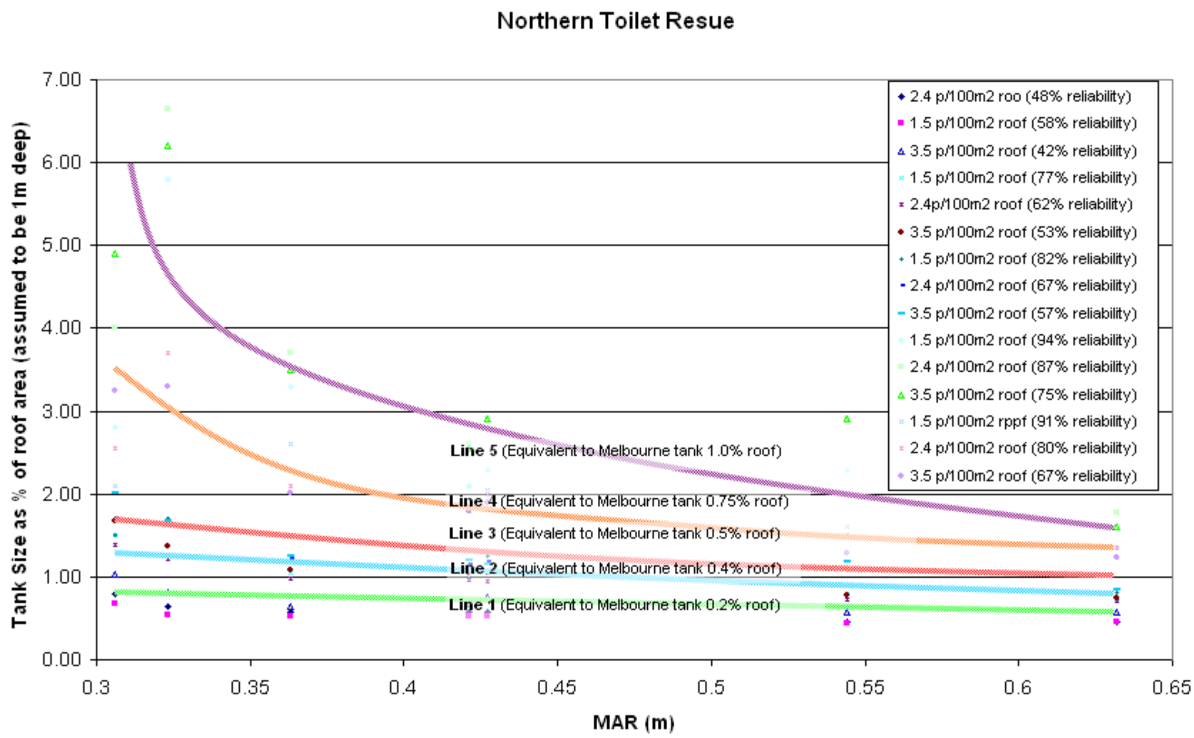

Figure C.6.1 Northern tank sizing region chart.

C.6.2 Central region

For the seven pluviographic stations within the central region, the tank size required to achieve the same reliability as the reference points described in Section 5 were plotted against MAR. A curve could be plotted through all the data points relating to each Melbourne tank size (Table C.6.2 and Figure C.6.2).

Table C.6.2 Table showing which reference points make up each curve on Figure C.6.2

No.	Equivalent Melbourne tank size (kL)	Demand (people/100m^2 roof)	Reliability
CURVE 1			
7	0.5	1.5	82
8	0.5	2.4	67
9	0.5	3.5	57
CURVE 2			
13	1	1.5	94
14	1	2.4	87
15	1	3.5	75
CURVE 3			
16	1.5	2.4	93
17	1.5	3.5	84
CURVE 4			
18	2	2.4	97
19	2	3.5	90

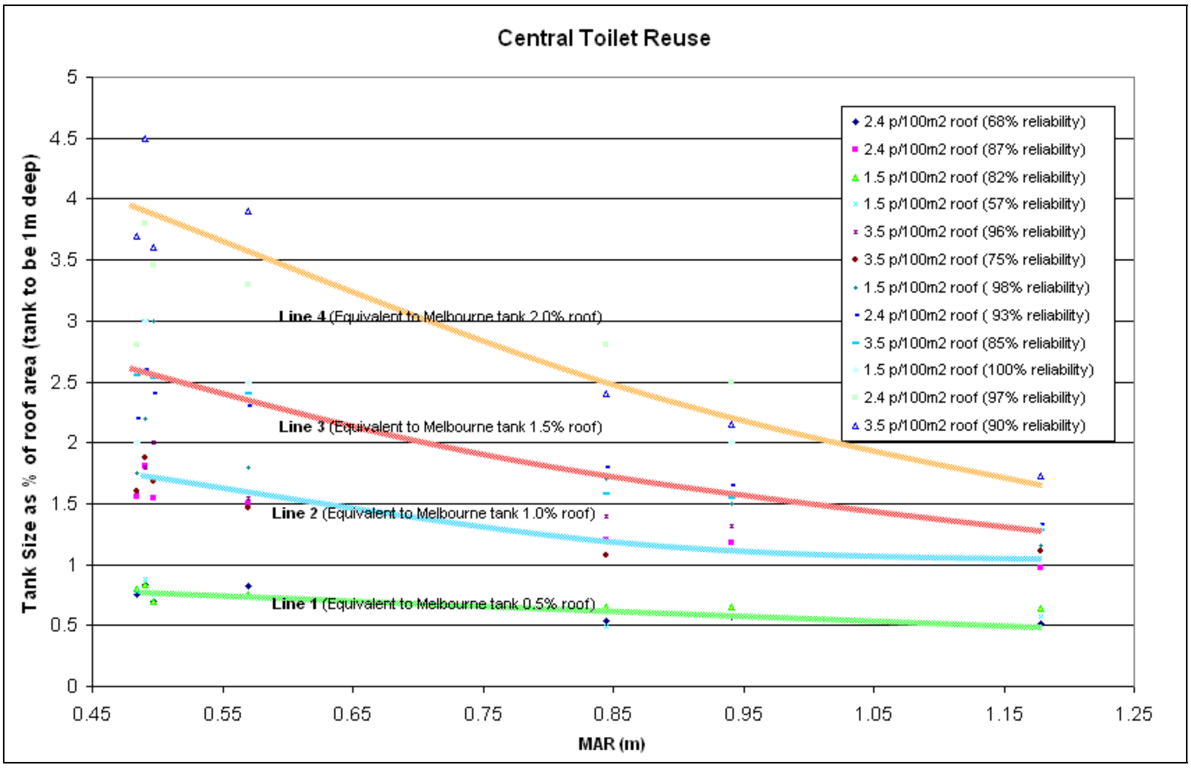

Figure C.6.2 Central tank sizing region chart.

C.6.3 Southern region

For the 31 pluviographic stations within the southern region, the tank size required to achieve the same reliabilities as the reference points described in Section C.5 were plotted againstMAR. A curve could be plotted through all the data points relating to a each Melbourne tank size (Table C.6.3 and Figure C.6.3).

Table C.6.3 Table showing which reference points that make up each curve on Figure C.6.3

No.	Equivalent Melbourne tank size (kL)	Demand (people/100 m^2 roof)	Reliability
CURVE 1			
7	0.5	1.5	82
8	0.5	2.4	67
9	0.5	3.5	57
CURVE 2			
13	1	1.5	94
14	1	2.4	87
15	1	3.5	75
CURVE 3			
16	1.5	2.4	93
17	1.5	3.5	84
CURVE 4			
18	2	2.4	97
19	2	3.5	90

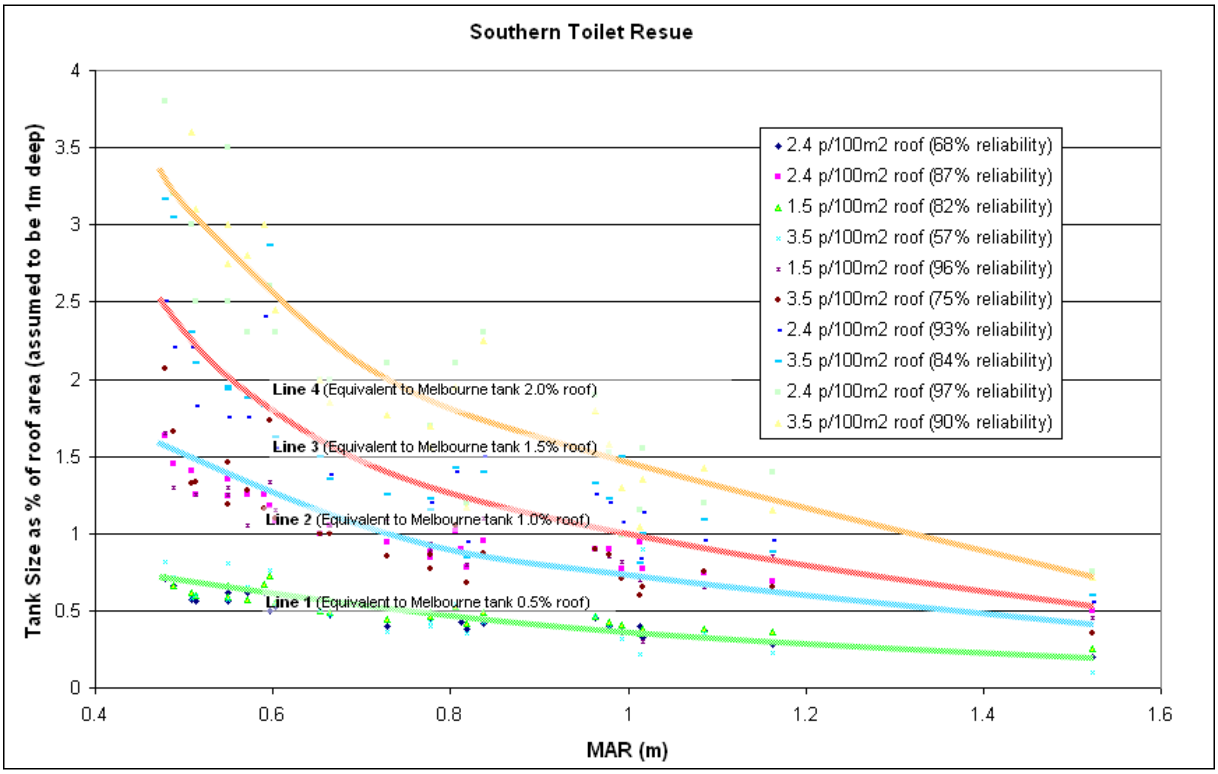

Figure C.6.3 Southern tank sizing region chart.

C.7 Recommended tank sizing curves

The curves in Figures C.7.1 to C.7.3 are the tank sizing curves recommended for the three regions in Victoria (as defined in Figure C.5.7) and used in Chapter 12 of this Manual.

C.7.1 Northern region

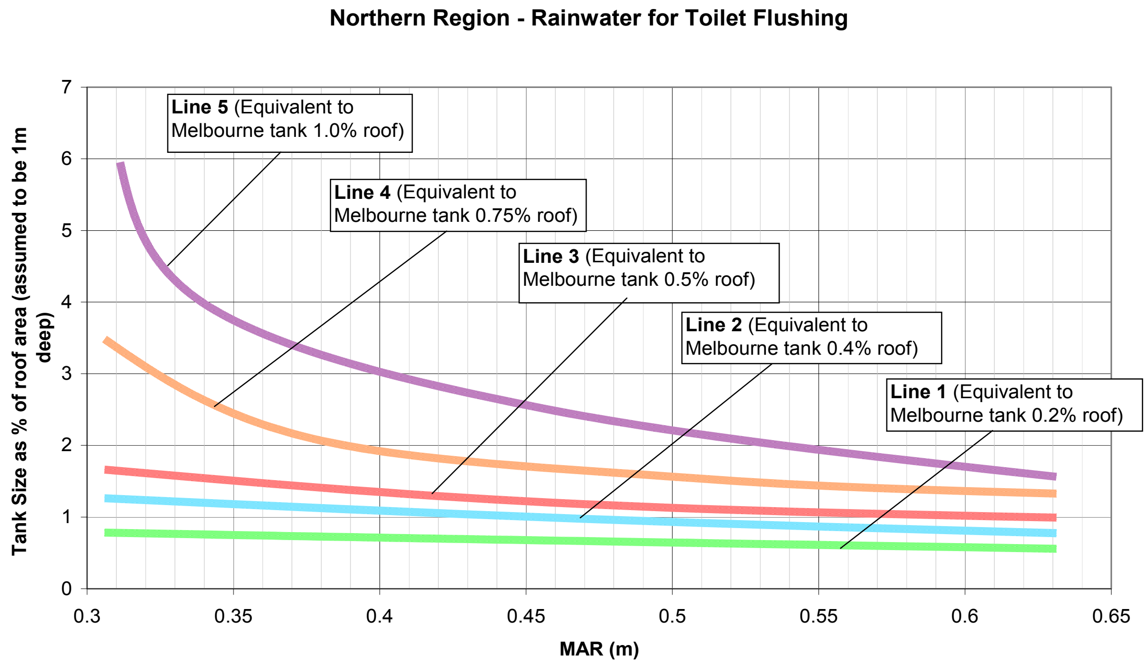

Figure C.7.1 Northern region tank sizing curves.

C.7.2 Central region

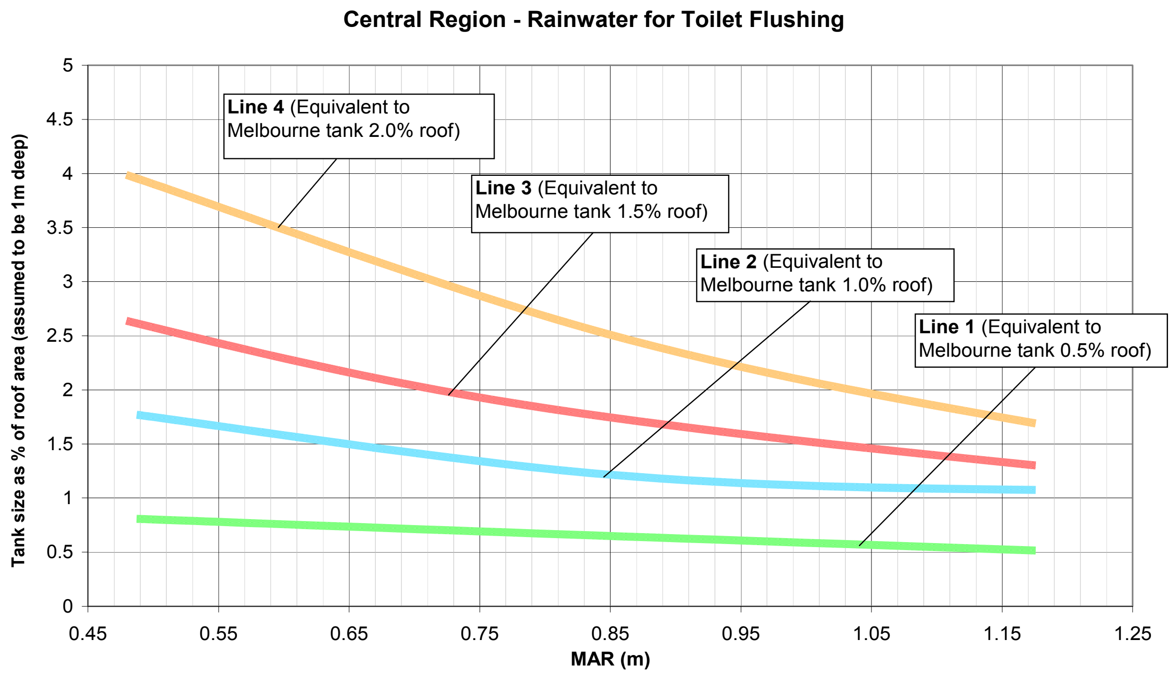

Figure C.7.2 Central region tank sizing curves.

C.7.3 Southern region

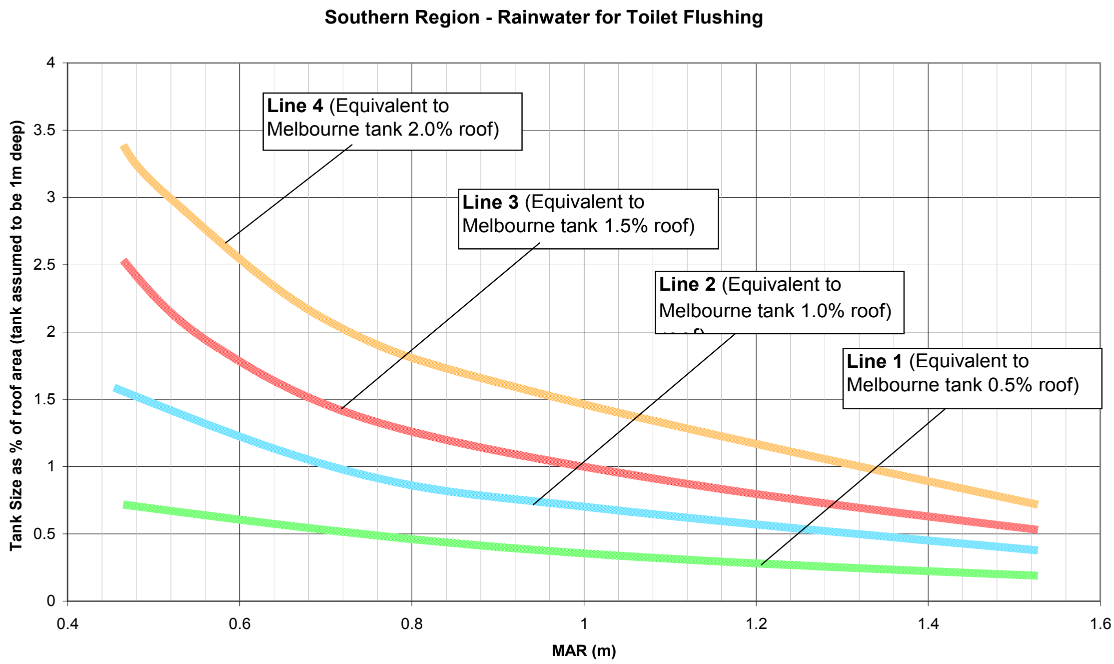

Figure C.7.3 Southern region tank sizing curves.

C.8 Example of the use of tank sizing equations

A family of three who live near Warrnambool Airport wish to collect rainwater to supplement at least 80% of their toilet flushing.

The MAR at Warrnambool Airport is 903 mm (i.e. 0.903 m). The area of roof available to collect water is about 125 m^2 (the occupancy density is therefore 2.4 people/100 m^2 roof).

1 Determine what size tank at the reference site will achieve an 80% saving in toilet water used (i.e. 80% reliability). If the following inputs are applied to the curve in Figure C.4.1 at:

- 2.4 people/100 m^2 of roof
- 80% reliability

interpolation from the curve gives a required tank size of 0.7% of the roof area (and 1 m deep). Therefore, a tank size of $0.007 \times 125 \times 1.0 = 0.9$ kL is required for the reference site.

2 Adjust the required tank size from the reference site to Warrnambool. If the values in Figure C.7.3 (Southern region) are applied with a tank size of 0.7% of roof area and MAR of 0.90 m, interpolation between Lines 1 and 2 with a rainfall of 0.90 m gives a tank size of 0.6% of roof area (1 m deep).

Therefore, required tank size = $0.006 \times 125 \times 1.0 = 0.75$ kL.

By way of comparison, if the family were in Horsham (450 mm MAR) the required tank size is calculated using Figure C.7.2 and is equivalent to 1.25% of roof area (1 m deep) which equates to a tank size of 1.6 kL.

C.9 Summary

A simple procedure for sizing rainwater tanks is proposed here. This procedure is based on defining three tank sizing regions within Victoria. More details of its application are presented in Chapter 12 of this Manual..

Three regional curves for estimating tank sizes are the result of pooling modelling results for relevant reference pluviographic stations (45 stations). To ensure a systematic application of the procedures, estimates of tank sizes should exclusively use the regional curves provided rather than values derived from a single station, irrespective of the proximity of the site in question to a reference pluviographic station. This would avoid situations where practitioners get to choose between the adjustment factor computed from the regional approach and that derived for the reference pluviographic station of close proximity to the site in question.

C.10 References

Coomes Consulting Group. (2002). Integrated Water Management Epping North Report. Report for Urban and Regional Land Corporation

Water Resources Strategy Committee. (2001). Water Resources Strategy for the Melbourne Area, 2001, Discussion Starter: Stage 1 in Developing a Water Resource Strategy for the Greater Melbourne Area.

Appendix D Cyanobacterial growth in constructed water bodies

Contents

D.1 Introduction

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

D.2 Factors influencing growth

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

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

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

D.2.1 Light

In Australian climatic conditions surface light is rarely a limiting factor for algal growth. Cyanobacterial responses to various light conditions differ between species. Turbidity and mixing conditions within a waterbody can determine the light environment that algal cells are exposed to by circulating them in and out of the euphotic zone. Typically, cyanobacterial growth rates are reduced under fluctuating light conditions such as those found in well-mixed water columns (Mitrovic et al. 2003).

Some cyanobacterial species can regulate cell buoyancy and migrate vertically, increasing their exposure to optimum light intensities. Cell buoyancy regulation offers cyanobacteria considerable advantage over other phytoplankton that are distributed evenly throughout the water column (Mitrovic et al. 2001). However, this buoyancy advantage depends on the mixing regime and degree of turbulence that the cells are exposed to within the water column (Brookes et al. 2003).

Depth of light penetration can be reduced by turbidity and therefore limit biomass development. The extent to which turbidity will reduce light availability to cells depends on the mixing patterns of the waterbody and the degree of cell buoyancy regulation.

D.2.2 Temperature

Temperature is an important factor in many cyanobacterial blooms in Australia. In temperate zones cyanobacterial blooms commonly occur in the warmer months. Cyanobacteria tend to have high optimal growth temperatures compared to green algae and diatoms and achieve maximum growth rates at around 25°C (Chorus and Bartram 1999).

D.2.3 Nutrients

Many cyanobacterial blooms are associated with elevated nutrient levels. However, nutrient availability in many aquatic environments is generally adequate to achieve cyanobacterial growth of bloom proportions when other factors such as temperature and hydrodynamics are also favourable. Many of the nuisance species of cyanobacteria are capable of fixing atmospheric nitrogen; however, this process requires considerable amounts of energy and may be limited in turbid environments (Chorus and Bartram 1999).

D.2.4 Hydrodynamics

A key parameter of aquatic ecosystems is hydraulic detention time (Harris 1996; Jorgensen 2003). Long detention times during warm weather in poorly mixed water bodies often leads to persistent stratification of the water column. Periods of stratification of a water body can also facilitate the release of nutrients from the sediments which can act to support algal growth. In lowland rivers and lakes, cyanobacterial blooms are more prevalent during periods of persistent stratification, a condition associated with low flows (Sherman et al. 1998). Cyanobacterial species that can regulate their buoyancy, and migrate vertically through the water column, have a competitive advantage over other phytoplankton under stratified conditions (Atlas and Bartha 1998). Buoyancy regulation allows cell movement between the nutrient-rich hypolimnetic waters and the euphotic zone so as to access both high nutrient and optimal light conditions.

In deep water bodies, hydraulic mixing and the breakdown of stratification can slow the growth of cyanobacteria and reduce the prevalence of excessive growth. Hydraulic mixing reduces growth rates by circulating cells below the euphotic zone for long enough to limit light availability, reducing carbohydrate accumulation and exhausting the energy supply required for growth and replication (Brookes et al. 2003).

In shallow water bodies, where the ratio of mixing depth to euphotic zone depth is < 3–5, mixing is typically insufficient to reduce growth (Oliver et al. 1999). Under such conditions, hydraulic detention time becomes a crucial factor in the control and prevention of excessive algal growth. When the hydraulic detention time is reduced the biomass becomes regulated by the rate at which it is removed from the lake by flushing (Reynolds 2003).

D.3 Growth rates

A model of algal growth can be developed using a simple relationship between time and growth rate at various temperatures, assuming adequate light and nutrient availability. The exponential growth rate equation is:

$$\mu = (1/t) \times \ln(N_t/N_0)$$

where μ = the growth rate per day
t = the number of days
N_t = final cell concentration
N_0 = the starting cell concentration.

This simple model can be used to determine how long it will take for an algal population to reach bloom proportions (15 000 cells/mL) and hence inform the development of guidelines on water body hydraulic detention time.

D.3.1 Common growth rate range

Under favourable growth conditions (20°C and light saturation) laboratory cultures of planktonic cyanobacteria have growth rates of between 0.21/day and 0.99/day, or 0.3 to 1.4 doublings per day, respectively (Chorus and Bartram 1999). Figure D.1 illustrates theoretical growth curves based on growth rates of laboratory grown cultures that have been adjusted to account for a slower growth rate (0.5 normal growth rate) at night (12 out of 24 h). The graphs are indicative of the range of growth rates both between species and between individual populations of the same species grown in laboratory cultures.

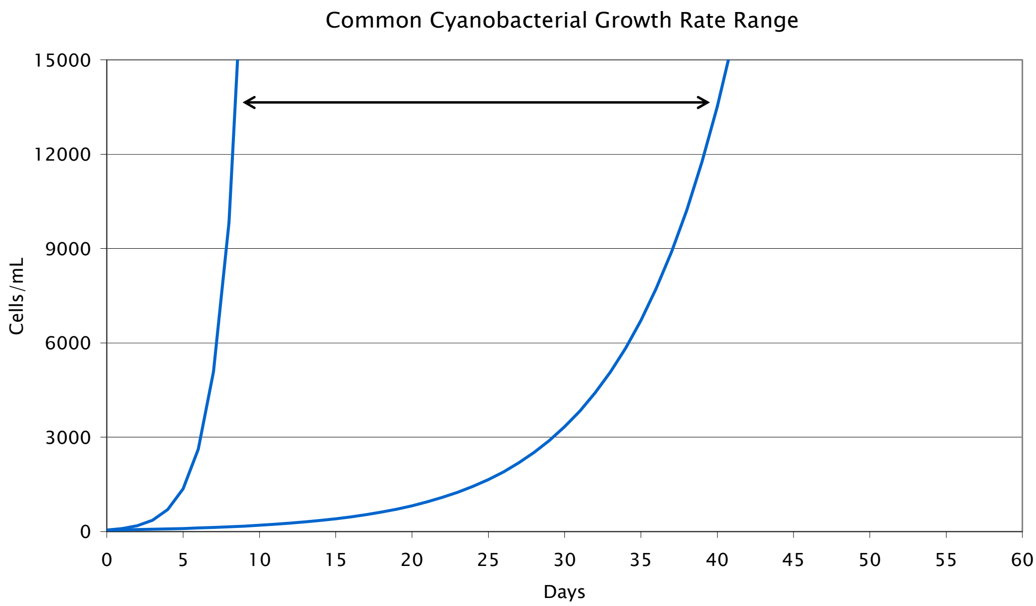

Figure D.3.1 The range of common cyanobacterial growth rates illustrated using theoretical growth curves based on growth rates of laboratory grown cultures (20°C and light saturation) adjusted for a 12 h:12 h light–dark cycle. Growth curves were constructed using an initial algal cell concentration of 50 cells/mL.

These results illustrate the wide range of growths rates that have been recorded for cyanobacteria and suggest that, under ideal conditions at 20°C, laboratory cultured cyanobacteria can achieve bloom conditions in 9–41 days depending on the species.

D.3.2 Laboratory cultures versus *in situ* growth rates

Physiological characteristics such as maximum photosynthetic capabilities, photoinhibition levels and flotation rates (speeds of vertical movement) vary considerably between cyanobacterial species and between individual populations within species. Growth rates also decrease with increasing cell or colony sizes (Reynolds 1984). Environmental variables, such as those discussed earlier, influence which species will dominate and the maximum growth rate. Typically, slower *in situ* growth rates occur as a result of these environmental variables. The relationship between laboratory growth rates and *in situ* growth rates is poorly understood. For example, *Microcystis* rarely grows in colonial form when grown in laboratory cultures; however, successful growth of colonies in culture have shown much slower growth rates than those recorded previously from unicellular cultures (Reynolds 1984). As a result, *in situ* growth rates are more desirable to use in models attempting to predict *in situ* conditions.

D.3.3 Mixing conditions

Westwood and Ganf (2004) measured the *in situ* growth of *Anabaena circinalis* in the Murray River at Morgan, Victoria (Table D.1). Growth was measured under well mixed and persistently stratified conditions and also under conditions that take into account a range of typical flotation velocities (or mixing conditions) recorded for *A. circinalis* populations (0.01–0.40 m/h).

Table D.3.1 *In situ* growth rates for *Anabaena circinalis* under various mixing conditions. From Westwood and Ganf (2004)

Hydrodynamic treatment	Growth rates per day
Persistent stratification	0.43
1.0 m/h mixing rate (diurnal stratification)	0.23
0.5 m/h mixing rate (diurnal stratification)	0.15
Well mixed	0.19

Figure D.3.1 has been constructed based on the *in situ* growth rates of *A. circinalis* recorded by Westwood and Ganf (2004). With starting cell concentrations of 50 cells/mL, the measured growth rates of neutrally buoyant populations under well-mixed conditions suggested the population would take about 31 days to reach bloom proportions. Under persistently stratified conditions, bloom proportions would be reached within 14 days. Populations of *A. circinalis* with flotation velocities of 0.5 m/h and 1.0 m/h[1], and under diurnally stratified conditions, would take longer than 25 days to reach bloom proportions.

Waterbodies incorporating best practice design features are assumed to be relatively shallow (< 2.5–3.0 m), have a flat bottom and be subject to wind mixing. These design features are assumed to prevent persistent stratification and create systems that are well mixed or only diurnally stratified. Where diurnal stratification occurs, mixing rates during the non-stratified period are expected to be relatively fast due to the shallow depth of the water body. As a result, *in situ* growth rates for a fully mixed system and *in situ* growth rates for a partially mixed system with a relatively fast mixing rate have been adopted. Figure D.3.1 shows the expected mixing conditions for waterbodies that incorporate the features of best management practice design.

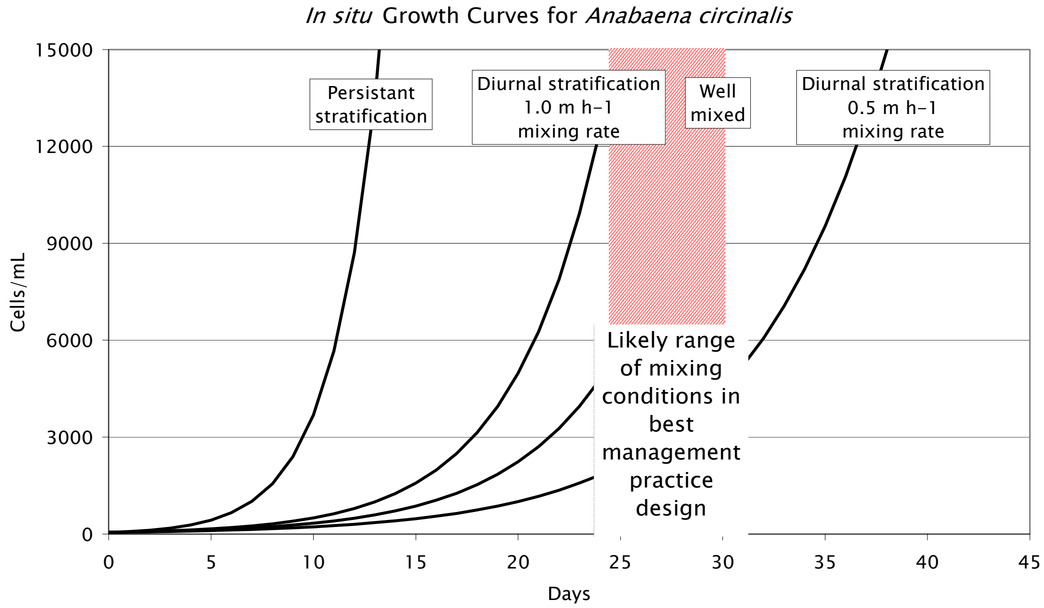

Figure D.3.2 Growth of *Anabaena circinalis* under various mixing conditions illustrated using growth curves constructed from data collected *in situ* (Westwood and Ganf 2004) and assuming starting cell concentrations of 50 cells/mL. Area of shading represents the range of mixing conditions likely to be found in best practice design systems.

D.3.4 Temperature effects

Provided that other factors (e.g. light, nutrients) remain non-limiting, maximum growth rates of cyanobacteria respond directly to changes in temperature. Specific responses to temperature changes differ between species but, typically, growth rates increase with increasing temperature (Reynolds 1984). The effect of temperature can be accounted for by adjusting growth rates using a temperature coefficient that represents the change over 10°C (Q_{10} values). Data presented in Table D.3.1 indicate that Q_{10} values can vary significantly between species.

Table D.3.1 Q_{10} values for a range of cyanobacteria species. Q_{10} is the temperature coefficient (Q_v) that represents the increase in growth rate that occurs with a 10°C increase in temperature

Genus	Q_{10} range	Temperature range (°C)	Reference
Asterionella, *Anabaena*, *Aphanizomenon* and *Oscillatoria*	1.8–2.9	10–20	Reynolds (1984)
Microcystis, *Merismopedia* and *Oscillatoria*	1.97–4.16	15–25	Coles and Jones (2000)

D.3.5 Starting concentration

The theoretical growth rate curves are constructed using initial cell counts of 2 cells/mL and 50 cells/mL which represent typical natural background levels. Webster et al. (2000) found blooms in the

Maude Weir pool forming from initial concentrations of 10 cells/mL. It is clear that the initial starting concentration can influence the time required to reach bloom proportions (although the degree of influence will be depend on the growth rate). For instance, for *A. circinalis* in well-mixed conditions and 20°C, starting concentrations of 2 cells/mL and 50 cells/mL result in bloom proportions of 15 000 cells/mL after about 33 and 51 days, respectively (Figure D.3.2).

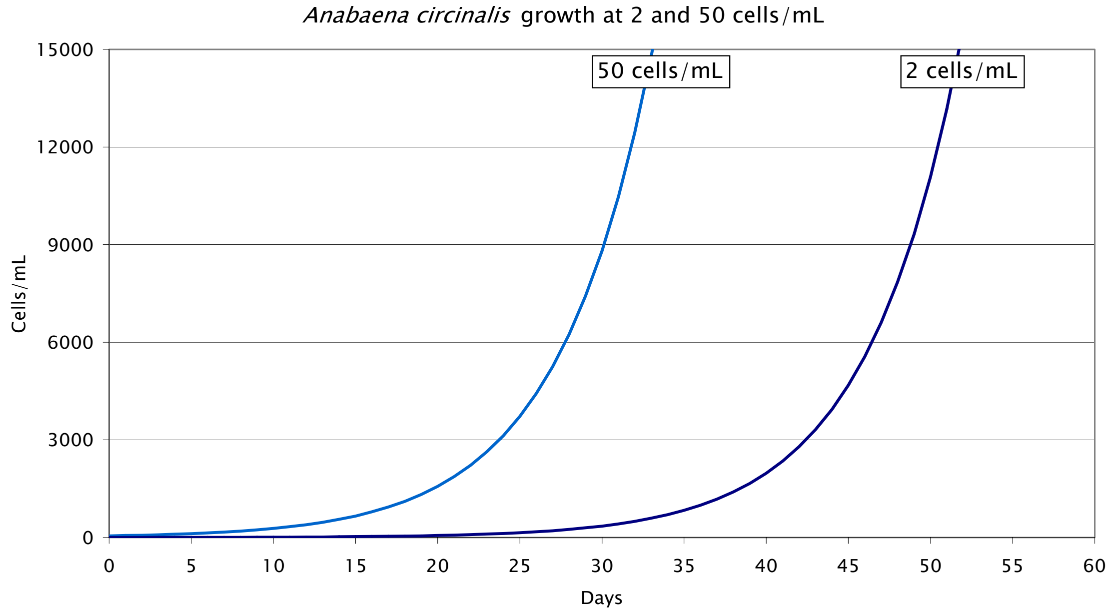

Figure D.3.3 Cyanobacterial growth curves at starting concentrations of 2 cells/mL and 50 cells/mL constructed using growth rates of *Anabaena circinalis* measured *in situ*, under well-mixed conditions (Westwood and Ganf 2004), adjusted for 20°C (Q_{10} 2.9). The number of days taken to reach bloom proportions varies from 33 to 51 days depending on the starting cell concentration.

D.3.6 Detention time

Reynolds (2003) recommends that the sensitivity of lakes to eutrophication, in relation to changes in external phosphorus loads, can be classified according to hydraulic detention time. Short detention times weaken the response of lakes to changes in external phosphorus loads. The weakened response of lakes to changes in phosphorus loads is due to the biomass becoming regulated by the rate at which it is removed from the lake by flushing, rather than the availability of phosphorus (Reynolds 2003). The most sensitive lakes are those with a detention time of greater than 30 days. Lakes with a detention time of 3 days to 30 days are only slightly sensitive to changes in external phosphorus loads, whereas lakes with a detention time of less than 3 days are insensitive to changes in phosphorus loads (Reynolds 2003).

In the Australian climate, designing constructed waterbodies with a detention time of less than 3 days is neither practical nor achievable. An upper limit of 30 days may be applied as a general precaution to ensure that waterbodies do not lie within the 'very sensitive' category of over 30 days detention time. Wagner-Lotkowska et al. (2004) recommend a hydraulic detention time of less than 30 days for the control of algal blooms in medium-sized reservoirs.

Wastewater treatment ponds could be viewed as ideal environments for algal growth (shallow, adequate light, high nutrients). However, experience has shown (e.g. Breen 1983) that it is rare to get cyanobacteria dominating the phytoplankton community in wastewater treatment ponds with detention times below 30 days.

D.3.7 Model parameters

The values presented in Table D.3.3 have been adopted to create a model appropriate for waterbodies with best management practice design. These systems are assumed to be shallow, have a flat bottom and are generally well mixed. A reasonable assumption is that the hydrodynamic conditions in a best management practice design varies somewhere between fully mixed and diurnally, partially mixed as represented by the shaded zone in Figure D.3.2.

Table D.3.3 Summary of model parameters

Variable	Value	Comment	Reference
Hydrodynamics	Well mixed to 1.0 m/h with diurnal stratification	Waterbodies incorporating best practice design are assumed to be relatively shallow, have a flat bottom and be easily mixed by wind. As a result, *in situ* growth rates for a fully mixed system and a partially mixed system with a relatively fast mixing rate have been adopted. From Figure D.3.2 this approach is considered conservative	Mixing values from Westwood and Ganf (2004)
Growth rate	0.19– 0.23/day	Adoption of *in situ* growth rate of a common nuisance cyanobacterial species (*Anabaena circinalis*) is considered reasonable given the frequency of *Anabaena* in blooms	Westwood and Ganf (2004)
Q_{10}	2.9	Adoption of the upper limit of the range of Q_{10} values recorded for various genera including *Anabaena* is considered a conservative assumption.	Reynolds (1984)
Temperature range	15–25°C	Likely temperature ranges of surface waters in Victoria	
Starting concentrations	50 cells/mL	Conservative, or likely upper limit, of background cell concentrations for cyanobacteria in waterbodies without chronic bloom problems	

D.3.8 Modelling results

The results of modelling are shown in Figures D.3.4 and D.3.5 for partially and well-mixed systems, respectively. The temperature ranges can be broadly interpreted in Victoria as follows:

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

The values represent summer water temperatures. Local water body temperature will clearly vary between sites within different years. Where local water temperature data are available they should be used to guide the selection of the critical detention time.

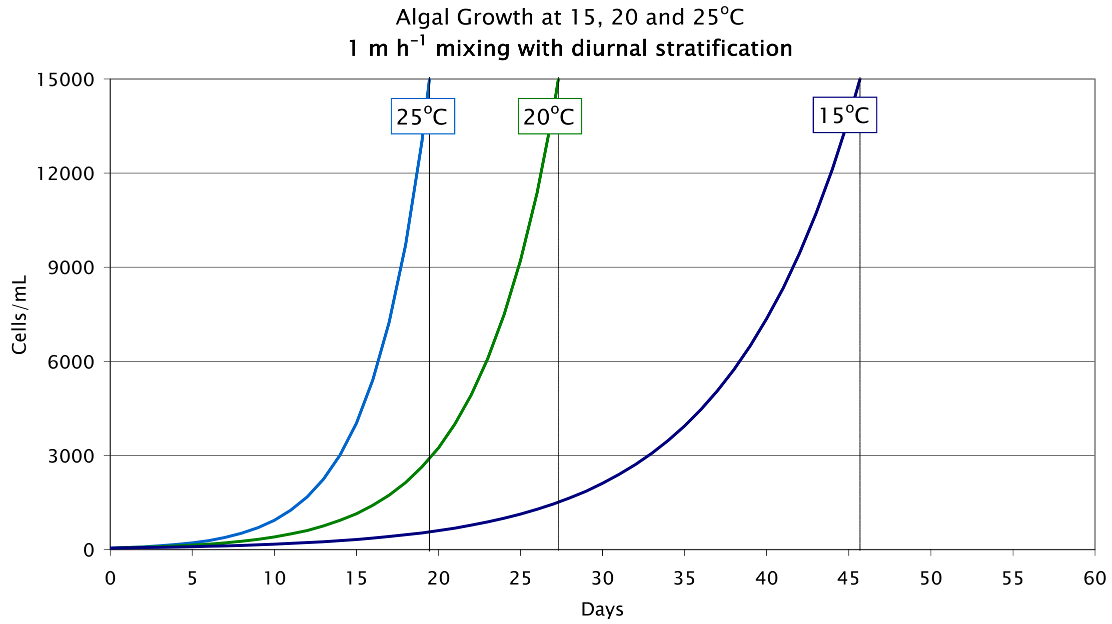

Figure D.3.4 Growth curves illustrating modelled times for cyanobacterial populations to reach bloom proportions under different temperature conditions and 1 m/h mixing conditions with diurnal stratification. Based on growth rates of *Anabaena circinalis* measured *in situ* (Westwood and Ganf 2004) adjusted for temperature, Q_{10} 2.9, and assuming 50 cells/mL starting concentrations.

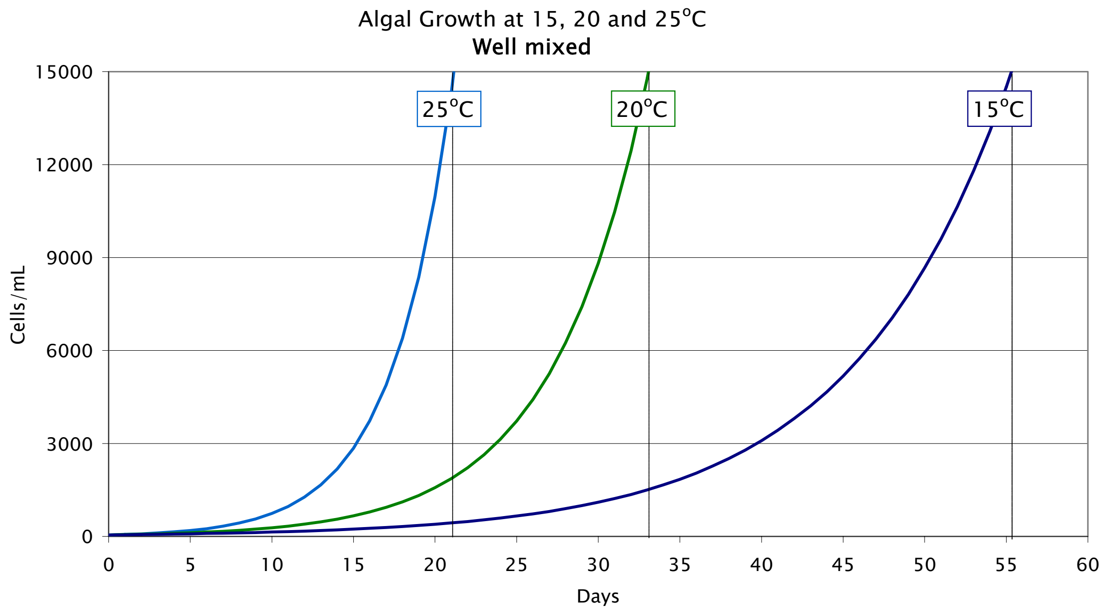

Figure D.3.5 Growth curves illustrating modelled times for cyanobacterial populations to reach bloom proportions under different temperature conditions and well-mixed conditions. Based on growth rates of *Anabaena circinalis* measured *in situ* (Westwood and Ganf 2004) adjusted for temperature, Q_{10} 2.9, and assuming 50 cells/mL starting concentrations.

Target detention times for the modelled temperature ranges are summarised in Table D.3.4 for both partially and well-mixed systems. The hydrodynamic state of waterbodies with best practice design would move between the proposed mixing conditions.

Table D.3.4 Modelled times for cyanobacterial populations to reach bloom proportions under different temperature conditions

Variables	Partially mixed			Fully mixed		
Temperature (°C)	15	20	25	15	20	25
Time (days)	46	27	19	55	33	21

The modelling approach taken in Table D.3.4 is considered to be reasonably conservative. For example, it adopts:

- non-limiting conditions for nutrient and light availability
- growth rates for a known nuisance species (*Anabaena circinalis*)
- summer temperature values (the main risk period)
- high starting population concentrations (50 cells/mL).

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

D.4 Recommended design criteria

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

- 50 days (15ºC)
- 30 days (20ºC)
- 20 days (25ºC).

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

D.5 Acknowledgements

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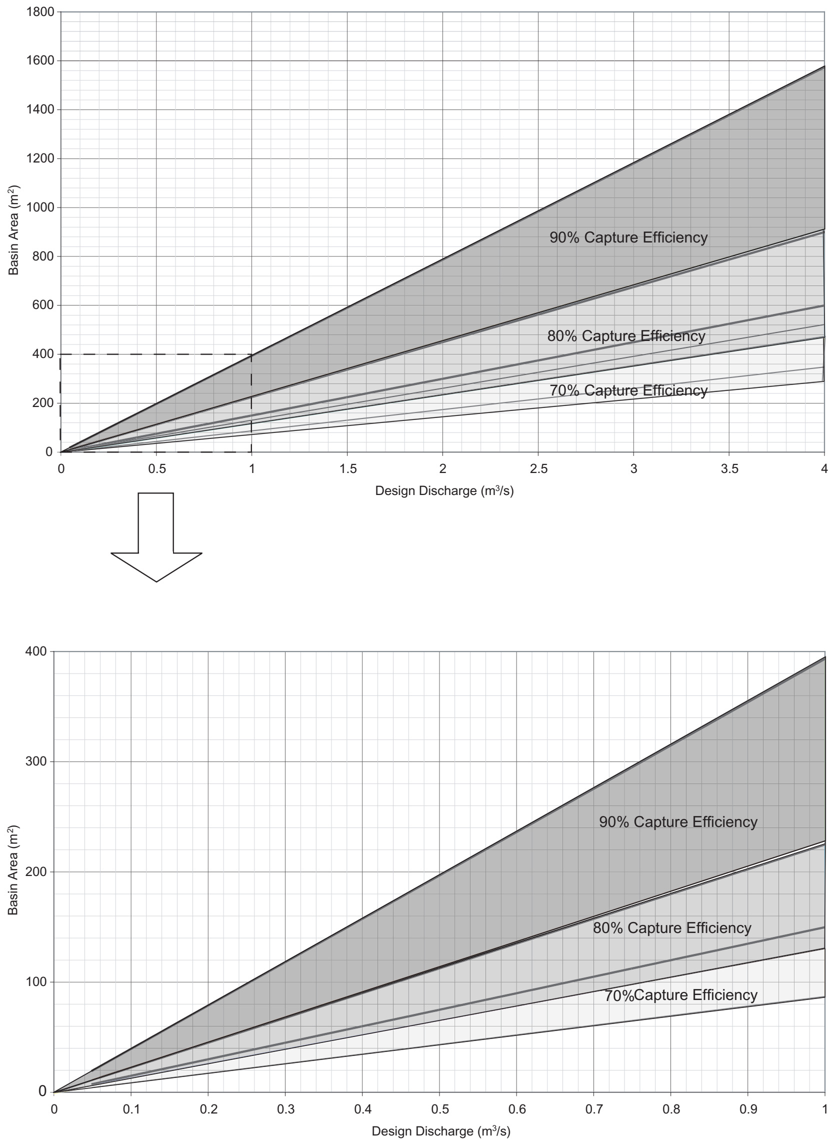

Figure 4.2 Sedimentation basin area versus design discharges for varying capture efficiencies of 125 μm sediment.

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

4.3 Design procedure: sedimentation basins

4.3.1 Estimating design flows

4.3.1.1 Design discharges

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

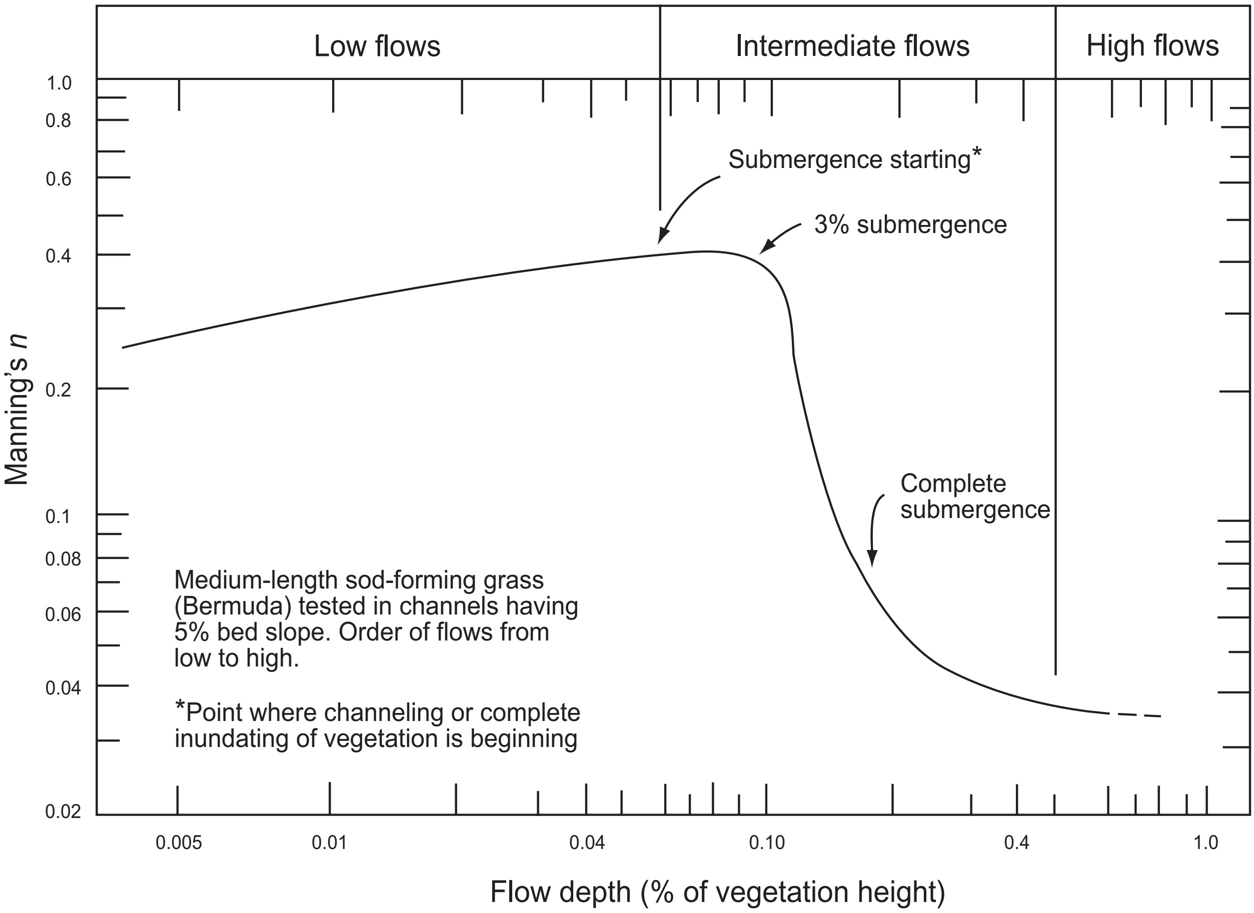

Figure 5.7 The effect of flow depth on hydraulic roughness (after Barling and Moore 1993).

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

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

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

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

5.3.2.2 Selection of Manning's *n*

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

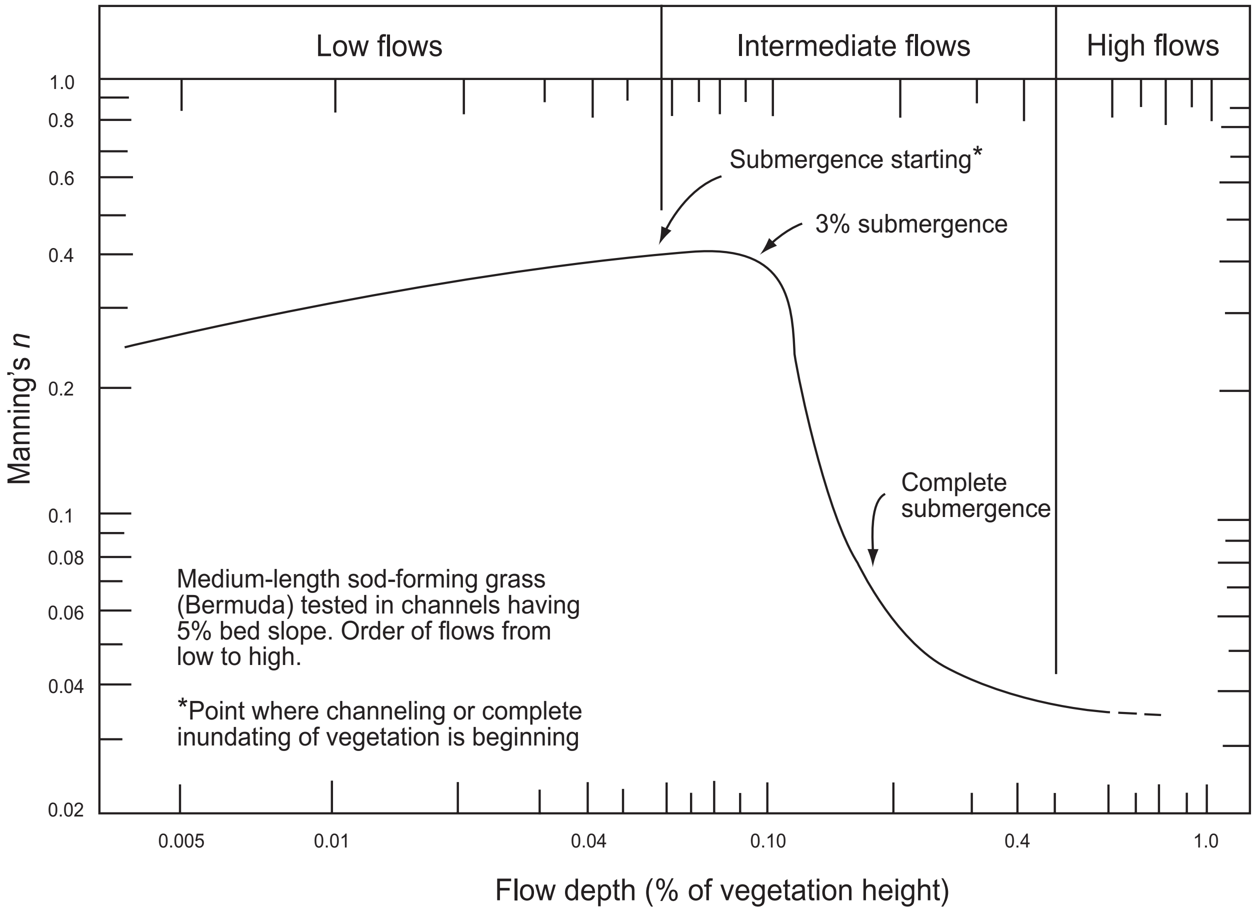

Figure 8.10 The effect of flow depth on hydraulic roughness (after Barling and Moore 1993).

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

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

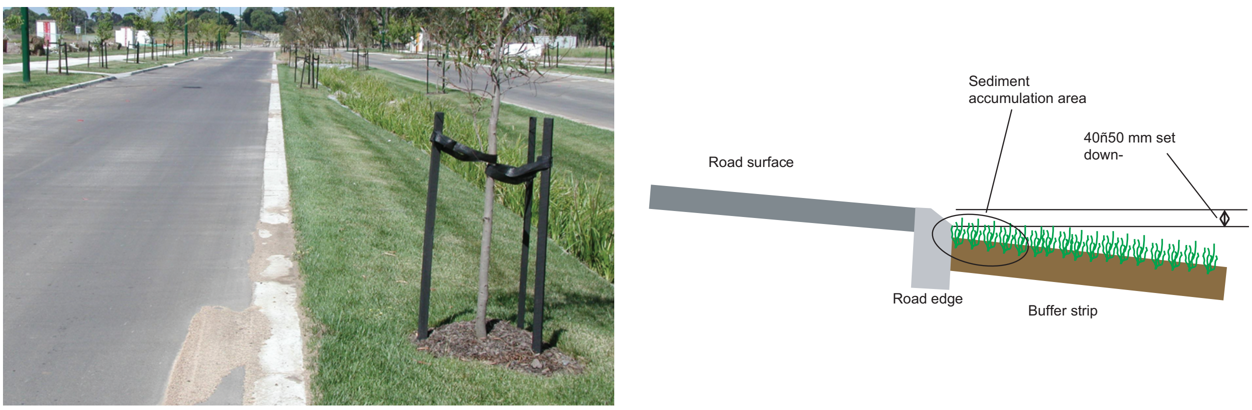

Figure 8.11 A flush kerb without setdown that shows accumulation of sediment on the street surface, and edge detail showing a recommended amount of setdown.