7. STORAGES FOR POLLUTION CONTROL (Category 2 systems)

7.1 GENERAL BACKGROUND

Perhaps the pollution control or filtration system which the general public most easily recognises in the urban landscape is the trash rack, GPT (gross pollution trap), or sediment (collection) basin constructed in a local drainage path or waterway. Such installations tend to be one of the earliest initiatives taken by municipal authorities newly faced with the task of managing urban stormwater. While these components have a valid role to play in total catchment stormwater management, the complete absence of *storage* associated with them, even short-term detention storage, disqualifies them from inclusion – as isolated elements – among Category 2 systems (see Section 4.3.1). 'At source' stormwater pollution control as interpreted in this Chapter may involve some member of this group, typically a GPT or area of permeable paving with built-in filter, but it must also be associated with a component **retaining** the cleansed storm-water.

Chapter 5 of the Handbook is devoted to meeting the need, primarily, to control flood flows in the urban environment using on-site retention (OSR) practices. The approach which must be used to design systems intended, primarily, for *pollution* control differs from the procedures described in Chapter 5 in a number of significant respects. The main differences are explained as follows :

All Category 1 (flood control) systems must include some form of filter located upstream of the required storage : this element of design is explained comprehensively in Section 1.3.3. Category 2 (pollution control) systems also require filters located upstream of storages : a typical arrangement is illustrated in Figure 3.5a. **The capacity flows of the respective filters provide the single most important difference between Category 1 systems and those of Category 2.** Both are matched to the highest flows, Q_{peak}, likely to pass through them in their respective critical design events, but it is the significantly different events themselves which provide the distinction :

- Q_{peak} of a treatment unit placed upstream of a flood control installation (Category 1) is determined by catchment area and the appropriate ARI/critical storm duration which, in combination (interpreted as a rainfall intensity) produce the most demanding **entry** condition for the device or system. Critical storm duration is therefore (usually) equal to t_C, as previously discussed (Section 4.2.6), and ARI may be drawn from the range 2-years to 10-years for 'minor' system components and 50-years to 100-years where the storage is a 'major' system component. [There are rare circumstances in which $T_{C(total)}$ and t_C coincide]
- Q_{peak} of a pollution control treatment unit (Category 2) is determined by catchment area and rainfall intensity, also for t_C , and ARI = 0.25-years (Australian suggested practice, NSW Dept of Housing, 1998). Or it may arise from 'continuous simulation' modelling aimed at achieving retention of a specific percentage of annual runoff volume : in this case, the peak flow before bypass is called Q_{lim} (see Table 3.2). An essential characteristic of Category 2 installations is that, normally, they bypass all flow exceeding their Q_{peak} or Q_{lim} capacities (see Figure 3.6).

Illustrations of the former (Category 1 installations) include **infiltration surfaces** which fulfil a highly effective stormwater cleansing role : much greater flow capacities than $Q_{0.25}$ or Q_{lim} can be justified in these installations because of their amenity value, for example as car-parking areas (see Example 5.1, Section 5.2). The range of **"leaky" devices** shown in Figures 1.3 and 1.5 and Example 5.2, have provision for pre-treatment (rainwater tanks, filters or sediment traps) or they retain incoming sediment in cases where these loads are too small to affect performance over their expected lifespan periods : these devices also must have relatively high filter-entry flow capacities. **Infiltration ponds** ('dry' ponds) integrate within their flood control capability a strong treatment role provided by the presence of a pond where sediment can settle, underlain by a permeable filter bed : these, too, are capable of accepting flows of relatively high order (see Example 5.4). The 'connectedness' or overlapping of design objectives – **firstly**, flood control and, **secondly**, pollution control – referred to in Section 4.1 is clearly indicated by these illustrations.

The types of installations falling into Category 2, where they are components of a water-sensitive urban design strategy, show a *reversal* of this priority with treatment of pollution emerging as the **primary** objective, and on-site storage of stormwater, **secondary**. The range of devices, systems, products and facilities available to achieve the primary goal is extensive and growing : the available offerings are described in many current documents [Water and Rivers (WA), 1998; NSW Dept of Housing, 1998;

CSIRO, 1999; Melbourne Water, 2005] and briefly reviewed in Chapter 2 of this Handbook. It is to this wealth of information and the accompanying performance data that designers must go to select the most appropriate, most cost-effective and most sustainable filter systems for their projects. Further detailed comment on this aspect of design/selection is outside the scope of the present chapter.

Treatment systems used in the process of managing urban stormwater *at source* are of three basic types : **first**, those which collect a wide range of urban litter components – leaves, cans, coarse sediment, etc. – and which require regular cleaning, for example, after each major storm. The **second** type retains medium/fine sediment at a filter layer integrated within the structure: removal of retained particulate matter in this case may involve regular sweeping and/or intense jet-vacuum surface cleaning as well as partial reconstruction of the installation including replacement of the filter layer after some (long) period of operation. The **third** type covers the wide range of vegetated filter systems such as filter strips, swales, reed beds, etc. Procedures 5A and 5B, described in Sections 7.2 and 7.4, following, are focussed on systems involving use of the *first* and *second* of these alternatives. The *third* is examined in Section 7.8 – Filter Strip Swale – which is offered as an alternative to the approach presented in Chapter 10 of the ARQ document (Engineers Australia, 2005).

The high standards of stormwater treatment required of installations discharging *directly* to sensitive receiving (water) environments cannot be delivered by the **first** type described above, used alone. There are three reasons for this: first, they are incapable of capturing the very fine particulate matter which is harmful to aquatic ecosystems; secondly, they are not designed to remove dissolved pollution components – phosphorous, nitrogen and heavy metals – and, thirdly, the irregular maintenance schedules frequently employed with them can lead to degradation of retained material, converting it into point source toxic pollution. Certainly, efficient and effective GPTs are important elements in the "toolkit" of the stormwater manager, but they should be well maintained and integrated with additional treatment components in any competently planned facility.

This 'additional treatment' can be provided, conceptually, by the gravel-filled (or similar) "soakaway" retention unit illustrated in Figure 3.5a. [Porous and permeable paving (the **second** type described above) incorporates such an element as sub-structure.] Its role in treatment is multi-faceted: it behaves in the manner of a 'trickling filter' – well understood and widely employed in the wastewater treatment industry – and shows impressive performance, in particular, in the removal of hydrocarbons (Pratt et al, 1998; Pratt et al, 2001). A further, important property of the "soakaway" can be its treatment – in appropriate circumstances - of any remaining dissolved pollutants (after passage through the gravel media) in the parent soil (Kotlar et al, 1996; Mottier et al, 2000; Datry et al, 2003; Weiss et al, 2006). 'Appropriate circumstances' referred to here includes clay soils, sandy clays and deep, fine sands but specifically excludes coarse, shallow sands with high groundwater tables (see Mikkelsen et al, 1997; Fischer et al, 2003).

Retention storages embraced by Category 2 **Strategy A** or **Strategy B** 'source control' systems (see Section 4.3.1), are predominantly **terminal** or off-line installations. The bulk of outflow from them (approximately 90% for Strategy A systems) passes, typically, into local parent soil, as explained above, rather than into a downstream drainage path. *Overflow* shown in Figure 3.5a represents only 3% – 5% of average annual runoff volume. In extreme circumstances (see Sections 7.3 and 7.5), some retained water *may* pass to an aquifer or be transferred by slow-drainage to a downstream surface waterway at very low flow rates.

The first half of Chapter 7 is devoted, mainly, to solving the problem posed by **Strategy A** systems :

What 'size' of sub-structure should be provided – following initial (coarse) filtration – to achieve retention and hence complete treatment of 90% average annual runoff from a specified catchment ?

The analysis, following, employs 'continuous simulation' modelling to answer this question taking account of the two factors which dominate such dimensioning – climate and local soil type. The second half of Chapter 7 is devoted to application of the results of this enquiry to the particular case of swales receiving surface runoff from, in particular, residential streets.

Of the three stormwater management strategies listed in Section 4.2.1, only the **yield-minimum** option meets the requirements of the filter/sub-structure integrated systems reviewed above. Category 2 (**Strategy A**) pollution control installations are therefore designed for storage of 90% of annual average runoff volumes generated in their contributing developed catchments without any account being taken of runoff volumes generated in the same catchments in their pre-developed states.

An apparent departure from this principle occurs when installations fulfilling the dual purposes of flood control and pollution control are to be designed. In such cases, the **regime-in-balance** strategy may be employed to fulfil the requirements of flood control with ARI fixed by 'minor' (2 – 10 years) or 'major' (50 – 100 years) system demands. However, the pollution control element of the installation (Category 2 system) should still – even under these circumstances – be designed according to the **yield-minimum** strategy. The outcome, in terms of size of sub-structure storage volume, etc., then involves the comparison and final decision process discussed in Section 7.6.2.

The most effective way to determine the basic information needed to design retention sub-structures such as "soakaways" for pollution control installations in the full range of soil types encountered at a particular location, is to follow the 13-step 'continuous simulation' procedure, and following, set out in Section 3.4. The outcome, in such circumstances, is a set of curves similar to those shown as Figure 3.7, but peculiar to the location of interest. The availability of these curves enables the designer to carry out Procedure 5A, below, with the preferred data base for the particular location.

It is presumed in what follows, however, that practitioners will take advantage of the 'generalised' approach expressed in the set of graphs included as Figure 7.1 and in Appendix C based on the five climate zones illustrated in Figure 3.8. These require knowledge of, only, location (represented by $i_{10,1}$, rainfall intensity), soil type (measured or estimated **site** hydraulic conductivity, k_h), and physical characteristics of the contributing catchment, to design 'simple', Strategy A pollution control retention systems. The basic design process involves the three steps set out in Procedure 5A, Section 7.2.2.

Before progressing further, however, there is need to identify the two options that face the designer with respect to basic filter systems, and to review water quality implications that follow the choice that is made. The wide range of filters and pollution removal/containment systems presently available (see discussion above) may be divided into two broad groups :

Type 1 : those that use processes drawn from the wide range of available technologies to retain gross pollution including coarse sediment, and that deliver **concentrated** or 'point' outflows, for example by pipe, and,

Type 2 : those that retain water-borne sediment by means such as sand and/or geotextile (non-woven) filtration and that deliver **distributed** outflows meeting acceptable 'cleansed water' standards. ["distributed" here means "from large plan area". Type 2 is particularly applicable to porous/permeable paving systems.]

These devices, used in combination with retention "soakaways" and detailed in the following sections, are capable of achieving **Strategy A** pollution control which implies *full treatment* including dissolved phosphorous, nitrogen and heavy metals, of approximately 90% of average annual runoff from a contributing catchment. An alternative, comparable option suited to circumstances where sediment control is the sole objective, is represented by the two 'treatment train' devices described in Sections 2.9 and 2.10 of the Handbook. Suspended particle treatment standards for these devices operating in real-world urban environments are :

suspended particle size: not greater than 20 μm, & TSS concentration: not greater than 20 mg/L.

These 'stand-alone' installations use non-woven geotextile fabric (Espinosa et al, 2001; Rommel et al, 2001; Jacobs, 2001) or sand/peat (Schueler, 1992b; Urbonas, 1999; Pitt et al, 1999; Pitt, 2002), respectively, as their central treatment media.

Another approach to stormwater treatment is provided in the following sections – termed **Strategy B**. This is based on 'first flush', only, retention which should be considered where a lesser standard than Strategy A treatment is acceptable.

7.2 DESIGN OF SIMPLE STRATEGY A SYSTEMS : PROCEDURE 5A

7.2.1 Introduction

The procedure starts by identifying the **climate band**, based on rainfall intensity, $i_{10,1}$, in which the location of interest lies : the graph-sets are found in Appendix C. The stormwater (cleansed) **retention efficiency, R = 90%** is recommended for all installations. This leads to the choice of an associated value for $\mathbf{q_{lim}}$ which is appropriate to the selected climate band. Taking this graph, the point determined by :

- site (modified) soil permeability (k_h), and,
- device-to-catchment (EIA) area ratio, A_R,

is identified.

Three possible cases arise :

Case 1 : the plan area available for the on-site retention device is *larger* than that required by the installation of minimum recommended depth – H_{min} = 0.30 m.

Case 2 : The plan area available for the on-site retention device corresponds to a device depth, H, falling *within* the recommended range – H = 0.30 m to 1.50 m, inclusive.

Case 3 : The plan area available for the on-site retention device is *smaller* than that required by the installation of maximum recommended depth – H_{max} = 1.50 m.

Note that area available, A_{avail}, is considered to be, typically, **within** the contributing catchment area, A_{EIA}.

STEP 2 of Procedure 5A explains the course of action which the designer must follow in each case.

In STEP 3, the q_{lim} flow determined for the climate band is interpreted into a catchment-related flow through :

$$Q_{lim} = 0.9 \times A_{EIA} \times q_{lim} \text{ L/s} \tag{7.1}$$

The designer is then required to 'match' this capacity flow (before bypass) to a preferred and/or available proprietary or other filter system. This process requires the designer to have available detailed technical information on a wide range of treatment units/systems which might be used upstream of the retention device and which admit (at least) coarse treated stormwater to the device.

Where the circumstances of a particular proposed installation fall into Case 3, above, and where Strategy A compliance is mandatory, then the process moves to Procedure 6A, Section 7.3, intended for more complex solutions. **Certain cautions/warnings need to be heeded in making this transition.**

The process reviewed to this point is focussed on graphs, etc. (Appendix C) derived from 'continuous simulation' modelling for "soakaway" devices filled with gravel. [The focus could have just as easily been placed on gravel/pipe combinations or, even, on 'dry' ponds.] The primary reason for this is recognition that the gravel-filled device is likely to be – for economic and treatment reasons – the most common form used. The second reason for the apparent bias is that use of a simple formula (Eqn. 7.4) enables the design outcome determined for a gravel-filled "soakaway" to be converted into the dimensions of an in-ground device of any other type, including a 'dry' pond. A more comprehensive explanation of this process is contained in Section 7.6.3.

The steps of Procedure 5A are explained in the next section.

7.2.2 PROCEDURE 5A : Strategy A design for sub-structure storages receiving *filtered* runoff from paved surfaces

Basic data requirements :

Location : Identify climate group based on range of $i_{10,1}$ rainfall intensity and, hence, graph-set selected from Appendix C (including Figure 7.1).

Value for R : The value recommended for retention efficiency in all design cases addressed in the Handbook is **R = 90%** [graphical information for other values of R is available from the ARQ document (Engineers Australia, 2005)].

Value for Q_{lim} : Identify the corresponding value of limiting capacity flow before bypass, q_{lim}, and hence Q_{lim} (see Eqn. 7.1).

Soil : Site hydraulic conductivity, k_h m/s, and Moderation Factor, U, which converts site (borehole) test results into values appropriate for installations (see Footnote, Section 5.1.2). Clay soils : U = 2.0; sandy clays : U = 1.0; sandy soils : U = 0.5.

Equivalent paved area of contributing catchment : A_{EIA} m^2.

Available space for device (within A_{EIA}) : A_{avail} in m^2. Hence, $A_{avail}/A_{EIA} = A_R$.

Void space ratio of proposed retention device, e_S **:**

gravel-filled : $e_S = 0.35$; gravel-pipe combination : $e_S = 0.5$

'milk crate' cells : $e_S = 0.95$; 'dry' ponds : $e_S = 1.00$.

Preferred/available treatment units file : An inventory of effective treatment/filter devices and systems together with their capacity flow characteristics should be consulted to select a filter unit (see Figure 3.5a) suited to the particular catchment circumstances. Its selection/design, including provision for **bypass** (see above), should be completed before Procedure 5A is commenced. The filter unit should have capacity, Q_{lim}, determined according to Eqn. (7.1).

The outflow from the (filter) device may be delivered as a concentrated entry (Type 1, see Section 7.1) or as a distributed entry from a porous/permeable paving surface (Type 2). In the latter case, attention must be given to the consequences for hydraulic conductivity, k_h, of sediment accumulation over the 'lifespan' of the paving (see Sections 3.1.2 and 3.1.3 and Procedure 1A or 1B, Section 5.1.).

Three-step process to determine retention device depth H:

STEP 1 :

Enter Figure 7.1 (illustrative graph) or graph selected from Appendix C, at the adopted value of site hydraulic conductivity, **adjusted in accordance with Moderation Factor, U**;

Enter Figure 7.1 at the value of A_R as applies.

STEP 2 :

The point of intersection in the k_h *versus* A_R plane (Figure 7.1) identified in STEP 1 lies :

Case 1 : *Above* the family of curves (point "1" in Figure 7.1), which implies that the plan area available for the on-site retention device is **larger** than required by the installation of minimum recommended depth, namely $H_{min} = 0.30$ m.

In this event, the designer may use this value (H_{min} = 0.30 m) for device depth, H, along with the corresponding **smaller** value of A_R given by the curve. In these circumstances the *adjusted* device plan area is called "A_{avail}" in STEP 3. The design process continues in STEP 3.

Case 2 : *Within* the family of curves (point "2" in Figure 7.1), which implies that the plan area available for the on-site retention device can be matched to a depth, H, falling **between** the recommended limits – H_{min} = 0.30 m to H_{max} = 1.50 m, inclusive; in this event, A_{avail} retains its initial value in STEP 3. The design process continues in STEP 3. **It is strongly recommended that the process go no further than STEP 3, particularly where *significant dissolved pollutants* are considered likely in the catchment runoff. Heeding this caution implies selecting a design depth from the family of curves represented by Case 2 *if at all possible*. The consequence of adopting a solution arising from Case 3, following, is a level of full treatment *less* than that recognised as Strategy A. [The ARQ document (Engineers Australia, 2005) allows this option.] A further WARNING to this effect is contained in Section 7.3.**

Case 3 : *Below* the family of curves (point "3" in Figure 7.1), which implies that the plan area available for the on-site retention device is **smaller** than required by the installation of maximum recommended depth, namely H_{max} = 1.50 m. In this event, adopt $\mathbf{H_{max}}$ = 1.50 m as the device depth and continue the design process in STEP 3 : A_{avail} retains its initial value. (This **must** be followed by Procedure 6A.) The designer may elect to use a value for H which is **smaller** than 1.50 m; the *process* to be followed is unaffected by this decision.

STEP 3 :

Report the dimensions and details of the filter device, including details of the limiting bypass flow, and of the retention system as plan area, A_{avail}, from STEP 2 and **H** (or $\mathbf{H_{max}}$), as applies, also from STEP 2. Provision for overflow must be included with these details.

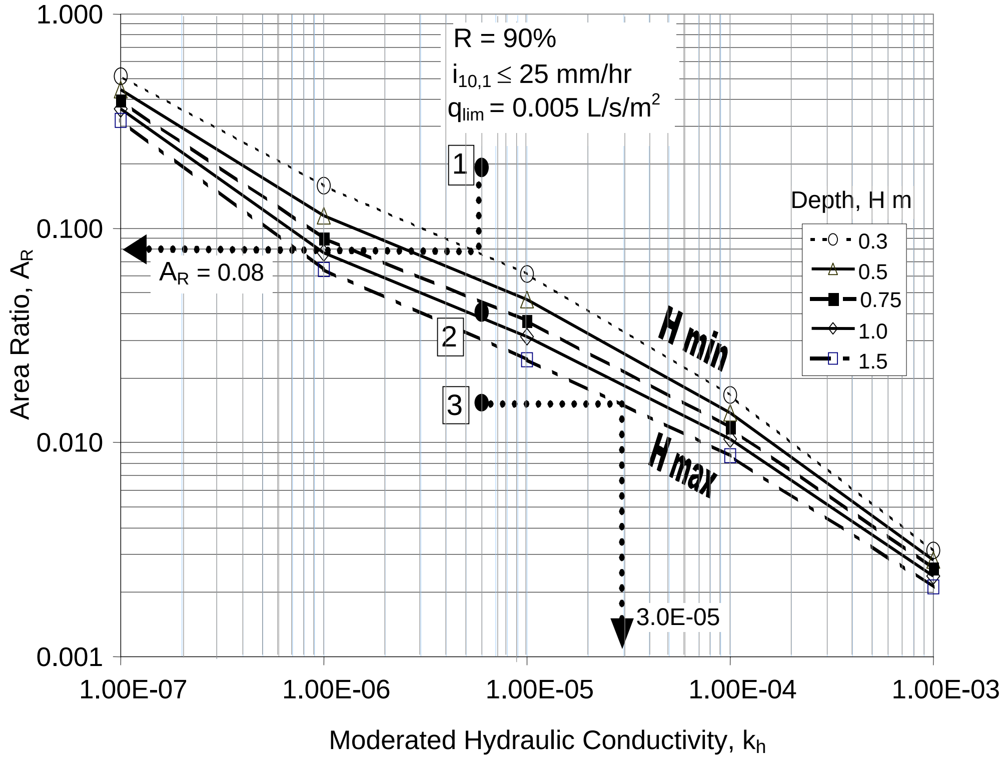


FIGURE 7.1 : Graph for retention efficiency, R = 90% (recommended value) for gravel-filled "soakaways" in Southern Australia (see Figure 3.8). "Soakaway" depth range : 0.30 – 1.50 m. Soils with moderated $k_{\mathbf{h}}$ **> 1.0 × 10^{-4} m/s are, typically, unsuitable for treating *dissolved pollutants* in catchment runoff** (see Mikkelsen et al, 1997; Fischer et al, 2003)

[The figures "1", "2" and "3" relate to Example 7.1, Section 7.7.]

7.3 MORE COMPLEX STRATEGY A SYSTEMS : PROCEDURE 6A

7.3.1 Introduction

In the ‘simple’ systems reviewed above, water removal is by percolation to the host soil, **only**. More complex systems require the designer to consider ‘hydraulic’ abstractions made while the routing process – inflow/storage/percolation – is taking place.

In these situations, retention device **depth** required – to achieve retention and on-site abstraction of 90% average annual runoff – may be greater than the practical maximum (depth) offered in the families of curves provided in Figure 7.1 or Appendix C. This problem can be solved through **accelerated water removal** (from storage) using bores or drainage pipes specially designed for the purpose. (This approach is introduced in Section 3.5.4.) The quest for these solutions lies at the heart of Procedure 6A.

Procedure 6A commences with STEP 4. This is intended to convey the clear signal to the designer that problems of filtered stormwater storage should be solved, first, as ‘simple’ systems (by the three steps of Procedure 5A) if at all possible before the move to a more complex solution is made. However, Procedure 6A also carries a WARNING: it should be avoided completely and design limited to Cases 1 and 2 (Procedure 5A) only, wherever *significant dissolved pollution* passing from the upstream filter is considered likely.

The issue facing the designer may be stated as follows : given the device plan area, $\mathbf{A_{avail}}$, and depth equal to $\mathbf{H_{max}}$ – for the particular site condition identified in STEP 2 (Case 3) of Procedure 5A – what rate of (water) abstraction from the installation, *other than by percolation*, is needed to produce a successful system? [The designer may adopt a lesser depth then 1.5 m – say 1.0 m – in the following process.]

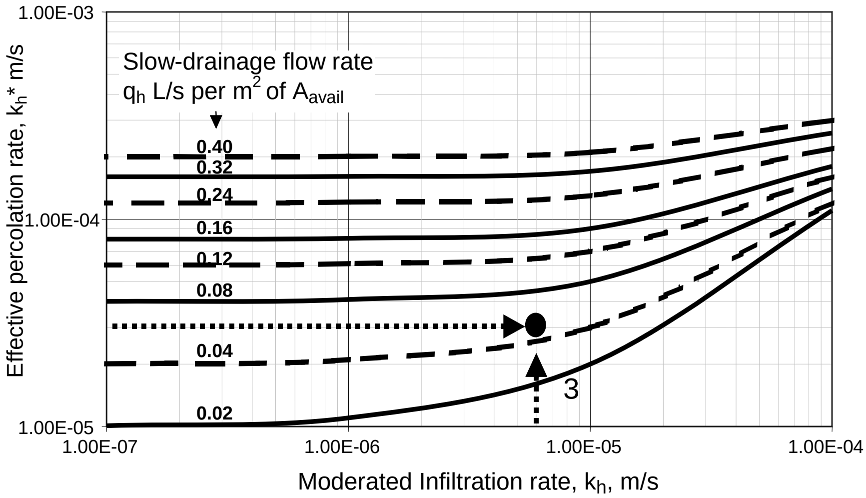


FIGURE 7.2 : Flow curves for accelerated water removal from in-ground retention devices

The two values, $\mathbf{A_{avail}}$, and $\mathbf{H_{max}}$, in any given set of circumstances, correspond to a **target** hydraulic conductivity value (‘to the right’ of point “3”) in the illustrative graph, Figure 7.1, which coincides with a successful outcome derived from the original ‘continuous simulation’ analysis. **The required, *total* water removal outflow corresponding to this case can be achieved through ‘hydraulic’ drainage acting conjunctively with percolation.** This *target* permeability (‘to the right’) is called “**effective percolation rate, $\mathbf{k_h}$***”.

The percolation flow rate into the host soil beneath the retention device may be computed from the available data (site moderated $\mathbf{k_h}$ and plan area $\mathbf{A_{avail}}$). Also, the *target* outflow rate may be calculated (from $\mathbf{A_{avail}}$ and $\mathbf{k_h}$***),** so the required ‘hydraulic’ flow rate, $\mathbf{q_h}$, can readily be found: it is the *difference* between the two calculated values: this value is presented for the full range of soil conditions in Figure 7.2, where: :

Horizontal axis : moderated hydraulic conductivity at site = (k_h) × Moderation Factor, U (used in STEP 1);

Vertical axis : effective percolation rate **, k_h***, stated in m/s - required with A_{avail} - obtained through the process described above and reported in STEP 4, following;

Family of curves (Figure 7.2) : recharge flow rate (or slow-drainage flow), q_h, stated in **L/s per m^2 of area, A_{avail},** which must be abstracted from the device whenever it contains filtered stormwater. A factor of safety of 2 has been introduced in the process of obtaining the curves presented in Figure 7.2. **If this outflow contains *significant dissolved pollutants* it is considered unsuitable for discharge to aquifers or other sensitive (water) receiving environments.**

Recognition of the presence of an aquifer beneath a site where recharge of cleansed storm runoff is considered possible, should be followed by an environmental impact assessment as discussed in Procedure 4A, Section 5.1. If favourable, a review of available geological-geotechnical reports to ascertain its basic characteristics should be conducted – depth, standing water level, water quality, potentiometric gradient, local use (potable, irrigation, industry, etc.), likely yield per bore and, hence, potential rate of recharge, **q_r**. [Potential rate of recharge may be taken as 50% rate of yield (Pavelic et al, 1992).]

In situations where the required accelerated emptying is to be achieved by slow-drainage pipeline, the main restriction on the quantity of flow which can be abstracted is likely to be head difference between the in-ground device and the pipe outlet. This option is therefore unavailable in terrain which is very flat; but the flows involved – Q_{hA} calculated using Figure 7.2 and Eqn. (7.2) – are, typically, so small that this constraint is rarely preclusive.

Steps in the design procedure are set out in Procedure 6A, Section 7.3.2, commencing with STEP 4, recognising that the design process is a continuation from STEP 3 (Case 3) of Procedure 5A.

7.3.2 PROCEDURE 6A : Strategy A design for sub-structure storages with :

(a) aquifer access, or

(b) potential for 'slow drainage',

receiving filtered runoff from paved surfaces or from filter-integrated systems.

At this point in the design process, the filter unit has been selected and dimensioning of the installation as a 'simple' system (plan area A_{avail} and depth, H_{max}) – including provision for bypass (Q_{lim}) and device overflow – have been completed (STEPS 1-3 of Procedure 5A) before Procedure 6A is commenced.

STEP 4 :

Locate 'point' in illustrative graph, Figure 7.1, or on the appropriate graph from Appendix C chosen at the commencement of Procedure 5A, where A_R, given by A_{avail}/A_{EIA}, coincides with "soakaway" depth, H = H_{max}. [The 'point' corresponds to a significantly **higher** value of k_h than the (moderated) site value used in STEP 1, above.] This (new) value is called, **"effective percolation rate", k_h***.]

Note : The designer may adopt a lesser value in place of H_{max} = 1.50 m, for example H = 1.0 m; this leads to a higher value of effective percolation rate, **k_h***, than that identified above. The procedure, below, is unaffected by this decision.

Locate 'point' in Figure 7.2 corresponding to :

- the **site** (moderated) hydraulic conductivity, k_h (horizontal axis); and,
- effective percolation rate, k_h* (vertical axis);

Determine the 'hydraulic' flow rate, **q_h**, in L/s per m^2 of retention device plan area, A_{avail} , given by the family of curves.

STEP 5a (aquifer access) :

Calculate number of bores, “n”, needed, with natural percolation, to achieve the required (total) outflow rate : this requires ‘hydraulic’ flow, $\mathbf{Q_{hA}}$, where :

$$\mathbf{Q_{hA} = [(A_{avail}) \times (q_h)]\ L/s} \qquad \textbf{(7.2)}$$

and, $\mathbf{n.q_r \geq Q_{hA}}$, where $\mathbf{q_r}$ = (individual) bore recharge rate, L/s; hence number (integer) of bores, “n”.

OR

STEP 5b (slow-drainage pipeline) :

Calculate slow-drainage flow rate, $\mathbf{Q_{hA}}$, by Eqn. (7.2).

Note : The *actual* flow rate in the slow-drainage pipeline may be any flow greater than Q_{hA} , however, if it is *much* greater, it may result in the outflow coinciding with the falling limb of the flood wave, causing the system to act more like an OSD facility than its intended use as a retention device. (See Special Note in Section 3.5.4 and, also, Section 5.4.)

STEP 6 :

Report details of the **complete** installation including filter system (Type 1 or Type 2, as described in Section 7.1), retention device dimensions etc from STEP 3, and, from STEP 5 :

(a) **“n”** bores at $\mathbf{q_r}$ m^3/s per bore; **OR** (b) ‘slow-drainage’ pipeline, flow $\mathbf{Q_{hA}}$ m^3/s.

7.4 DESIGN OF SIMPLE STRATEGY B SYSTEMS : PROCEDURE 5B

7.4.1 Introduction

Under **Strategy B** (see Section 4.3.1) pollution control using retention installations, only the ‘first flush’ of all storm runoff is treated and, hence, only the runoff volume which conveys this pollution to the device need be stored. Design of ‘simple’ systems following this approach, is the subject of Procedure 5B.

The above considerations – a focus on ‘first flush’ runoff – lead to site ‘time of concentration’, t_C, as the storm duration of significance used in the design process. The peak flow entering the device before bypass operates, Q_{lim} is set equal to $Q_{0.25}$ based on storm duration t_C (see Section 3.7.1). The design of in-ground, “leaky” installations receiving cleansed storm runoff, becomes a simple application of Eqn. (5.5a or b), as applies : in this case,

$$\tau = 2\ t_C \text{ (see Figure 3.4).}$$

Volume, $\forall$, is set equal to the runoff volume generated in the ARI, Y = 0.25-years storm of duration t_C, only.

The design process involves two steps which are applicable to both Type 1 and Type 2 treatment/filter units or systems (see Section 7.1) followed by retention installations.

An alternative to Procedure 5B, as reviewed above, is to use the technology of infiltration or ‘dry’ ponds for pollution control/retention in small urban catchments (see Section 2.11 and Procedure 3, Section 5.1.4). The procedure is based on the ‘design storm’ approach with application opportunities more limited than for “soakaways” : infiltration ponds are recommended for sandy and sandy-clay sites, only, in order to satisfy appropriate emptying time criteria. Nevertheless, these devices should not be overlooked by designers who should include them among alternative options in the initial stages of planning. An illustrative example of design, Example 7.4, is included in Section 7.7.

7.4.2 PROCEDURE 5B : Strategy B design for sub-structure storages receiving cleansed runoff from paved and other ground-level filter-integrated surfaces

In this case, the choice of treatment unit (see Figure 3.5a) has been made and its design, including provision for **bypass**, completed before Procedure 5B is commenced. The filter/treatment unit should have capacity, Q_{lim}, matched to the contributing catchment (critical storm duration, t_C) with ARI, Y = 0.25-years.

The outflow from the treatment device may be delivered as a concentrated entry (Type 1, see Section 7.1) or as a distributed entry from a porous/permeable paving surface (Type 2). In the latter case, attention must be given to the consequences for hydraulic conductivity, k_h, of sediment accumulation over the 'lifespan' of the paving (see Section 3.1.2 and Procedure 1A or 1B, Section 5.1.)

STEP 1 :

Compute runoff volume $\forall$ m^3 from the contributing catchment in the ARI, Y = 0.25-years storm for duration $t = t_C$, catchment 'time of concentration'.

Given the space available for the "leaky" device, identify **available** plan area, A_{avail} m^2 set equal to area "a" in Eqn. (5.5b, below); also identify void space ratio, e_S, of the selected "soakaway" type, hydraulic conductivity of the site soil, k_h, and the appropriate value of Moderation Factor, U (U = 0.5 for sand; U = 1.0 for sandy clay; U = 2.0 for clay).

With $\forall$, a (= A_{avail}), e_S , k_h and U assigned, determine "soakaway" depth, H, from :

$$H = \frac{\forall}{a.e_s} - \frac{60}{e_s}.k_h.\tau.U \qquad \textbf{(5.5b)}$$

where $\tau = 2t_C$.

The depth H given by Eqn. (5.5b) should fall, for practical reasons, within the range 0.30 m to 1.50 m (see Figure 7.1). If H is **smaller** than 0.30 m, then the area A_{avail} is "too large" and a smaller area may be used for A_{avail} and STEP 1 repeated.

If Eqn. (5.5b) gives depth H **greater** that 1.50 m, then the area A_{avail} is too small for a 'simple' solution to emerge from Procedure 5B, and recourse to Procedure 6B is inevitable. In this circumstance, the value of depth H (greater than 1.50 m) goes **directly** to Procedure 6B, STEP 4. **Every effort should be made to avoid this design direction where *significant dissolved pollutants* are considered likely in the catchment runoff (see Section 7.2.2 – Case 2).**

STEP 2 :

Test the result (for depth H) from STEP 1 for acceptable emptying time using :

$$T \approx \frac{2H.e_s}{k_h}, s \qquad \textbf{(3.32)}$$

If $T \leq 12$ hours, then **accept** result as final value for depth H;

If $T > 12$ hours, then **reject** result for depth H and proceed directly to Procedure 6B, STEP 4.

STEP 3 :

Report the dimensions and details of the filter device or system adopted, including details of the limiting bypass flow, and of the retention device using values for A_{avail} and H associated with the successful outcome (STEP 2). Provision for overflow must be included with these details. Re-design of a rejected system (STEP 2) takes place under Procedure 6B, Section 7.5.2, following, commencing with STEP 4.

7.5 MORE COMPLEX STRATEGY B SYSTEMS : PROCEDURE 6B

7.5.1 Introduction

Design of systems involving aquifer access or slow-draining pipes and meeting **Strategy B** requirements (see Section 4.3.1), that is, control of 'first flush' runoff and retention of the associated cleansed runoff volume, only, follows the same broad principles set out for Strategy A – complex systems – compliance, above (Sections 7.3.1 and 7.3.2). The main difference lies in the manner in which the aquifer recharge or slow-drainage flows are determined. In the present case, the flows are calculated on the basis of devices emptying in an 'acceptable time', set at 12 hours.

As in the parallel Strategy A procedure, recognition of the presence of an aquifer beneath a site where recharge of cleansed storm runoff is considered possible, should be followed by an environmental impact assessment (see Figure 1.4a and Section 3.8). If favourable, a review of available geological-geotechnical reports to ascertain its basic characteristics should be conducted – depth, standing water level, water quality, potentiometric gradient, local use (potable, irrigation, industry, etc.), likely yield per bore and, hence, potential rate of recharge, $\mathbf{q_r}$. [Potential rate of recharge may be taken as 50% rate of yield (Pavelic, et al, 1992).]

In situations where the required accelerated emptying is to be achieved by slow-drainage pipeline, the main restriction on the quantity of flow which can be abstracted is likely to be head difference between the in-ground device and the pipe outlet. This option is therefore unavailable in terrain which is very flat; but the flows involved – Q_{hB} calculated using Eqn. (7.3) – are, typically, so small that this constraint is rarely preclusive.

However, the WARNING contained in Section 7.3.1 relating to the likely presence of *significant dissolved pollution* in effluent passing from the upstream filter applies here also. Procedure 6B should therefore be avoided in these circumstances and design confined to the scope offered by Procedure 5B only.

The design is executed according to the steps of Procedure 6B, Section 7.5.2, following, commencing with STEP 4. This is intended to convey the clear signal to the designer that problems of cleansed stormwater runoff should be solved, first, as 'simple' systems (by the three steps of Procedure 5B) if at all possible before the move to a more complex solution is made.

7.5.2 PROCEDURE 6B : Strategy B design for sub-structure storages with :

(a) aquifer access, or

(b) potential for 'slow drainage',

receiving cleansed runoff from paved surfaces or ground-level filter-integrated systems.

At this point in the design process, the treatment unit has been selected and dimensioning of the installation as a 'simple' system – including provision for bypass and device overflow – has been completed (Procedure 5B) before Procedure 6B is commenced. The outcome of the previous design segment is either depth, H, greater than the recommended maximum of 1.50 m (see Procedure 5B, STEP 1) or **failure** to meet the required emptying time criterion (STEP 2). The design process continues :

STEP 4 :

Revisit the design previously prepared under Procedure 5B **and which produced a depth, H > 1.50 m or which failed the emptying time test** and determine, for this design, an acceptable emptying flow rate, Q_{hB} :

$$\mathbf{Q_{hB} = (a.H.\ e_S)/(12 \times 3600)\ m^3/s} \qquad \textbf{(7.3)}$$

where the parameters a, H and e_S are identical to the values used in or resulted from STEP 1 (Procedure 5B).

STEP 5a (aquifer access) :

Calculate number of bores, “n”, required to recharge a flow of *at least* Q_{hB} (Eqn. 7.3) to the aquifer, that is:

$$\mathbf{n.q_r \geq Q_{hB}} \qquad \textbf{(7.3a)}$$

where $\mathbf{q_r}$ = (individual) bore recharge rate, m^3/s; hence number of bores, “n” , an integer.

OR

STEP 5b (slow-drainage pipeline) :

Calculate slow-drainage pipe flow rate, $\mathbf{Q_{hB}}$**,** by Eqn. (7.3).

STEP 6a :

Re-evaluate depth H by direct substitution into Eqn. (5.9b) with parameter Q_r **replaced** by a flow given by n × (permissible recharge rate per bore), that is ($n.q_r$); this flow should be ≥ Q_{hB}, determined in STEP 4. All other parameters remain the same as those used in or resulting from Procedure 5B, STEP 1. This process leads to a slightly shallower installation with emptying time less than 12 hours.

OR

STEP 6b :

Re-evaluate depth H by direct substitution into Eqn. (5.9b) with Q_r **replaced** by Q_{hB}, obtained from Eqn. (7.3). All other parameters remain the same as those used in or resulting from Procedure 5B, STEP 1. This process leads to a slightly shallower installation with emptying time less than 12 hours.

[While a value of Q_{hB} greater than that determined in STEP 4 is permissible, the *excess* should be small to achieve the purpose of ‘slow drainage’ (see Special Note, Section 3.5.4)].

STEP 7 :

Prepare a system design using values for a (= A_{avail}), H, n and q_r **or** Q_{hB} associated with the successful outcome from STEP 6.

7.6 INFILTRATION SYSTEMS : FURTHER DESIGN ISSUES

7.6.1 Introduction

Design of pollution control systems (Strategy A and Strategy B, see Section 4.3.1) are treated in Chapter 7 to this point as, effectively, **isolated** systems, i.e. cases addressing the joint problems of filtration followed by cleansed stormwater retention including further treatment. The major focus, clearly, is on the provision of storage in sub-structures, often beneath treatment surfaces such as sand/gravel filters, permeable paving layers, etc. An important element of such systems which have pollution control as their focus is provision for bypass in events producing runoff flows greater than Q_{lim} (Strategy A systems) or $Q_{0.25}$ (Strategy B systems).

In practice, such system isolation is unusual : it is far more common for a comprehensive interpretation of ‘source control’ to be sought involving containment of flow quantity (flood control systems) as well as pollution control. Interaction between the two systems (flood control and pollution control) raises important design considerations and consequences, explored in the following sections.

Another issue of importance for the designer is how to adjust the information contained in the Strategy A graphs derived from ‘continuous simulation’ modelling – the illustrative graph, Figure 7.1 (and Appendix C) – for retention sub-structures **other** than gravel-filled “soakaways”. This raises two differences of importance :

- design of **"soakaways"** constructed from pipe/gravel materials or from 'milk crate' and other types of units, and design of "dry" ponds;
- design of gravel-filled **trenches** or trenches constructed from pipe/gravel materials or from 'milk crate' etc. units.

A further matter of uncertainty which is high on the agenda of every practitioner in the field of WSUD, is that of 'lifespan' of retention installations receiving storm runoff from urban catchments.

Each of these issues is considered below.

7.6.2 Flood control, pollution control interaction

In relation to **Strategy A :**

1) Current Australian practice ['design storm' approach, IEAust., 1987)] requires that **flood control** systems for specific sites be designed using a 'critical storm duration' which is, normally, related to the (local) catchment longest travel time; in other cases, this duration is set equal to site 'time of concentration', t_C. **Pollution control** system design procedures (5A and 6A) have no such focus in terms of storm duration because of their basis in 'continuous simulation' modelling; also,

2) **Flood control** systems are, typically, designed with ARI = 2 to 10 years ('minor' networks) or 50 to 100 years ('major' networks), while **pollution control** systems (Procedures 5A and 6A) have no clearly-identifiable ARI : a 'surrogate' ARI of, perhaps, 0.25- to 1-year probably applies.

Consideration (1), above, results in storages required for flood control installations which are **smaller** than those required for pollution control systems, *if the Y-years frequency were taken as the same in both instances*. But this is not the case, as indicated by consideration (2), above. The significantly greater storm runoff volumes, $\forall$, generated in design storms used in flood control scenarios (Y > 2-years), typically out-weigh the corresponding $\forall$ resulting from ARI, Y = 0.25- to 1-year events associated with pollution control installations. This is common but not universal, so the designer is advised to check each case to determine the dominant condition where problems involve both flood control and pollution control, which frequently is the case. While a universal 'rule' cannot be formed on this matter, the greater sub-storage volume is more likely to arise from flood control considerations than those of pollution containment. Illustrative Examples 7.2 and 7.3 (Section 7.7) shed more light on this issue.

In relation to **Strategy B :**

The storage volumes required for Strategy B cases (Procedures 5B and 6B) in normal circumstances are significantly smaller than those which result from Procedures 5A and 6A. The reason for this is the combination of short (design) storm duration – t_c , 'time of concentration' only – and low ARI, Y = 0.25-years used in the 'first flush' procedure. It may therefore be reasonably assumed that any flood control design, correctly executed with ARI, Y = 2-years or greater, will *inter alia* subsume the corresponding pollution control cleansed stormwater requirements of Strategy B.

In relation to **bypass :**

It is mandatory for an on-site installation falling within the ambit of **Type 1** – filter device with concentrated exit flow (see Section 7.1) – to incorporate bypass of all flows greater than $Q_{.lim}$. Devices falling within this classification are usually proprietary products matched, by design/selection, to field circumstances.

However, **Type 2** installations – typically, with grass or permeable paving treatment surfaces designed according to Procedures 1A or 1B or 5A/6A – are not, necessarily, so limited. Indeed, it would be most unusual to impose a $Q_{.lim}$ bypass limitation, for example, on a permeable paving car park, to achieve purely pollution control objectives. This recognition, taken with flood/pollution control interaction discussed above, puts the **design** of porous/permeable paving systems primarily in the domain of flood control with pollution containment achieved as a subsidiary benefit. Study of Example 7.2, Section 7.7, should provide some valuable insights on this issue.

It may be concluded from this brief excursion into the realm of stormwater quantity and quality management interaction, that **systems designed with *flood control* as the primary objective, rather than pollution containment, tend to achieve Strategy A or Strategy B compliance as a beneficial by-product of competent design.**

This does not mean that the detailing of Procedures 5A, 6A, 5B and 6B is without value – far from it – but it does mean that their **exclusive** use to design installations is likely to be unusual and their vital role in reviewing compliance with Strategy A or Strategy B objectives, as in Examples 7.2, 7.3 and 7.4 (Section 7.7), far more common. ‘Filter strip’ swales, described in Section 7.8, are members of the “unusual” class referred to here.

7.6.3 Design of non-gravel retention devices

One of the most important elements in the ‘continuous simulation’ modelling which led to Figure 7.1 (and the graphs of Appendix C) is that of water level drop with time – the recession relationship – of an operating retention device.

The relationship is revealed in Eqn. (3.32), emptying time :

$$T \approx \frac{2H.e_s}{k_h} \text{ , s} \qquad \textbf{(3.32)}$$

It is clear from this formula that emptying time of a gravel-filled “soakaway” is related, directly, to the product of device depth, H, and void space ratio, e_S. But this product represents, simply, the **absolute** depth of water contained in the device, that is, depth h_a , *unhindered by the presence of the gravel*.

Thus, the time taken for the water depth in a “soakaway” of any type to fall from

depth, $(h_a)_1$ to depth, $(h_a)_2$,

is **the same**, regardless of space taken up by solid material, provided depth h_a is set equal to the product of actual depth, h, and e_S. This reasoning may be applied equally to infiltration or ‘dry’ ponds : Eqn. (3.32) can be employed in the design of ponds so long as e_S is set equal to 1.0.

It follows that design of a “soakaway” of **any** type and, also, ‘dry’ ponds, can be determined from the sets of graphs referred to above provided a simple conversion between void space ratio, e_S, of the proposed device and e_S = 0.35, representing the gravel-filled case, is made (Eqn. 7.4).

It is recommended in these circumstances, that design of a “soakaway” other than gravel-filled, or of a ‘dry’ pond, be carried out in two steps :

STEP 1 :

Design as for a gravel-filled “soakaway” with the data appropriate to the particular case – location, soil type, catchment EIA, available space, etc., using Procedures 5A and 6A, as required.

STEP 2 :

Convert the final (gravel-filled) “soakaway” depth, H_{gravel} (Procedure 5A, STEP 3; or Procedure 6A, STEP 6, as applies), into a final ‘new’ device depth through the formula :

$$H_{new} = H_{gravel} \times [0.35/(e_S)_{new}] \qquad \textbf{(7.4)}$$

An illustration of this process is to be found in Example 7.3, Section 7.7.

All other aspects of the design remain the same, except for the following qualifications applicable to infiltration or ‘dry’ ponds.

It is recommended that the full range of soils which is suitable for “soakaways” is not appropriate for infiltration ponds : these should be limited in application to sand and sandy clay sites only (see Procedure 3, Section 5.1.4). Another difference relates to upstream treatment : storage and the permeable bed of a pond provide, together, an effective sedimentation/filtration system. Upstream treatment, in the case of infiltration ponds, can therefore be limited to collection/retention of gross pollutants such as leaf matter, urban debris, etc. A vegetated swale is ideal for this duty (see Chapter 10, ARQ document, Engineers Australia, 2005).

A further difference between “soakaways” and ‘dry’ ponds relates to the application of the Procedure 6A or 6B component of design. Directly applied, this would lead to injection of ponded water into aquifers or removal of (pond) water by slow-drainage pipeline. Because of inadequate pre-treatment provided to retained water, this mode of abstraction practice *directly* is **not** recommended (see Important Note, Section 5.1.5).

7.6.4 Design of Retention Trenches

A ‘correct’ procedure for the design of retention trenches of various types – gravel-filled, ‘milk crate’, etc. – using the same ‘continuous simulation’ modelling approach which produced Figure 7.1, etc. – could certainly be developed. However, it would result in a folio of graphs – equivalent to Appendix C – for *each* trench width of interest. This has not been attempted : in its place is a simplified design procedure which is conservative and based on the existing Procedures 5A and 6A. It involves three steps :

STEP 1 :

Design as for a gravel-filled “soakaway” with the data appropriate to the particular case – location, soil type, catchment EIA, available space – using Procedures 5A and 6A, as required.

STEP 2 :

Accept the final (gravel-filled) “soakaway” depth, H_{gravel}, and plan area, A_{avail} (Procedure 5A, STEP 3, or Procedure 6A, STEP 6, as applies) in cases where a **gravel-filled** trench is sought; convert the device depth into a final ‘new’ device depth through Eqn. (7.4) in **all other cases**.

STEP 3 :

Arrange area A_{avail} in any (trench) plan layout that is suited to the location, keeping trench depth the same as given by STEP 2, above. Trenches designed according to these guidelines will tend to empty more rapidly than the equivalent “soakaways” upon which they are based.

7.6.5 Strategy A systems : consequences for ‘lifespan’ and maintenance

The origins of sediment in the urban landscape and observed impacts of water-borne particulate matter on the performance and ‘lifespan’ (years of service before reconstruction or reinstatement is necessary) of components of retention systems, were reviewed briefly in Section 1.3.3. The main lesson which emerges from these considerations is the need for great caution to be exercised by designers/practitioners in dealing with sediment mobilised in urban catchments following rainfall, particularly during the construction phase of any development.

Treatment/filter systems designed to achieve the Strategy A level of pollution control for any nominated retention efficiency such as R = 90% recommended in this Handbook, may be grouped into Type 1 or Type 2 applications (see above). The **former** includes the array of proprietary and public domain installations which collect/retain water-borne sediment and, therefore, require an ongoing commitment to cleaning, maintenance and eventual replacement. Some advice on the consequences of these aspects of GPTs is included in Chapter 2.

Type 2 installations, on the other hand, are “lost sediment” systems by design, meaning that they collect/retain sediment within their structures for the duration of their effective ‘lifespans’ and are, therefore, maintenance-free as regards sediment removal. Of course, regular inspection and corrective maintenance such as mowing (grassed surfaces) or sweeping/vacuuming [block paving, see Dierkes et al

(2002)], to remove *surface*-retained sediment is strongly recommended, but the task of removing *internally*-retained particulate matter is not a required activity. This aspect makes them popular with local government and other municipal agencies. The issue of their widespread use hinges, however, on the ‘lifespan’ which can be expected for such installations.

This issue is explored in Section 3.1.2 where the findings of Rommel et al (2001) are reported. Rommel et al’s results can be re-interpreted to provide another insight to the problem at hand. Assuming that (see Urbonas and Stahre, 1993) :

1) 80% of the total pollution load is contained in the first 20% of runoff (the ‘first flush’); and,

2) the 200 mg/L used in the tests represents a typical ‘first flush’ (average) concentration for established residential neighbourhoods; and,

3) the design standard average recurrence interval likely to be used in design is ARI, Y = 2-years *at least*, is adopted,

then it can be shown that a ‘lifespan’ of at least 25 years without (**internal**) filter system replacement is attainable. This outcome, taken with the observation made in the previous section that “… systems designed with *flood control* as the primary objective, rather than pollution containment, tend to achieve Strategy A or Strategy B compliance as a beneficial by-product of competent design” offers the designer hope that such systems, using ARI values of 2-years and above, carry with them the assurance of (relatively) maintenance-free filter performance for periods of up to 25 years.

There is an important qualification incorporated into this line of reasoning which needs to be clearly understood, namely, that the suggested performance applies to “…established residential neighbourhoods…”. As pointed out by Argue (2000), poor on-site management of construction-phase sediment can produce complete blockage of a Type 2 filter system before it has even begun its intended ‘lifespan’ in an established urban setting.

7.7 EXAMPLES : PROCEDURES 5A and 6A

7.7.1 Introduction

It was pointed out in Section 7.6.1 that cases of isolated design, for example flood control, only, without stormwater treatment, or pollution control without associated flood control measures, are rare in practice. The examples which follow include a mix of illustrations, some involving dual objectives, others addressed to single goals. The aim of this section is not to present complete design detailing of case study installations but, rather, to provide practitioners with sufficient information to enable them to undertake such (complete) tasks drawing together aspects and components of design appropriate to the demands of particular situations.

Four illustrative examples are included :

- Example of the use of the Figure 7.1 graph (or Appendix C) in pollution control design or checking;
- Treatment surface designed, also, as a stormwater quantity management facility;
- “Soakaway” designed as a flood control facility and checked for Strategy A pollution control compliance;
- Residential sub-division with infiltration pond designed for treatment of ‘first flush’ pollution load.

7.7.2 Example 7.1a : Use of Figure 7.1 graph (also Appendix C)

Procedure 5A (Strategy A)

Location : Adelaide, Southern Australia;

Soil : medium clay, $k_h = 3 \times 10^{-6}$ m/s**;** Moderation factor, U = 2.0;

Catchment : paved area, $A_{EIA} = 2{,}500$ m^2 ;

Space available : $A_{\mathbf{avail}} = 500$ m^2; hence, $A_R = 0.20$;

Retention device : gravel-filled "soakaway", $e_S = 0.35$.

Retention efficiency, R = 90%,

STEP 1 :

Moderated hydraulic conductivity :

$$k_h = (3 \times 10^{-6}) \times U = 3 \times 10^{-6} \times 2.0 = 6 \times 10^{-6} \text{ m/s;}$$

Plan space ratio, $A_R = 0.20$;

Locate in Figure 7.1; see point "1" on graph.

This corresponds to a "soakaway depth significantly **less** than H_{min}. Therefore adjust "$A_{\mathbf{avail}}$" to value corresponding to depth, $H = H_{min} = 0.30$ m; (see 'shift' in Figure 7.1 to $A_R = 0.08$);

Hence, (new) $\mathbf{A_{avail} = 0.08 \times 2{,}500\ m^2 = 200\ m^2}$**.**

STEP 2 : Case 1

"Soakaway" depth, H = 0.30 m, from STEP 1.

STEP 3 :

Report conclusions, R = 90% :

$q_{lim} = 0.005$ L/s per m^2 of A_{EIA}, hence, treatment system capacity flow :

$$Q_{lim} = 0.9 \times A_{EIA} \times q_{lim} \text{ L/s} \qquad \textbf{(7.1)}$$

$$= 0.9 \times 2{,}500 \times 0.005 = \mathbf{11.25\ L/s;}$$

Plan area of "soakaway" = **200 m^2**; "soakaway" depth, H = **0.30 m.**

7.7.3 Example 7.1b : Procedure 5A (Strategy A)

Location : Adelaide, Southern Australia;

Soil : medium clay, $k_h = 3 \times 10^{-6}$ m/s**;** Moderation factor, U = 2.0;

Catchment : paved area, $A_{EIA} = 2{,}500$ m^2;

Space available : $A_{\mathbf{avail}} = 100$ m^2 ; hence, $A_R = 0.04$;

Retention device : gravel-filled "soakaway", $e_S = 0.35$.

Retention efficiency, R = 90%.

STEP 1 :

Moderated $k_h = 6 \times 10^{-6}$ m/s, as above;

Plan space ratio, $A_R = 0.04$;

Locate in Figure 7.1; see point “2” on graph.

This corresponds to a “soakaway depth of H = 0.9 m

STEP 2 : Case 2

“Soakaway” depth, H = 0.90 m.

STEP 3 :

Report conclusions :

$$q_{lim} = 0.005 \text{ L/s per m}^2 \text{ of } A_{EIA},$$

hence, treatment system capacity flow :

$$Q_{lim} = 0.9 \times A_{EIA} \times q_{lim} \text{ L/s} \quad \textbf{(7.1)}$$

$$= 0.9 \times 2{,}500 \times 0.005 = \mathbf{11.3\ L/s;}$$

Plan area of “soakaway” = **100 m^2**; “soakaway” depth, H = **0.90 m.**

7.7.4 Example 7.1c : Procedures 5A and 6A

Location, soil, catchment, “soakaway” and retention efficiency and q_{lim} as for Example 7.1b, above.

Space available : $A_{\mathbf{avail}}$ = 37 m^2 ; hence, A_R = 0.015;

STEP 1 :

Moderated $k_h = 6 \times 10^{-6}$ m/s, as above;

Plan space ratio, $A_R = 0.015$;

Locate in Figure 7.1; see point “3” on graph.

STEP 2 : Case 3

“Soakaway” depth, H > H_{max} on R = 90% graph

$$q_{lim} = 0.005 \text{ L/s per m}^2,$$

hence,

$$Q_{lim} = 0.9 \times A_{EIA} \times q_{lim} \quad \textbf{(7.1)}$$

$$= 0.9 \times 2{,}500 \times 0.005 \text{ L/s} = 11.3 \text{ L/s.}$$

Hence nominate “soakaway” depth, H = $\mathbf{H_{max}}$ = **1.50 m**.

STEP 3 :

Report part-conclusions :

$$Q_{lim} = 0.9 \times A_{EIA} \times q_{lim} \text{ L/s} \quad \textbf{(7.1)}$$

$$= 0.9 \times 2{,}500 \times 0.005 = \mathbf{11.3\ L/s;}$$

Plan area of “soakaway” = **37 m^2**;

“soakaway” depth, H = **1.50 m.**

Procedure 6A – STEP 4 (slow-drainage) :

Moderated $k_h = 6 \times 10^{-6}$ m/s, as previously;

Plan space ratio, $A_R = 0.015$, as in STEP 1; see point "3" on graph, Figure 7.1.

Locate 'point' (horizontal shift from "3") in Figure 7.1, where $A_R = 0.015$ coincides with "soakaway" maximum recommended depth, $H_{max} = 1.5$ m. This 'point' corresponds to :

$k_h = 3.0 \times 10^{-5}$ m/s, now called **effective percolation rate, k_h*.**

Locate point "3" in Figure 7.2 corresponding to :

the **site** (moderated) hydraulic conductivity, $k_h = 6 \times 10^{-6}$ m/s (horizontal axis); and,

effective percolation rate, $k_h^* = 3.0 \times 10^{-5}$ m/s (vertical axis).

The value given by the family of curves is, say

$$\mathbf{q_h = 0.055 \text{ L/s per m}^2 \text{ of } A_{avail},}$$

STEP 5 :

Calculate slow-drainage flow rate :

$$Q_{hA} = [(A_{avail}) \times (q_h)] \text{ m}^3\text{/s} \qquad \textbf{(7.2)}$$

$$= [(37) \times (0.055)] \text{ L/s} = \mathbf{2.04 \text{ L/s}}.$$

STEP 6 :

Report conclusions :

Q_R = 90%, as above,	= **11.3 L/s**;
Plan area of "soakaway"	= **37 m²**;
"soakaway" depth, H	**= 1.50 m**;
Slow-drainage flow, Q_{hA}	= **2.04 L/s.**

7.7.5 Example 7.2 : Infiltration surface without ponding

The case study used to illustrate this aspect of 'source control' of stormwater is based on the car park at St. Elizabeth Church, Warradale, SA, illustrated in Figure 7.3 (previously Figure 5.1), below.

Details of the catchment draining to the grassed ("Grasspave") hard-standing area located **within** the total car park area, are as follows (see Example 5.1) :

Total area of car park (40 spaces + carriageway) EIA = 1,220 m^2

site time of concentration, $t_C = 10$ minutes;

design storm intensity (ARI = 3-years, $F_y = 0.9$), $i = 50$ mm/h;

hence, design $Q_{peak} = 0.014$ m^3/s.

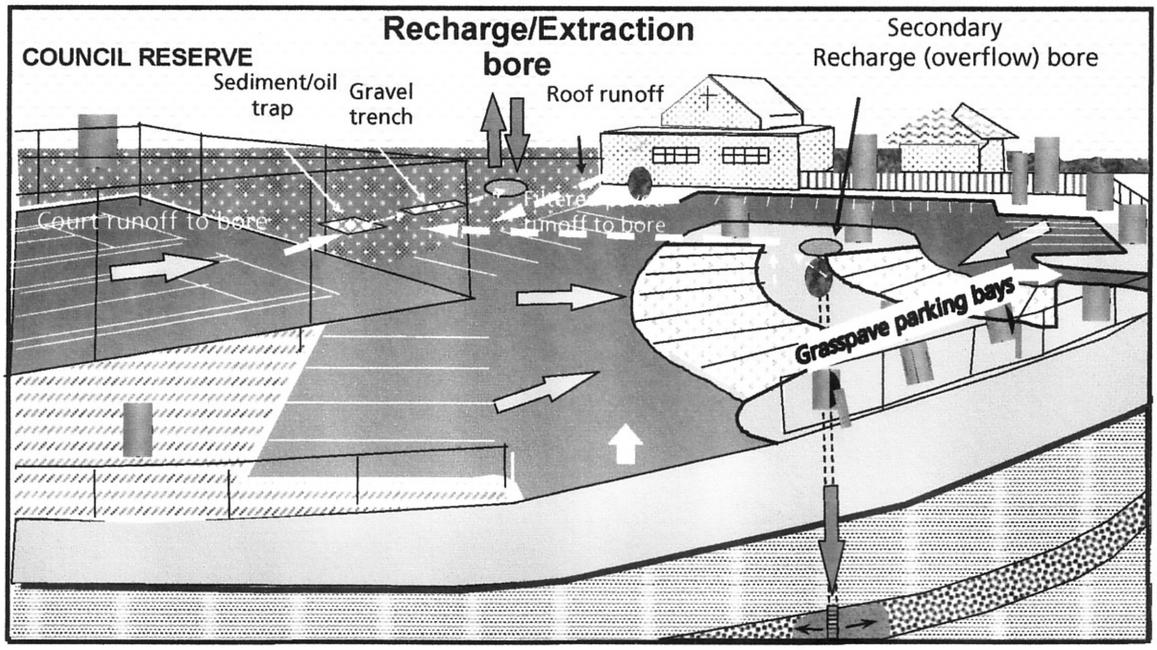


FIGURE 7.3 : General layout of car park at St. Elizabeth Church, Warradale, SA

Procedure 1B (Section 5.1.2) : infiltration surface (without ponding) accepting runoff from total area, A m²; the infiltration surface is *internal,* that is located *within* the area set aside for the car park . Infiltration surface area A_S = 407 m^2 (see Example 5.1, Section 5.2).

With plan area (equivalent to A_{avail} in the context of Procedures 5A and 6A), determined, the next step is to find the depth, H, of "soakaway" required **beneath** the treatment surface to accommodate cleansed storm runoff. This involves Procedure 5A :

Location : Warradale, Southern Australia;

Soil : medium clay, $k_h = 3 \times 10^{-6}$ m/s**;** Moderation factor, U = 2.0;

Catchment : paved area (total), $A_{EIA} = 1{,}220$ m^2; it should be noted that although one-third of the total car park area is grassed, storm 'runoff' into the gravel-filled sub-structure beneath this component (by infiltration) will be close to 100%. It therefore behaves, effectively, as an impervious surface.

Space available : $A_{\mathbf{avail}} = 407$ m^2; hence, $A_R = 0.33$;

Retention device : gravel-filled "soakaway", $e_S = 0.35$.

Retention efficiency, R = 90%.

STEP 1 (Procedure 5A) :

Moderated hydraulic conductivity :

$$k_h = (3 \times 10^{-6}) \times U = 3 \times 10^{-6} \times 2.0 = 6 \times 10^{-6} \text{ m/s};$$

Plan space ratio, $A_R = 0.33$;

Locate in Figure 7.1 (**above** point "1" on graph used in Example 1a).

This corresponds to a "soakaway" depth significantly **less** than H_{min}. Adjust "$A_{\mathbf{avail}}$" to value corresponding to depth, $H = H_{min} = 0.30$ m; (see 'shift' in Figure 7.1 to $A_R = 0.08$);

hence, (new) $A_{avail} = 0.08 \times 1{,}220$ m^2 = 98 m^2.

STEP 2 : Case 1

"Soakaway" depth, H = 0.30 m, from STEP 1.

STEP 3 :

Report conclusions :

$$q_{lim} = 0.005 \text{ L/s per m}^2 \text{ of } A_{EIA},$$

hence, treatment system capacity flow :

$$Q_{lim} = 0.9 \times A_{EIA} \times q_{lim} \text{ L/s} \qquad \textbf{(7.1)}$$

$$= 0.9 \times 1{,}220 \times 0.005 = \mathbf{6.1 \text{ L/s};}$$

Plan area of "soakaway" = **98 m²**;

"soakaway" depth, H = **0.30 m.**

The practical implications of this outcome are :

- 407 m^2 of "Grasspave" surface must be provided to meet the storm runoff (quantity) control requirements, and to ensure the longevity of the pervious surface in the face of sediment accumulation;
- the volume required for storage of cleansed runoff (Strategy A requirement) delivered by the treatment surface to the in-ground "soakaway" (0.30 m deep), can be readily accommodated, occupying only **one-quarter** of the area of the treatment surface;
- the Q_{lim} capacity flow for Strategy A compliance – 6.1 L/s – is easily accommodated by the "Grasspave" treatment surface which has a capacity of more than 18 L/s;
- it is interesting to note that stormwater (quantity) control at the car park site (design computations are presented in Example 5.5, Section 5.2) requires a gravel-filled "soakaway", 0.50 m deep, with bore, under one-third of the "Grasspave" hard standing area, but attention should be given to "Important additional note", Example 5.5.

7.7.6 Example 7.3 : "Soakaway" at Plympton Anglican Church

Stormwater runoff from a large paved area and from six buildings at the Plympton Anglican Church in Adelaide, South Australia, passes, after cleansing, into a "soakaway" constructed from a combination of gravel and perforated pipes illustrated in Figure 7.4. Details of the catchment draining to the retention installation are as follows (see Example 5.3, Section 5.2) :

site area (total)	2,640 m^2
total connected roof areas	1,600 m^2
total connected paved area	360 m^2
total connected pervious areas (C_{10} = 0.10)	680 m^2
site time of concentration, t_C	10 minutes
critical storm duration (Council specification)	30 minutes
design storm intensity [ARI = 100-years] (Council specification)	73 mm/h

Procedure 2C (see Section 5.1.3) : "soakaway" part-occupied with impervious material, pipes, etc., with cleansed water inflow : required area, a = 406 m^2 (see Example 5.3, Section 5.2).

[This example is parallel to Example 5.3, which is focussed on flood control design under the regime-in-balance strategy (Section 4.2.1). The present design review for pollution control, employs the yield-minimum strategy, even though regime-in-balance is specified for stormwater quantity management on the site. This apparent contradiction is explained in Section 7.1.]

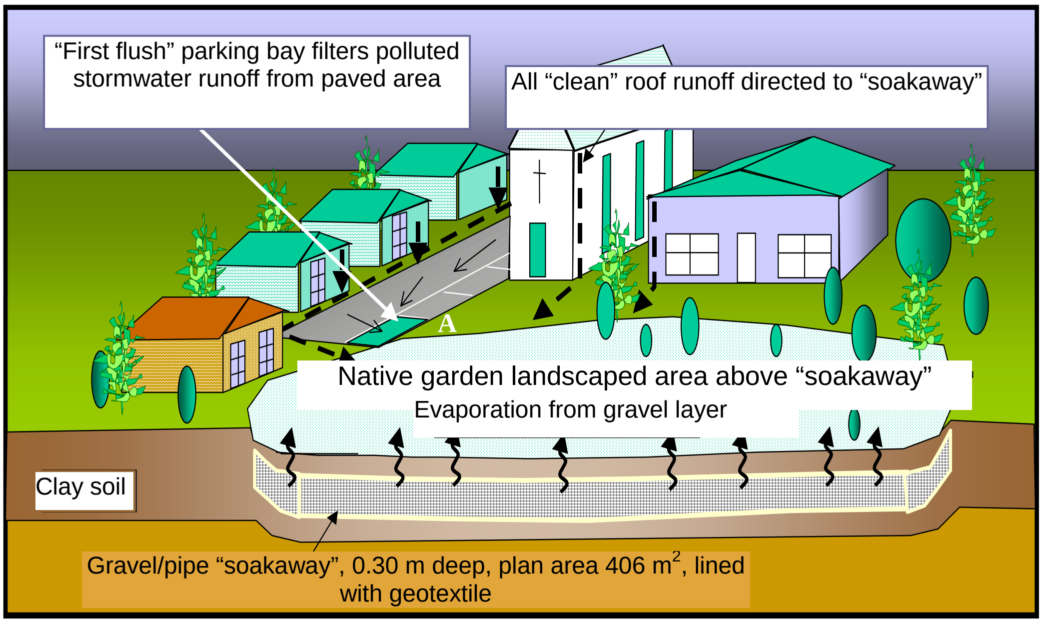


FIGURE 7.4 : General layout of site and "soakaway" at Plympton Anglican Church.

With plan area a = 406 m^2 (equivalent to A_{avail} in the context of Procedures 5A and 6A), determined, the next step is to check the design to ensure that it complies with Strategy A pollution control/retention requirements under Procedure 5A :

Location : Plympton, Southern Australia;

Soil : medium clay, $k_h = 1 \times 10^{-6}$ m/s**;** Moderation factor, U = 2.0;

Catchment : paved area, $A_{EIA} = 2{,}028\ m^2$;

Space available : $A_{\mathbf{avail}} = 406\ m^2$; hence, $A_R = 0.20$;

Retention device : gravel/pipe "soakaway", $e_S = 0.50$.

Note : Procedure 5A is followed using information from Figure 7.1, arriving at a solution for the gravel-filled "soakaway" case. The conversion described in Section 7.6.3 is then carried out enabling the final check/comparison to be made for the pipe/gravel installation.

STEP 1 (Procedure 5A) :

Moderated hydraulic conductivity :

$$k_h = (1 \times 10^{-6}) \times U = 1 \times 10^{-6} \times 2.0 = 2 \times 10^{-6} \text{ m/s};$$

Plan space ratio, $A_R = 0.20$;

Locate in Figure 7.1 (**above** the curve for H_{min}).

This corresponds to a "soakaway" depth significantly **less** than H_{min}. Therefore adjust "$A_{\mathbf{avail}}$" to value corresponding to depth, $H = H_{min} = 0.30$ m;

hence, (new) $A_{avail} = 0.12 \times 2{,}028\ m^2 = 243\ m^2$.

STEP 2 : Case 1

"Soakaway" depth, H = 0.30 m, from STEP 1.

STEP 3 :

Report conclusions :

q_{lim} = 0.005 L/s per m^2 of A_{EIA},

hence, treatment system capacity flow : $Q_{lim} = 0.9 \times A_{EIA} \times q_{lim}$ L/s **(7.1)**

= $0.9 \times 2{,}028 \times 0.005$ = **9.1 L/s;**

Plan area of "soakaway" = **243 m^2**;

"soakaway" depth, H = **0.30 m.**

Conversion step (see Section 7.6.3) :

The dimensions reported in STEP 3, above, apply to a Strategy A gravel-filled installation requiring inlet treatment flow capacity, Q_{lim} = 9.1 L/s, plan area equal to 243 m^2 (or greater) and depth, H = 0.30 m. The conversion, Eqn. (7.4), enables the gravel-filled depth to be converted to depth, H, for a pipe/gravel "soakaway" equal to :

$$H_{new} = 0.3 \times [0.35/0.50] = 0.21 \text{ m} \qquad \textbf{(7.4)}$$

The practical implications of this design are :

- 406 m^2 of pipe/gravel "soakaway" of depth 0.30 m required for flood control (Example 5.3, Section 5.2) easily embraces the requirements for Strategy A, pollution control compliance, exceeding those requirements in terms of both plan area and device depth;
- the Q_{lim} capacity flow required for Strategy A compliance – 9.1 L/s – provides the basis for selecting/designing a treatment/filter unit or system to be located upstream of the in-ground device. In the case of the installation at Plympton, this takes the form of a "Grasspave" treatment area located at the end of the driveway; it can easily accommodate a flow of 9.1 L/s.

7.7.7 Example 7.4 : Design of Strategy B System

Stormwater runoff from a residential sub-division, plan area 3.0 ha, for which equivalent impervious area, A_{EIA} = 1.50 ha, is required to be treated to Strategy B standard, that is **'first flush'** treatment, only (Council specification). Local soil is sandy-clay, so a "dry" pond appropriate to the pollution control/retention requirements of Council is to be designed.

Location : Newcastle, NSW;

Soil : sandy-clay, $k_h = 3 \times 10^{-5}$m/s**;** Moderation factor, U = 1.0;

Catchment : paved area, A_{EIA} = 15,000 m^2 ;

Time of concentration : t_C = 20 minutes;

Rainfall intensity : $i_{0.25} = 0.5 \times i_1$ **(3.38)**

= 0.5×47.7 mm/h = 24 mm/h

Space available : A_{avail}= 250 m^2;

Retention device : infiltration or 'dry' pond.

The design procedure is similar to Procedure 5B :

STEP 1 :

Determine runoff **volume** from catchment in the ARI = 0.25-year storm, duration, t = 20 minutes;

$\forall \quad = C \times A_{EIA} \times 0.50 i_1 \times t$

$\quad = 0.9 \times 15{,}000 \times (24/1{,}000) \times (20/60) = 108 \text{ m}^3.$

STEP 2 :

Determine **depth** of infiltration pond, d , from :

$$d = \frac{\forall}{A_P} - 60 k_h . \tau U + \frac{i(\tau - t_c)}{6 \times 10^4} \qquad \textbf{(5.6b)}$$

where $\forall$ = 108 m^3;

A_p = 250 m^2;

k_h = (3×10^{-5}m/s); U = 1.0;

τ = $2t_C$ = 40 minutes, in this case;

$i_{0.25}$ = 24 mm/h

hence, **d = 0.37 m**

Check emptying time :

$$T \approx \frac{2H.e_s}{k_h}, s \qquad \textbf{(3.32)}$$

where H = d = 0.37m; e_S = 1.0;

hence, T = 6.9 hours – satisfactory for ARI, Y = 0.25 years (see Table 3.3).

Report :

‘Dry’ pond treating storm runoff from residential sub-division, area 3.0 ha, to Strategy B standard :

pond area, A_p = **250 m^2**;

pond depth, d = **0.37 m**;

emptying time in ARI, Y = 0.25 years event :

T = 6.9 hours.

Note: This design approach might be considered – in appropriate (soil) circumstances - as a ‘lesser’ alternative to the Filter Strip Swale option reviewed in the remaining sections of this chapter.

7.8 'FILTER-STRIP' SWALES : GENERAL BACKGROUND

7.8.1 Introduction

Swales are shallow grassed channels – typically 0.30 to 0.50 m (maximum) deep, 5 to 6 m wide in residential streets – with longitudinal slopes less than 3% (see Chapter 2). They have wide application in water-sensitive urban design for three main reasons :

1. they can be instrumental in retaining runoff through bed infiltration;
2. they can be effective in retaining pollutants conveyed in stormwater; and,
3. they can fulfil a role in stormwater harvesting through soil moisture enhancement and, possibly, aquifer recharge and recovery (see Figure 2.4).

The full potential of swales therefore includes each of the primary goals of stormwater 'source control' as defined in Section 1.3. Clearly, this scope is very great and, in a handbook such as this, warrants an entire chapter : the following treatment is confined to 'filter strip' swales which extend Procedures 5 and 6 into a realm of stormwater management that presents WSUD with one of its greatest challenges – that of controlling pollution in residential streets (see Section 1.4.3). In terms of the four systems defined in Section 3.4.2 to which 'continuous simulation' modelling is applicable, 'filter strip' swales are closest to Number 2 – "formal inlet control (by design); storage, including on-site disposal; and overflow". Because of their primary role – indeed their **only** 'source control' role – as pollution reservoirs, 'filter strip' swales earn a place among Category 2 systems described in Section 4.1 (see Section 7.6.2, above).

Like the bulk of catchment components embraced by Procedures 5 and 6, 'filter strip' swales handle flows exceeding Q_{lim} essentially as bypass. 'Filter strip' swales can therefore be expected to provide treatment for approximately 95% of average annual runoff (see Section 4.3.1). The configuration of a 'filter strip' swale in relation to a residential street carriageway is shown in Figure 7.5.

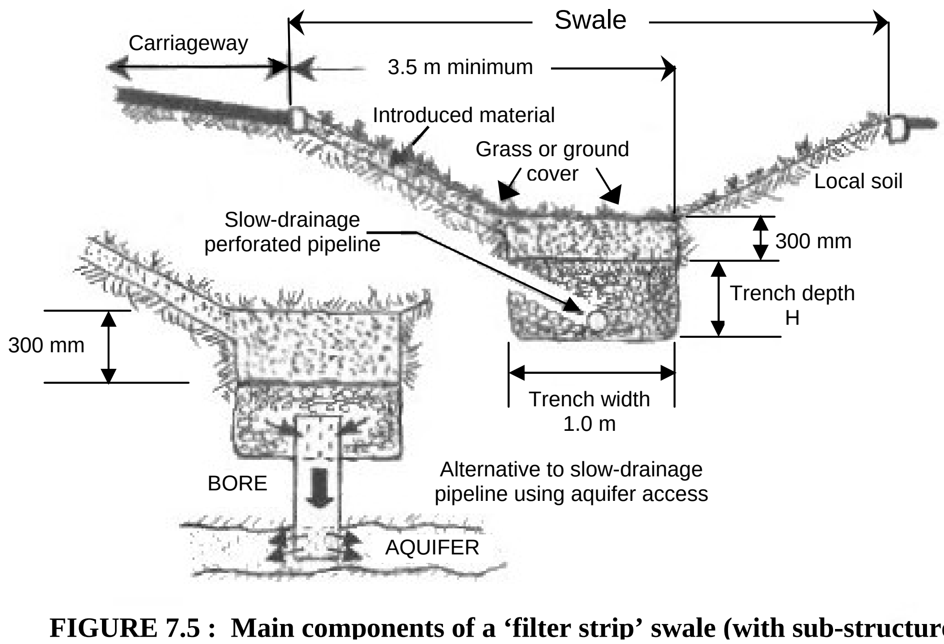


FIGURE 7.5 : Main components of a 'filter strip' swale (with sub-structure)

Swale systems of the type examined in this section abstract all flows up to the Q_{lim} limit – by design – and bypass all exceedances as open channel flow conveyed downstream within the boundaries of the swale, or perhaps channelled to the site of a 'dry' pond located within the streetscape (see final Note, Example 7.4). The process of abstraction is achieved through infiltration alone or by infiltration combined with sub-structure retention (gravel-filled trench or similar), and 'hydraulic' disposal to aquifers or local waterways (slow-drainage) if necessary. Application of the design procedures described earlier in this chapter to the particular case of 'filter strip' swales warrants the creation of two new design methods – Procedures 7 and 8 – parallel to Procedures 5 and 6 respectively. Before presenting details of the new procedures, however, it is appropriate to review the range of installations that result from their application.

7.8.2 ‘Filter-strip’ swales : general description

There are three broad types of ‘filter strip’ swales :

(i) Swales whose cross-section includes a surface layer of sand and gravel mixture (100 mm thick) laid from the contributing carriageway edge to and across the bed of the excavated channel;

(ii) Swales whose cross-section includes the layer of introduced material described in (i), above, plus a gravel-filled trench located beneath the swale invert;

(iii) Swales whose cross-section includes the layer of introduced material and gravel-filled sub-structure, described in (i) and (ii), above, plus ‘hydraulic’ means of removing/conveying accumulated, cleansed stormwater either :

a) by direct disposal/recharge to a conveniently located aquifer; or,

b) by access to a ‘slow drainage’ perforated pipeline located in the base of the gravel-filled trench.

In this case the WARNING contained in Section 7.3.1 relating to the likely presence of *significant dissolved pollution* should be heeded and design limited to the scope provided by types (i) and (ii), above, only.

These components are illustrated in the compound ‘filter strip’ installation shown in Figure 7.5.

The use of these components as (i) only, or (i) and (ii) only, or all three, depends on the following factors :

- climate zone in which the swale is located;
- soil properties of the site;
- streetscape catchment area draining to the swale;
- availability of suitable aquifer (depth, permeability etc.) to receive recharge;
- availability of suitable receptor for cleansed stormwater.

It is possible to prepare a comprehensive step-by-step procedure similar to those given for Procedures 5 and 6, but adapted for application to the particular case of ‘filter strip’ swales. However, this approach has been rejected in favour of information derived from ‘continuous simulation’ modelling, enabling ‘filter strip’ swales to be designed for the great bulk of streetscape scenarios. This information is presented in a set of graphs appropriate to the five climate zones identified in Figure 3.8 and covering the range of native soils found in these zones with hydraulic conductivities from $k_h = 1 \times 10^{-7}$ to $k_h = 1 \times 10^{-3}$ m/s, inclusive.

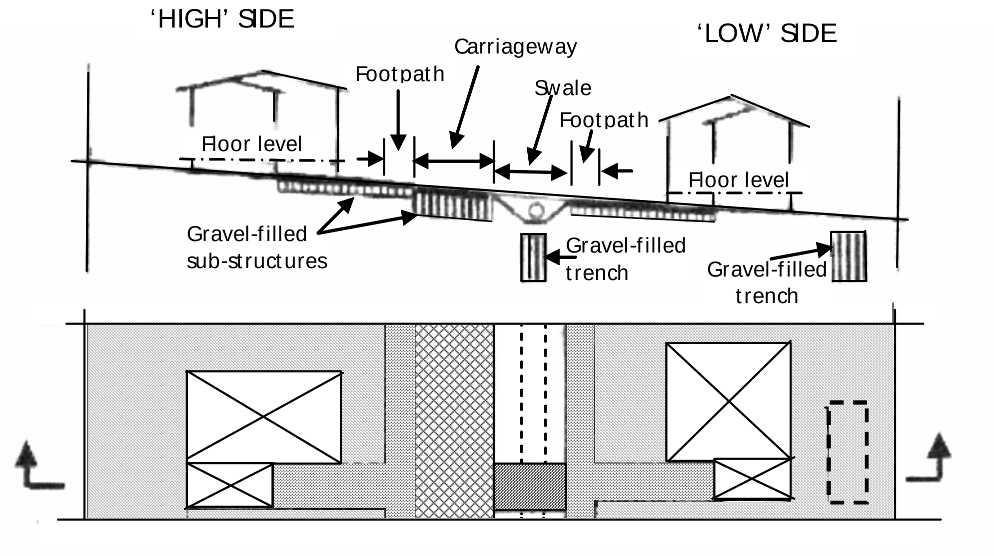
7.8.3 Catchment area

The basic information (apart from climate zone and soil permeability) that the designer must bring to the task of planning a ‘filter strip’ swale using the graphs is : **catchment area (expressed as equivalent impervious area) draining to a typical 15 m segment of the proposed swale.** This raises some important issues about the management of storm runoff in the residential streetscape for which the design is being prepared. Figure 7.6 illustrates a strip of residential neighbourhood in which allotments are **15 m wide.** [In cases where average frontages are other than 15 m wide, a factor must be applied to reduce them to typical or ‘standard’ 15 m swale segments in order to use the graphs directly.]

Every effort should be made to minimise the area of paving directly draining to the swale and, hence, the quantity of runoff generated in the typical 15 m segment.

Measures which might be taken to achieve this include (see Figure 7.6) :

- backyard ‘soakaways’ in allotments remote from the swale streetscape but *up-slope* from it;
- permeable paving for allotment driveways including crossings;
- permeable paving for footpaths;
- permeable paving for the carriageway.



15 m wide segment of residential street with swale

Within the segment there are six different surface covers or land uses :

- roof areas (house, garage) .... shown
- allotment driveways and other outdoor 'hard' surfaces (possible porous/permeable paving) .... shown
- pervious outdoor areas .... shown
- footpath (possible permeable paving) .... shown
- carriageway (possible permeable paving).. shown
- paved 'crossing' within swale (possible permeable paving)................ shown

The remaining area within the segment is taken up by the swale.

In addition to these surface covers or land uses are sub-structures as follows :

- below allotment driveways and other 'hard' surfaces :
  shallow, gravel-filled trench encased in geotextile .... shown
- below footpath contiguous with driveway & carriageway on 'high' side; and below footpath (isolated) on 'low' side :
  shallow, gravel-filled trench encased in geotextile .... shown
- below carriageway on 'high' side :
  deep, gravel-filled trench encased in geotextile .... shown
- below invert of swale :
  deep, gravel-filled trench encased in geotextile .... shown
- 'Low' side roof runoff directed to deep, gravel-filled trench encased in geotextile .... shown

FIGURE 7.6 : Swale streetscape segment with full stormwater retention options excluding rainwater tanks (see alternative layout in Figure 2.4)

The presence of rainwater tanks in such developments further assists the process of reducing the overall equivalent impervious area. It is difficult to satisfactorily quantify this in a given situation : it is therefore acceptable to consider this element as a 'margin of safety'. By following this approach it is found that typical, medium-density residential streetscapes, yield **equivalent impervious areas between 300 m^2 and 500 m^2 per 15 m segment**. [This includes the *entire* filter strip area itself and the filter strip portion of the crossing (see Figure 7.5), which although partly pervious and partly paved, absorbs the full rainfall input and, therefore, acts as though it were fully paved.] The swale layouts which can be produced from the design graphs cover this range.

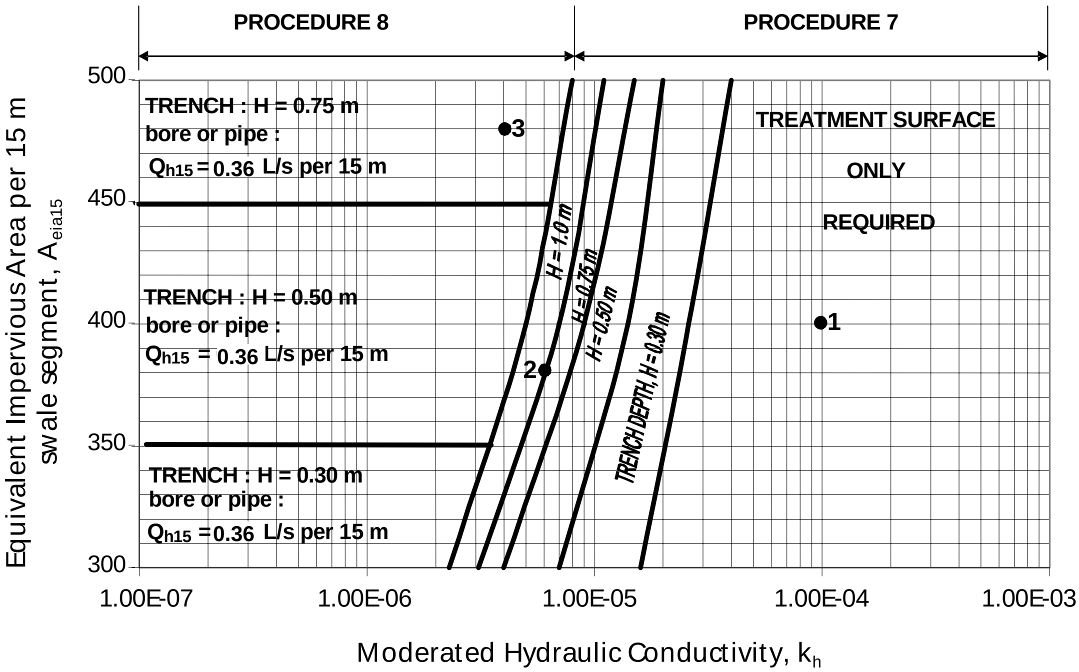


FIGURE 7.7 : Graph for design of 'filter strip' swales – Southern Australia ($i_{10.1}$ rainfall intensity range: up to and incl 25 mm/h). Soils with moderated k_h > 1.0 × 10^{-4} m/s are, typically, unsuitable for treating *dissolved pollutants* in catchment runoff (see Mikkelsen et al, 1997; Fischer et al, 2003)

7.8.4 Procedures 7 and 8 : introduction

The focus of Procedures 7 and 8 in any design task is the graph which has been prepared for the location's climate zone. This is selected, as in Procedure 5A, on the basis of the $i_{10,1}$ value for the site, and its identification with one of the five climate zones illustrated in Figure 3.8.

The graph for the Southern Australia zone is Figure 7.7; graphs for the other zones are presented in Appendix D. Each graph (Catchment EIA *vs* soil moderated hydraulic conductivity) comprises three distinct areas (see Figure 7.7) :

- Swales which require a 100 mm layer of introduced sand/gravel, indicated as **treatment surface only** in each graph;
- Swales which require surface treatment and a gravel-filled (trench) sub-structure. The **width** of trench is 1.0 m in every case; the **depth** is read from the family of curves covering the range 0.3 m (minimum) to 1.0 m (maximum).

The information presented in these two areas of each graph is pertinent to the application of Procedure 7. The information derived from application of Procedure 8 is also shown in the graph and is necessary for :

- Swales which require surface treatment, and gravel-filled trench, as well as provision for disposal of cleansed stormwater into an aquifer, if available, or to a receiving drainage channel or waterway. The quantity of flow (L/s) per 15 m segment of swale (to be conveyed) can be read from the 'bands' set out under "Procedure 8" in each graph. **Use of Procedure 8 requires that *significant dissolved pollution* is absent (see Section 7.8.2).**

7.8.5 Specification

The **carriageway** delivering flow to the swale should show a fixed cross-slope towards the swale (no crown). A uniform longitudinal slope is not required, but changes in (longitudinal) grade – positive and negative, associated with 'dips' – should be avoided. The lower edge of the carriageway should be formed as a continuous concrete overflow kerb.

Introduced treatment surface : This component is responsible for filtering all surface runoff passing to the swale from the streetscape catchment. It is a mixture of coarse sand (propagating sand, used in the building industry, is preferred) and single-size gravel (20 mm to 30 mm) in a 100 mm thick layer underlain with non-woven geotextile fabric and **placed at optimum moisture content**. It is permissible for this mixture to be enriched with a small quantity of loam providing the base for drought-tolerant vegetation (grass or ground cover).

The minimum length of the treatment surface is 3.5 m measured from the edge of the roadway overflow kerb to the remote edge of the bed of the swale (see Figure 7.5). The grade of the introduced surface is typically 1 in 5 to 1 in 8. This grade as well as the length of the swale side-slope itself may be determined by conveyance capacity considerations: the swale must be designed to carry flood flows corresponding to ARI, Y = 50- to 100-years events in the streetscape catchment (see Section 7.8.9).

The swale's side-slope remote from the carriageway plays no part in the filtering process. Its grade may match that of the near-carriageway slope or it may be steeper depending on channel geometry required for flood conveyance. It is permissible in appropriate circumstances for the remote slope to be steep and take the form of a "rockery" containing suitable plants and shrubs. This could be a useful and attractive option in circumstances where the space available for a swale is severely limited.

The graphs which have been prepared for use in Procedures 7 and 8 in Australian climate zones (Figures 7.7 and D1 – D4 in Appendix D), are based on treatment surface area of 42 m^2 corresponding to 12.0 m $\times$ 3.5 m (see Figure 7.5). This allows for a 3.0 m-wide driveway crossing in each 15 m length of swale segment, the most extreme case that is likely to occur.

Aspects of treatment surfaces reviewed above – slopes, geometry, construction, etc. – are of less importance to the success of a 'filter strip' swale than the landscaping and horticultural expertise which must be available. Indeed, it is recommended that 'filter strip' swales be ignored as a possible stormwater 'source control' measure unless this expertise including knowledge of local native and/or drought-tolerant vegetation is readily accessible. Even where these conditions of opportunity and expertise are met, it is probably best to 'hasten slowly' with the technology and gain experience with a relatively small demonstration section of swale before applying it in a wholesale manner.

The harsh growing environment presented by introduced sand/gravel as a base for vegetation, particularly in the semi-arid climate zone, or during prolonged dry spells in temperate Australia, calls for grass or ground cover capable of good performance in these conditions without continuing irrigation after establishment. Three ground covers – *Marsillea drumondii, Centella asiatica* and *Viola hederacia* – and two wetland reeds *Juncus kraussii* and *Typha orientalis* have demonstrated the required robust capabilities in Southern Australia and may give similar satisfactory performance in the wetter climatic environments of Intermediate and Northern Australia. Of course, locally available drought-tolerant species of grasses and ground covers should be sought to provide the vegetation cover required of 'filter strip' swales.

Gravel-filled (trench) sub-structure : This component is similar to the gravel-filled trenches reviewed among the Category 1 installations in Chapter 5 and illustrated in Figures 1.3, 1.4 and 1.5. The gravel should be sharp-edged and of uniform size, typically, 20 mm – 30 mm nominal size, encased in non-woven geotextile fabric. All trenches specified in the design graphs are 1.0 m wide. The depths covered in the design graphs (Figures 7.7 and D1 – D4, Appendix D) are 0.30 m, 0.50 m, 0.75 m, and 1.0 m : these are minimum depths for the particular design conditions in each case.

'**Hydraulic' cleansed stormwater disposal** : As explained above, two options may be explored for this component :

(a) access (recharge) to an aquifer; or,

(b) 'slow-drainage' to a local formal or natural waterway.

This option is required throughout Australia in all cases where native soils show very low permeability; in Northern Australia the range of requirement extends to soils of medium permeability. The flow given by Procedure 8 in the design graphs (Figures 7.7 and D1 – D4, Appendix D), is expressed in terms of **L/s per 15 m length of swale segment**. The lengths of typical streetscapes where 'filter strip' swales may be used represent multiples of 15 metre segments, side-by-side. The **total** flow which must be managed in such cases is therefore equal to the q_{h15} flow (per 15 m swale segment) multiplied by the number of 15 metre segments : this gives Q_{hA}. Where aquifer access is available and environmentally appropriate, the number of bores required and hence the spacing of bores along the swale alignment should be determined from a knowledge of the bore recharge rate, q_r. Hence, the number of bores :

$$n \geq \frac{Q_{hA}}{q_r}\ , \qquad \text{where "n" is an integer.} \qquad \textbf{(7.5)}$$

In the case of a slow-drainage pipeline draining cleansed stormwater to a formal or natural drainage water-way outside the streetscape, the perforated pipeline capacity at its discharge point should be at least Q_{hA}. The flows involved are relatively small compared to flows normally experienced in storm drainage networks and are unlikely to be greater than 120 L/s. 225 mm to 300 mm PVC (perforated) pipes enclosed in non-woven geotextile fabric are suitable for this duty.

7.8.6 'Lifespan'

Stormwater generated in the catchment draining to a swale will convey runoff from a number of surfaces including roof areas, residential paved surfaces, footpath, carriageway, pervious areas, etc., some of which are relatively clean and others represent potential sources of pollution, particularly sediment. This latter component of runoff determines the speed with which the treatment surface (introduced material) blocks, leading, ultimately, to the need for its replacement or reinstatement. The term 'lifespan', introduced in Section 3.1.2, is applied to the time taken for progressive blocking to reach finality.

This process, as it takes place in 'filter strip' swales, is similar to the "blockage front" phenomenon described in Section 3.1.3 and quantified in Table 3.1. Information given in that table can be used to estimate 'lifespan' in these swales. It is related to :

- the **ground-level** impervious area, I, draining to a porous (or permeable) surface, such as a 'filter strip' swale;
- the area, P, of the filter strip receiving the runoff; and,
- the particular environmental conditions of the site – proximity to trees, wind-driven sand supply to the site from beaches, etc.

The first and second of these variables are expressed in the ratio (I/P). It is important to note that the value of "I" used in this ratio is **not** the same as A_{eia15} which includes roof areas. Also, "I" does not include ground-level areas devoted to porous or permeable paving.

The area of swale receiving runoff in all cases modelled to produce the graphs (Figures 7.7 and D1 – D4) is fixed at 42 m^2 (3.5 m $\times$12.0 m) per 15 m segment, so that ratios of (I/P) may range from 1.0 or less, where permeable paving is used extensively up to 6.0 or more where little consideration, if any, has been given to retaining runoff from ground-level paved surfaces. Estimates of 'lifespan' can be made from Table 3.1 which shows a "times 5" factor for vegetated porous surfaces. In fact, the field observations at St. Elizabeth Church Car Park, Adelaide (see Section 3.9) which led to the recommended factor reported no significant 'blocking' after 6 years of service in an (I/P) = 40 environment! Use of the "times 5" factor (Table 3.1) leads to 'lifespan' estimates ranging from about 15 years up to 50 years in typical cases. Based on experience at St. Elizabeth Church, these estimates are likely to be conservative.

It is important for practitioners to realise that 'lifespan' values quoted here are derived from data for **established neighbourhood** conditions only. Poor monitoring/control of construction phase activities can reduce 'lifespan' to a fraction of the values indicated in the above discussion (see Section 1.3.3).

7.8.7 Procedures 7 and 8 and Figure 7.7 – Illustrations

ILLUSTRATION 1 – APPLICATION OF PROCEDURE 7

Consider the requirements for a ‘filter strip’ swale in the following circumstances :

- Location : Adelaide, $i_{10,1}$ = 25 mm/h, therefore Southern Australia zone – Figure 7.7.
- Local soil hydraulic conductivity, $k_h = 2 \times 10^{-4}$ m/s (sand); Moderation Factor, U = 0.5.
- Catchment EIA : 400 m^2 per 15 m length of swale segment, A_{eia15}.

Hence, Moderated hydraulic conductivity :

$$k_h = (2 \times 10^{-4}) \times U = 2 \times 10^{-4} \times 0.5 = \mathbf{1.0 \times 10^{-4}\ m/s};$$

Catchment equivalent impervious area :

$$A_{eia15} = \mathbf{400\ m^2}.$$

Locate this data pair in Figure 7.7 at ‘1’.

Interpretation : The ‘filter strip’ swale requires treatment surface only comprising sand/gravel mixture, 100 mm deep, installed in the swale slope adjacent to the roadway reserve carriageway and extending 3.5 m (minimum) across the bed of the swale (see Figure 7.5).

ILLUSTRATION 2 – APPLICATION OF PROCEDURE 7 :

- Location : Adelaide, $i_{10,1}$ = 25 mm/h, therefore Southern Australia zone – Figure 7.7.
- Local soil hydraulic conductivity, $k_h = 3 \times 10^{-6}$ m/s; Moderation Factor, U = 2.0.
- Catchment EIA : 380 m^2 per 15 m length of swale segment, A_{eia15} .

Hence, Moderated hydraulic conductivity :

$$k_h = (3 \times 10^{-6}) \times U = 3 \times 10^{-6} \times 2 = \mathbf{6.0 \times 10^{-6}\ m/s};$$

Treatment equivalent impervious area :

$$A_{eia15} = \mathbf{380\ m^2}.$$

Locate this data pair in Figure 7.7 at ‘2’.

Interpretation : The ‘filter strip’ swale requires treatment surface as described in Illustration 1, above, with gravel-filled trench 1.0 m wide (all cases) and H = **1.0** m (from graph).

ILLUSTRATION 3 – APPLICATION OF PROCEDURE 8 :

- Location : Adelaide, $i_{10,1}$ = 25 mm/h, therefore Southern Australia zone – Figure 7.7.
- Local soil hydraulic conductivity, $k_h = 2 \times 10^{-6}$ m/s; Moderation Factor, U = 2.0.
- Catchment EIA : 480 m^2 per 15 m length of swale segment, A_{eia15} .

Hence, Moderated hydraulic conductivity :

$$k_h = (2 \times 10^{-6}) \times U = 2 \times 10^{-6} \times 2 = \mathbf{4.0 \times 10^{-6}\ m/s};$$

Catchment equivalent impervious area :

$$A_{eia15} = \mathbf{480\ m^2}.$$

Locate this data pair in Figure 7.7 at ‘3’.

Interpretation : This ‘filter strip’ swale requires treatment surface as described in Illustration 1, above, with gravel-filled trench 1.0 m wide and H = 0.75 m, and ‘hydraulic’ (additional) disposal at the rate of 0.36 L/s per 15 m swale segment, q_{h15}.

7.8.8 ‘Filter-strip’ swales in Australian environments

Application of Procedures 7 and 8, as set out above, in the five climate zones recognised in Figure 3.8 leads to a mix of swale outcomes. The ‘mix’ is determined not only by climate variation but also by the soil type in which the ‘filter strip’ swale is located and the presence or absence of aquifers.

Table 7.1 lists the likely outcomes for three of these zones for streetscape swales designed according to Procedures 7 and 8. It should be borne in mind that the design approach which underpins these procedures is **Strategy A**, described in Section 4.3.1. This represents a very high standard of pollution containment and carries with it the promise of relatively long ‘lifespan’, as reviewed above, before filter strip reinstatement might be required. This excludes regular inspection and maintenance of the treatment surface to keep it functioning including mowing, attending to ground cover and repairing scour channels or sheet erosion whenever these occur. The sub-structure system of gravel-filled trench and bore or slow-drainage pipeline – where these components are included – is virtually isolated from the impact of sediment and, therefore, can be expected to give service of indefinite length.

TABLE 7.1

‘FILTER STRIP’ SWALES PROVIDING A HIGH LEVEL OF POLLUTION CONTROL (STRATEGY A) IN SOUTHERN, MID-INTERMEDIATE AND NORTHERN AUSTRALIA

<table>
<tr><th colspan="2" rowspan="2">PARENT SOIL TYPE</th><th colspan="3">ZONE</th></tr>
<tr><th>SOUTHERN AUSTRALIA</th><th>MID-INTERMEDIATE AUSTRALIA</th><th>NORTHERN AUSTRALIA</th></tr>
<tr><td rowspan="2">SANDY SOILS</td><td>Coarse*</td><td rowspan="2">Filter strip only required (no sub-structure)</td><td>Filter strip only required (no sub-structure)</td><td>Filter strip and trench sub-structure, 1.0 m deep</td></tr>
<tr><td>Fine</td><td>Filter strip and trench sub-structure, 1.0 m deep</td><td>Filter strip, trench sub-structure, 1.0 m deep with ‘hydraulic’ assistance, Q_{h15} = 3.6 L/s**</td></tr>
<tr><td colspan="2">SANDY CLAYS</td><td>Filter strip and trench sub-structure, 1.0 m deep</td><td>Filter strip, trench sub-structure, 1.0 m deep with ‘hydraulic’ assistance, Q_{h15} = 1.6 L/s**</td><td>Filter strip, trench sub-structure, 1.0 m deep with ‘hydraulic’ assistance, Q_{h15} = 8.0 L/s**</td></tr>
<tr><td colspan="2">ALL CLAYS</td><td>Filter strip, trench sub-structure, 0.75 m deep with ‘hydraulic’ assistance, Q_{h15} = 0.36 L/s**</td><td>Filter strip, trench sub-structure, 1.0 m deep with ‘hydraulic’ assistance, Q_{h15} = 1.6 L/s**</td><td>Filter strip, trench sub-structure, 1.0 m deep with ‘hydraulic’ assistance, Q_{h15} = 8.0 L/s**</td></tr>
</table>

* See note re sites with coarse sand and significant ***dissolved pollution*** in Figure 7.7 and Appendix D.
** All ‘hydraulic’ assistance flows quoted are *per 15 m swale segment*. Warnings relating to injection of these flows into aquifers or waterways should be heeded (see Sections 7.8.2 and 7.8.4).

NOTE : Introduced filter strip sand/gravel mix must have hydraulic conductivity, k_h, *at least* 2.0×10^{-4} m/s. Propagating sand with 20 mm gravel is suitable for this duty.

Outcomes of particular interest from Table 7.1 are :

- successful ‘filter strip’ swales can be constructed without gravel-filled sub-structures or ‘hydraulic’ assistance, in sandy soils in much of the southern regions of the continent;
- filter strips with introduced sand/gravel and gravel-filled trenches up to 1.0 m deep, only, are required in the coarse sands of Northern Australia, the fine sandy soils of much of Intermediate Australia and in the sandy clays of Southern Australia;
- the full ‘filter strip’ swale structure, including treatment surface, gravel-filled sub-structure and ‘hydraulic’ assistance, is required in all other cases;
- the quantity of cleansed stormwater which must be removed (“hydraulic assistance”) from the base of the formal structure varies from 0.36 L/s per 15 m swale segment in Southern Australia up to 8.0 L/s per segment in Northern Australia.

7.8.9 Design for flows greater than Q_{lim}

The basis for design of ‘filter strip’ swales by Procedures 7 and 8 is focussed on control of flows up to and including, *nominally,* those of ARI, Y = 0.25 years and **bypass** of all flows greater than this limit. Such (bypass) flows appear from time to time as surface flows exceeding the infiltration capacity of the filter strip, and follow moderate to large storm burst rainfall. The **total** flood flow involved, Q_Y, can be managed in a swale by considering it as, effectively, two flows, partitioned as follows :

- flow equal to $Q_{0.25}$ managed within the filter strip and its sub-structure; and,
- a “gap” flow of ($Q_Y - Q_{0.25}$) confined to the swale channel itself (see Procedure 7, STEP 5, following).

The former component is an element of design, adequately catered for within Procedures 7 and 8. The latter (component) can be managed as open channel flow passing to some downstream receiving domain such as a local drainage path or waterway. Its conveyance characteristics – depth, velocity, etc., – may be determined by reference to Manning’s formula or some similar, recognised hydraulic procedure such as Izzard’s triangular flow formulation (Argue, 1986).

It will be found that swale channels 0.30 m deep can accommodate major flood “gap” flows (ARI, Y = 50- to 100-years) provided the street lengths are not excessive. In Southern Australia, residential street lengths up to 300 m can apply the technology; the corresponding limit in Northern Australia is about 200 m. Alternatively, it may be possible to collect the flow and pass it to a site set aside as a “dry pond” within the streetscape reserve or, perhaps, off-line : Procedure 3 (Section 5.1) could be applied to provide a satisfactory resolution to the problem of disposal in these circumstances.

7.8.10 Procedures 7 and 8 – the step-by-step process

The material presented above provides the basis for a step-by-step design process commencing with data requirements and climate zone selection leading to ‘filter strip’ swale design including sub-structure dimensioning and ‘hydraulic assistance’ flow detailing, where required, and also including hydraulic conveyance capacity considerations taking account of the flood flows for which the swale channel must be designed.

Preliminary considerations – site data, etc.

The procedure must commence with a clear description of the circumstances in which the design process is to take place. This involves identifying :

- The climate zone in which the streetscape is located.
- Components of the typical allotment and its portion of fronting roadway reserve draining to the swale. This list must identify, in particular, roof areas, paved surface areas and whether they are impervious, porous or permeable, pervious areas, etc.
- Physical properties of the streetscape – allotment frontage (typical), street length and longitudinal slope.
- Streetscape time of concentration, t_C. Hydrological (flood control) aspects of the design (STEP 5, below) come under the provisions of ‘site’ drainage (see Sections 4.2.4 and 4.2.6), in particular “entry works” which do not involve storage considerations. External critical storm durations – $T_{C(total)}$ and $T_{C(local)}$ – are therefore not relevant to swale design. Design intensity for t_C in the major storm set by local council requirements (ARI, Y = 50 to 100-years) should be listed.

PROCEDURE 7 – STEP 1 :

Determine equivalent impervious area of typical unit contributing runoff to the filter strip. This produces a total EIA for the typical allotment including a corresponding portion of the fronting roadway reserve and a portion of the swale corresponding to (allotment frontage length × 3.5 m), explained in Section 7.8.3. These data lead to determination of $\mathbf{A_{eia15}}$, the equivalent impervious area which impacts on the filter strip component of the swale.

STEP 2 :

Identify climate zone of site based on $i_{10,1}$ rainfall intensity and select appropriate design graph from Figure 7.7 (Southern Australia) or from Appendix D (Figures D-1 to D-4).

Determine moderated hydraulic conductivity : $k_h \times$ (Moderation factor), as previously – see Section 7.7.

Locate 'point' in selected design graph corresponding to moderated hydraulic conductivity (X-axis) and A_{eia15} (Y-axis). The 'point' will fall either within the range covered by Procedure 7 or within the scope of Procedure 8 (see Section 7.8.7).

STEP 3 :

Determine required components of the filter strip – the treatment surface (always) **and** gravel-filled sub-structure, if applicable – and their (minimum) dimensions if and only if the 'point' falls within the scope of "Procedure 7" on the graph. Illustrations 1 and 2, Section 7.8.7, demonstrate this process. The plan area of the treatment surface – **42 m^2** (minimum) – is universal and required in STEP 4, all cases.

However, if the 'point' lies within the domain of "Procedure 8", all **dimension** information indicated in the graph is bypassed at this stage and re-visited under STEP 6, below. The design process for these cases continues, however, through STEPS 4 and 5 which are parts of both Procedures 7 and 8.

STEP 4 :

Estimate the 'lifespan' of the filter strip. This requires area I to be determined from the basic case data. Area I is the sum of EIAs of all *ground-level* components delivering runoff to the filter strip : roof areas and areas of porous or permeable paving are specifically excluded (see Section 7.8.6). From these data, a value of the parameter (I/P) is calculated and a value for 'lifespan' interpolated from Table 3.1. It is important in this operation to take note of the factor "5 times" which applies to vegetated treatment surfaces receiving stormwater.

STEP 5 : Swale hydraulic capacity (flood flow) considerations

This step involves calculating the major flood flow (design ARI in the range Y = 50- to 100-years) which must be conveyed by the streetscape swale without surcharge, as set by council requirements. This can be achieved by calculating the flood flow peak passing from the streetscape considered as a 'site' case **not** involving storage (see above, this section). The peak flow is determined :

$$\mathbf{Q_Y} = \frac{(CA)\, i_Y}{0.36} \ \textbf{L/s} \qquad \textbf{with (CA) in hectares} \tag{7.6}$$

where
$$(CA) = \frac{A_{eia15} \times \text{the number of 15 m segments in the streetscape}}{10{,}000} .$$

The swale conveyance capacity is calculated using conventional open channel flow formulae such as Manning's Equation or Izzard's formulation (Argue, 1986). Q_Y must be significantly below the open channel (swale) capacity to ensure passage of the Y-years event without surcharge.

It should be noted that no allowance has been made in these calculations for flow abstracted into the filter strip during passage of a major flood in the streetscape. Account could be taken of this flow (nominally $Q_{0.25}$), but its omission is intentional and represents a 'margin of safety' which is considered to be warranted under circumstances of major flooding.

Another hydraulic constraint which should also be checked is that relating to culvert capacity at driveway crossings where these are used. The critical condition is likely to occur at the downstream extremity of the street where the entire flood flow, Q_Y, must pass through the last crossing. This installation should be designed as a culvert system with, possibly, multiple pipes. In this event, crossings upstream may have appropriately reduced numbers of pipe-culverts. Culverted crossings should incorporate rip-rap or gabion (scour) protection where the pipes discharge downstream.

In extreme circumstances where the presence of culverted crossings is likely to cause channel surcharge, such installations must give way to simple ‘low level’ driveways of the type illustrated in Figure 2.4.

The completion of STEP 5 concludes the design process for cases falling within the scope of Procedure 7. Those which have been shown (STEP 3) as “Procedure 8 cases” continue to STEP 6 where the issues of detailed dimensioning and setting of ‘hydraulic assistance’ flows are addressed.

STEP 6 : PROCEDURE 8 – revisit STEP 3

All ‘filter strip’ swale cases continuing to this stage comprise three components :

- Filter strip – 100 mm deep, sand/gravel mix, minimum 3.5 m length (see Figure 7.5).
- Gravel-filled sub-structure, 1.0 m wide (all cases), minimum depth, H, given by ‘point’ on the design graph noted in STEP 3.
- ‘Hydraulic assistance’, also indicated by the position of the ‘point’ plotted in STEP 3.

Illustration 3, Section 7.8.7 demonstrates this process.

Sufficient basic information is available to the designer through the six-step process, above, to enable him/her to complete detailed design of a ‘filter strip’ swale for any location in Australia.

7.8.11 Construction : some recommendations

Construction of a ‘filter strip’ swale should be integrated with construction of the carriageway with which it is associated. However, it should **under no circumstances** be called upon to operate in an environment of *unregulated* general building/construction activity that frequently follows completion of a carriageway. ‘Unregulated’ here means “inadequate sediment control associated with the off-site movement of construction equipment”. ‘Filter strip’ swales are capable of long service receiving and processing stormwater runoff from **established** residential street settings, only : the warning contained in the last paragraph of Section 7.6.5 should not go unheeded.

The most vulnerable (to physical damage) component of the ‘filter strip’ swale is undoubtedly the sand/gravel treatment surface, particularly during the period of its construction and sowing/planting with grass or selected ground cover. Coincidence of a significant storm with the sand/gravel laying and sowing/planting processes could result in erosion of sand in the immediate vicinity of the carriageway edging as well as ‘scouring off’ of upper level sand from the treatment surface itself. The first of these problems can be ongoing unless preventative action is taken; the second is temporary, until such time as the grass is established or the ground cover is thriving.

The most effective action which can be taken to prevent edge erosion in the vulnerable area – say the 0.5 m width beside the carriageway – is to provide some form of **porous** paving along this strip. There are a number of proprietary products which are suited to this duty including plastic ring-matrix and masonry systems (see Figure 3.1a; St. Elizabeth Church car park, Section 3.9). The essential criteria which should be sought in selecting a suitable product are strong grass integration with the system and a high level of porosity.

Some repair of the surface may, of course, be necessary, particularly where the construction schedule coincides with a large storm – say a “20-year” or larger event – and/or wherever a concentrated entry to the sloping carriageway surface from an upstream residence results in jet-impact on the swale surface matrix. The use of hay bales to break up such jet action is advised.

Clearly, it is better – if at all possible – for construction of such flood-sensitive components to take place when the risk of major storm rainfall occurring is minimum. In the case of Northern Australia, this is during the “dry” (winter) season. In Southern Australia, winter is also the preferred period as storm burst rainfall – while more frequent than in summer – is less intense; also, winter sowing avoids the intense heat of summer. Local climatic conditions of Intermediate Australia can be related, broadly, to these two extremes.

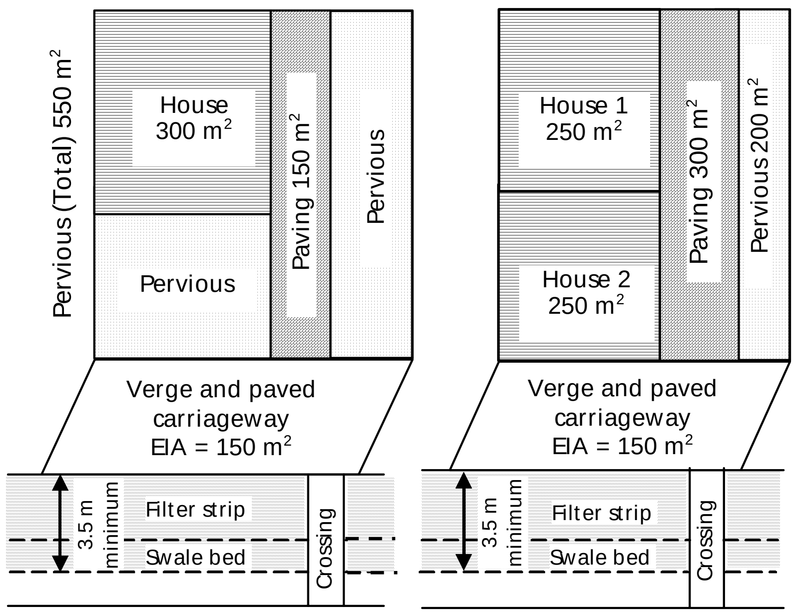
An alternative to sowing/planting is to use established (roll-out) grass to cover the entire or particularly vulnerable parts of the swale cross section. Provided the type of grass is well-suited to the climatic and irrigation conditions in which it is expected to survive, this is a satisfactory though more expensive solution.

There is one further aspect of construction and management of ‘filter strip’ swales, as described above, which is worthy of attention. This relates to use of the sloping swale surface for off-carriageway parking. This has been recognised as a major problem for municipal agencies responsible for maintaining swales. Solutions to date have been preventative, such as the border fencing illustrated in Figure 2.4. The sand/gravel mixture recommended for the filter strip in Procedure 7 and described in more detail above, provides a cost-effective alternative to this solution. The sand/gravel matrix with grass or ground-cover, forms a surface suitable for standing vehicles provided this use is rare. It is recommended, therefore, that irregular or occasional vehicle parking on swale slopes – constructed as outlined above – rather than regular use, could be tolerated without seriously impacting on the performance of the system. Notices advising offenders of the imposition of a fine for unlawful use of the filter strip, together with some vigilance, may be all that is needed to ensure minimal use of swales for car parking.

7.9 ‘FILTER-STRIP’ SWALES : ILLUSTRATIVE EXAMPLES

7.9.1 Introduction to Examples 7.5 and 7.6 : swales for two residential streetscapes

The following examples relate to the tasks of designing ‘filter strip’ swales for two streetscape scenarios in which the alternative types of development illustrated below are characteristic. Both allotments (Figure 7.8) are 1,000 m^2 in area (‘quarter acre’ block) with 30 m frontages. The first (Figure 7.8a), is typical of conventional development in the older suburbs of Australian cities; the second (Figure 7.8b), represents the same site area, this time accommodating two medium density residences. The two cases re-visit Examples 5.6 and 5.7.



(a) Conventional development with swale (b) Medium-density development with swale

FIGURE 7.8 : Layout of typical residential streetscape developments used in Examples 7.5 and 7.6

7.9.2 Example 7.5 : Conventional development streetscape with swale

The task involved in designing a 'filter strip' swale streetscape in this case has two parts :

- design of the filter strip to trap pollutants conveyed in runoff from the carriageway; and,
- design of the swale to convey the full range of stormwater (flood) flows from ARI, Y = 0.25-years up to Y = 100-years.

In this example, the streetscape is 180 m long, located in an "…average suburb with some trees…" in Brisbane, Northern Australia. Other site and catchment characteristics, in particular, details of catchment components draining to the swale along each (typical) residential allotment frontage are :

- total connected roof area ....300 m²
- allotment paved area (permeable paving) ....150 m²
- allotment pervious area (runoff coeff, C_{10} = 0.70) ....550 m²
- verge, footpath and carriageway (runoff coeff, C_{10} = 0.90) ....150 m²
- soil k_h, hydraulic conductivity (sand)..5×10^{-4} m/s
- allotment frontage, 30 m; streetscape length, 180 m; longitudinal slope, 1.0 %
- streetscape time of concentration, t_C ....25 minutes
- critical storm duration, $(T_C)_{total}$, Council specification.........................90 minutes
- design storm intensity (ARI, Y = 100-years, Council specification) ...173 mm/h (25 mins storm)

Notes :

- there is no runoff contribution to the swale from 'low' side residences (see Figure 7.6);
- the Q_{100} flood peak estimated for the streetscape is based on 'time of concentration', t_C, for the streetscape, not total or local critical storm duration, see Section 4.2.6 ("On-site entry works").

PROCEDURE 7 : 'Filter strip' swale for conventional residential development (see Section 7.8.10)

STEP 1 :

Determine EIA (equivalent impervious area) of typical unit contributing runoff to the swale :

- roof area .... = 300 m^2
- allotment paved area (permeable paving) .... = 0 m^2
- allotment pervious area = 550 m^2 × 0.7 .... = 385 m^2
- verge and paved carriageway .... = 150 m^2
- filter strip – 30 m × 3.5 m (see Section 7.8.3).......................... = 105 m^2

TOTAL.................................. = 940 m^2, 30 m frontage

Hence, A_{eia15} = 470 m^2 per 15 m (standard) swale segment

STEP 2 :

Identify climate zone of site based on $i_{10,1}$ rainfall intensity (= 71 mm/h),

hence Northern Australia, Appendix D, graph D-4.

Determine moderated hydraulic conductivity : k_h × (Moderation factor)

$$= 5 \times 10^{-4} \text{ m/s} \times (0.5) = 2.5 \times 10^{-4} \text{ m/s}$$

EIA per 15 m segment .... **A_{eia15} = 470 m^2.**

STEP 3 :

Determine required components of filter strip and their dimensions (from Figure D-4) :

Filter strip (sand/gravel mix), 100 mm deep, underlain with geotextile, at least 3.5 m wide;

plan area **– 42 m^2** , minimum; this information is used in STEP 4, following.

Gravel-filled trench beneath swale bed, 1.0 m wide, **H = 0.75 m** (just!).

STEP 4 :

Determine 'lifespan' of filter strip system identified in STEP 3.

Ground-level surfaces contributing runoff to the filter strip are :

- allotment pervious area = 550 m^2 × 0.7 .... = 385 m^2
- verge and paved carriageway .... = 150 m^2

TOTAL................................I = 535 m^2, 30 m frontage

Hence, I/P ratio (ground-level runoff) = $\dfrac{535\ \text{m}^2 \div 2}{42\ \text{m}^2}$ = **6.4**

Interpolating in Table 3.1 – "…average suburb with some trees" **–** and noting "times 5" factor for vegetated porous treatment surfaces, estimated 'lifespan' is 5 × 2.8 years = **14 years (minimum).**

STEP 5 : Swale hydraulic capacity (flood flow) considerations

Determine (streetscape) Q_{100} flood flow and conveyance capacity of swale :

$$Q_{100}\ \text{peak flow} = \frac{(CA)i_{100}}{0.36}\ \text{L/s} \quad \text{with (CA) in hectares,}$$

$$\text{hence,}\quad \frac{(470 \times 12 \div 10{,}000) \times 173}{0.36} = \mathbf{271\ L/s}$$

Swale basic dimensions : depth = 0.30 m; bed width = 1.0 m; side-slope (filter strip) = 1 in 8;

minimum hydraulic radius = 0.18 m; bed slope, S_o = 0.010. **Manning's equation, Q = 0.38 m^3/s or 380 L/s** ("n" = 0.06), see Argue (1986). **Result :** the swale will convey the Q_{100} peak flow without surcharge.

Driveway crossings (six) – culvert required to convey 271 L/s (end of street flow) :

4 × 225 mm diam concrete pipes (consult culvert handbook) will convey this flow. Normal rip-rap or gabion (scour) protection should be provided downstream of crossings. Number of pipe culverts required at crossings may be reduced upstream in proportion to contributing catchment.

Completion of STEP 5 concludes basic design of a 'filter strip' swale requiring application of Procedure 7, only, as determined by consultation of the relevant design graph, in this case Figure D-4.

7.9.3 Example 7.6 : ‘Filter strip’ swale for medium density residential development

The task involved in designing a ‘filter strip’ swale streetscape for this case has two parts :

- design of the filter strip to trap pollutants conveyed in runoff from the carriageway; and,
- design of the swale to convey the full range of stormwater (flood) flows from ARI, Y = 0.25-years up to Y = 100-years.

In this example, the streetscape is 180 m long and is located in an “…average suburb with some trees…” in Brisbane, Northern Australia. Other site and catchment characteristics, in particular, details of catchment components draining to the swale along each (typical) residential allotment frontage are :

- total connected roof area ....500 m²
- allotment paved area (permeable paving) ....300 m²
- allotment pervious area (runoff coeff, C_{10} = 0.70)...............................200 m^2
- verge, footpath and carriageway (runoff coeff, C_{10} = 0.90)................150 m^2
- soil k_h, hydraulic conductivity (medium clay) ....3×10^{-6} m/s
- allotment frontage, 30 m; streetscape length, 180 m; longitudinal slope, 1.0 %
- streetscape time of concentration, t_C .... 25 minutes
- critical storm duration, $(T_C)_{total}$, Council specification......................... 90 minutes
- design storm intensity (ARI = 100-years, Council specification) ....173 mm/h (25 mins storm)

Notes :

- there is no runoff contribution to the swale from ‘low’ side residences (see Figure 7.6);
- the Q_{100} flood peak estimated for the streetscape is based on ‘time of concentration’, t_C, for the streetscape, not total or local critical storm duration, see Section 4.2.6 (“On-site entry works”).

PROCEDURE 7 : Design of filter strip components of swale

STEP 1 :

Determine EIA (equivalent impervious area) of typical unit contributing runoff to the swale :

- roof area .... = 500 m^2
- allotment paved area (permeable paving) .... = 0 m^2
- allotment pervious area = 200 $m^2 \times 0.7$ .... = 140 m^2
- verge and paved carriageway .... = 150 m^2
- filter strip (see above) .... = 105 m^2

TOTAL.................................. = 895 m^2, 30 m frontage

Hence, A_{eia15} = 448 m^2 for 15 m (standard) swale segment.

STEP 2 :

Identify climate zone of site based on $i_{10,1}$ rainfall intensity (= 71 mm/h),

hence Northern Australia, Appendix D, graph D-4.

Determine moderated hydraulic conductivity : $k_h \times$ (Moderation factor)

$$= 3 \times 10^{-6} \text{ m/s} \times (2.0) = 6.0 \times 10^{-6} \text{ m/s}$$

EIA per 15 m segment .... A_{eia15} = 448 m^2.

STEP 3 :

Determine required components of filter strip and their dimensions (from Figure D-4). Although this case falls, clearly, into the domain of Procedure 8, the filter strip dimensions are universal. Hence :

Filter strip (sand/gravel mix), 100 mm deep, underlain with geotextile, at least 3.5 m wide,

plan area – **42 m^2**, minimum.

This information is used in STEP 4, following.

STEP 4 :

Determine 'lifespan' of filter strip system identified in STEP 3.

Ground-level surfaces contributing runoff to the filter strip are :

- allotment pervious area = 200 m^2 × 0.7 .... = 140 m^2
- verge and paved carriageway .... = 150 m^2

TOTAL................................I = 290 m^2, 30 m frontage

Hence, I/P ratio (ground-level runoff) $= \dfrac{290 \text{ m}^2 \div 2}{42 \text{ m}^2} = \mathbf{3.4}$

Interpolating in Table 3.1 – "…average suburb with some trees" – and noting factor of 5 for vegetated porous treatment surfaces, estimated 'lifespan' is 5 × 4.5 years = **22 years (minimum).**

STEP 5 : Swale hydraulic capacity (flood flow) considerations

Determine (streetscape) Q_{100} flood flow and conveyance capacity of swale :

$$Q_{100} \text{ peak flow} = \frac{(CA)i_{100}}{0.36} \text{ L/s with (CA) in hectares,}$$

$$\text{hence, } \frac{(448 \times 12 \div 10{,}000) \times 173}{0.36} = \mathbf{258 \text{ L/s}}$$

Swale basic dimensions : depth = 0.30 m; bed width = 1.0 m; side-slope (filter strip) = 1 in 8;

minimum hydraulic radius = 0.18 m; bed slope, S_o = 0.010. **Manning's equation, Q = 0.38 m^3/s or 380 L/s** ("n" = 0.06), see Argue (1986). **Result :** the swale will convey the Q_{100} peak flow without surcharge.

Driveway crossings (six) – culvert required to convey 271 L/s (end of street flow) :

4 × 225 mm diam concrete pipes (consult culvert handbook) will convey this flow.

STEP 6 : Procedure 8 – consult Figure D-4

Moderated hydraulic conductivity : $k_h \times$ (Moderation factor)

$$= 3 \times 10^{-6} \text{ m/s} \times (2.0) = \mathbf{6.0 \times 10^{-6}\ m/s}$$

EIA per 15 m segment............. $\mathbf{A_{eia15} = 448\ m^2}$**.**

Filter strip requirements with 'hydraulic' assistance :

- Filter strip (sand/gravel mix), 100 mm deep, underlain with geotextile, at least 3.5 m wide;
- Gravel-filled trench beneath swale bed, 1.0 m wide, H = 0.75 m (just !);
- 'Hydraulic' assistance (bore or slow-drainage pipeline) : q_{h15} = 7.2 L/s (flow per 15 m segment).

The latter requirement must be interpreted into the context of the site (12 segments).

Where aquifer access is available (and environmentally appropriate), the **total** 'hydraulic' flow for the streetscape, Q_{hA}, should be determined ($12 \times q_{h15}$ = 86.4 L/s) and the number of bores, n, calculated from a knowledge of the permissible recharge rate per bore, q_r :

$$n \geq \frac{Q_{hA}}{q_r} \qquad \mathbf{(7.5)}$$

where "n" is an integer.

Alternatively, where aquifer access is denied, a slow-drainage (perforated) pipeline may be installed to convey the entire 'hydraulic assistance' flow to disposal outside the streetscape. In this case :

Total 'hydraulic' flow – as above, Q_{hA} – is 86.4 L/s; this can be conveyed in a 300 mm diam PVC pipe, perforated, except where it discharges from the streetscape into a downstream receiving domain.

Completion of STEP 6 concludes basic design of a 'filter strip' swale requiring application of Procedures 7 and 8 as determined by consultation of the relevant design graph, in this case Figure D-4.

8 STORAGES FOR STORMWATER HARVESTING (Category 3 Systems)

8.1 INTRODUCTION

8.1.1 Current drainage practice

Discharge of roof and other surface runoff to the street drainage system is a common stormwater management practice in Australian cities. Although expedient for on-site drainage control, discharge of roof and allotment runoff directly to the street reflects a thoughtless attitude to the undesirable consequences of the practice, as reviewed in earlier chapters including :

- its significant contribution to street runoff peak flows, which increases the burden placed on drainage infrastructure;
- greatly increased **frequency** of flow for a specified flow rate : typically, development increases the frequency of 'minor' runoff events by at least an order of magnitude – this can markedly increase the rate of scour and erosion in receiving watercourses and degrade their ecological value;
- the greatly increased **volume** of catchment runoff, which can impact on the receiving water environment. Reduced volumes may also improve the efficiency of stormwater quality control measures, such as wetlands, due to increased retention time in such measures (or, in the case of wetlands, a smaller footprint to achieve the same level of treatment);
- the addition of pollutants such as sediment, animal faeces, fertilisers and pesticides – from domestic residences – and heavy metals, nutrients, hydrocarbons and other toxins from industrial properties;
- waste of a locally-available water resource.

The practice of diverting roof runoff directly or indirectly to the street represents, generally, poor water-sensitive urban (residential) design practice, except in the special circumstances of the **yield-maximum** strategy referred to in Section 4.2.1.

8.1.2 Chapter outline and qualification

This chapter explores, primarily, the potential of **Roof Runoff Harvesting Systems (RRHSs)** to harvest residential roof runoff, in particular, for use and thereby to minimise the waste of a potentially valuable resource which is a by-product of residential development. The term 'rainwater tank' is avoided as there is a growing number of alternatives to the traditional tank storages, for example enlarged (roof) gutter storages and roof runoff storage systems integrated with metal or polyethylene fencing, that also deserve consideration. In fact RRHSs should be considered as much more than a simple storage, but a 'package' system that matches roof-runoff harvest potential to the 'best overall' RRHS for the specific requirements and constraints of a given site. Rainwater tanks also play a role in stormwater quality improvement through their association with 'first flush' devices; furthermore, overflows provide cleansed water input to "leaky" on-site retention devices (see Section 5.1.3).

The focus of this chapter is on *small footprint* RRHSs, for example systems involving small rainwater storages located, typically, on individual allotments or integrated with a cluster of dwellings. The reason for this bias is the fact that the domestic sector makes greater demand on mains water supplies in Australian cities than all other sectors – industrial, commercial, etc. – combined. Any significant reduction in this demand can therefore be translated into major savings in the cost of water supply infrastructure as well as maintenance of these facilities.

Large (neighbourhood scale) rainwater storages providing supply to domestic consumers are considered impractical and usually unacceptable for general use in urban (residential) development, however, such units, integrated with industrial, commercial or, perhaps, educational sites, can be viable. The procedures for determining the capture/use performance of such facilities, developed later in this chapter, are equally applicable to these installations : only roof area, daily demand rate, and a handful of other parameters need be altered to produce a valid design.

The primary purpose of RRHSs is considered in this chapter to be for mains water replacement. However there is also potential for these components of urban infrastructure to co-incidentally reduce urban catchment stormwater volumes and mitigate street runoff peak flows. Investigations by Allen (1993), van der Wel (2000), Coombes et al (2002), Allen (2002), Kuczera and Coombes (2002), clearly demonstrate that RRHSs can be dimensioned to reduce peak flows resulting from 'minor' storm events, the magnitude of the reduction being locality dependent. Under some circumstances even quite small rainwater storages can reduce flood peaks for ARI, Y = 5 – 10 years flows, but they will have little or no effect in major storms such as the "50 or 100 years" events, unless used in conjunction with other flood mitigation measures such as large retention or detention storages.

Where RRHSs are **designed** to achieve street drainage peak flow mitigation as well as harvesting objectives they are more properly described as Category 4 systems (see Section 4.1). This topic is revisited in Section 8.7

8.2 ASPECTS OF RESIDENTIAL ROOF RUNOFF HARVESTING

8.2.1 Storage size – practical considerations

A common misconception is that RRHS storages must be very large in order to capture most of the roof runoff. It is true that large storages are required in areas without access to other supplies, where their role is to provide a high level of secure supply even during prolonged dry spells. However in urban areas, mains water is available as a 'backup' and security of supply is not a major concern.

The main consideration for RRHSs in urban areas is therefore not how large a tank must be to supply the demand 99-100% of the time, but how much roof runoff can be captured and put to beneficial use from a storage of 'practical' capacity. The focus is therefore on small storages of 1 to 5 kL capacity as these are suitable even for very small residential sites and should therefore be quite acceptable to the public. However, depending on the type of storage device, allotment size and individual householder preferences, storages of 5 kL to 10 kL may also be appropriate, so these are also considered.

Although storages larger than 10 kL may not be suitable for the bulk of contemporary domestic situations, even these have their place: for example, the 'Healthy Home' on the Gold Coast, uses a 22 kL concrete cistern installed below the low-set house.

8.2.2 Considerations for maximising roof runoff capture

In order to limit RRHS storage to the minimum whilst capturing as large an amount of roof runoff as possible, it is necessary to ensure there is as much free storage available as possible at the onset of rainfall. This can only be achieved by ensuring that water is regularly drawn from storage and that the rate at which it is abstracted is quite high, particularly during the rainfall season. Although a high rate of use might seem at odds with the water-conserving approach of WSUD there is a practical limit to the rate of domestic water use that can be predicated on factors such as household number and the purpose and pattern of water use. Failure to regularly draw water from storage will, of course, result in frequent overflows, which represent a loss of the resource through site runoff.

The two most popular uses of roof runoff in Australia are for drinking (typically, for tea- or coffee-making) and garden watering. The former provides almost negligible use (representing less than 1 percent of residential water demand) while the latter is predominantly a 'dry season' activity when small storages are likely to be empty. Furthermore, water use is not 'guaranteed' but relies on the householder's volition to use tank water in preference to mains water. It is apparent, therefore, that neither drinking nor garden watering uses realise the full potential of this alternative supply. Despite these criticisms and recommendations that other uses be considered, it should be added that **any** use – from the water conservation and flood control points of view – is better than no use at all. Alternative methods that increase the use of roof runoff by a significant amount are discussed in the next section.

8.2.3 Appropriate uses of roof runoff

In harvesting terms, roof runoff is better used for in-house, year-round demand rather than for drinking or garden watering. The means for increasing roof runoff harvesting for in-house supply have been considered by a number of Australian researchers and at least one RRHS product designer. Four domains are recognised :

- gravity supply;
- hot water supply;
- all-house supply;
- all-house supply excluding drinking water.

Each of these is considered in the context of dual supplies, i.e. supported by mains or other reliable water supply, when the rainwater storage is empty.

Gravity supply : Roof runoff can be captured in an elevated tank (on a stand), or at roof (eaves) level by means of an enlarged gutter storage which comprises a screened box gutter that stores water at roof level for suitable gravity-fed uses such as toilet cistern supply. The advantage of such elevated storage is that it enables rainwater to be delivered under gravity (to some uses) as opposed to needing a pumped supply. Water can be supplied, for example, to lower-floor bathrooms, laundries and toilets of multi-storey units – although plumbing requirements for devices receiving a gravity fed (low pressure) water supply need attention. There were more than 100 single-storey residences in Sydney successfully using this system at the end of 2003. With enlarged gutter storage, the capacity of the gutter depends on its length and cross-sectional area, but for a typical installation will be in the order of 1 to 3 kL.

Hot water supply : Although in-house uses such as bathroom and laundry demand can make good use of roof runoff, a considerable cost disadvantage is incurred by the need to provide dual plumbing for mains and RRHS supplies. Allen (1993) suggested an alternative technique to reduce the amount of dual plumbing : plumbing roof runoff (via pumped supply) *and* mains supply to the gravity hot water service. Allen suggested that there will be an incidental advantage in that the water is 'pasteurised' in the hot water tank under normal operating temperatures, thereby improving the microbial quality of roof runoff supply. [Water quality is discussed in the next section].

Taking Adelaide as an example, model results have shown that rainwater tanks could provide 30 to 60 kL per year to households if they are plumbed into the hot water service, resulting in a 10 to 20 percent reduction in total domestic water use. In many other Australian (non tropical) cities the amount of water savings will be much greater than in Adelaide due to their higher rainfalls and more uniform seasonal distributions. Water use savings are discussed later in the chapter.

The Urban Water Resources Centre (University of SA), trialled the hot water service concept during 1995-96 at the 'Intelligent Home', an experimental house at Regent Gardens Estate, Adelaide. A 2 kL rainwater tank supplied roof runoff, via a pump, to a gravity-type hot water tank which was fitted with two float valves : one for activating the rainwater tank supply, when possible, and another, low-level float, that provided mains water (via another line) when the rainwater tank was empty.

Following the success of the trial, UWRC incorporated a variation of the design into the Concept Design for "Figtree Place", a residential re-development in inner Newcastle (Argue, 1997). Computer modelling based on historical rainfall records suggested that a 2 kL storage per residential unit would supply nearly 50 percent of the combined hot water and toilet flushing needs for the site, based on assumed use rates for hot water and toilet flushing. At "Figtree Place", hot water and toilet flushing are supplied from underground rainwater tanks (shared between clusters of, typically, four residential units, see Section 3.9). The tanks are topped up with mains water when the rainwater storage falls to a pre-set level, thereby avoiding the need for dual plumbing for the hot water service and/or toilets. Monitoring of "Figtree Place" by Dr. Peter Coombes, University of Newcastle, has proven the efficacy of the concept as a roof runoff harvesting technique. A 45 – 50 percent reduction in (in-house) mains water use has been achieved (Coombes et al, 1999; Coombes, 2002). Furthermore, water quality tests have proven the capability of the hot water supply concept to improve the microbial quality of roof runoff water.

All-purpose supply (including drinking supply) : RRHS storages can be plumbed directly into the household mains to provide *all* domestic uses. The main advantages of this system are :

- it maximises the capture and use of roof runoff;
- it minimises tank overflows; and
- it overcomes the difficulty and cost of dual plumbing.

Two perceived drawbacks are the risk of rainwater being back-siphoned into the public water supply system, and the potential health risks associated with using roof runoff for drinking water supply.

The potential risk of contaminating the mains supply is negated by ensuring that the installed system conforms to the regulatory requirements for rainwater tank-mains interconnections. This matter is the subject of Australian Standard AS 3900, which requires the installation of the 'RZPD valve' in mains/ rainwater inter-connections. [This requirement was observed at "Figtree Place".] Standards change from time-to-time and the local water supply authority should be consulted for up-to-date information on regulatory requirements and advice on mains/rainwater tank inter-connections.

The use of roof runoff for drinking and/or washing – both personal and clothes – requires a thoughtful evaluation on the part of the householder of the risks involved. These are discussed in Section 8.2.4. While there may be **potential** risks in using roof runoff for drinking and washing, the relative simplicity of connecting a RRHS – fitted with mains top-up supply – to supply the entire in-house needs is certainly attractive.

However, the possibility of contamination of the entire body of stored water as a consequence of some external cause – perhaps a dead animal on the roof or severe discolouration during leaf or blossom fall from a tree up-wind of the roof – should alert householders to the need for 'instant' alternative supply from mains sources.

A patented tank-mains diversion valve and associated components enabling a mains-rainwater tank inter-connection and pumped supply, which also incorporates automatic switch-over to the mains when the tank is empty or during a power failure (which would inactivate the pump) or as a consequence of contamination of the types outlined above, has recently been developed by an Australian company.

All-purpose supply (excluding drinking water) : Several alternative 'all-purpose' concepts that exclude the need to use roof runoff directly for drinking purposes are :

- all-purpose supply, but with bottled water used for drinking;
- all-purpose supply, but with householders storing water from the hot water supply and refrigerating it in preparation for drinking;
- mains top-up supply to the RRHS storage with a separate mains-water line to the kitchen. This approach has additional merit in being able to provide water inside the house (kitchen) even in the event of a pump malfunction, power blackout or period of rainwater tank contamination.

8.2.4 Roof runoff quality considerations

Roof runoff quality depends on many factors such as the local air environment, roof and storage tank material and whether the roof, gutter and storage tank are maintained in good condition. An excellent, detailed reference on factors that affect the quality of harvested roof runoff is the "Guidance on the Use of Rainwater Tanks" monograph by the National Environmental Health Forum (Cunliffe, 1998). The publication includes information on :

- rainwater tank microbial and chemical quality;
- Australian standards pertaining to RRHSs, including materials;
- roof and tank materials;

- installation considerations;
- procedures to improve water quality : maintenance and repair;
- components to improve water quality : inlet screens, 'first flush' diversion devices, filters, UV disinfection;
- chemical disinfection;
- mosquito control.

The monograph can be obtained from state and territory public health authorities and can (at the time of publication) be downloaded from some state health agency web sites. In addition to the monograph, the advice of state and local public health authorities can be sought for up-to-date information on regulatory and policy matters.

Microbiological considerations : Roof runoff that is stored in a rainwater tank or other storage device is quite likely to contain micro-organisms from one or more of a number of sources, for example :

- soil or leaf litter accumulated in roof gutters;
- faecal material deposited by birds, cats, mice, etc.

Most micro-organisms are completely harmless to humans and although there have been documented instances of gastrointestinal infections attributed to drinking rainwater, the risks appear to be extremely low. The National Environmental Health Forum monograph, which is endorsed by all Australian state and territory health authorities states : "the perception is that rainwater is safe to drink and this is probably true if it is clear, has little taste or smell and, importantly, that the source of water is from a well maintained tank and roof catchment system".

In Adelaide, survey data show that roof runoff is the primary source of drinking water for about 28 percent of the population (Heyworth et al, 1998). This is the highest rate of use of any Australian state or territory capital and suggests that almost 300,000 persons in Adelaide use rainwater as the primary drinking supply. Most of these people prefer the taste of roof runoff water to the filtered mains water which is sourced from local catchments and from the River Murray.

A recent study on the health of South Australian children (Adelaide and rural communities) who drink raintank water, compared to those who drink mains water, found no greater incidence of illness among the children who drank tank water. In fact the study data suggest a slightly higher incidence of gastrointestinal illness among children exposed, exclusively, to mains water (Heyworth et al, 1999; Heyworth, 2001).

Chemical considerations : The National Environmental Health Forum monograph (Cunliffe,1998) provides information on chemical risks, including those associated with various roof materials, paints and coatings, industrial pollution and wood-burning stoves.

In general, the risks are extremely low even in urban areas. Nevertheless there can be exceptions : in Port Pirie, South Australia, where there is a long history of lead smelting with associated airborne pollution, lead levels in rainwater tanks have been detected at concentrations exceeding drinking water guidelines (Fuller et al, 1981; Body, 1986).

While state health authorities do not advocate the use of roof runoff for drinking where mains water is available, many provide information on ways to reduce the risks associated with drinking rainwater : for example, recommending that 'first flush' diversion devices be installed and emphasising good maintenance practices. Authorities may also provide warnings in relation to 'at-risk' persons such as the infirm or immuno-compromised persons. Boiling water before ingestion, of course, achieves disinfection, but over-night storage of water taken from the hot water system is a simple and energy-saving way to achieve the same objective.

It is **strongly** recommended that the National Environmental Health Forum monograph, and other information obtainable from state and territory public health authorities and local council authorities, be consulted before embarking on a proposal to collect roof runoff for drinking.

Health warning for underground tanks : There are documented cases where rainwater tanks installed below ground level have been contaminated by surface runoff. There is also the potential for accidental cross connection with other sources of water – even domestic wastewater or (in unsewered areas) septic tank water. In one instance 89 people supplied with drinking water from an underground tank became ill as a result of the tank being contaminated by overflow from a septic tank (Lester, 1992).

Monitoring at “Figtree Place” initially indicated microbial contamination of the subsurface rainwater tanks. This was caused by poor installation (sealing) of the tanks that enabled surface runoff and soil to enter the rainwater tanks. (Soil naturally contains some types of microbial organisms.) Fortunately, ‘pasteurising’ by the hot water services improved the quality of the water to a high standard (Coombes et al, 1999).

Surface runoff may potentially also contain traces of pesticides, fertilisers or other chemicals. Devices such as UV units and carbon filters may assist in reducing the health risk, but do not guarantee that water in sub-surface tanks will be free of contamination.

It is **strongly** recommended that all storages and downpipes to storage devices be entirely above ground level, particularly where the water is used for drinking. If this is not possible, then all components should be subject to close and continued scrutiny. Advice on the need or otherwise for water quality monitoring should be sought from the relevant state health authority.

8.2.5 Practical considerations for RRHSs

General

There are many issues that require consideration when it comes to designing, operating and maintaining a RRHS :

- For new housing, ensure that all roof areas intended to drain to the storage device actually do so! Gutter support structures must be correctly designed and installed to ensure this occurs.
- It is often difficult to drain runoff from a large roof to a single storage device. In fact, because a single drainage point is often insufficient to prevent gutters from overflowing, it is common practice to drain roof runoff to two, three or more points around the roof perimeter. The use of a deep box gutter, previously described, avoids these problems and may allow excess roof run-off (exceeding the box gutter capacity) to drain to a single tank storage point by way of a downpipe to a conventional rainwater tank.
- Regularly check that all RRHS components operate satisfactorily;
- Where space is limited, alternative innovative storage techniques can be considered (separately or in combination) including :
    - modular ‘slim line’ rainwater tanks;
    - box gutter storage;
    - rainwater fences : a patented ‘ultra slim’ modular storage device that doubles as a fence. Modules can be connected to increase storage capacity (and length of fence).

Pressure supply system

- The possibility of pump noise should be considered when selecting and locating a pressure pump.
- Pump reliability : poorer quality pumps can be relatively cheap but may break down more frequently. In the longer term these may be less cost effective.
- It is recommended that the Master Plumbers Association be consulted in relation to aspects and advice on pressure pump supply systems.
- It may be prudent to use a plumber’s experience in installing pressure pump supply systems : contact the Master Plumbers Association for advice.

8.3 AVERAGE ANNUAL YIELD : BACKGROUND DISCUSSION FOR PROCEDURE 9

8.3.1 Approach

A daily water balance procedure which calculates daily inflows, outflows and changes in the volume of water in a RRHS storage device over a period of years, is the basis of estimating the average annual harvest potential (yield) from a RRHS.

Procedures based on time increments longer than one day are generally less suitable in that they provide a poorer estimate of the annual yield. Time periods of less than one day are unnecessarily complex for basic procedures and unlikely to provide a more accurate estimate of the average annual yield than a daily-based model. A comparison between rainwater tank yields determined by a daily rainfall model, and another using pluviometer data, was undertaken by Allen (1994). The latter approach involved modelling very short time steps (a few minutes) which, theoretically, should provide a more accurate estimate of annual yield than the daily model. However the yield estimates obtained from the models were almost identical (within 1 to 2 percent) indicating that daily rainfall is appropriate for estimating average annual yields.

This finding has been confirmed in more recent studies by UWRC researchers (Barton et al, 2003). There are also several disadvantages with using pluviograph data which relate to record accuracy and the relatively few stations with long-term pluviograph data compared to those providing daily data (600 stations compared with over 7,000 in Australia). However, use of the DRIP program (Henneker et al, 2001) has widened the opportunities for designing rainwater tank storages using time steps of short duration [6-minutes, 15-minutes, etc. (see Section 3.4.3)].

8.3.2 RRHS average annual yield model

Allen and Argue (1998) describe daily water balance procedures for two situations :

- RRHSs that do not incorporate a ‘low-level’ mains top-up supply to the storage; and
- RRHSs incorporating a low-level mains top-up supply, for example via a ‘low level’ float-activated valve that supplies mains water to a rainwater tank if the tank water level falls to a predetermined minimum.

Although these procedures differ in some respects, comparisons have shown that the difference in average annual yields calculated by the above procedures are very small (within several percent), which is well within the bounds of other uncertainties (such as the assumed average daily demand, discussed later). Consequently only water balance procedures for RRHSs **without** mains top-up supply are described in this Handbook. Further information on the procedures for RRHSs incorporating a low-level mains top-up supply, can be found in Allen and Argue (1998).

A simple daily RRHS model, which is provided (disk) with this Handbook, is based on a RRHS without mains top-up supply to the storage tank (see Section 8.4). Although the model is self-explanatory, it is strongly recommended that the following sections be read prior to using the model.

8.3.3 Model requirements

The information required to estimate the average annual yield is :

- historical daily rainfall record;
- effective roof area connected to the RRHS storage;
- RRHS storage capacity;
- RRHS assumed initial storage on the first day of the record;
- required daily demand.

Daily Rainfall Data

For urban areas, local historical daily rainfall data should be used. If these are not available, data from a nearby representative station with a similar seasonal and annual rainfall should be used. If unsure what data are available or appropriate, the Bureau of Meteorology should be contacted for advice. The data record should cover as long a period as possible – at least 10 years is recommended and preferably 20 years or more. If a requirement is also to calculate other, possibly useful information such as the 'once in 50 years' lowest annual yield, then a longer period of record will be needed. Long records of historical daily rainfalls are available for over 7,000 Australian locations and can be obtained, usually for a small fee, from Bureau of Meteorology offices. CSV or XLS format should be requested for the model accompanying this Handbook. The daily rainfall data should be requested from the Bureau of Meteorology in column format (which is standard) and should **include** days of no rainfall.

A check is recommended to ensure there are no gross errors in the data set. Daily rainfalls for each year can be summed for comparison with monthly or yearly totals, also obtainable from meteorological offices.

Often, daily rainfalls are not recorded on weekends (or public holidays), although the sum for the previous few days is usually recorded at 9.00 a.m. each Monday for the preceding three days from 9.00 a.m. Friday. This has minimal effect on the calculated average annual yield and if anything, provides a marginally conservative yield estimate. Occasionally there may be large gaps in a rainfall record due to loss of original records, temporary closure of the station, or other reasons. Where significant gaps occur, it is necessary to ascertain if it is due to a prolonged period without rain, or due to lost or incomplete data. If significant hiatus occur, consideration should be given to replacing the missing record with data from a suitable nearby station for the same period, or to excluding the 'broken' year's rainfall altogether.

Effective Roof Area

The roof area connected to a RRHS storage is an important parameter affecting supply. As a rule, the connected area should be as large as practical to increase the likely capture. Other roofs including verandahs, garages, carports or sheds can be included in the total area also connected to the storage tank. It is not always feasible to connect the entire roof to a single rainwater storage unless it is below ground level – however, note the warning in Section 8.2.4 in relation to sub-surface storages. A better way to increase roof runoff capture may be to use an enlarged eave-gutter, described previously, the overflow from which is more easily connected to a single above-ground storage.

Not all rainfall striking the roof surface contributes to storage tank inflow : losses occur as a result of some uptake by semi-porous roof materials, sun and wind-induced evaporation, ponding in roof gutters, gutter overflows and gutter leaks. Even where impervious roof materials such as colour-bonded steel, galvanised iron or glazed tiles are used, the effective (connected) roof area may be only 80 to 90 percent of the nominal roof area. In determining harvesting potential for RRHSs, the National Environmental Health Forum monograph (Cunliffe, 1998) assumes a capture efficiency of 80 to 85 percent of the connected roof area.

Storage Capacity and Initial Storage

Initial storage refers to the assumed volume of water in storage at the beginning of the first day of the rain-fall record being modelled. For small storages the assumption as to whether the storage is full, or empty, or between these extremes, has no significant effect on the estimated average annual yield performance. However where a model is used to assess the harvesting performance of extremely large storages the initial storage assumption may produce misleading estimates. For this reason, it is recommended that the initial stored volume should be assumed as :

- where the date of the first day of record occurs in the dry season : zero;
- where the date of the first day of record occurs in the rainy season : the full storage capacity, but not exceeding a maximum of 10 kL.

In locations which are not characterised by distinct "dry" and "wet" seasons, the initial storage should be set at half full.

Daily Demand – In-house

Daily demand (the rate of water use) can have a very significant influence on the yield. On a daily basis some types of in-house uses exhibit a high degree of variation, for example clothes washing, which might be undertaken on one or two days per week. However, in-house demand (for a given household) is generally fairly constant seasonally or annually so that the assumption of a constant daily demand is generally acceptable. Where data exist which indicate a significant seasonal influence in in-house water use – for example, higher use during summer – this should be accommodated in modelling.

The major factor influencing in-house water demand is the number of household occupants, which is often associated with dwelling type : townhouses, for example, are more likely to have an average occupancy of one to two persons, while large homes may have an average occupancy of three to four persons (two adults and one or two children) in new sub-divisions. It is therefore important to link average in-house daily water demand to the type of development or expected average number of occupants.

Unfortunately the rate of water use also varies considerably between Australian cities so that it is necessary not only to consider the average number of occupants per dwelling, but also location. Of the state and territory capitals, the highest per capita water uses occur in Darwin (very high), Hobart and Brisbane, and the lowest in Adelaide, Sydney and Melbourne. Canberra and Perth fall between these groups.

For most cities, detailed information on in-house water use patterns are not readily available. Potential sources of information are the local water authority and other state government agencies, the Australia Bureau of Statistics, university researchers and the Water Services Association of Australia (WSAA). Limited ‘indicative’ information is provided in the following sections for some state capitals.

Brisbane

Estimates of average in-house consumption by use type are summarised in Table 8.1. Unfortunately these data do not provide a breakdown for dwelling types or number of occupants. However a satisfactory estimate can be obtained by dividing the average in-house use by the average household occupancy for Brisbane to obtain per capita averages which can then be multiplied by the estimated average number of occupants for the dwelling type in question. It is suggested that the following average occupancy rates be used :

- Apartments and townhouses of one or two bedrooms : 2 persons. [Although apartments and townhouses may have an average occupancy of less than two persons, some evidence suggests that a higher per capita rate of in-house water use occurs in these developments. The assumption of two person occupancy is therefore recommended].
- Traditional large allotment residential housing with two bedrooms : 2.5 persons.
- Traditional large allotment residential housing with three bedrooms : 3 persons.

TABLE 8.1
TYPICAL DAILY WATER DEMAND (IN-HOUSE USE), BRISBANE

IN-HOUSE USE	AVERAGE DAILY DEMAND (L/DAY)
Shower/bath	193
Hand basin	28
Toilet	186
Laundry	135
Kitchen (incl. dish-washing)	44
Total In-house	**586**

Source : Urban Water Research Association of Australia (Jeppeson, 1996); data precede significant uptake of dual-flush toilet cisterns. Toilet flushing rates should be reduced by a factor of two where 6/3 litre dual flush toilets are installed – i.e. to 93 L/day.

Perth

An extensive 1986 survey by the Water Authority of Western Australia (WAWA, 1987) provided in-house water demand for Perth.

Table 8.2 shows the WAWA-derived relationships for in-house water demand for specified uses and numbers of occupants. The WAWA data were obtained by in-house metering, validated by homeowner surveys; consequently the data reflect in-house water use and in-house losses.

TABLE 8.2

DAILY WATER DEMAND AND NUMBER OF OCCUPANTS, PERTH

IN-HOUSE USE	QUANTITY FOR FIRST HOUSEHOLD OCCUPANT (L/DAY)	ADDITIONAL QUANTITY PER EXTRA PERSON (L/DAY)
Bathroom (incl. hand basin)	101.0	37.3
Toilet	81.8	31.4
Laundry	41.4	29.4
Kitchen (incl. dish-washing)	20.2	6.9
Total In-house	**244.4**	**105.0**

Source : Water Authority of Western Australia, 1987; data precede significant uptake of dual-flush toilet cisterns. Toilet flushing rates should be reduced by a factor of two where 6/3 litre dual flush toilets are installed.

Adelaide

Table 8.3 provides indicative information for in-house use for Adelaide dwelling types.

TABLE 8.3

ESTIMATED WATER USE BY DWELLING TYPE, ADELAIDE

IN-HOUSE USE	TOWNHOUSES	SMALL 'VILLA' HOMES	MODERATE SIZE NEW DWELLINGS	OLDER, 'THREE-BEDROOM' DWELLINGS
	LITRES PER DAY			
Bathroom (incl. hand basin)	110	110	220	180
Toilet	35	35	70	70
Laundry	65	65	130	100
Kitchen (incl. dish-washing)	40	40	50	50
Total In-house	**250**	**250**	**470**	**400**
Outdoor water use, L/day	**70**	**160**	**400**	**600**

Source : Adapted from Allen, 1993. Assumes low flow dual flush WCs for the first three dwelling categories and partial uptake of dual flush WCs for the fourth dwelling category. For the fourth category, reduce WC use to 50 L/day if 6/3 litre dual flush WCs are installed.

A more recent study of Adelaide domestic consumption by Barton (2003) provides general support for the values presented in Table 8.3.

Other State and Territory Capitals

In the absence of appropriate local information, the following 'indicative' in-house water use estimates will provide satisfactory RRHS yield estimates :

- Darwin : 1.5 times the Brisbane rate
- Sydney : 1.3 times the Adelaide rate
- Canberra : 1.3 times the Adelaide rate
- Melbourne : 1.2 times the Adelaide rate
- Hobart : 1.2 times the Adelaide rate.

For other locations the local water authority should be contacted for advice.

Daily Demand – influence of outdoor use

Outdoor demand is the most difficult component of all to estimate as it will vary considerably from house to house and year to year, as well as between geographic locations. Factors that influence garden watering include the type of vegetation (native or exotic), number of hot days (particularly days over 30°C), irrigated area, watering method (hand, sprinkler, dripper, automatic irrigation, etc.), socio-economic factors, mains water price, whether dwellings are owned or rented, and the householder's level of interest in gardening. In some areas, notably Perth, bore water is extensively used; there is likely to be much greater use of water where there is access to (free) bore water.

For many Australian cities the wet and dry weather periods are relatively distinct, particularly for cities such as Darwin and Brisbane. Because garden watering is largely a mid-dry season activity when rainfall is lowest, garden watering is less likely to influence RRHS yield (for small storages) than significant in-house consumptive uses, particularly for dwellings with small gardens. In the absence of other information, a conservative yield estimate for RRHSs designed to supply all-household demands can therefore be obtained by a number of methods such as :

- Assume no summer irrigation use : that is, modelling the in-house water use **only**. This is a simple approach which should give an accurate estimate of yield for locations where there is a pronounced wet-dry season, garden irrigated areas are quite small, and the RRHS storage is small, e.g. 1 to 2 kL capacity. The approach will provide a conservative estimate of yield for **any** situation where the rainwater storage is intended to supply water for all household demands including outdoor use.

OR

- Assume an average irrigation rate of 4 mm per day (4 litres/m^2/day) per square metre of irrigated, **exotic** garden area (lawn plus garden beds) during the driest three months. This will enable garden use to be estimated for either a known or assumed irrigated garden (exotic) area. A daily water balance model can be established that incorporates the daily irrigation water use during the irrigation season (adding this to the constant in-house demand). This approach is most suited to estimating yield for sites where there is a less well-defined season, high garden water use and larger capacity storages.

OR

- Assume an average irrigation rate of 1 mm per day (1 litre/m^2/day) per square metre of irrigated, **native or drought-tolerant** garden area (lawn plus garden beds) during the driest three months. This will enable garden use to be estimated for either a known or assumed low water-use garden area. Best practice management of native, drought-tolerant or Mediterranean-style vegetation involves one or two "soakings" per month totalling 25 – 30 mm. A daily water balance model can be established as outlined above.

Daily Demand – hot water use

The amount of water supplying domestic hot water needs can vary considerably between dwellings. In the (likely) absence of data, a conservative estimate of hot water use is to assume 50 percent of bathroom water use and 20 percent of kitchen and laundry use is supplied by the hot water service. The water demand for bathroom, laundry and kitchen uses should be determined from local information, where available, or Tables 8.1 to 8.3 with respect to Brisbane, Perth and Adelaide. For other state and capital cities, the rates should be estimated using the factors suggested in the preceding discussion. Estimates based on this approach are included in Table 8.5.

Significance of the selected demand rate

While the actual daily demands may in reality be very different from the estimated demand rates discussed in the preceding sections, in most cases this is unlikely to result in significant difference between the ‘actual’ and the ‘predicted’ average annual yield, providing that harvested roof water is used for all suitable consumptive purposes during the wet season. These include hot water supply, bathroom and/or toilet use as well as laundry uses. This is because, beyond a certain rate of daily water use, the average annual yield becomes less sensitive to daily water use. This is apparent from the charts of indicative yields for Australian state and Territory capitals presented in Section 8.5, which show that beyond a certain demand rate which varies between cities, an increased demand has increasingly less effect on the yield.

8.4 AVERAGE ANNUAL YIELD : PROCEDURE 9

A daily water balance model is easily established using spreadsheet format such as Excel with calculations undertaken by cell formulae. An example layout of the first few rows of a spreadsheet-style calculation is shown (Table 8.4). Explanation of the column requirements is provided in the notes.

Each of the rows shown in Table 8.4 represents the water balance for one day and therefore these form only the first of thousands of rows of a spreadsheet model that should use at least 10 years of local daily rainfall record. Nevertheless, other than Column B (daily rainfall data) it is only necessary to set cell formulae in the first few rows of the record and then copy the “relative cell” formulae to the other rows. [To do so requires a basic knowledge of Excel or any other suitable spreadsheet program; the disk referred to above has built in cell formulae.] A sum for Columns D, Column F or Column I will provide, respectively :

- a sum for the entire record of rainwater tank inflow (effective roof runoff);
- overflow volume for the entire record;
- volume of the demand met by the RRHS for the entire record, i.e. summed yield for the entire record.

The summed values can be divided by the number of years of record to obtain average annual values. If desired, totals and averages for each year, or even for each season, are relatively easily extracted. Once the basic summary data are obtained, other parameters such as percentage of the required demand met by the RRHS, or percentage of days in which the tank storage can or cannot meet the full daily demand for the modelled use (from Column J), can be easily determined.

The model that is provided with the Handbook automatically calculates parameters such as average annual yield, percentage reduction in runoff to the street drainage system from the connected roof area, and percentage of the daily demand met by the RRHS.

PROCEDURE 9 : Method for determining the average annual yield

STEP 1 : Obtain the relevant daily rainfall record and adjust, if necessary, for missing data (see Section 8.3.3). Input this into the model provided and run the model for appropriate roof sizes and storage capacities as explained in the model.

OR

STEP 2 : Establish a tabular spreadsheet with headings similar to those shown in Table 8.4 and copy/paste the daily rainfall record into Column B.

STEP 3 : Establish the appropriate formulae in the cells in the spreadsheet. The effective roof area contributing to the RRHS, storage capacity, assumed daily demand (if necessary, varied seasonally), and other parameters must be considered as these need to be input into cell formulae. [This step requires a basic knowledge of the spreadsheet package – alternatively, the disc accompanying this Handbook includes the appropriate cell formulae and parameters in the RRHS model.]

STEP 4 : Sum the results in columns D, F and I over appropriate time periods, for example, the full length of record, or for each year or season, etc.

STEP 5: Repeat Steps 3 and 4 using other effective roof areas, storage capacity, assumed daily demand etc., if necessary. It is suggested that a number of storage capacity sizes be trialled.

TABLE 8.4

SIMPLE SPREADSHEET BASED MODEL FOR ESTIMATING RRHS YIELD AND OTHER PARAMETERS OF INTEREST

(A) DAY IN RECORD	(B) RAIN-FALL	(C) INITIAL	(D) INFLOW	(E) SUM	(F) OVER-FLOW	(G) VOLUME	(H) VOLUME	(I) DEMAND	(J) FLAG
1	0	2200	0	2200	0	2200	1960	240	1
2	0	1960	0	1960	0	1960	1720	240	1
3	0	1720	0	1720	0	1720	1480	240	1
4	2.8	1480	378	1858	0	1858	1618	240	1
5	51.8	1618	6993	8611	5611	3000	2760	240	1
etc.									

Column A : Day number in the record (1 = first day in the record).

Column B : Daily rainfall, in mm (preferably, the long-term historical record for the location).

Column C : Volume in rain tank (litres) at beginning of that day. Other than the first day of record (day 1) this value is the volume remaining in the rain tank at the end of the previous day (i.e. Column E minus Columns F and I, from the previous day). For the assumed volume for the first day of record, refer Section 8.3.2.

Column D : Tank inflow (litres) is the volume of rainfall (mm) from Column B, multiplied by the effective connected roof area (m^2).

Column E : Sum (litres) of the initial tank storage volume (Column C) and day's inflow (Column D). Note this may exceed the tank capacity.

Column F : The tank overflow (litres). If the value in Column E is greater than the tank size, the overflow is the value of Column E minus the tank volume expressed in litres. Otherwise the value is 0 (no overflow).

Column G : Volume after overflow (litres). If no overflow occurred on this day (Column F value is zero), this value equals the value of Column E. If overflow did occur on this day (Column F value is greater than zero), then Column G value is the tank capacity.

Column H : Value (in litres) of Column G minus the daily demand (in litres). The value of Column H may be negative.

Column I : Volume of demand (litres) actually met by the tank during that day. If the volume in Column H is negative, the value of Column 9 is the value of Column G. If the value of Column H is greater or equal to zero, the value of Column I is the required daily demand.

Column J : Various "flags" can be devised in order to determine other parameters, if so desired. The flag shown in Column J, for example, may be set at a value of "1" if the full daily demand was met by the tank (i.e. value in Column 9 is the required daily demand), or a value of "0" if the full daily demand was not met by the tank. Analysis of these values for the entire record will indicate how many days the tank was able to supply the full daily demand (for the modelled type of demand).

8.5 INDICATIVE YIELDS FOR STATE AND TERRITORY CAPITALS

The charts presented in Figures 8.1 to 8.4 provide ‘first estimates’ of the long-term average annual yields of RRHSs for state and territory capitals. They should be considered as providing indicative estimates only of yield prior to obtaining location-specific daily rainfall data for modelling. The charts were developed using data obtained from a more sophisticated, but essentially equivalent approach to that outlined in Table 8.4. For each city the rainfall stations from which data were taken to develop the charts had greater than 50 years available data. It was assumed that the effective connected roof area (see Section 8.3.3) is 90 percent of the roof area connected to storage, i.e. for the first chart, Adelaide 50 m^2, the effective roof area is 45 m^2. Of particular interest are :

- the efficacy of small storages which, in many situations, will provide the same or almost the same yield as larger storages. This has significance for the practicality and cost-effectiveness of RRHSs;
- there is often a relatively pronounced ‘flattening’ of the curves for higher demands (typically, those exceeding about 300 L/day for most roof areas).

Demand rates should preferably be based on local data (if available) from the water supply authority, other sources, or estimated for the site in question. In the absence of such information Table 8.5 provides indicative average daily demands for in-house and hot water use for state and territory capitals, determined from Tables 8.1 to 8.3 and related discussion. The values in Table 8.5 can be used with the charts on the following pages to determine indicative average annual yields for various locations, storage sizes and connected roof areas.

TABLE 8.5

INDICATIVE DEMAND FOR VARIOUS DWELLING TYPES, STATE AND TERRITORY CAPITALS

CITY	ALL IN-DOOR DEMANDS, L/DAY			HOT WATER ONLY DEMAND, L/DAY		
	TOWNHOUSE AND ‘VILLA’	MODERN DETACHED HOUSE	OLDER DETACHED HOUSE	TOWNHOUSE AND ‘VILLA’	MODERN DETACHED HOUSE	OLDER DETACHED HOUSE
Adelaide	250	470	400	75	145	120
Brisbane	420	835	630	105	210	155
Canberra	325	610	520	95	190	155
Darwin	630	1255	940	155	315	235
Hobart	300	565	480	90	175	145
Melbourne	300	565	480	90	175	145
Perth	350	560	455	90	140	115
Sydney	325	610	520	95	190	155

Note : Modern detached refers to new detached houses with an average occupancy of approximately four persons (two adults, two children); older detached houses have an assumed average occupancy of three persons.

Note that the chart for Darwin has a greater range of demand than the other cities due to the particularly high in-house water use noted for that city.

ADELAIDE (Average annual rainfall : 545 mm)

BRISBANE (Average annual rainfall : 1128 mm)

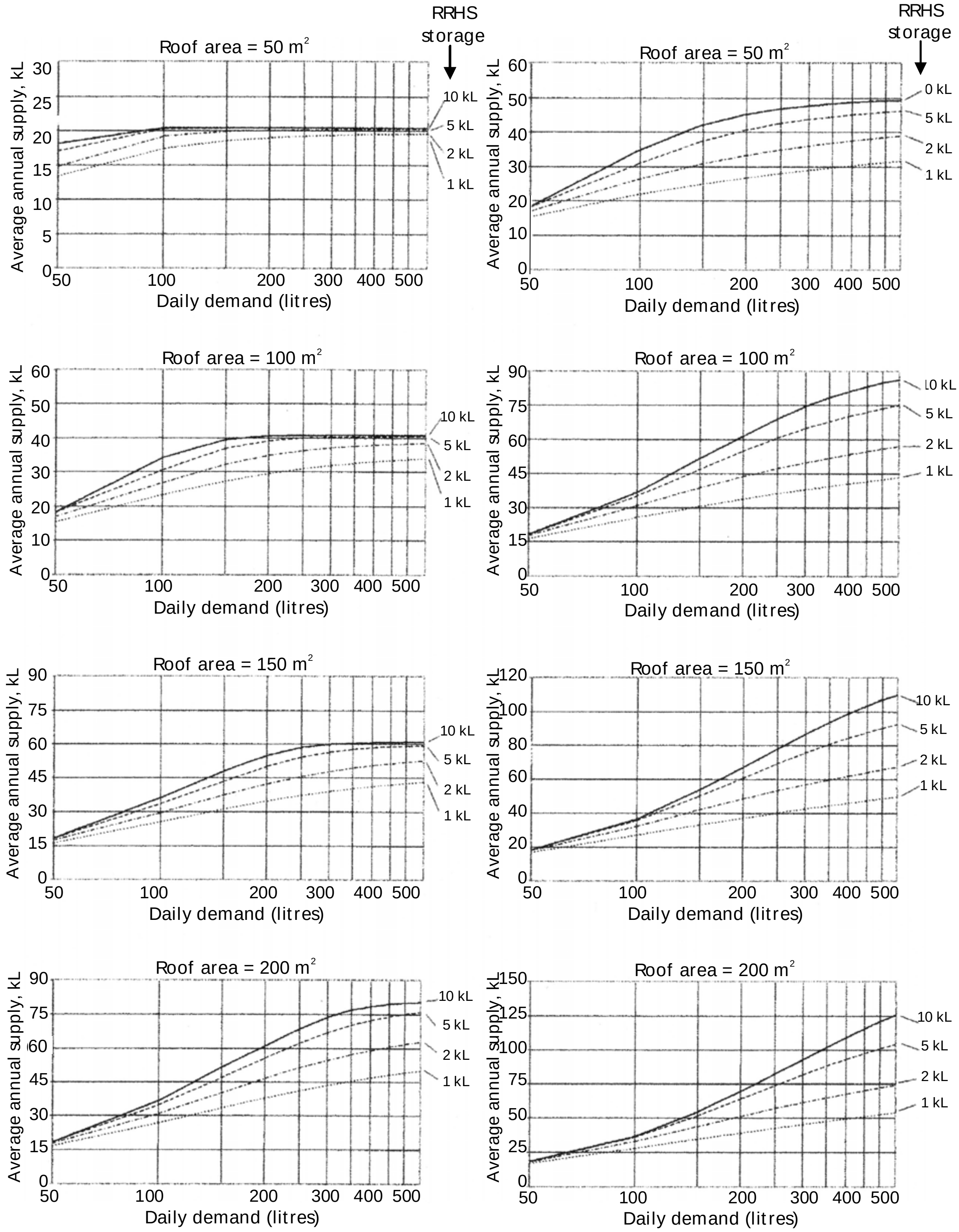


FIGURE 8.1 : Indicative Supply (Yield) from Roof Runoff Harvesting : State and Territory Capitals

CANBERRA — (Average annual rainfall : 628 mm)

DARWIN — (Average annual rainfall : 1623 mm)

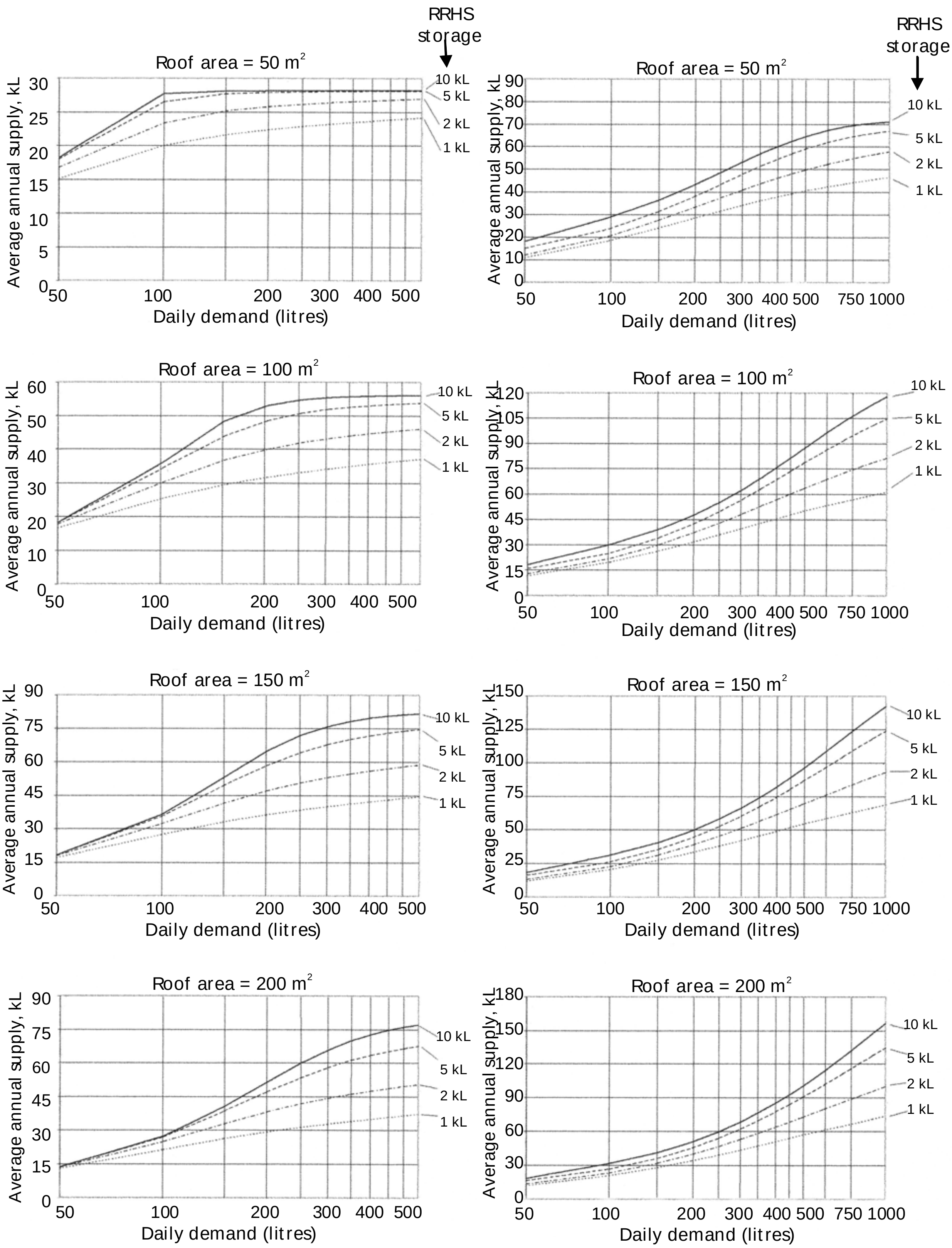


FIGURE 8.2 : Indicative Supply (Yield) from Roof Runoff Harvesting : State and Territory Capitals

HOBART | **MELBOURNE**

(Average annual rainfall : 595 mm) | (Average annual rainfall : 660 mm)

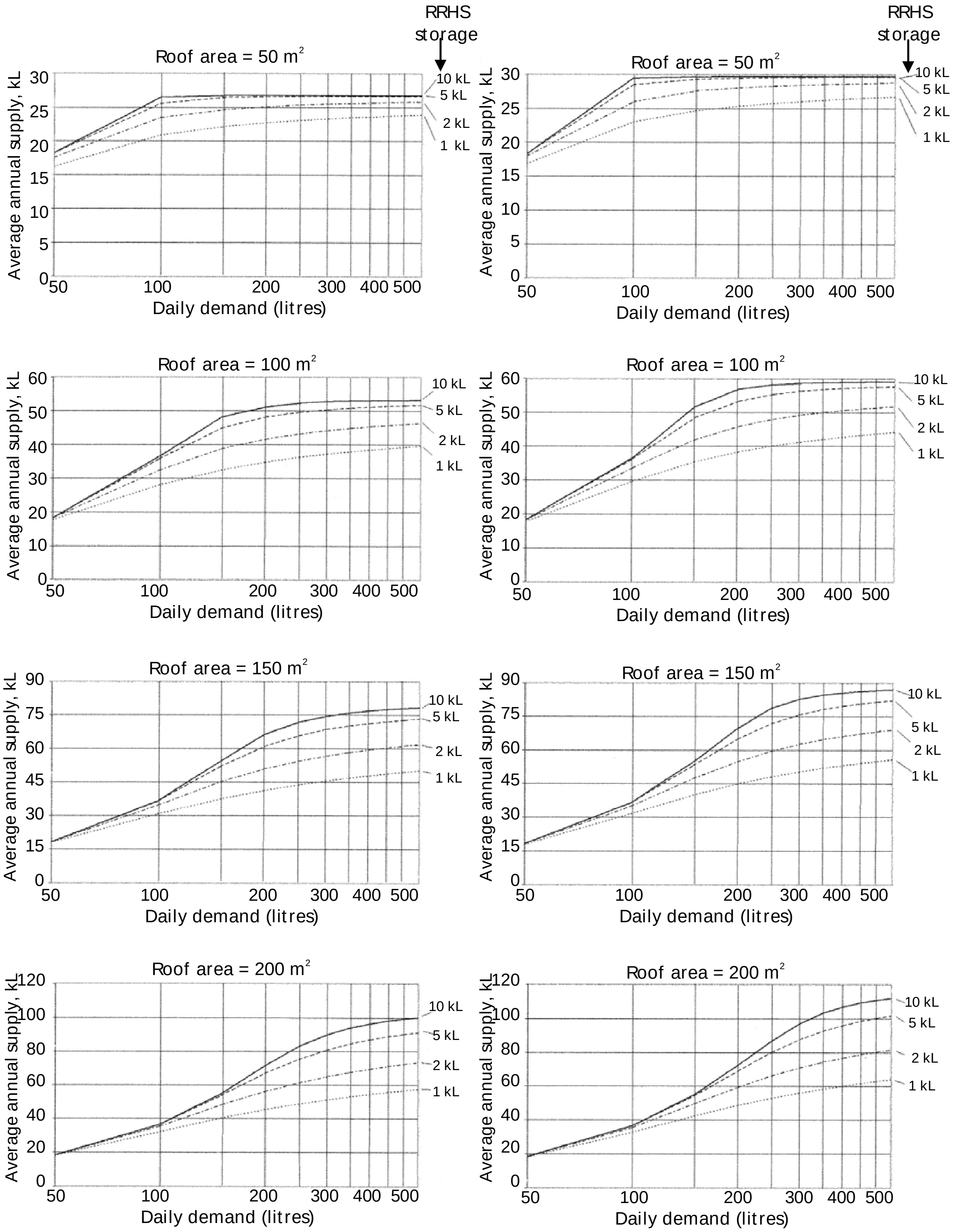


FIGURE 8.3 : Indicative Supply (Yield) from Roof Runoff Harvesting : State and Territory Capitals

PERTH

(Average annual rainfall : 870 mm)

SYDNEY

(Average annual rainfall : 1240 mm)

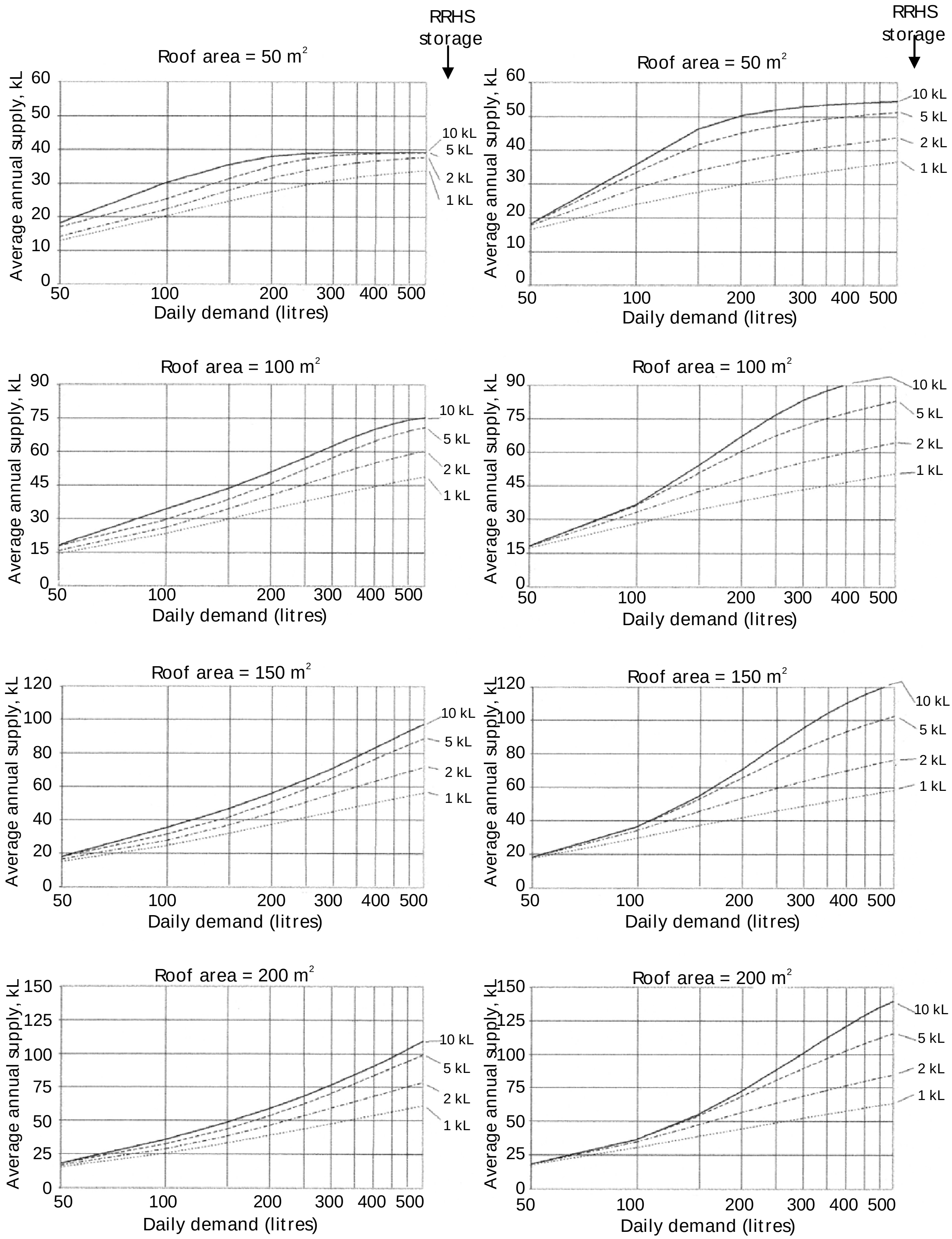


FIGURE 8.4 : Indicative Supply (Yield) from Roof Runoff Harvesting : State and Territory Capitals

8.6 A REGIONAL STORMWATER HARVESTING MODEL : PROCEDURE 10

8.6.1 Introduction

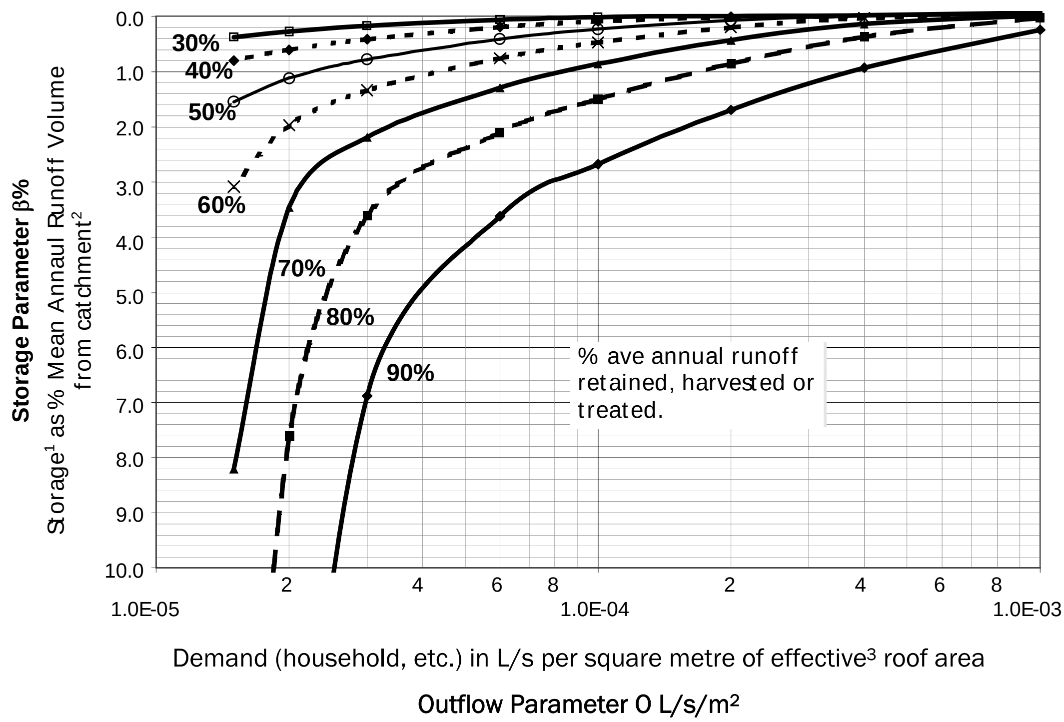
An alternative procedure to that reviewed above for fixing the sizes of rainwater and other storages can be developed from 'continuous simulation' modelling of the type introduced in Section 3.4.2 : results are presented in Figure 3.7, repeated here as Figure 8.5 (Southern Australia) for convenience. The process which lies behind the preparation of Figure 8.5 is explained in Table 3.2 and, for this reason, is not repeated here.

The design method associated with the use of this information is Procedure 10. It enables storage sizes for various harvesting devices and installations to be fixed **without** recourse to the continuous daily rainfall record for a particular location of interest which characterises Procedure 9. Procedure 10 provides general outcomes which are applicable in each of the climate zones identified in Figure 3.8. A graph for each zone is presented in Figure 8.5 or in Appendix E. Design by Procedure 10 is therefore simpler than by Procedure 9 but, because of its regional generality, is not as accurate.

8.6.2 Figure 8.5 explained

The independent variable (X axis) of Figure 8.5 represents **outflow, O,** from the storage device. This is (household or industrial) average daily demand – as reviewed in Section 8.3.3, above, expressed as "L/s/m^2 of **effective** roof area (EIA)". *Effective* roof area is the actual catchment yielding runoff directly to the rainwater tank.

In the case of a storage pond, to which Procedure 10 can also be applied, **outflow, O,** is the sum of the three parameters – average daily extraction (for use), daily seepage quantity and daily average evaporation. The first of these parameters may be known; the second and third components must be estimated. This 'sum' must then be expressed as "L/s/m^2 of catchment area (EIA)" draining to the storage facility.



NOTES :

1 : **Device** may be a pond or a rainwater tank.
2 : **Catchment** may be a ground-level paved area or a roof.
3 : Effective roof area for rainwater tanks may be as low as 80% of nominal roof area (see Section 8.3.3).

FIGURE 8.5 : General stormwater harvesting design graph – Southern Australia

The dependent variable (Y axis) gives the **storage, β,** expressed in terms of *percentage of mean annual runoff volume* discharged from the catchment (EIA). This is, simply, mean annual rainfall, with an appropriate runoff coefficient applied typically 0.75, × catchment area. The factor 0.75, and not 0.90 or 0.95, is suggested because the great bulk of annual yield from a catchment results from rainstorms of small magnitude which show relatively high losses for individual events.

All of the graphs associated with Procedure 10 (Figures 8.5 and E-1 to E-4, Appendix E) have been derived for small catchment area cases – maximum 2.0 ha. They are therefore satisfactory for residential or industrial rainwater tank sizing, but are limited in their application to pond storages providing significant use/demand : they are appropriate for dimensioning ornamental ponds, for example.

The family of curves gives average annual retention (stormwater harvesting) at seven levels between 30% and 90% inclusive, and shows that for any given percentage of runoff harvesting, the size of storage required to deliver a fixed level of demand **decreases** as the demand itself **increases**. For some, this may be a counter-intuitive outcome.

8.6.3 Procedure 10

Preliminary considerations – location, catchment, etc.

The procedure must commence with a clear description of the circumstances of the particular application.

This involves defining :

- The **climate zone** in which the catchment is located.
- **Average annual rainfall** for the site of the installation.
- **Components** of the catchment draining to the installation. This list must identify, in particular, roof areas, ground-level areas and whether they are impervious, porous or permeable, or pervious, etc. This leads to $\mathbf{A_{EIA}}$.
- **Outflow** from the device or installation (see above) converted to L/s/m^2 of equivalent impervious area, A_{EIA}.

PROCEDURE 10 – STEP 1 :

Determine equivalent impervious area of the catchment contributing runoff to the device (rainwater tank, storage pond, etc.). These data lead to determination of catchment area, $\mathbf{A_{EIA}}$.

STEP 2 :

Identify climate zone of site based on $i_{10,1}$ rainfall intensity, and select appropriate design graph from Figure 8.5 (Southern Australia) or from Appendix E (Figures E-1 to E-4).

Identify **outflow, O,** as L/s/m^2 of equivalent impervious area, A_{EIA}.

Nominate a **target percentage** of (average) annual runoff to be harvested: available values – 30%, 40%, 50%, 60%, 70%, 80% and 90%.

Locate 'point' in selected design graph corresponding to the specified value of outflow (X-axis) and target harvesting percentage and identify corresponding **storage, β,** within the range 0 to 10%.

STEP 3 :

Calculate the average annual runoff (volume) from the contributing catchment. This is :

$$\forall_A = \textbf{(average annual rainfall in metres)} \times 0.75 \times A_{EIA}\ \mathrm{m}^3 \qquad \textbf{(8.1)}$$

Required storage volume is given by $\forall_h = \beta \times \forall_A\ \mathrm{m}^3$ **(8.2)**

STEP 4 : Review the design volume, $\forall_h$

In the case of a rainwater tank, the volume, $\forall_h$, must be interpreted into a plan area and height, either of which may be considered unacceptable in terms of space available or appearance in certain circumstances leading to consideration of further (volume) options.

In the case of a pond intended for the dual purposes of providing water for daily use as well as amenity, a necessary compromise between acceptable depth (safely issues) and available space (plan area) may have to be reached requiring the exploration, also, of other volume options. This task is made difficult because evaporation from the pond cannot be satisfactorily estimated until its (final) plan area is known : only an approximation for this component of outflow can therefore be substituted into STEP 2.

Such situations involve returning to STEP 2 and repeating the design process using a different value of **target percentage** (harvesting) to arrive at a more acceptable outcome. This process is iterative and continues until such time as a satisfactory final solution or compromise is reached in each case.

This process of "… iteration and … compromise …" has been developed into a software package which is reviewed, briefly, in Section 8.6.5.

This terminates Procedure 10.

8.6.4 Illustrative examples

Example 8.1

The storage volume of a rainwater tank is required for a residence in Brisbane ($i_{10,1}$ = 71 mm/h) with effective roof area draining to the tank equal to 150 m^2. The average annual rainfall at the Brisbane site is 1164 mm. Household estimated average daily water use from the rainwater tank is 350 litres.

Preliminary considerations

- The catchment is located in Brisbane – Northern Australia climate zone.
- Average annual rainfall is 1.164 m.
- Catchment area – *effective* roof area - $\mathbf{A_{EIA}}$ = 150 m^2.
- Outflow based on household average daily water use (350 L/day) and roof area A_{EIA} = 150 m^2 leads to outflow, O = 2.70×10^{-5} L/s/m^2.

PROCEDURE 10 – STEP 1 :

Catchment area, $\mathbf{A_{EIA}}$ **=** 150 m^2.

STEP 2 :

The site location is Brisbane : climate zone – Northern Australia, design graph E-4.

Outflow, O = 2.70×10^{-5} L/s/m^2 of equivalent impervious area, A_{EIA}.

Target percentage of (average) annual runoff to be harvested : Trial 1, use 30%. Note that in the case of Northern Australia, it is not possible to harvest high percentages of annual (roof) runoff unless the daily quantity used is, similarly, large. In the present case – with only 350 L/s/m^2 of A_{EIA} being used – it is only possible to harvest around 30% of average annual runoff.

This gives (Figure E-4, Appendix E) **storage, β = 3.6%.**

STEP 3 :

Average annual runoff (volume) from the contributing catchment is :

$$\forall_A = (1.164) \times 0.75 \times 150 \text{ m}^3 = 131 \text{ m}^3 \tag{8.1}$$

Required storage volume is $$\forall_h = 0.036 \times 131 \text{ m}^3 = 4.7 \text{ m}^3 \tag{8.2}$$

STEP 4 : Review the design volume, $\forall_h$

Rainwater tank volume 4.7 kL is acceptable.

This terminates Procedure 10.

Example 8.2

A storage gutter system ("Rainsaver" see Section 2.2) is required for a residence in Sydney ($i_{10,1}$ = 60 mm/h) with effective roof area draining to the device equal to 200 m^2; the length of perimeter eave-guttering is 70 m. The average annual rainfall at the Sydney site is 1,226 mm. Household estimated average daily water use from the storage guttering is 350 litres.

Preliminary considerations

- The catchment is located in Sydney – Upper-Intermediate climate zone.
- Average annual rainfall is 1.226 m.
- Catchment area – roof area – $\mathbf{A_{EIA}}$ = 200 m^2.
- Outflow based on household average daily water use (350 L/day) and roof area A_{EIA} = 200 m^2 leads to outflow, O = 2.03×10^{-5} L/s/m^2.

PROCEDURE 10 – STEP 1 :

Catchment area, $\mathbf{A_{EIA}}$ = 200 m^2.

STEP 2 :

The site location is Sydney : climate zone – Upper-Intermediate, design graph E-3.

Outflow, O = 2.03×10^{-5} L/s/m^2 of equivalent impervious area, A_{EIA}.

Target percentage of (average) annual runoff to be harvested : Trial 1, **use 40%**.

This gives (Figure E-3, Appendix E) **storage, β = 2.3%**.

STEP 3:

Average annual runoff (volume) from the contributing catchment is :

$$\forall_A = (1.226) \times 0.75 \times 200 \text{ m}^3 = 184 \text{ m}^3 \qquad \textbf{(8.1)}$$

Required storage volume is $$\forall_h = 0.023 \times 184 \text{ m}^3 = 4.2 \text{ m}^3 \qquad \textbf{(8.2)}$$

STEP 4 : Review the design volume, $\forall_h$

In the case of the residence in Sydney with 70 m length of eave-guttering, 4.2 m^3 of storage translates into $\frac{4,200}{70}$ = 60 L per lineal metre. This is greater than the capacity available in the standard "Rainsaver" installation (23 L/m).

Return to **STEP 2** and select percentage of (average) annual runoff to be harvested as 30%. This leads to storage, **β = 0.9%. Hence (STEP 3) :**

$$\forall_A = (1.226) \times 0.75 \times 200 \text{ m}^3 = 184 \text{ m}^3 \qquad \textbf{(8.1)}$$

Required storage volume is $$\forall_h = 0.009 \times 184 \text{ m}^3 = 1.66 \text{ m}^3 \qquad \textbf{(8.2)}$$

This storage volume distributed around 70 m of eave-guttering represents 23.6 L/m which corresponds, approximately, to the storage capacity of the standard "Rainsaver" unit.

This terminates Procedure 10.

Example 8.3

An ornamental pond is to be designed for the grounds at the rear of the church site illustrated in Figure 5.3, located in Adelaide: the 'normal level' in the pond will be fixed by harvesting (not flood control) considerations. The pond will receive all storm runoff from the catchment defined in Example 5.3. Outflow from the pond will include seepage into the native clay soil of the site and evaporation from the pond surface including an allowance for evapotranspiration from native water-plants. No additional use of the stored water is contemplated other than during periods of high rainfall when pond excess *up to the defined overflow level* (flood control) may be applied to open space irrigation.

Preliminary considerations

- The catchment is located in Adelaide – Southern Australia climate zone.
- Average annual rainfall is 0.58 m.
- Catchment :
  total connected roof area .... 1,600 m^2
  total connected paved area .... 360 m^2
  total connected pervious area (C_{10} = 0.10) .... 680 m^2
- Outflow : for preliminary (trial) estimate, assume pond surface area, A_p = 50 m^2 :
  seepage = 1×10^{-6} m/s × (Moderation factor = 2.0) × (pond area = 50 m^2) .... 8,640 L/day
  evaporation/transpiration (allow 3.0 m per annum, 50 m^2 pond).................................. 410 L/day

PROCEDURE 10 – STEP 1 :

Catchment area, $\mathbf{A_{EIA}}$ **=** 1,600 + 360 + (0.10 × 680) = **2,028 $\mathbf{m^2}$**.

STEP 2 :

The site location is Adelaide : climate zone – Southern Australia, design graph Figure 8.5.

Outflow, O **=** (8,640 + 410) = 9,050 L per day = 0.1047 L/s

= 5.16×10^{-5} L/s/m^2 of equivalent impervious area, A_{EIA}.

Note : Evaporation/transpiration is not constant throughout the year but is assumed so here to determine Outflow O. This approximation is acceptable because it represents less than 5% of O.

Target percentage of (average) annual runoff to be harvested: Trial 1, use 90%.

This gives (Figure 8.5) **storage, $\boldsymbol{\beta}$ = 3.8%**.

STEP 3:

Average annual runoff (volume) from the contributing catchment is :

$$\forall_{\mathbf{A}} = \mathbf{(0.58) \times 0.75 \times 2{,}028\ m^3 = 882\ m^3} \qquad \mathbf{(8.1)}$$

Required storage volume is $$\forall_{\mathbf{h}} = \mathbf{0.038 \times 882\ m^3 = 33.5\ m^3} \qquad \mathbf{(8.2)}$$

STEP 4 : Review the design volume, $\forall_{\mathbf{h}}$

Based on 90% stormwater harvesting, pond storage volume 33.5 m^3 leads to pond depth, 0.67 m, for the 50 m^2 plan area pond. This may be considered too deep – for safety reasons – and a shallower depth preferred.

Return to STEP 2 and use harvesting percentage 80% instead of 90%. This leads to :

storage, β = 2.2% and $\forall_h$ = 0.022 × 882 m^3 = 19.4 m^3,

hence, pond depth, **H = 0.39 m**, which is acceptable.

The review process in the case of an ornamental pond receiving surface runoff, must also take account of stormwater generated in minor and major storm events. This requires re-design of the facility as a 'dry' pond using Procedure 3, Section 5.1. Correct process with this calculation requires the storage available in the ornamental pond to be *ignored* (assumed always full) and the 'dry' pond to be located, effectively, above it. The computation (ARI, Y = 100-years design requirement, following the approaches presented in Examples 5.3 and 5.4, Section 5.2) leads to a design for the 'dry' pond (many options) :

$$A_p = 240 \text{ m}^2 \text{ with depth, } H = 0.30 \text{ m.}$$

This outcome could be interpreted into the site as an attractive landscaped area, 240 m^2 in extent and 'sunk' 0.30 m below natural surface (the 'dry' pond), within which the stormwater harvesting pond, 50 m^2 in area and a further 0.40 m deep, is located.

The following points should be noted in relation to this addendum (the 'dry' pond) :

- Inundation of the 'dry' pond area (surcharge from the stormwater harvesting pond) is likely to occur about four times per year because its capacity is closely associated with ARI, Y = 0.25-years.

- The design approach suggested above may be unacceptable on the grounds of cost and a design ARI significantly less than Y = 100 years required, for example ARI, Y = 5 years. In this event, consideration should be given to the provisions of the last four paragraphs of Section 4.2.6 and a design prepared taking account of the full range of storm durations from 10 minutes to 72 hours (see Example 5.5 in Section 5.2).

This concludes Procedure 10.

8.6.5 Rainwater tank design software package

It will be clear from Examples 8.1 to 8.3 inclusive that the process of using any of Figures 8.1 to 8.4 inclusive, or information from a graph such as Figure 8.5 (or any of the graphs from Appendix E), does not lead to a *direct* solution in any given practical case. As stated in Section 8.6.3, the design process involves iteration and compromise. A software package – publicly accessible – has been developed to aid this process. It is available from the Centre for Water Science and Systems, University of South Australia website at : http://www.newdev.unisa.edu.au/water/UWRG/publication/raintankanalyser.asp

The primary use of the package is for matching a rainwater tank size (volume) to any given set of **domestic** circumstances (location, roof catchment area, daily demand, etc.) including irrigation if required, but it is equally capable of providing tank sizes (up to 20 kL capacity) associated with industrial settings. The software brings together elements of both Procedures 9 and 10. Its main features are :

- Provides, directly, a **suggested** tank size based on *hydrological* considerations; the user is also presented with information enabling an alternative, optimum *economic* size to be selected. The process is illustrated, directly, for each of the state and territory capital cities (nominal tank sizes).
- Includes capability for users to 'paste in' continuous (daily) rainfall data and local monthly irrigation demand for any location.
- Includes capability to fix tank storages for in-house only or outdoor only uses, or combination of in-house/outdoor uses.
- Each application of the software reports :
  - average annual in-house and irrigation demand in kL/annum;
  - **suggested** tank size (hydrological considerations) in kL;
  - average annual yield in kL/annum;
  - average number of days when less than full demand is provided;
  - average number of days with zero supply;
  - percentage of total demand supplied by *suggested* tank.

8.7 RAINWATER TANKS AND URBAN DRAINAGE : CONSEQUENCES FOR FLOOD CONTROL

It has long been held that rainwater tanks or similar installations, operated in the manner described in the foregoing sections of this chapter in an urban catchment, can result in significant reductions in stormwater peak flows generated in that catchment. Local government agencies have, on the other hand, been traditionally unimpressed by these claims and have, instead, maintained that reduction of peak flow through the presence of rainwater tanks cannot be relied upon because streamflow peaks are likely to coincide with the tanks being full and therefore incapable of providing benefit by way of detention storage. This 'case' is further strengthened when it is recognised that conventional use of rainwater tanks by past generations has been to supply minimal in-house demand, for example tea-making, with the consequence that rainwater tanks have remained full or nearly so for the greater part of the time.

Steadily growing population and profligate use of water to maintain exotic lawn and garden species, coupled with forecasts of adverse climate change, have led to the prospect of serious water shortages occurring in the foreseeable future in many of our major urban concentrations. Water restrictions imposed in the early years of the new century are evidence of this recognition. Two potential solutions are being widely canvassed – rainwater (or similar) on-site storages becoming mandatory, and conversion of the Australian garden culture from its preoccupation with exotic plants and lawn grasses to native and drought-resistant varieties (Argue and Barton, 2004).

In this climate of water conservation and cultural change, the types of devices and systems reviewed in Section 2.2 are likely to assume an importance in urban Australia of the future not seen since the days of the pioneers when the only water which could be relied upon was that available on-site – roof runoff, recycled water and groundwater, where available. It follows that future use of roof runoff collection/ storage systems will not be limited to the trivial uses of the past, such as tea-making but, rather, to purposeful and strategic exploitation of the resource which it should be remembered is delivered "free" at good quality to every residence and commercial/industrial building in the nation. Earlier sections of Chapter 8 provide the basis for bringing about this change in rainwater collection/use; but its consequences for flooding have not been addressed.

So how is the predicted change likely to impact on flood management in the urban landscape?

There are complex issue here – interaction between two rainfall-dependent but significantly different phenomena (rainwater collection/use and catchment peak flows) – which can only be solved by analyses which have firm bases in the 'continuous simulation' modelling of both the collection/use aspect as well as catchment storm runoff flow estimation. Research has been undertaken in recent years on the problem [Allen (1993); van der Wel (2000); Coombes and O'Loughlin (2001), Allen (2002)] but has not satisfactorily answered the principal questions : What quantifiable effect can operating rainwater tanks have on catchment outflow peaks? And, in particular, can the presence of domestic rainwater tanks operated as outlined above be relied upon to absolve a municipal authority from the need to upgrade stormwater infrastructure as urban development/re-development takes place?

An exploration of these questions was conducted by Coombes (2002) who used 'continuous simulation' modelling to compare the volumes of conventional OSD tanks required on a residential allotment near Newcastle, NSW, to achieve flood mitigation objectives, against rainwater tank storages designed to satisfy allotment stormwater harvesting as well as (the same) flood mitigation objectives. 'Continuous simulation' was then applied to a proposed 39 ha sub-division, also near Newcastle, which incorporated allotment-sized rainwater tanks throughout. Stream flows computed for the proposed development showed reductions (below values determined for a 'traditional' drainage layout) of between 10% and 44%. Coombes described his analysis as an "initial foray" with improved performance likely through optimisation.

Pezzaniti (2003) investigated the consequences for flood mitigation of domestic rainwater tanks in a 50 ha urban catchment in Adelaide. He used 'continuous simulation' modelling based on over 100 years of rainfall records and took account of antecedent conditions in the rainwater tanks at the onset of critical design storm events. The capacity of the (existing) catchment drainage network was assessed to be ARI, Y = 2-years. The study showed that it was possible for this stormwater infrastructure to convey ARI, Y = 5-years peak flows if large (4.5 kL) rainwater tanks were fitted to each allotment and in-house water demand (from the tanks) was set at 300 L/day. This value of use is in the middle of the range for typical Adelaide households (see Table 8.3) and, therefore, quite feasible. Modelling of lower in-house daily demand by the households did not result in any significant benefits for the drainage infrastructure. Also, small rainwater tanks (2 kL capacity) showed no drainage benefit at all.

A further, interesting outcome of the study related to the spatial distribution of rainwater tanks within the catchment. The same drainage infrastructure benefits in terms of peak flow conveyance could be achieved by applying rainwater tanks to only the upper two-thirds of the catchment. The economic benefit to a drainage authority of incorporating rainwater tanks into a new catchment is relatively small. However, the benefits of retrofitting tanks in the favourable circumstances reviewed above, namely 4.5 kL tanks with 300 L/day use, significantly outweigh the costs of upgrading existing drainage infrastructure in the face of widespread re-development.

Hardy, Coombes and Kuczera (2004) investigated the problem of rainwater tank impact on flood management in hypothetical, medium-density residential catchments in four Australian capital cities – Adelaide, Brisbane, Melbourne and Sydney. The catchments ranged in size from 2.5 ha to 10 ha. The developments incorporated street drainage networks matched to the critical ARI, Y = 5-years design standard. Two domestic scenarios were investigated :

- Residences with no rainwater tanks (the "no tank" option); and,
- Residences with 5 kL rainwater tanks (the "tanks" option).

Rainwater abstracted from the tanks was applied to both indoor and outdoor uses in the model. Peak flows generated in the catchments were determined for design storms ranging from ARI, Y = 0.25-years up to and including 100-years events; the storm duration range was 5-minutes up to 72-hours.

The study sought answers to two main questions :

1. What storage is available in a 5 kL rainwater tank prior to the arrival of a significant storm event? and,

2. What differences in peak flows can be expected between catchment outflows in the "no tanks" and "tanks" cases ?

Results of the modelling showed quite similar storage volumes being available in the rainwater tanks of the southern Australian cities (Adelaide and Melbourne) over the full range of ARIs normally associated with 'minor' and 'major' stormwater drainage systems. However, Brisbane (representing Northern Australia) showed indications of an ARI-dependent (direct) relationship with respect to available on-site storage. Results for Sydney (Upper-Intermediate climate zone) fell between these extremes.

Results (Hardy et al) from the second issue investigated, above, showed that rainwater tanks can be associated with significant peak flow reductions in the low rainfall regions of Australia, but not in the tropics. Outcomes for Intermediate Australia (represented by Sydney), again, fall between these extremes.

Collating all of these results, it may be concluded that the beneficial impact which the presence – and daily use – of rainwater stored in tanks is likely to have on flood peaks in the waterways of an urban catchment in Australia is related to :

- The common tank size and (daily) extraction rate;
- the spatial distribution of tanks within the catchment;
- whether new development ("greenfields") or re-development of old housing stock is involved;
- the geographical location (climate) of the catchment.

This brief review serves to inform us that the use of rainwater tank storage in the context of flood management is complex and, certainly, not amenable to any simple rule, such as "5 kL rainwater tanks installed throughout an urban catchment will reduce main channel peak flows from ARI, Y = 5-years, to ARI, Y = 2-years", highly desirable though this might be. It is our judgement that such a rule will never emerge for nation-wide application, although some guideline of this nature may be possible in specific locations under particular circumstances of urban development.

9 CONCLUDING DISCUSSION

9.1 OVERVIEW

The **vision** of stormwater management proposed in the Handbook, ranging from conservation of natural drainage lines and their associated fauna and flora ecosystems in forested catchments facing development, through to restoration of former drainage paths in catchments 'lost' to overdevelopment, was firmly set in Chapter 1 (see Section 1.4). The remaining chapters have provided the tools by which that vision can be realised. Indeed, it may be argued that this is the primary contribution which stormwater 'source control' can make to WSUD, not only because of its role in reducing the impacts of storm runoff, particularly in catchment 'bottom lands', but also because of the improved amenity and heightened community and environmental values which it brings.

9.2 SOME ISSUES

9.2.1 Relationship of the Handbook to Current Australian Practice

9.2.2 Resistance to change

9.2.3 Differences in Northern, Intermediate and Southern Australian design outcomes

9.2.4 Pollution control in Australian urban catchments

9.3 THE NEED FOR CASE STUDIES

9.3.1 Stream management scenario/strategies

9.3.2 Detention/retention in the urban landscape

9.3.3 Rainwater tanks and flood control

9.3.4 Cost and policy matters

9.4 RESEARCH DIRECTIONS

9.4.1 Acceptable 'emptying time' for OSR devices

9.4.2 Detention/retention : spatial distribution

9.4.3 Problems of water retention and 'reactive'soils

9.4.4 Pollution control – fine sediments

9.5 CONCLUSION

[These sections omitted from the Student Edition]