3. THEORY, DATA AND SOME IMPLEMENTATION ISSUES

3.1 INFILTRATION OF STORM RUNOFF

3.1.1 Basic theory

Direct **infiltration** of storm runoff into a residential allotment topsoil surface, a permeable pavement, a grassed hard-standing area or into a well-vegetated loamy area specially intended for this purpose, represents a simple application of Darcy's Law as expressed in the following equation :

$$q = k_h . I \qquad \textbf{(3.1)}$$

where q = infiltration flow rate per unit surface area ($m^3/s/m^2$ = m/s)

k_h = hydraulic conductivity (m/s)

I = hydraulic gradient, $\frac{\Delta h}{\Delta \ell}$ (m/m).

As in all storm runoff infiltration and seepage analyses, the 'fully drained' condition is assumed :

$$I = \frac{\Delta h}{\Delta \ell} = 1.0 \qquad \textbf{(3.2)}$$

Measurements made at many field sites and retention installations show this assumption to be, generally, satisfactory. The qualification which should be noted, however, is that I = 1.0 represents the limiting condition to which the hydraulic gradient tends during the wetting-up process. Field situations where high infiltration rates are observed in the initial stages of wetting-up are produced **not** by hydraulic gradients greater than 1.0 but, rather, as a consequence of water filling the void spaces between coarse particles.

Stormwater flow infiltration, being intermittent, frequently shows void-filling, as described above, before progressing to the fully drained condition. Design procedures which employ the I = 1.0 assumption are therefore, generally conservative.

It follows that direct infiltration of surface runoff into a facility specifically set aside for this purpose can be accomplished by matching the peak surface design runoff flow from the contributing catchment, Q_{peak}, to its capacity for infiltration, thus :

$$Q_{peak} = q.A_i \qquad \textbf{(3.3)}$$

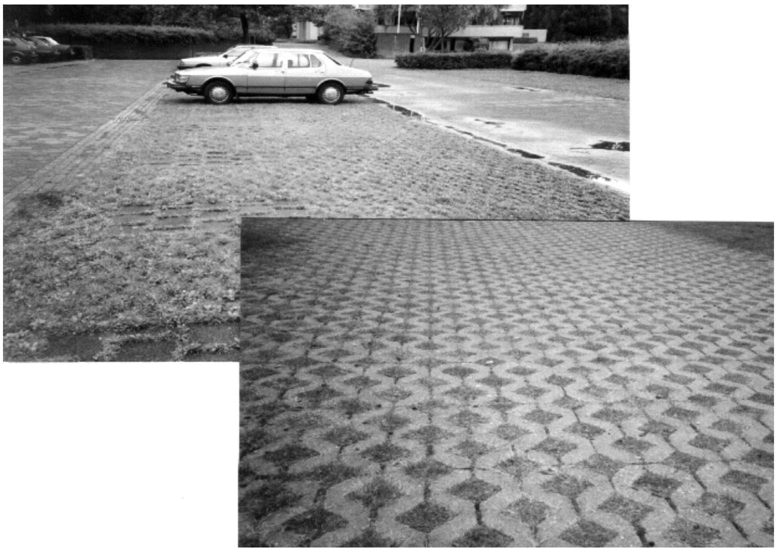
where $q \equiv k_h$, the design hydraulic conductivity [see Eqns. (3.1) and (3.2) above],

and A_i = surface area available for infiltration.

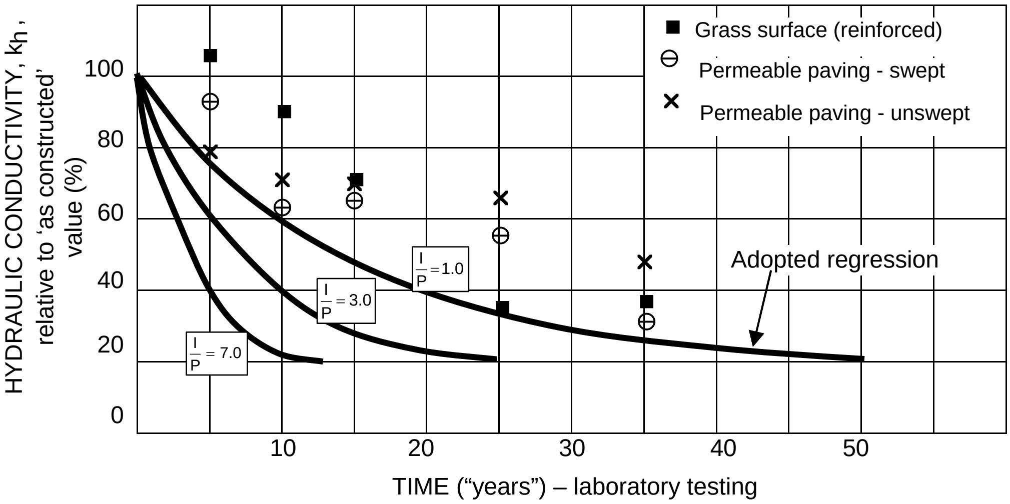
The design of permeable paving surfaces – typically, concrete block pavers with vertical 'slots' or gravel-filled 'shafts' – is amenable to the above theory without modification, provided the overall or global value of hydraulic conductivity is known for the product and its underlying sub-structure system. Application of the (above) theory to hard-standing areas of the type illustrated in Figure 3.1a must incorporate a **blockage factor, Ψ**, to take account of the impervious surface area which is **not** subject to infiltration. A blockage factor, $\Psi = 0$ may be applied in cases where the paving is uniformly porous, and to permeable paving systems for which *overall hydraulic conductivity is known*.

However, the issue of greatest concern for designers of such porous/permeable surface systems, is the effect that **sediment accumulation** has on permeability (hydraulic conductivity, k_h) as the installations "age" in service: this important issue is addressed in Sections 3.1.2 and 3.1.3.

The matching of peak flow rate to infiltration capacity (with zero site overflow discharge) is a valid approach for minor drainage systems designed to accept the typical ARI, Y = 2, 5 or 10 years storm runoff. The technology has also been demonstrated beyond this range (to ARI, Y = 20-years, 50-years and even 100-years events) with 'sheet' flow applied to residential allotment topsoil surfaces (Riley, 2001).



(a) : Example of block-reinforced grass used for vehicle hard-standing areas



(b) : Hydraulic conductivity, k_h , deterioration trend curve for porous and permeable paving filters subject to (fine) sediment loading, based on laboratory results
(after Rommel et al, 2001; UWRC, 2002a and b)

FIGURE 3.1 : Two aspects of stormwater infiltration into porous and permeable paving

3.1.2 'Lifespan' and the effect of sediment accumulation on permeability

The origins of sediment in the urban landscape and observed impacts of water-borne particulate matter on the performance of components of retention systems, were reviewed briefly in Section 1.3.3. The clear lesson which emerges from those considerations, particularly for systems of the type under discussion here – porous and permeable surfaces – is that they can be expected to show steadily reducing permeability (infiltration rate or hydraulic conductivity, k_h) as their years of service lengthen towards eventual reconstruction or reinstatement. The term **'lifespan'** is used to describe and quantify this phenomenon**.**

Reliable data on 'lifespan' or rates of permeability reduction in operating systems are either non-existent or anecdotal (Pratt, 1997): the studies by Rommel et al (2001) and UWRC (2002a and b) aimed to resolve these uncertainties. Testing by Rommel et al of a proprietary block paving system with internal geotextile filter layer used in a simulated (laboratory test rig) car park catchment element **with equal areas of impervious and permeable paving,** that is **I/P = 1.0**, showed a characteristic regression in filter permeability with time. [The simulation experiments applied one "year" equivalent of Adelaide, South Australia, rainfall plus sediment to the rig in each 7-hour period of testing.] The sediment application rate was more than 200 mg/L for the full period: this is a relatively high value for car parks and established (but not construction phase) urban catchments. The graph of hydraulic conductivity normalised to 'as constructed' values, versus testing time in "years", is presented in Figure 3.1b. Results from tests on an identical rig with grass (growing in a sand/gravel mixture : see Section 7.8.5) receiving the runoff, are also plotted in Figure 3.1b. 'As constructed' values for k_h were : 1.0×10^{-3} m/s for permeable paving and 2.5×10^{-4} m/s for the sand/gravel mixture.

The first important conclusion which can be drawn from Figure 3.1b is that permeability reduced to around 40% of 'new' product value over a period of some 30 – 35 (experimental) "years" : the adopted regression shows a drop to 20% of 'as constructed' value at 40 – 50 years. Taking this period as an acceptable 'lifespan' for porous/permeable paving systems leads to the following design guideline for surface infiltration, based on the adopted regression (I/P = 1.0) :

Guideline 1 : Design procedures for porous and permeable infiltration systems based on Eqn. (3.3) should adopt hydraulic conductivity, k_h, equal to **20% of 'new' product (or system) value for permeability** to ensure acceptable 'lifespan' performance.

Important qualification : This guideline applies to **fine** sediment input retained on an internal geotextile layer in the case of permeable paving (see Figure 2.3), and *on the vegetated surface* in the case of the porous system. A 'lifespan' of at least 40 years can be expected for an installation such as a car park with impervious-to-porous/permeable paving ratio, I/P = 1.0 (the geometrical ratio reported by Rommel et al and UWRC). It would be quite wrong to assume the same performance in a car park with significantly different impervious-to-porous/permeable ratio. For example, if I/P is increased to 3.0 thereby doubling the load on the filter system, then the 'lifespan' is reduced to half, as indicated by the derived curve in Figure 3.1b. Again, if I/P is increased to 7.0, 'lifespan' is not likely to be more than 12 years. These (latter) operating periods are not 'long-term', but may nevertheless be satisfactory in many applications and/or may be acceptable periods between major reconstruction or reinstatement of the paving system down to the level of the geotextile layer, inclusive. The 20% 'new' product value (of permeability) is recommended for universal use, but practitioners must recognise that the 'lifespan' which results is likely to vary depending on the I/P value of each case, as explained above.

3.1.3 Surface blockage and 'lifespan'

Pollutant input to a porous or permeable paving system including coarse sediment, vegetative matter (leaves, blossoms, berries, etc.) as well as fine sediment is frequently **not** evenly distributed over the entire receiving surface. Instead, the pollutant supply may enter the paved area as a 'line source' along the edge of a carriageway or as a jet discharged from an upstream gutter. In either case, the porous/permeable surface where the flow makes first contact suffers the full impact of the pollutant stream, and may reach a situation of complete blockage by particulate matter at a relatively early stage in the life of the facility. The type of blockage referred to here is evidence of the porous/permeable system acting as a GPT (gross pollution trap). With time, the area affected spreads downstream as a 'blockage front'.

In vegetated porous systems, this front is almost imperceptible, being obscured by the grass or other groundcover; furthermore, the filtering action of the grass coupled with regular removal of retained material through mowing, significantly diminish the impact of the "blockage front" in these systems. But, in surfaces finished with permeable paving, it may be quite apparent soon after commissioning. This should not cause alarm or be the signal to initiate major reinstatement works! The permeable paving system **downstream** will readily cope with the flow entering the facility perhaps for many years, provided it has been designed in accordance with Guideline 1 (and following qualification) above, and also, that the area sacrificed to blockage is located **in the uppermost segment of the porous/permeable paved surface**.

However, a new guideline is necessary to take account of this category of surface blockage, particularly where regular inspection and maintenance of the porous or permeable surface cannot be guaranteed and/or where no provision is made for interception of this type of particulate matter passing from the upstream, impervious catchment.

Guideline 2 : Porous and permeable infiltration systems receiving untreated runoff from contributing, ground-level impervious catchments, *and where regular inspection/maintenance cannot be guaranteed*, should have impervious to porous/permeable area ratios (I/P) not greater than the values given by Table 3.1, to ensure the indicated 'lifespan' performance.

TABLE 3.1
SURFACE BLOCKAGE : I/P RATIOS AND ADVISORY 'LIFESPANS'*
Systems based on **permeable paving** : use 'lifespan' (table) values directly
Systems based on **vegetated porous surfaces** : apply factor of 5 to tabulated 'lifespan' values

CATCHMENT DESCRIPTION	I/P RATIO 0	I/P RATIO 1.0	I/P RATIO 2.0	I/P RATIO 3.0	I/P RATIO 5.0	I/P RATIO 7.0
Parking bays in coastal suburb beside park (many trees)	lifespan : 10 years	lifespan : 5 years	lifespan : 3.3 years	lifespan : 2.5 years	lifespan : 1.7 years	lifespan : 1.2 years
Parking bays or car park in coastal suburb : site without trees	lifespan : 20 years	lifespan : 10 years	lifespan : 6.7 years	lifespan : 5 years	lifespan : 3.3 years	lifespan : 2.5 years
Parking bays or car park in average suburb : site with some trees	lifespan : 20 years	lifespan : 10 years	lifespan : 6.7 years	lifespan : 5 years	lifespan : 3.3 years	lifespan : 2.5 years
Parking bays or car park in average suburb : site without trees	lifespan : 40 years	lifespan : 20 years	lifespan : 13.3 years	lifespan : 10 years	lifespan : 6.7 years	lifespan : 5 years

* Based on observations of field installations in Adelaide over five years period.

This Guideline (and Guideline 1, above) is derived from testing regimes which represented – in the field (and laboratory) – 'established neighbourhood' conditions, much less severe than 'construction phase' conditions. However, Guideline 2 should be interpreted, with respect to I/P ratios as involving flow contributed from **ground-level paved surfaces** only to establish values for I. In these circumstances, Guideline 1 still needs to be applied with full account taken of the **total** catchment contributing runoff, but Guideline 2 can be restricted to consideration of ground-level paved surfaces only and the contributing roof area can be ignored. Note that I/P = 0 corresponds to a fully paved porous/permeable site.

Another circumstance in which the I/P ratios employed in Guidelines 1 and 2 may differ is where special provision is made to trap coarse sediment before it passes onto a permeable surface. If a system capable of retaining all pollution components greater than 300 μm can be installed (upstream) to perform this duty, then design according to Guideline 1, only, is required. Of course, maintenance of the type(s) described by Dierkes et al (2002) will prolong the 'life' of these systems. Such maintenance, will also improve their appearance : this may be a determining factor where permeable paving is used in a high profile urban environment.

The same principle of I/P ratio recognition explained in **Important Qualification** following Guideline 1, above, explains the entries in Table 3.1 for I/P ratios other than 1.0. Also, it should be noted that 'lifespan', as it applies in Guideline 2, can be interpreted as the time gap between reconstruction/reinstatement operations affecting **the surface components only**. While inspection/assessment of the geotextile layer within a permeable paving system being upgraded in this way is strongly advised, it is quite possible that its replacement is unnecessary (Adelaide observation).

Total system design therefore involves use of an appropriate procedure, for example Procedure 1A or 1B (Section 5.1), applying Guideline 1 and relevant I/P information from Figure 3.1b, followed by Guideline 2 and advice from Table 3.1. This will normally result in **two** values for area in permeable systems where 'lifespan' is specified OR, two values of 'lifespan' where (I/P) ratios are fixed. The maintenance/reinstatement consequences of these alternatives should inform the decision which is taken in either of these two cases. Design of vegetated porous paving, on the other hand, does not involve such alternatives because the filtration process which takes place in these systems occurs at one level only – the grassed surface.

The task of reconstructing/reinstating permeable paving (surface) blocks at the end of a 'lifespan' period, while certainly labour-intensive, can be performed without wholesale lifting, cleaning, stockpiling and relaying. Unskilled workers removing, cleaning and replacing the blocks two or three at a time can reinstate a paved area at the rate of about one square metre per 45 minutes. Porous paving (grassed hard-standing areas), on the other hand, are much more difficult to reinstate. However, the growth process itself, as well as regular mowing of these systems, represents a form of 'forced maintenance' which accounts for the 5-fold multiplier advised for 'lifespan' periods in Table 3.1.

Sediment loads generated during local construction activity (see Section 1.3.3) are unpredictable other than in general terms, so that the best information obtainable on when remedial action such as reinstatement/ reconstruction of an installation should be carried out comes from careful monitoring of the installation itself. In this respect it should be recognised that a paving installation is performing satisfactorily as a whole (system) if, in a design or significant storm event, the entire storm runoff is managed without overflow, even though failure (complete blockage) may be apparent in a limited area.

3.2 SOIL PERMEABILITY AND PERCOLATION AT DEPTH

3.2.1 Percolation of storm runoff

The disposal of storm runoff into percolation devices such as "leaky" wells or gravel-filled trenches draws on the same hydraulic theory as that used for surface infiltration. The modelling difficulties which arise in this case, however, stem from the more complex area through which the flow passes as it leaves the device on its outwards/downward path towards the watertable.

The basic augerhole case is considered first. The assumptions which lie at the heart of this derivation are that :

- a well-defined wetting front develops from the walls and floor of the augerhole as flow seeps from it and that the flow space occupied by this seepage is completely saturated;
- the seepage outflow from the augerhole is acting under 'free draining' conditions, i.e. hydraulic gradient, I = 1.0 applies;
- seepage from the augerhole, once it has commenced its journey through the soil matrix, does not return to the augerhole.

Applying these assumptions to outflow from a cylindrical augerhole with **low** watertable (Figure 3.2a) gives (Jonasson, 1984) :

Seepage outflow volume $d\forall$, from an augerhole, diameter = $2r_o$, while depth changes from h to (h – dh) in time dt is given by :

$$d\forall = \pi r_o^2 . dh \quad \textbf{(3.4)}$$

Also, from Darcy's Law, $d\forall = k_h I A_i \, dt$

where k_h = hydraulic conductivity

I = hydraulic gradient

A_i = area through which flow is percolating.

Hence,

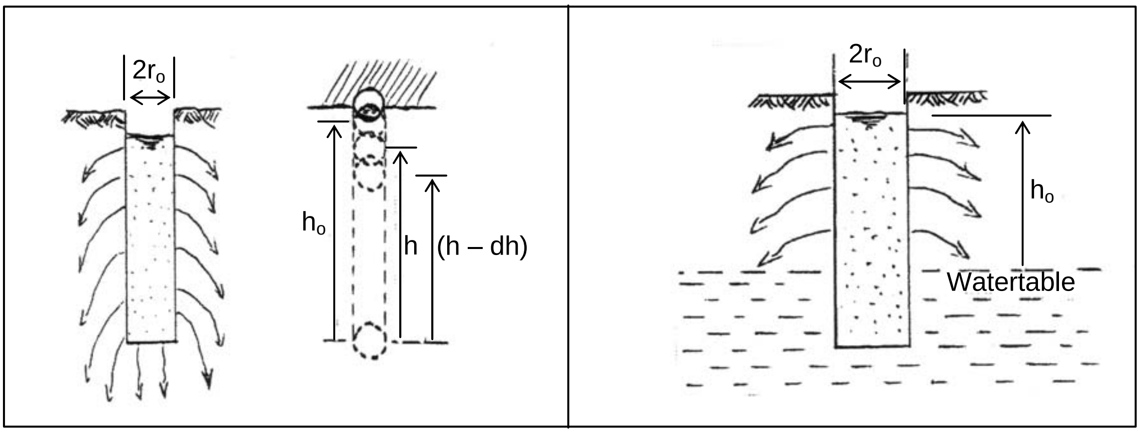
$$d\forall = k_h I \left[\pi r_o^2 + 2\pi r_o h\right] dt \quad \textbf{(3.5)}$$

Equating (3.4) and (3.5) :

$$\pi r_o^2 . dh = k_h I \left[\pi r_o^2 + 2\pi r_o h\right] dt \quad \textbf{(3.6)}$$

i.e.

$$dh = k_h I \left[1 + \frac{2h}{r_o}\right] dt = k_h I \left[\frac{r_o + 2h}{r_o}\right] dt \quad \textbf{(3.7)}$$



(a) Low watertable case **(b) High watertable case**

FIGURE 3.2 : Augerhole analysis

i.e. $$k_h I.dt = \left[\frac{r_o}{r_o + 2h}\right]dh = \frac{r_o}{2}\left[\frac{1}{\frac{r_o}{2} + h}\right]dh \qquad \textbf{(3.8)}$$

i.e. $$\frac{2k_h I}{r_o}\int_{t_1}^{t_2} dt = -\int_{h_1}^{h_2} \frac{dh}{\left(\frac{r_o}{2} + h\right)} \qquad \textbf{(3.9)}$$

i.e. $$\frac{2k_h I}{r_o}\left[t_2 - t_1\right] = -\left[\log_e\left(\frac{r_o}{2} + h\right)\right]_{h_1}^{h_2} \qquad \textbf{(3.10)}$$

i.e. $$\frac{2k_h I}{r_o}\left[t_2 - t_1\right] = -\left[\log_e\left(h_2 + \frac{r_o}{2}\right) - \log_e\left(h_1 + \frac{r_o}{2}\right)\right]$$

$$= \left[\log_e\left(h_1 + \frac{r_o}{2}\right) - \log_e\left(h_2 + \frac{r_o}{2}\right)\right] \qquad \textbf{(3.11)}$$

i.e. $$k_h = \frac{r_o}{2I\left(t_2 - t_1\right)}\log_e\left[\frac{h_1 + \frac{r_o}{2}}{h_2 + \frac{r_o}{2}}\right]$$

$$= \frac{2.3 r_o}{2I\left(t_2 - t_1\right)}\log_{10}\left[\frac{h_1 + \frac{r_o}{2}}{h_2 - \frac{r_o}{2}}\right] \qquad \textbf{(3.12)}$$

Assuming free draining seepage conditions, i.e. I = 1.0, gives :

Hydraulic conductivity $$k_h = \frac{1.15 r_o}{\left(t_2 - t_1\right)}\log_{10}\left[\frac{h_1 + \frac{r_o}{2}}{h_2 + \frac{r_o}{2}}\right] \qquad \textbf{(3.13)}$$

A modified form of Eqn. (3.13) is used where the watertable is **high** and intersects the augerhole (Figure 3.2b). In this case, all heights, h, are measured **from the watertable surface** :

The analysis follows that presented above **except** that the πr_o^2 term is dropped from Eqn. (3.5), and subsequently; outflow is assumed to take place through the walls, only, of the borehole. The resulting expression for hydraulic conductivity is, therefore :

$$k_h = \frac{1.15 r_o}{\left(t_2 - t_1\right)}\log_{10}\left[\frac{h_1}{h_2}\right] \qquad \textbf{(3.14)}$$

The inverse augerhole equations, above, are used in two important ways in stormwater management technology. The first is in field testing to establish site hydraulic conductivity values (see Section 3.2.2, below); the second is in performance review studies such as those upon which the scale effect Moderation Factor, U, is based (see footnote, Section 5.1.2).

3.2.2 Hydraulic conductivity and its measurement

The performance of a practice or technique for managing stormwater by in-ground retention is influenced by the hydraulic conductivity of site soil. Five broad soil permeability classifications are recognised – these are :

Sandy soil : $k_h > 5\times10^{-5}$ m/s

Sandy clay : k_h between 1×10^{-5} and 5×10^{-5} m/s

Medium clay and some rock : k_h between 1×10^{-6} and 1×10^{-5} m/s

Heavy clay : k_h between 1×10^{-8} and 1×10^{-6} m/s

Constructed clay : $k_h < 1\times10^{-8}$ m/s,

where k_h is the value of hydraulic conductivity determined by Jonasson's (1984) 'falling head' augerhole method, Eqns. (3.13) and (3.14), above. Brief descriptions of these five soil classifications are contained in Section 1.3.4.

Prior to conducting a falling head test it is important that the soil surrounding the borehole is **wet**. This is achieved by maintaining an inflow to the hole and checking its outflow rate until fairly constant. (A borehole of any diameter and depth may be used, but diameter = 125 mm and depth = 2.0 m is preferred : the borehole should be lined with 110 mm diameter perforated PVC pipe or equivalent.)

This process also provides the opportunity for obtaining a constant head soil permeability value based on the same theory presented in Section 3.2.1. In this case:

$$Q = q.A_i = (k_h.I)A_i \qquad \textbf{(3.15)}$$

where $$A_i = \left[\pi r_o^2 + 2\pi r_o h\right]$$

hence $$k_h = \frac{Q}{\left[\pi r_o^2 + 2\pi r_o h\right]}$$

where Q = rate of topping-up of borehole (m³/s). Site testing to obtain a satisfactory value of k_h should be repeated up to six times using 20-minute topping-up durations for each test.

When the borehole soil environment is satisfactorily wet following the constant head test, it is convenient to commence the falling head test by simply ceasing to supply further inflow to the bore. The characteristic h versus t curve, illustrated in Figure 3.3, results from subsequent observations.

The relevant basic equations from which the permeability value may be calculated are :

$$k_h = \frac{1.15 r_o}{t_2 - t_1} \log_{10}\left[\frac{h_1 + \frac{r_o}{2}}{h_2 + \frac{r_o}{2}}\right] \text{ m/s} \quad \text{low water table case} \qquad \textbf{(3.13)}$$

and $$k_h = \frac{1.15 r_o}{t_2 - t_1}\left[\log_{10}\frac{h_1}{h_2}\right] \text{ m/s} \quad \text{high water table case} \qquad \textbf{(3.14)}$$

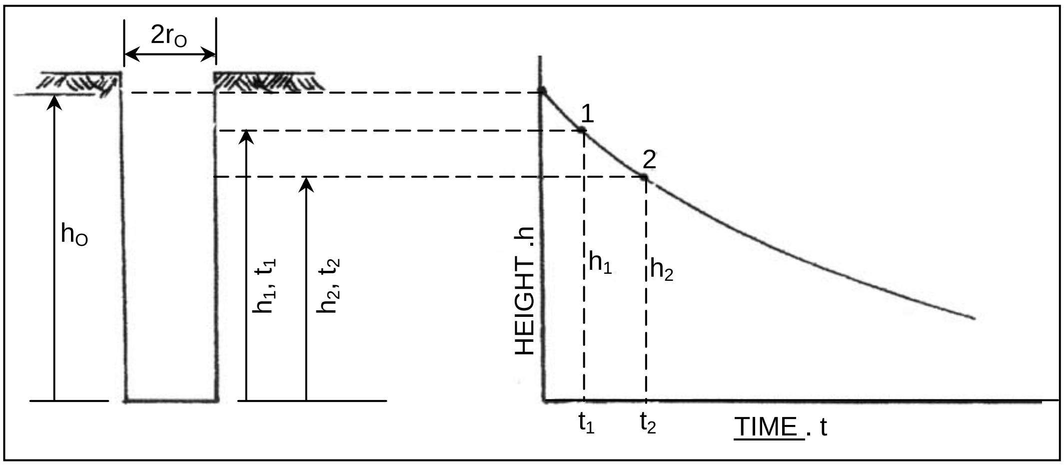


FIGURE 3.3 : Inverse augerhole test – definition of terms and typical "h" versus "t" relationship

A plastic, cylindrical float with attached measuring rod may be used to obtain readings of "h" as the water level drops. Time, t, is measured from the instant inflow to the bore ceases (following the constant head test).

The main disadvantage of hydraulic conductivity determination by a falling-head procedure is the (apparent) variation in k_h values which results when successive t-h observations are substituted into Eqns. (3.13) or (3.14). For this reason a standard 60 minute value for hydraulic conductivity, k_{60}, is used in clay soils. In sandy soils, the rate of fall of the water level may be quite rapid: in such cases the hydraulic conductivity is calculated for depth-time conditions around h = 1.70 m.

Appendix A includes a more comprehensive treatment of this topic and a Specification for hydraulic conductivity testing.

3.3 RUNOFF VOLUME AND TIME BASE τ IN 'SOURCE CONTROL' TECHNOLOGY

The broad class of underground, on-site retention installations described as "leaky" devices includes those illustrated in Figure 1.3 and installations of similar type which incorporate proprietary units specially manufactured to store water and release it slowly into the surrounding soil. Atlantis and Ausdrain ("milk crate") cells and Everglas units are examples of the latter : infiltration or "dry" ponds are another member of this class of retention device. "Leaky" wells may be 2 – 3 metres deep; trenches have a maximum depth, typically, of 1.5 metres with 0.3 m of backfill cover; "dry" ponds – for safety reasons – are rarely deeper than 0.50 m.

These (in-ground) devices are effective in soils of high and medium permeability, but perform poorly in low permeability soils. Such soil conditions call for installations employing the same range of materials and proprietary products (gravel, slotted pipes, "milk crate" cells, etc.), but arranged in a mattress configuration, usually 0.3 – 0.5 m deep : the term "soakaway" is applied to these devices (see Figure 1.4b).

Engineering design of all types of installations referred to in this section depends on two main factors :

- the **volume, $\forall$,** of stormwater runoff which **enters** the device during the design storm period of runoff; and,
- the **time base, τ,** of the design storm runoff hydrograph.

[The Handbook uses the 'design storm' method in some procedures and the 'continuous simulation' modelling approach in others. A comprehensive explanation of the issues involved in these uses is presented in Sections 3.4 and 3.5.]

There is, of course, a strong link between stormwater runoff volume $\forall$ and the design storm parameters, **storm** duration**, T_C,** and (site) **time of concentration, t_C**. This inter-relationship is illustrated in Figures 3.4a and 3.4b, and explained in greater depth in Section 4.2. These parameters and the relationships which exist between them, are important ingredients of the design procedures presented in Chapter 5.

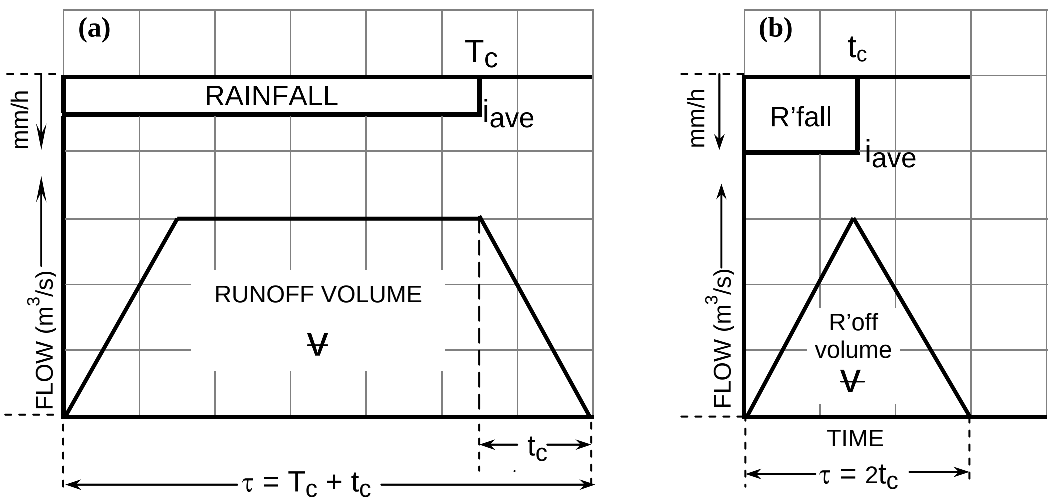


FIGURE 3.4 : Definition of 'design storm' parameters T_C, t_C, τ and $\forall$ for two catchment cases

3.4 AN EXCURSION INTO 'CONTINUOUS SIMULATION' MODELLING

3.4.1 Introduction : storm successions and the 'design storm' approach

Design procedures employed to create traditional street drainage networks have used the design storm approach assuming, typically, that runoff generated in the critical (design) event is unaffected by antecedent channel flow conditions or by floodwater held in temporary storage following a previous event (IEAust, 1999). The justification for this assumed behaviour was that flood wave movement through the urban landscape is so rapid that any remnants of a preceding event are unlikely to have a significant effect on the next storm runoff wave, even one of 'design' magnitude.

When **detention** techniques were introduced in the 1960s, the same assumption was invoked by the bulk of practitioners who ignored the changed conditions, even though the new technology involved significant elements of temporary ponding. Justification for ignoring the possible effects of interaction between storm successions was again based on the speed with which flood waves move, even though – in these new circumstances – they involved routing through temporary storage.

The introduction of **retention** technology puts the issue of storm successions and the inability of the design storm approach to deal satisfactorily with them, once more under the spotlight. The case against the design storm method – used widely in Australian and overseas contemporary practices (IEAust, 1999) – rests on two grounds :

1. users of the method are required to make professional judgements relating to the likely state of on-site storages – typically empty or half-full – at the commencement of the design storm, and,
2. design storm temporal patterns, determined by current analytical procedures are, in fact, "embedded" storm bursts : dimensions of storages determined from these (storm) profiles are therefore likely to be undersized (see Rigby et al, 2003).

Perhaps the strongest, comprehensive criticism of the design storm approach is contained in a Keynote Address delivered by Professor George Kuczera at the 2003 Engineers Australia Hydrology and Water Resources Symposium in Wollongong (Kuczera et al, 2003). Professor Kuczera concludes that there is "…already considerable evidence that specification of average initial conditions in the design storm approach is problematic and that the design storm approach can be seriously biased". Professor Kuczera and other recognised researchers have proposed 'continuous simulation' modelling as the best remedy for these deficiencies (of the design storm method). These issues are explored for the case of initial loss estimation for flood studies in Heneker et al, 2003.

Given this authoritative assessment and the fact that many of the procedures proposed in later sections of the Handbook use a *modified* form of the design storm approach, it is, therefore, important that readers understand how the two design methods relate to each other and why the decision to *modify* the conventional design storm method was taken. The following content (Sections 3.4 and 3.5) provide an explanation of these matters. 'Continuous simulation' modelling is considered first, illustrated with aspects that are relevant to stormwater 'source control' in urban drainage systems.

3.4.2 'Continuous simulation' modelling : some background

'Continuous simulation' modelling of runoff events using historical records, was introduced to the literature via the Stanford Watershed Model (Linsley and Crawford, 1960; Crawford and Linsley, 1962). Boughton (1966) was the first in Australia to use this form of modelling on a daily time-step basis; this was followed by Chapman (1968) using sub-daily time steps. More recent contributions of significance have been provided by Guo and Urbonas (1996) and Urbonas et al (1996) and by Dr. Tony Wong of the CRC in Catchment Hydrology, Monash University. The US and Wong's work have focussed almost exclusively on problems of wetland 'sizing' and design, for example detention time (Somes and Wong, 2000). More recently, Coombes and Kuczera (2001) developed the PURRS program, based on 'continuous simulation' modelling and applied it to the design of rainwater tank systems.

'Continuous simulation' modelling uses historical rainfall records, typically, obtained from a long period – usually 20 years or more – together with a catchment rainfall/runoff model, to produce a continuous (rainfall) data-based streamflow record for the catchment – usually ungauged. While it is true that the resulting flow sequence is 'constructed' rather than 'real', its base in recorded rainfall information makes it a valuable tool in seeking solutions to many runoff-related hydrological problems. It should be noted that the rainfall record used in the modelling observes the gaps (periods of no rainfall) between stormbursts as strictly as the stormbursts themselves.

Two applications of 'continuous simulation' modelling of interest to WSUD are explored here :

- in-ground "leaky" storages required to retain, on-site, storm runoff which has been cleansed through upstream filter/treatment units (see Figure 3.5a); and,
- rainwater storages receiving roof runoff: a household or industrial/commercial demand on the retained water is an essential element of the modelling process.

A simple general layout of the components which are common to both of these cases, is presented in Figure 3.5b.

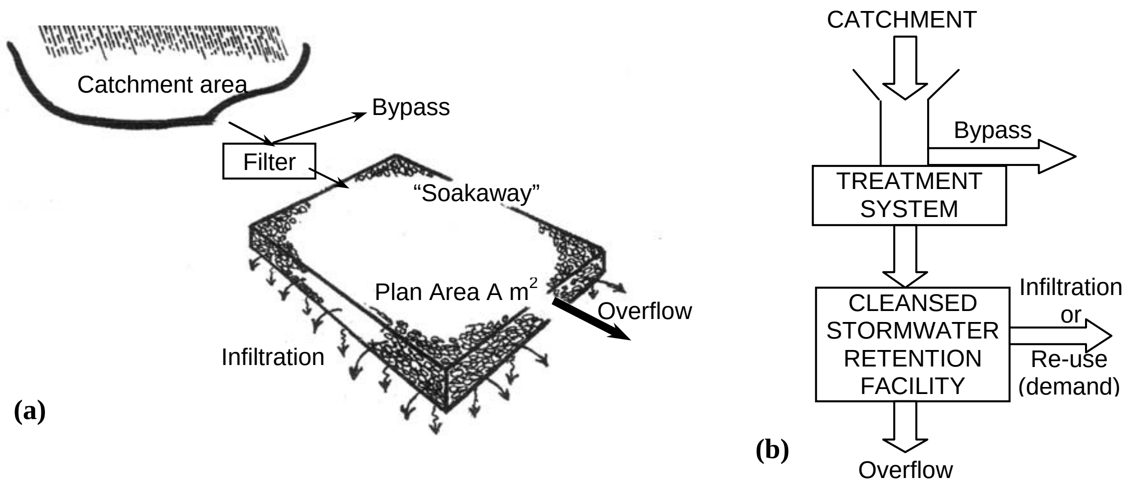


FIGURE 3.5 : Sketch of stormwater-retaining device and system schematic

It may assist the process of understanding these (two) applications if their essential components are recognised and the manner in which these relate to corresponding elements of other stormwater best practice installations involving storage is noted. The cases to which 'continuous simulation' modelling have been applied may be divided into four broad categories of systems :

1. **outlet control (only) with storage :** this includes detention basins with the full range of flood flows controlled at the basin outlet;

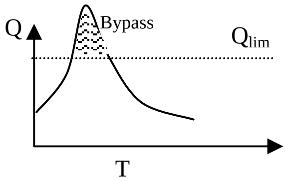
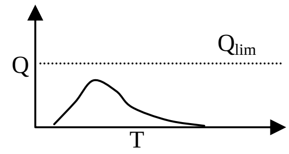
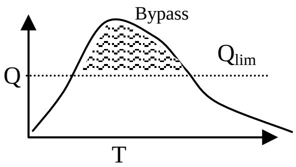
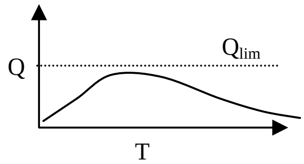
2. **formal inlet control (by design); storage, including on-site disposal; and overflow :** systems with inlets incorporating treatment/filter units with set flow capacities (hence bypass), before in-ground storage, typically with on-site abstraction by percolation, and overflow. This case is explored below;

3. **informal inlet control; storage (and on-site re-use); and overflow :** this covers rainwater tanks with upstream bypass of major flows taking place at roof guttering, before tank storage (with on-site abstraction for re-use) and overflow. This case is explored below;

4. **formal inlet control (by design); storage; and formal outlet control (by design) :** this represents the typical wetlands case with inlets (some treatment) having set flow capacities (hence bypass), before storage and formal outlet control (weir, 'riser', siphon, etc.).

There are four possible modes of operation of systems 2 and 3, above. These are illustrated in Figure 3.6.

The relationships between catchment area, storage, percolation rate, water (supply) demand, etc. which arise in the two cases (2 and 3, above), are, of course, quite different, however, the modelling steps involved in the process of arriving at correct relationships, have much in common. These are presented as a series of 13 steps in Table 3.2. A graphical outcome from the Table 3.2 procedure is presented in Figure 3.7.

RUNOFF EVENT TYPE *WITH* BYPASS	LOSS MODE : BYPASS AND/OR OVERFLOW	RUNOFF EVENT TYPE *WITHOUT* BYPASS	LOSS MODE : BYPASS AND/OR OVERFLOW
 High flow rate Low runoff volume	**BYPASS and NO OVERFLOW***	 Low flow rate Low runoff volume	**NO BYPASS and NO OVERFLOW**
 High flow rate High runoff volume	**BYPASS and OVERFLOW ** **	 Low flow rate High runoff volume	**NO BYPASS and OVERFLOW****

* Retention storage antecedent condition may cause overflow to occur.

** Retention storage antecedent condition will affect overflow characteristics

FIGURE 3.6 : Illustration of modes of operation of some on-site stormwater retention systems

3.4.3 Application of Figure 3.7 : the "soakaway" and 'dry' pond cases

Information from Figure 3.7 can be used directly to determine, for any small catchment in Adelaide described in terms of its equivalent impervious area and known site soil permeability, the dimensions of **"soakaway" (unobstructed internal space) or 'dry' pond** required to achieve a target retention efficiency within the range 30% – 90%. This information is part, only, of the full interpretation which can be made of the 13-step procedure (Table 3.2) described above relating to "soakaways". For example, it is possible, using Figure 3.7 and a simple multiplier, to dimension other retention device types, such as gravel-filled or 'milk crate' or combination gravel/pipe "soakaways" (see Figures 1.3 and 1.4b) receiving storm runoff from urban catchments located in the Adelaide climate zone. Of course the *procedure* is not limited to Adelaide climate conditions but can be applied using appropriate continuous rainfall data to any location.

The particular advantage of the procedure is its independence from the assumptions and potential errors inherent in the design storm method, as reviewed in Section 3.4.1. Its disadvantages include the limited rainfall (pluviograph) data directly available to carry out 'continuous simulation' modelling: directly useable data are available from only 600 stations across Australia. However, this disadvantage can be overcome using the DRIP program (Heneker et al, 2001), opening access to many of the 7,000 daily-read data sets available from Commonwealth Bureau of Meteorology sources. Other disadvantages of 'continuous simulation' modelling are the uncertainties which must surround any attempt to mathematically model catchment processes and in-ground retention installation (device) performance/behaviour, and the time and expertise required to undertake the analyses for individual cases.

For these reasons, a more general and simplified approach to the dimensioning of "soakaways" and 'dry' ponds is offered in the Handbook for locations in the Northern and Southern climate zones of Australia, and for three climate 'bands' covering Intermediate Australia. These (five) groupings are based on $i_{10,1}$ similarity. [$i_{10,1}$ is the ARI, Y = 10 years, 1-hour rainfall average intensity used in the AR&R – 1987 (IEAust, 1999) linear interpolation procedure to determine values of hydrological parameters for use in locations in Intermediate Australia (see Figure 3.8 and Eqn. 3.34)]. Five graphs derived using the 13-step procedure (Table 3.2), modified for special application to "soakaways" and 'dry' ponds, are presented in Appendix C. They form the basis for Procedure 5A in Section 7.2.

TABLE 3.2
BASIC STEPS IN TWO STORMWATER RETENTION CASE SCENARIOS

VARIABLES

"LEAKY" DEVICES	RAINWATER TANKS
A_{EIA} : catchment equivalent impervious area, m^2	A : roof area, m^2
H : device depth, m	V : rainwater tank volume, m^3
A_D : device plan area, m^2	R : volumetric retention efficiency, %
R : volumetric retention efficiency, %	D : demand, L per household per day
k_h : hydraulic conductivity of host soil, m/s	q_{20} : gutter/downpipes capacity flow expressed as L/s per m^2 of area A.
q : bypass in L/s per m^2 of catchment (A_{EIA})	
q_{lim} : limit of bypass, q, for each R (see STEP 10)	
Q_{lim} : ($q_{lim} \times A_{EIA}$) for each value of R	
e_s : device void space ratio – 0.35 for gravel, etc.	

Q_I : cumulative volume of runoff which **inflows** to device/tank in Y years
Q_B : cumulative volume of runoff which **bypasses** device/tank in Y years
Q_O : cumulative volume of runoff which **overflows** device/tank in Y years
$Q_{B/O}$: cumulative volume of runoff which **bypasses or overflows** device/tank in Y years
Q_{ret}% : percentage (total) of runoff which is retained by device or tank

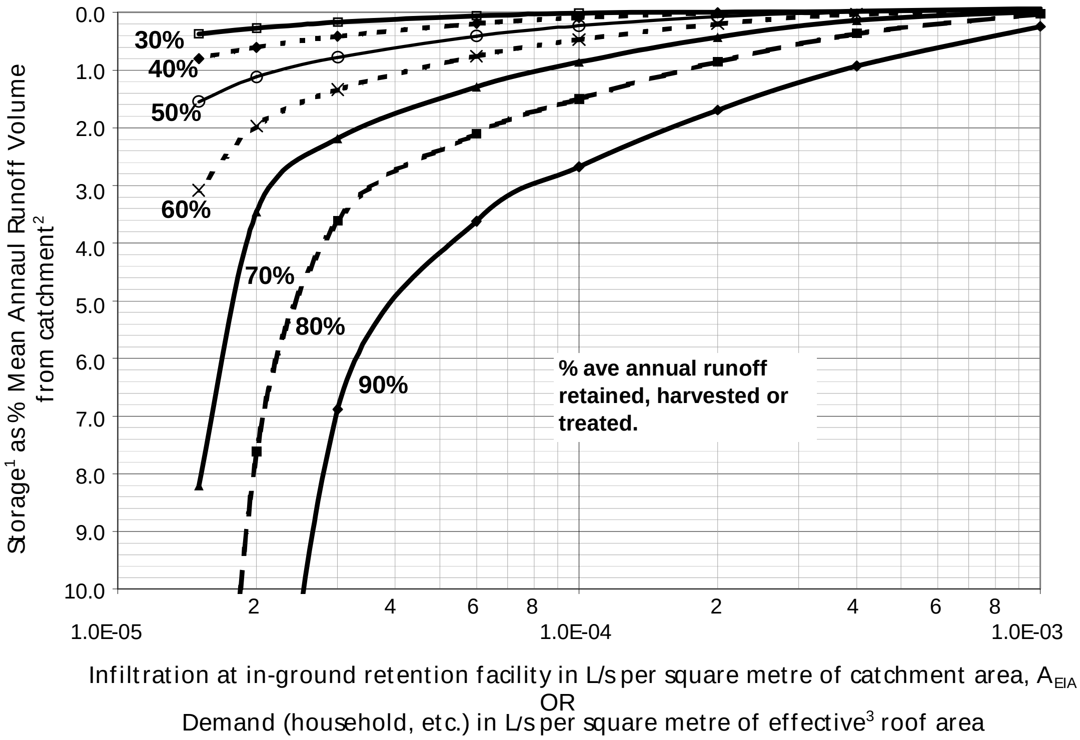
"LEAKY" DEVICES	STEPS IN THE PROCESS	RAINWATER TANKS
Base mathematical model on equivalent impervious area, A_{EIA} : recommend A_{EIA} = 1,000 m^2	**STEP 1 :** Establish mathematical model of the catchment rainfall/ runoff process.	Base mathematical model on roof area, A : recommend A = 50 m^2.
Nominate depth, H, and **trial** value of device plan area, A_D; adopt a target value of R. **Outflow** in this case, is represented by percolation equal to $A_D \times k_h$, where k_h is a value selected within the soil range.	**STEP 2 :** Establish mathematical model of retention device for nominated dimensions and conditions of operation, in particular, **outflow**. Also nominate a target value for R.	Nominate volume, V, of rainwater tank; adopt a target value of R. **Outflow** in this case, is **trial** daily demand, D, made on water stored in the tank (50 L per day).
Bypass flow, q L/s per m^2 of area, A_{EIA} is assigned; all flow greater than ($q \times A_{EIA}$) bypasses. All flow reaching the retention device and exceeding its capacity, given by ($e_s \times A_D \times H$), **overflows**.	**STEP 3 :** Nominate a **bypass** condition applied upstream of the retention device and, also, an **overflow** condition.	**Bypass**, ($q_{20} \times A$) is determined by the capacity flow of the roof guttering/downpipes, typically, ARI = 20 years. All flow stored at any time in the rainwater tank above spill level – fixed by trial tank volume, V – **overflows**.
This step is applied to area, A_{EIA} = 1,000 m^2.	**STEP 4 :** Apply rainfall data (N-years record). Determine the **cumulative (total) volume** of runoff from the catchment over the N-years period, Q_I.	This step is applied to roof area, A = 50 m^2.
	STEP 5 : Determine, for the values of parameters nominated in STEP 2 and conditions applied in STEP 3, the **cumulative (total) volume** of runoff which bypasses (Q_B) or overflows (Q_O) the retention device (in Y-years). Call this $Q_{B/O}$.	
Increase trial value of Retention device plan area, A_D. Repeat STEPS 4, 5 and 6.	**STEP 6 :** Calculate the Retention percentage $Q_{ret}\% = 100\ [(Q_I - Q_{B/O})/Q_I]\%$ and compare with target of R. If Q_{ret}% is less than R, return to STEP 2 and adopt larger value for the appropriate parameter. Repeat STEPS 4, 5 and 6 until $Q_{ret}\%$ = R.	Increase trial value of daily demand, D. Repeat STEPS 4, 5 and 6.

Continued….

TABLE 3.2 (continued)

"LEAKY" DEVICES	STEPS IN THE PROCESS	RAINWATER TANKS
Device depth, H (STEP 2); Soil type, k_h (STEP 2); Device plan area, A_D (STEP 6). Hence area ratio, $A_R = A_D/A_{EIA}$.	**STEP 7 :** Report **adopted** parameter values, condition, etc., following successful completion of STEP 6.	Tank volume, V (STEP 2); Demand, D_A (STEP 6).
Range of H : H = 0.30 m, 0.50 m, 0.75 m, 1.00 m, 1.50 m. Range of soils : k_h = 1.0×10^{-7} m/s; 1.0×10^{-6} m/s; 1.0×10^{-5} m/s; 1.0×10^{-4} m/s; 1.0×10^{-3} m/s. A value for area ratio A_R emerges for each (H, k_h) pair.	**STEP 8 :** Repeat STEPS 2 to 7 inclusive using the full range of installations of interest. Use the same bypass and overflow conditions set in STEP 3. Output from this step is a particular value for the **'trial'** parameter nominated in STEP 2.	Range of tank sizes, V : 2.0 kL; 3.0 kL; 4.0 kL; 10.0 kL. A value for demand, D, emerges for each tank size.
Repeat STEPS 2 to 8, inclusive, with greater value of bypass capacity flow, q, set in STEP 3.	**STEP 9 :** Report parameter values, conditions, etc., from successful completion of STEP 8, **for the target value of R** set in STEP 2. [**STEP 9A :** It may be necessary to repeat STEPS 2 to 8 inclusive, for a different bypass condition from that set in STEP 3.]	Report final values (for retention efficiency, R) covering tank volumes 2 kL to 10 kL inclusive, hence demand, D – one value for **each** tank volume. Demand : 50, 60, 70, etc., L/day. **Note :** Bypass flow, q_{20}, used in the raintank analysis, is fixed for all cases; STEP 9A is therefore unnecessary.
Report final values covering ranges : H = 0.30 m to 1.50 m, incl.; k_h = 1.0×10^{-7} m/s to 1.0×10^{-3} m/s, incl.; q = 0.002 L/s etc., per m^2 A_{EIA}. It is discovered that different values of bypass flow, q, lead to different relationships between H and k_h for a given value of R. However, a **limiting** value, q_{lim}, can be identified : the relationships between depth, H, and k_h (permeability) associated with this limiting value are adopted as those which are unique to R.	**STEP 10 :** Report parameter values, conditions, etc., emerging from successful completion of STEP 9A.	N/A
Adopt R values 90%, 80%, 70%, 60%, 50%, 40%, 30%.	**STEP 11 :** Adopt a new target value for R in STEP 2 and repeat STEPS 2 to 10 inclusive, as appropriate.	Adopt R values 90%, 80%, 70%, 60%, 50%, 40%, 30%.
N/A	**STEP 12 :** Adopt a new value for roof area, A, in the raintank analysis; repeat STEPS 1 to 11 inclusive.	Adopt A = 100 m^2, 150 m^2, 200 m^2, etc.
Report final values : H = 0.30 m to 1.50 m, incl.; k_h = 1.0×10^{-7} m/s to 1.0×10^{-3} m/s, inclusive; and q_{lim}; R = 90%, 80%, 70% etc.	**STEP 13 :** Report parameter values, conditions, etc., emerging from successful completion of STEPS 11 or 12, as appropriate.	**Report final values :** R'tanks : 2.00 to 10.0 kL, incl. Demand : 50, 60, 70 etc., L/day. Roof areas : 50, 100, 150 etc., m^2. R = 90%, 80%, 70% etc.

Outcomes from STEP 13 – for both "leaky" devices and rainwater tanks – can be combined and presented on a **single** graph for the location to which the N-years record, employed in STEP 4, applies. The graph for Adelaide, South Australia, is presented in Figure 3.7.



NOTES :

1 : **Device** may be an unobstructed (internal volume) "soakaway", 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 3.7 : Relationship between infiltration, demand and retention storage – general case (Adelaide)

In-ground installations dimensioned according to the Table 3.2 procedure (and graphs similar to Figure 3.7) incorporate an outflow process which is entirely natural, namely percolation. Situations frequently arise in such systems where site or other requirements or constraints cannot be accommodated. In these circumstances it may be necessary to assist the emptying process by providing (additional) **slow-drainage** of stored water by systems similar to those illustrated in Figure 1.4b. Such provision is, in essence, the basis of on-site detention (OSD) practice, familiar to Australian urban drainage practitioners. Application of this technology to infiltration devices, however, differs significantly from OSD in the length of time during which the drainage process takes place. It is normal practice for detention installations to empty in periods of two to four hours; slow-drainage is calculated to take place over periods of 12 hours or longer.

Slow-drainage should only be considered after the preliminary design for an installation, dimensioned as explained in Section 7.2, is not practicable (because of site or other difficulties, referred to above). Details of how this aspect is managed in the design of pollution control retention installations are explained in Section 7.3.

3.4.4 Application of Figure 3.7 : the rainwater tank cases

Information from Figure 3.7 can also be used to determine, for any roof area in Adelaide and in-house daily (water) demand, the required **rainwater tank volume** needed to achieve a target retention efficiency within the range 60% – 90%. Of course the procedure is not limited to Adelaide climate conditions but can be employed using appropriate 'continuous' rainfall data applied to any location.

In a way which parallels the procedures outlined above (Section 3.4.3) for "soakaways" and 'dry' ponds, a more general and simplified approach is also offered in the Handbook for rainwater tank sizing. This is realised, also, for five climate zones or 'bands' in Australia and presented in Appendix E. They form the basis for Procedure 10 in Chapter 8.

3.4.5 Catchment-wide modelling and catchment performance

The task of identifying a critical flow peak and hence ‘stage’ condition (for a nominated ARI) at a particular location in an urban catchment – called a “point of singular interest” in the present discussion (see Section 4.2) – has been determined by practitioners in the past using a suite of design storm hyetographs applied to the catchment by way of an appropriate rainfall/runoff model. The outcome was a corresponding suite of hydrographs : the peak of the envelope curve enclosing the set of hydrograph maximum values gave the required peak flow (and hence ‘stage’), as well as **duration of the storm that was critical in the contributing catchment** (IEAust, 1999, Section 9.2.4).

This method, based on isolated, hypothetical ‘design’ storms, takes no account of the consequences of interaction between successive (real) storms. However, it has served more than a generation of urban drainage designers reasonably well, because its basic assumption – that there is no significant interaction between runoff generated in successive storms (because of rapid storm runoff wave movement in conventional urban catchments) – is reasonably true.

So what lies ahead for Australian practice if it is to take the ‘continuous simulation’ modelling path, only, from the ‘crossroads’ identified by Kuczera et al (2003)? First, a wealth of research and analysis development must occur before a procedure emerges which can truly take the place currently occupied by the design storm approach referred to above. To quote Hardy, Coombes and Kuczera (2004) : “These questions serve to motivate strongly the need for new modelling tools capable of multi-scale investigations in a continuous simulation framework. It will only be with the development of these new tools that more definitive and informative investigations of these and other questions can take place”. The PURRS program of Coombes and Kuczera (2001) is, undoubtedly, a major contribution towards this goal but more progress is needed before the goal is finally reached.

When it is (reached), its application into any typical municipal situation is likely to be complex and costly to produce and well beyond the up-front financial investment in design that a developer could – or should – be reasonably expected to bear in typical development circumstances. Argue and Pezzaniti (2004) predict that such ‘tailor-made’ (catchment-wide) packages, wherever they are developed, will be held by local government agencies, only, and that their use in the process of designing competent on-site works associated with development/re-development projects will take place as a collaborative exercise involving Council-owned software and consultants using the software on behalf of developers.

Although the advent of such models may put to rest the disquiet now harboured concerning the validity of the design storm method in the light of the earlier criticisms (Section 3.4.1), their widespread use – should that become a reality – is unlikely to yield any dramatically different outcomes when applied to general urban landscape flood study assignments requiring 20-years, 50-years, 100-years, etc., flood contours. The justification for this prediction is the relatively minor role played, typically, by non-channel storage in *normal* urban landscapes.

All of these considerations – currently (2004) the source of much professional discussion – would be matters of no great moment in a Handbook on basic stormwater ‘source control’ measures were it not for the fact that conversion of Australian practice exclusively into the domain of ‘continuous simulation’ modelling would involve the loss of a very useful catchment property, namely, catchment **critical storm duration** (see Section 4.2). This concept has no place in ‘continuous’ modelling but is a fundamental component of the design storm method, a *modified* form of which is used in some later procedures in this Handbook (see Section 5.1).

It need hardly be said that contemporary practice should be based on wise use of all available practices and technologies, recognising their advantages and their shortcomings. With this in mind, the following long-term approach to resolving the issues raised in the foregoing discussion is offered and represents, we believe, an acceptable solution leading to an urban drainage design practice which is both credible and easy for practitioners to apply.

STEP 1 : Use ‘continuous simulation’ modelling with a comprehensive rainfall/runoff model of the developing urban catchment of interest, as it is or was (the ‘benchmark year’ concept, Sections 4.2.1 and 4.6.3), together with streamflow frequency analysis, to determine the Q_{peak} *versus* frequency relationship and, hence, the target peak flow (and hence ‘peak stage’) condition at a “point of singular interest” (see Section 4.2.1) for a selected ARI = Y-years. Representatives of all stakeholders in the catchment should reach consensus on these matters involving a fair balance between flood security, heritage, economic, environmental, etc. interests;

STEP 2 : Use the design storm approach [suite of design storm hyetographs, as in AR&R – 1987 (IEAust, 1999)] applied to the same rainfall/runoff model used in STEP 1, above, to determine the catchment **critical storm duration** for the same nominated ARI.

STEP 3A : Determine the dimensions of on-site flood control storages (detention or retention) by application of 'continuous' rainfall data to the comprehensive runoff model (developed and used in STEP 1) taking the sites to be developed or re-developed one at a time and using an iterative approach ('trial and error'); successful on-site design for appropriate development/ re-development, gives the same Q_{peak} *versus* frequency relationship for the catchment determined in STEP 1.

OR

STEP 3B : Base the design of on-site flood control (detention or retention) storages on use of the *modified* design storm method described below.

It is unlikely that the peak flows produced in STEPS 1 and 2 will be the same: that produced in STEP 1 should dominate. However, the critical storm duration – absent from STEP 1 – is an acceptable and valuable outcome from STEP 2, regardless of the difference in peak flows.

3.5 THE *MODIFIED* DESIGN STORM METHOD

3.5.1 Introduction

Professor Kuczera's Keynote Address (Kuczera et al, 2003) places Australian practice at the crossroads with regard to its use of the design storm method and/or 'continuous simulation' modelling. The complexities of the latter as well as its uncertainties, reviewed in Section 3.4, taken with the familiarity which the design storm method enjoys across all national practices – not to mention its simplicity – provide considerable incentive to explore the criticisms which have been levelled against this method of design, and attempt to resolve them. If this can be achieved, then both methods can take their rightful and valued places in Australian practice.

So what are the substantive criticisms made of the design storm method? From Section 3.4.1 :

1. "users of the method are required to make arbitrary assumptions about the state of on-site storages – typically empty or half-full – at the commencement of the design storm; and,

2. design storm temporal patterns, determined by current analytical procedures are, in fact, 'embedded' storm bursts : dimensions of storages determined from these (storm) profiles are therefore likely to be undersized."

Consideration and resolution of the **first** criticism is offered in the remainder of the present section; the **second** is addressed in Section 3.7.2.

The essence of the *first* criticism stems from uncertainty experienced by the designer concerning the initial volumetric status of any significant storage in an urban catchment following (unknown) previous rainfall – full?, empty?, part-full? etc. – as he/she commences to apply the appropriate design storm to the modelled catchment. [The separate issues of 'shape' and magnitude of the (design storm) hyetograph are aspects of the *second* criticism.]

However, this criticism **collapses** if the designer can be reasonably certain that the storage in question is, *in fact*, completely empty : this implies acceptance of some risk that it may **not** be the case. It is important to note that the discussion, following, applies to all types of retention storages *other than pollution control storages and rainwater tanks*. Some form of 'continuous simulation' modelling is mandatory in both of these cases (see Chapters 7 and 8).

The principal issue then becomes : what mechanisms operate naturally or can be introduced by design to retention storage devices and installations to ensure that they have a high likelihood of being empty prior to commencement of the (theoretical) design storm? Two candidates are suggested :

- natural water removal from storage installations by way of infiltration and/or percolation; and,
- 'hydraulic' means of abstraction such as bores transferring stored water to aquifers, or slow-drainage pipes releasing water into a nearby waterway or formal drainage path.

Both of these alternatives lead on to the concept of **emptying time** ('ponding time' or 'drain time' are employed in US terminology) : how long will it take for a retention device of given type and dimensions to empty from full under natural or 'hydraulic' emptying conditions? Theory associated with 'natural' emptying of typical systems is presented in Section 3.5.2; issues relating to aquifer access or slow-drainage provision ('hydraulic' emptying from storage) are explained in Section 3.5.4.

3.5.2 Emptying time – 'natural' drainage

The theory which leads to the basic emptying time formulae for typical retention devices is illustrated for the case of a "leaky" well, diameter D, height H :

$$\text{wetted area} = \pi Dh + \frac{\pi D^2}{4} \qquad \textbf{(3.16)}$$

Seepage outflow, Q, from the well when water depth equals "h", assuming the fully drained condition is :

$$Q = k_h\left(\pi Dh + \frac{\pi D^2}{4}\right) \qquad \textbf{(3.17)}$$

Let water level drop Δh in time Δt :

then, seepage outflow in time Δt :

$$d\cancel{V} = Q.dt = k_h\left(\pi Dh + \frac{\pi D^2}{4}\right).dt \qquad \textbf{(3.18)}$$

also, $$d\cancel{V} = -\frac{\pi D^2}{4}.dh \qquad \textbf{(3.19)}$$

Hence, $$k_h\left(\pi Dh + \frac{\pi D^2}{4}\right).dt = -\frac{\pi D^2}{4}.dh \qquad \textbf{(3.20)}$$

hence, $$k_h\left(h + \frac{D}{4}\right).dt = -\frac{D}{4}.dh \qquad \textbf{(3.21)}$$

i.e. $$dt = \frac{-\frac{D}{4}}{k_h(h + \frac{D}{4})}.dh \qquad \textbf{(3.22)}$$

Integrating : $$\int_0^t dt = -\frac{D}{4k_h}\int_H^h \frac{dh}{\left(h + \frac{D}{4}\right)} \qquad \textbf{(3.23)}$$

where H equals well-full depth.

i.e. $$t = -\frac{D}{4k_h}\left[\log_e\left(h + \frac{D}{4}\right)\right]_H^h \qquad \textbf{(3.24)}$$

hence, time for well depth to fall from H to h :

$$t_{H\to h} = -\frac{D}{4k_h}\left[\log_e\left(h + \frac{D}{4}\right) - \log_e\left(H + \frac{D}{4}\right)\right] \qquad \textbf{(3.25)}$$

i.e. $$t_{H\to h} = -\frac{D}{4k_h}\log_e\left[\frac{h+\frac{D}{4}}{H+\frac{D}{4}}\right] = -\frac{2.3D}{4k_h}\log_{10}\left[\frac{h+\frac{D}{4}}{H+\frac{D}{4}}\right] \qquad \textbf{(3.26)}$$

hence, time for well to **empty**, i.e. h = 0 :

$$T = -\frac{2.3D}{4k_h}\log_{10}\left[\frac{\frac{D}{4}}{H+\frac{D}{4}}\right], \text{ s} \qquad \textbf{(3.27)}$$

A corresponding expression may be derived for emptying time in a gravel-filled trench – it is :

$$T = -\frac{2.3Lb.e_s}{2k_h(L+b)}\log_{10}\left[\frac{Lb}{Lb+2H(L+b)}\right], \text{ s} \qquad \textbf{(3.28)}$$

where L = trench length; b = trench width; H = trench depth; e_S = void space ratio $\left[\frac{\text{volume of voids}}{\text{total volume occupied}}\right]$. In "soakaways" or shallow trenches having large plan area, Eqn. (3.28) simplifies to :

$$T \approx \frac{H.e_s}{k_h}, \text{ s} \qquad \textbf{(3.29)}$$

Field measurement of infiltration devices in Adelaide, together with the observations of Lee and Taylor (1998), suggests that the latter three formulae **underestimate** emptying time, T, by a factor of about two. Hence, recommended formulae for emptying time are :

- "leaky" wells : $$T = -\frac{4.6D}{4k_h}\log_{10}\left[\frac{\frac{D}{4}}{H+\frac{D}{4}}\right], \text{ s} \qquad \textbf{(3.30)}$$

- gravel-filled (or similar) trench : $$T = -\frac{4.6Lbe_s}{2k_h(L+b)}\log_{10}\left[\frac{Lb}{Lb+2H(L+b)}\right], \text{ s} \qquad \textbf{(3.31)}$$

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

The latter formula can also be used for open water recession cases for example 'dry' ponds, by setting e_S, void space ratio, equal to 1.0.

3.5.3 Emptying time criteria – analyses

The emptying time concept is virtually unknown in current Australian practice (IEAust, 1999) but is a component of other practices. Not surprisingly, emptying time criteria, where they are used, vary. UK uses "50% empty 24 hours after cessation of rainfall" (Bettess, 1996); US practice, generally, uses "completely empty, 72 hours after cessation of rainfall"; Auckland City Council, New Zealand, has, possibly, the most conservative criterion – "completely empty, 24 hours after cessation of rainfall".

The essence of statistical hydrology which can be brought to bear on the problem of emptying time recognises, firstly, that the **minimum** time gap which might be expected between two unconnected, random, meteorological events, varies inversely as the frequency of the events. Thus, it is hardly remarkable if two, unrelated, ARI, Y = 1-year storm events occur at a particular location on consecutive days, but it would be very unexpected for two (unrelated) ARI, Y = 100-years stormbursts to do so. Clearly, there must be some relationship (unknown) linking ARI and the minimum time likely between unconnected random events having the same average frequency of occurrence.

But historical records of stormbursts **do** show, not only ARI, Y = 1-year (magnitude) rainfall events occurring on consecutive days, but also very rare occasions when ARI, Y = 20-years, 50-years or even 100-years events show similar, apparent behaviour. However there is an important difference: the consecutive "1-year" events may be regarded as reasonably unconnected because they are relatively small; the likelihood of two "100-years" events occurring on consecutive days *and being unconnected* is remote. The phenomenon of their occurrence, when it does occur, is explained as two (internal) storm peaks on consecutive days as parts of a single storm having significantly greater magnitude than that of the "100-years" event, say, the once in 500-years or 1,000-years storm, and **not** two, independent occurrences.

The best way to produce a credible set of emptying time criteria for use with the design storm procedure is to apply ‘continuous simulation’ modelling to a range of catchments, each discharging to a retention (or detention) facility matched to the contributing catchment area and its physical characteristics, and taking account of the rainfall experienced at that catchment location. The link between size of installation (retention/detention facility), frequency of overflows (hence ARI) and emptying flow rate can be established: these data can then be interpreted into a relationship between ARI and emptying time, T. This process has not been carried out at the time of producing this Edition of the Handbook. However, the following interim relationship (Table 3.3), based on presently employed criteria is recommended for use pending the outcome of the study referred to above.

TABLE 3.3

INTERIM RELATIONSHIP BETWEEN ARI AND ‘EMPTYING TIME’

Ave Recurr. Interval (ARI), Y-years	1-year or less	2-years	5-years	10-years	20-years	50-years	100-years
Emptying time, **T** in days	0.5	1.0	1.5	2.0	2.5	3.0	3.5

3.5.4 The *modified* design storm method

The essence of the procedure offered as the *modified* design storm method has two basic elements :

- design of the required storage, volume $\forall_d$, by any conventional design storm method or by one of the (design storm) procedures presented in the Handbook; followed by,
- a check on emptying time, T, from the storage to ensure that it meets the appropriate criterion listed in Table 3.3 for the assigned ARI.

In simple retention systems (see Figure 1.3), such as “soakaways” and ‘dry’ ponds draining naturally – that is by infiltration or percolation – the check involves application of one of the formulae Eqns. (3.30), (3.31) or (3.32). Not all cases succeed and provision is made in the Handbook for this outcome : Procedure 4 (Section 5.1.5) enables the dimensions of simple devices and “soakaways” to be determined taking account of ‘hydraulic’ assistance (bore recharge or slow-drainage by pipeline) needed to meet the appropriate Table 3.3 criterion. [‘Slow-drainage’, the term used in the Handbook, is referred to as **extended detention** in US literature (US Dept. of Transportation, 1996)].

The Handbook advises users to design ‘dry’ ponds employing ‘natural’ emptying only and to avoid introducing ‘hydraulic’ assistance directly to these systems (see Section 5.1.4).

More detail on these matters is provided in the Handbook : the following notes are intended as an introduction only.

‘Natural’ drainage systems (emptying by infiltration or percolation) :

STEP 1 : design the retention storage device or installation, volume $\forall_d$, using the normal design storm approach for required ARI, critical storm duration, etc. (IEAust, 1999) or by Procedures 2 or 3 from the Handbook (see Chapter 5). Note that $\forall_d$ is the volume of stored stormwater;

STEP 2 : check to ensure that outflow by ‘natural’ means will result in emptying from full in a time, T, approximating that given – for the appropriate design ARI – in Table 3.3. This will normally involve use of one of the formulae (3.30) or (3.31) or (3.32).

In the event of **failure** to meet the Table 3.3 criterion, then the designer may explore either or both of the options outlined below :

Sites where aquifer access is available :

STEP 3A : check that all required conditions in terms of recharge water quality, environmental impact and costs can be met, then conduct an exploration to determine (single) bore recharge capacity, q_r , at the site;

STEP 4A : design the retention storage device or installation according to Procedure 4A in the Handbook (Section 5.1.5) using one bore;

STEP 5A : calculate emptying time, T, [Eqn. (3.33), below]. The outcome must then be compared with the value given in Table 3.3 for the appropriate ARI. If the indicated emptying time is **exceeded**, then two, three or more bores should be considered [revisit Eqn. (3.33)] until a satisfactory result emerges.

Sites where aquifer access is NOT available : slow-drainage should be explored :

In field cases where 'natural' means of storage-emptying are insufficient to meet the criteria listed in Table 3.3 and aquifer access is denied, then recourse must be made to slow-drainage as the last available option other than emptying by pumping. This involves, simply, providing a pipeline from the storage to dispose of the retained water into a local waterway or formal drainage path.

STEP 3B : taking the value of "T" (converted to seconds) given for the appropriate ARI in Table 3.3, and the device (stored water) volume, $\forall_d$, calculated previously (see STEP 1, above), determine slow-drainage pipeline flow, Qr :

$$Q_r = \forall_d/T \tag{3.33}$$

STEP 4B : design the retention storage device or installation according to Procedure 4B in the Handbook (Section 5.1.5) using Q_r as the release flow rate carried by the slow-drainage pipeline, but taking account of the conditions contained in the Special Note, following**.**

Special Note (slow-drainage case) :

A typical result for "Q_r" in STEP 2, above, is a flow less than 1.0 L/s, requiring a pipeline of extremely small diameter. The normal engineering design response to this result is to specify a pipe of some 90 mm size (unthrottled) on the ground that the target outflow rate will be easily achieved by such a conduit.

Certainly, with a 90 mm (unthrottled) pipe in place, the outflow, typically, will be considerably greater than required for satisfactory emptying, and the storage device or installation is likely to empty in much less time than set out in Table 3.3. In fact, the outflow rate, under these circumstances, may approach that of an uncontrolled site, defeating, totally, the purpose of having a retention storage. The criteria set out in Table 3.3 should therefore be interpreted – **in slow-drainage cases** – as target durations which should not be exceeded but which should, also, be **not significantly under-achieved.** The practical difficulty involved in balancing these seemingly conflicting demands is reviewed in Section 5.4 and a practical solution illustrated in Figure 5.6.

3.5.5 Catchment performance and the *modified* design storm method

The previous section on 'continuous simulation' modelling concluded with a discussion of the future direction that urban storm drainage design might take when analytical procedures have been developed to cope with the difficulty presently posed by catchment-wide modelling using the 'simulation' approach (Section 3.4.5). It offered a 3-step procedure, grounded in catchment-wide 'continuous simulation' modelling, but interpreted finally – at the scale of the individual site – in application of the complex catchment-wide 'continuous simulation' model (STEP 3A) **OR** the *modified* design storm method (STEP 3B). It remains for the latter approach to the design/dimensioning of on-site storages to be explained in the context of catchment-wide flood control objectives.

Consider the physical environment of a partly developed urban catchment which is assessed by consensus among all stakeholders to be operating **satisfactorily**. This implies that storm runoff in the Y-years ARI event (Y-years agreed by all interested parties) is accommodated by the existing drainage infrastructure with an *acceptable* level of inconvenience and flooding. This does not, typically, imply a "flood free" condition – although that could be the case if such a level of (flood) protection were to be demanded by stakeholders – but, more likely, acceptance of a trade-off between the financial burden of very high quality flood security and a modest level of community discomfort on rare occasions.

It follows (in this situation) that the surface runoff *volume* which passes from each component of the urban landscape to the existing drainage path **in the storm of critical duration** creates no unacceptable level of nuisance in the community. Thus, the goal of flood management by the local government agency within whose jurisdiction the partly urbanised catchment lies should be to preserve this status indefinitely through application of appropriate policy instruments. In such a regulatory environment, it is possible for a high level of development and re-development to occur without overloading the existing drainage infra structure : a vital condition of this scenario is competent maintenance – not upgrade – of the drainage infrastructure. The vision of “a high level of development/re-development” may, of course, be constrained by other (non-flood oriented) goals of stakeholders relating to issues of amenity, heritage, habitat, environmental values, biodiversity, etc.

Design of on-site works within the context described here becomes, then, simply a matter of ensuring that the ‘before and after’ runoff *volume* state of each component of development/re-development **in the storm of critical duration** remains the same. The various forms of on-site retention, including harvesting/re-use of the collected, cleansed stormwater (rainwater tanks, etc.) as well as opportunities for aquifer recharge and maintaining soil moisture, provide the means whereby this design objective can be achieved. The approach to design underpinning this framework for managed urban change is the *modified* design storm method.

What risk does the designer face in taking the path outlined above?

The design storm has characteristics determined through the process described in Section 3.4.5. If it were to occur on the catchment of interest, and **all on-site (designed) storages were empty** at its commencement, then equality of ‘before and after’ development/re-development runoff *volumes* would result and the overall system would perform as planned. However, if the storages were **not** empty at the commencement of the event, as consequences of the two (numbered) issues raised in Section 3.4.1, then runoff flows greater than those of the ‘before’ condition would result and the drainage system would undoubtedly overload. Adherence to the emptying time criteria set out in Table 3.3 offers the designer the best available security against this occurring.

The design storm is certainly a useful tool for drainage designers to use, but the likelihood of it ever occurring in a practical catchment case is remote, so the above scenario, in fact, is unrealistic. Critical flood conditions do occur – those that the stakeholders (see above) have identified as Q_{peak} (or peak ‘stage’) together with its associated ARI – but they result, typically, not from a design storm but from close sequences of small storms draining to storages which are not empty. Only correct ‘continuous simulation’ modelling is able to reveal these critical conditions.

This interplay between the *modified* design storm and ‘continuous simulation’ approaches is vital to a simple but credible urban drainage design practice. It may best be described as a design storm method calibrated to give the ‘continuous simulation’ outcome. To date (2004) only the “Heritage Mews” project in Western Sydney has been designed by both methods (see Section 3.9). The outcome showed the *modified* design storm method to require 7% greater storages than given by ‘continuous simulation’ modelling (Argue et al, 2003; Coombes et al, 2003).

3.6 RUNOFF COEFFICIENTS

Typical urban catchments in Australia have undergone a sequence of development/re-development phases in which primeval forests have been cleared, firstly, to make way for agricultural activity – crops or grazing. At a later stage, large-area farms located on the edges of urban concentrations were divided into small rural holdings of 5 – 20 acres (2 – 8 ha) apiece; these, in turn, were divided – typically – into quarter-acre allotment housing sub-divisions in the earliest phase of “greenfields” development, or given over to industrial estates.

The most recent stage of development/re-development which continues to take place following major planning changes on the part of State and Commonwealth governments in Australia to combat ‘urban sprawl’ (Dept. of Housing and Regional Development, 1995), is :

- “greenfields” developments in urban fringe areas with (housing) allotment sizes in the range 300 m^2 – 600 m^2;
- “greenfields” industrial estates in urban fringe areas; and,
- urban infill involving re-development of old housing stock – typically quarter-acre blocks – as sites for two, sometimes three, residential units or townhouses.

The effect of the urban evolution, outlined above on the **structure** of upper-level soils in the regions of development/re-development has been profound, in particular, progressive loss of natural seepage paths along root systems, worm holes and animal burrows. The consequence is a very different response to rainfall : runoff coefficients for urban soils are greater than values for forested catchments. In Southern Australia (see Figure 3.8) the difference is not great, as Table 3.4 shows : but in Northern Australia the corresponding figure (runoff coefficient for pervious areas) is seven times its Southern Australian counterpart.

The explanation for this is not significantly different soils, but, rather, significantly different climates in which the critical conditions for flooding take place. Most **urban** flooding in Australia occurs as the consequence of cyclonic activity characterised by high intensity rainfall of relatively short duration in summer, typically December to March, inclusive. In Southern Australia, this occurs when the soil is in a hot/dry state resulting in low runoff rates; in Northern Australia, the cyclonic rain falls on catchments which may have experienced prolonged hot and wet conditions. The consequences are reflected in the basic information from “Australian Rainfall and Runoff” (IEAust, 1999) presented in Table 3.4.

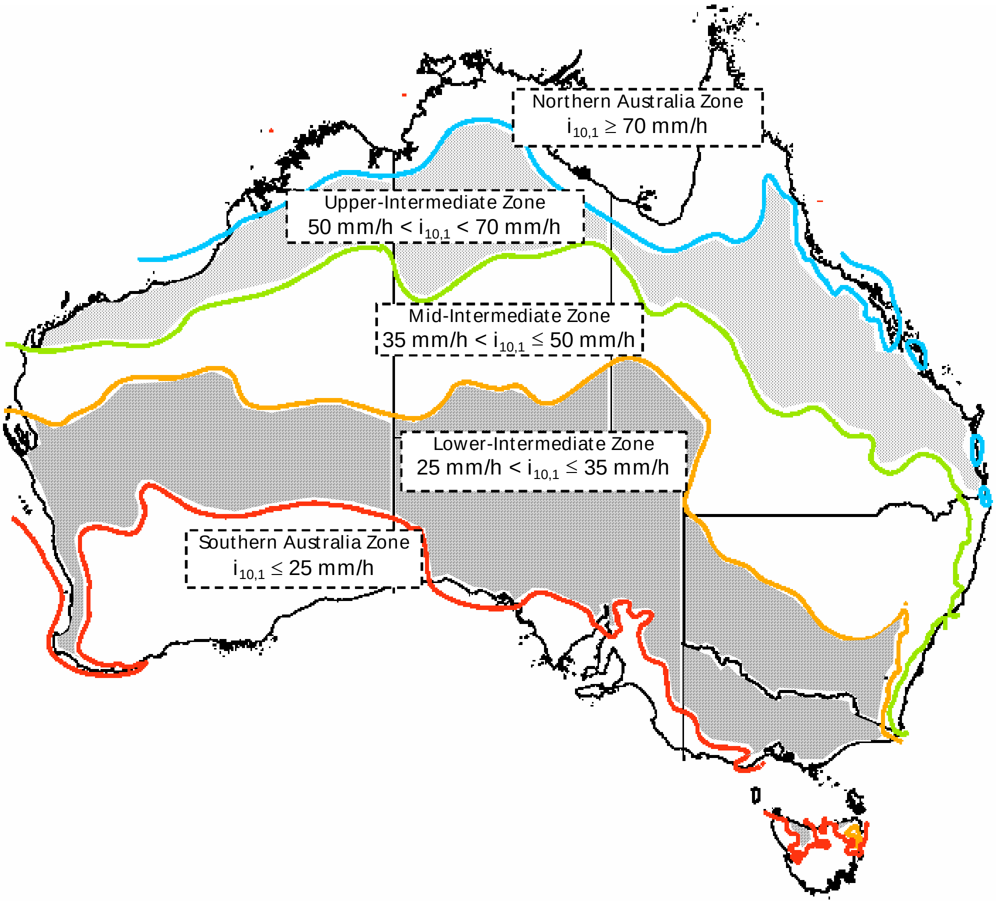


FIGURE 3.8 : Australian climate zones based on 10-year, 1-hour rainfall
Courtesy Bureau of Meteorology, Melbourne Office, gratefully acknowledged.

TABLE 3.4
BASIC RUNOFF COEFFICIENTS (C_{10})* FOR URBAN CATCHMENT SURFACES

SURFACE CLASSIFICATION	NORTHERN AUSTRALIA ZONE	SOUTHERN AUSTRALIA ZONE
Connected paved areas : • roadways and roofs	C_{10} = 0.90	C_{10} = 0.90
Unconnected paved areas and pervious areas : • mixed with paved areas as in residential land use; • major urban open space areas, parks, etc.	C_{10} = 0.70	C_{10} = 0.10

*** C_{10}** is the runoff coefficient for ARI, Y = 10-years storm conditions.

The explanation, above, for the sequence of development stages through which the urban landscape has evolved in the great bulk of cases, and the effect this has had on the values for pervious area runoff coefficient listed in Table 3.4, are relatively undisputed. What is less certain are the values of C_{10} which should be used in situations where the process is short-circuited and forest catchments are converted directly to urban development – typically residential in character – without passage through the agricultural phase. [Reference to Table 3.4 should, undoubtedly, be made for values of C_{10} to be applied to pervious areas integrated *within* such developments, despite their immediate pre-history. The reason for this is the totally destructive effects which land development processes have on soil structure in such circumstances.]

However, the interpretation of WSUD into such a scenario – in keeping with the principle explained in Section 1.4.2 – requires the designer to recognise the pre-development pervious area not as Table 3.4 presents it but, rather, as a **forest land use**. [AR&R – 1987 (IEAust, 1999) is silent on this issue.] This would see use of $C_{10} \cong 0.10$ for all (forest) catchment development cases, regardless of location in Australia. This value of runoff coefficient (for forest catchments) follows Canadian practice (Stephens et al, 2002; Gilliard, 2004).

The major Australian reference in the area of urban runoff (IEAust, 1999) provides a linear interpolation procedure based on the (location) ARI, Y = 10-years, 1-hour stormburst intensity, for calculating paved and pervious area runoff coefficients for any place in Australia between the Northern and Southern zones (see Figure 3.8). Steps in the procedure to determine pervious area C_{10} for location X are as follows :

STEP 1 : Determine Northern Australia and Southern Australia values for the runoff coefficient of interest – C_N and C_S , respectively from Table 3.4;

STEP 2 : Determine the 10-year, 1-hour average rainfall intensity at location X, i.e. i_X ;

STEP 3 : Compute the required value for C_X :

$$C_X = C_S + \frac{(i_X - 25)}{(70 - 25)}(C_N - C_S)$$

$$= C_S + 0.022\ (i_X - 25)(C_N - C_S) \qquad \textbf{(3.34)}$$

Example :

Find the runoff coefficient for pervious areas, design ARI = 10-years, i.e. $[C_{10}]_{perv}$ for Newcastle, N.S.W.

STEP 1 : $[C_{10}]_{perv}$ for Northern Australia = 0.70
$[C_{10}]_{perv}$ for Southern Australia = 0.10
(from Table 3.4)

STEP 2 : the 10-year, 1-hour, average rainfall intensity at Newcastle, $i_{(10,1),\ N'castle}$ = 49.6 mm/h

STEP 3 : $[C_{10}]_{perv}$ for Newcastle = 0.1 + 0.022 (49.6 – 25) (0.70 – 0.1 0) = 0.43 **(3.35)**

One further step is required for the designer to be able to determine runoff coefficients in all practical situations. This relates to the conversion of C_{10} values into runoff coefficients for frequencies other than 10 years. The required **Frequency Conversion Factor, F_Y,** information is presented in Table 3.5 (Argue, 1986). The listed factors are applied directly to the appropriate C_{10} value(s) calculated by the procedure described above, with the over-riding constraint that the (derived) runoff coefficient should never exceed 1.0

TABLE 3.5

FREQUENCY CONVERSION FACTOR, F_Y
(after Argue, 1986)

ARI (years)	1 and <1	2	5	10	20	40	60	80	100
Conversion Factor, F_Y	0.8	0.85	0.95	1.0	1.05	1.13	1.17	1.19	1.20

3.7 RAINFALL

3.7.1 Rainfall and the ARI, Y = 0.25-year stormburst intensity

Each of the design procedures included in the Handbook involves use of Australian rainfall information in one of its many available forms. Because of the simplicity of the procedures set out in Chapters 5 and 7, peak runoff flow can be determined, satisfactorily, from average stormburst intensity, $\mathbf{i_{ave}}$, and runoff volume, ∀, can, similarly, be calculated using data from the basic IFD chart or table readily available from the Commonwealth Bureau of Meteorology or other sources for any location within Australia. Examples of these data sets are given in Appendix B representing Southern Australia, Intermediate Australia and Northern Australia (see Figure 3.8).

More detailed rainfall information required for 'continuous simulation' modelling in connection with raintank sizing (Procedure 9 in Chapter 8) calls for the use of daily rainfall records of long duration. These are also readily available, again from Bureau sources for over 7,000 stations where long-term records have been kept. No procedures included in the Handbook require users to employ small-interval, for example 6-minute or 15-minute, continuous rainfall information. Where such data bases are necessary to deliver a satisfactory outcome, the authors have undertaken the necessary modelling and produced results in forms (graphs and charts) which are directly useable by practitioners.

The only domain of rainfall information where the procedure in the Handbook departs, significantly, from current practice relates to the ARI, Y = 0.25-years rainfall (average) intensity, $i_{0.25}$, and its fraction or proportion of the ARI, Y = 1-year value. [The peak flow generated in a catchment in the ARI, Y = 0.25-years event is widely used in Australia as the basis for design of GPT installations, swales and stormwater quality improvement wetlands.] This relationship – in the interests of simplicity – needs to be 'globalised' and an acceptable average factor struck for universal use across the nation.

Australian practice to date has followed the lead of NSW Dept of Housing (1998) by setting $i_{0.25}$ equal to 0.25 times i_1 , the ARI, Y = 1-year rainfall (average) intensity. The factor 0.25, used in this context, is incorrect for Australia-wide use: examination of IFD data from stations across the continent shows the factor which needs to be applied to i_1 is 0.50 or an even greater value. The following simple relationship is, therefore, adopted in the Handbook and recommended for general use :

$$\text{0.25-years stormburst average intensity, } i_{0.25} = 0.50 \times i_1 \qquad \textbf{(3.36)}$$

3.7.2 The rainfall data base : some current issues

"Embedded storms" : Recent years have seen disquiet expressed by some researchers concerning both the IFD (intensity-frequency-duration) data used in current Australian practice and, more particularly, the design temporal patterns recommended by AR&R – 1987 (IEAust, 1999). [See Rigby et al (2003).] The principal complaint centres on the claim that the analysis which has given us both data sets extracted "storms from within storms" (the so-called "embedded storms") and that when these data are applied into the context of design, they lead to systematic *underestimates* of flows and, hence, underestimates of storages.

While the voracity of these concerns will be fully explored in the course of preparing the next edition of "Australian Rainfall and Runoff", it is considered prudent to draw this issue to the attention of readers and, by so doing, to alert them to possible changes in practice which may ensue when these matters are resolved.

The procedures presented later in the Handbook and which employ the *modified* design storm approach, stand outside this controversy because they leave responsibility for estimating Q_{peak} or runoff volume, ∀, entirely in the hands of current Australian practice as set by Engineers Australia. It is recommended that practitioners take interest in the discussion outlined above, but in all matters of day-to-day design, follow the most up-to-date version of AR&R available from the professional body.

Continuous simulation : Another matter relating to rainfall which is currently under discussion in professional engineering circles, is the data base which should be used in 'continuous simulation' modelling. Two types of data are available :

- Long period daily rainfall records; and,
- Long period, short time increment (typically, 6 minutes, 15 minutes, etc.) continuous rainfall data.

Records of the first type are available from the Commonwealth Bureau of Meteorology for over 7,000 stations throughout the nation. These are used for sizing rainwater tanks in the spreadsheet procedure set out in Chapter 8.

The second category of data is available for some 600 Bureau stations: however it is possible to extend this data set to many of the 7,000 daily-reading stations using the DRIP program of Heneker et al (2001). Users of the Handbook are encouraged to proceed along these lines and follow the 13-step process described in Table 3.2 to obtain the required relationships for individual locations of interest. However, the authors have undertaken this task using recorded data for selected stations to produce design charts for pollution control installations (Appendix C) and rainwater storages (Appendix E) for climate groupings covering the entire Australian land mass. This is explained in Sections 7.2 and 8.6 leading to Procedures 5/6 and 10 respectively.

3.8 ON-SITE RETENTION (OSR) TECHNOLOGY : SOME DO'S AND DON'TS

1. **Unsuitable soils :** Don't put OSR devices ("leaky" wells, gravel-filled trenches, etc.) in soils which are predominantly "wind-blown" sands. This does not exclude well-compacted dune sands but loose, aeolian sands should be avoided. There are also clay (calcareous) soils which collapse when in direct contact with retained water and acid sulphate and 'sodic' soils which should also be avoided. Good practice demands that sites where 'source control' technology by retention is contemplated must be visited for permeability testing (see Section 3.2) and soil assessment to determine suitability.

2. **Clearance distances :** Don't put OSR devices closer to building footings or to boundaries than the recommended clearance distances. These are included with the soil classification information (five classes of soils) in Section 1.3.4. The consequences of ignoring this advice may be fracture of footings particularly domestic footings, and severe cracking of walls, both internal and external. The "Water-reactivity and 'clearance'" paragraph in Section 1.3.4 should, however, be noted.

   The need to observe clearance distances to boundaries arises from the possibility that a neighbouring building may be placed on the boundary and be adversely affected by an OSR device placed next door.

3. **Rock and shale :** Don't put OSR devices in rock which has zero or near-zero permeability. This includes most non-sedimentary rock and some sedimentary rock such as shale. However, the issue of suitability should not be decided on the evidence of geological (map) information alone : permeability testing on site should be carried out and may reveal an apparently impermeable stratum to be severely weathered or fractured and, therefore, suitable for infiltration technology. The permeabilities of some sandstones have been found to be comparable to those of medium clays, encouraging the view that OSR technology can be considered in sandstone sites also (see "Sites with rock or shallow soil cover over rock", Section 1.3.4).

4. **Shallow soil cover over rock :** Great care must be exercised before applying OSR technology directly to shallow-soil sites. This is because of the likelihood that water stored on or near impervious bedrock will provide a stream of flow along the soil/rock interface and proceed down-slope. The plane of emergence of this interface can be predicted from study of the local geology : this needs to be checked and a conclusion reached as to its importance. If the plane is remote from dwellings or roads, etc., then a water-retaining device can be considered. However, if detailed exploration produces the possibility that emerging seepage will create nuisance or hazard for those downstream, then the prospect of using an OSR device in such circumstances should be abandoned.

   The possibility that the underlying rock is permeable, severely weathered or fractured should be explored : a positive finding in this regard may lead to a situation similar to that addressed in item 3, above.

5. **Steep terrain :** Use of OSR devices in steep terrain is, generally considered unwise. British practice places a limit of 5% on the land-slope where on-site water-retention is recommended. The reasons for this limitation are not so much slope-dependent, but are related, rather, to the soil/rock conditions most likely to be encountered in steep terrain. These are reviewed in items 3 and 4, above. Recommended practice would therefore be to exclude water-retaining installations from steep-slope sites in the absence of thorough exploration of site and down-slope geology. A simple guideline is : soil depth (suitable soil) of at least 3 m should be available **throughout** a down-slope, developed hillside before OSR practice should be contemplated.

6. **Watertable interaction with OSR :** The presence of a high (unconfined) watertable certainly limits the potential for use of OSR devices but does not, of itself, preclude them. Provided groundwater levels are **stable**, apart from seasonal fluctuations, they can be associated with quite successful water-retention systems (see “High groundwater environments”, Section 5.3.4). Serious problems can arise, however, at sites where groundwater levels show a systematic rise explained by, perhaps, removal of forest vegetation or the presence of an artificial lake or some other global cause. The inclusion of OSR devices in such circumstances will only aggravate the situation, accelerating the inevitable “waterlogging” of the region. OSR systems should be avoided in any terrain which exhibits a rising water table, particularly where it is **highly saline**. This problem – salinisation – is particularly serious in parts of Western Sydney, Wagga Wagga, NSW, and south-west Western Australia (see WSROC, 2003).

   Another problematical aspect of interaction between watertables and OSR practice is where components of the built environment such as basements and undercroft garages, intersect shallow aquifers. Construction of such components must take account of seepage encountered under these circumstances using, for example sump pumps. It would be quite unacceptable for additional water to be introduced into the aquifer through on-site stormwater retention practices applied in the same area. A possible solution is to install OSR devices draining directly to a **lower** aquifer, but it would be imperative that the standing water level (SWL) of the lower aquifer be significantly below the upper aquifer, **and that it remained so** indefinitely.

Watertable affected by upstream OSR : Care must be taken where groundwater levels meet the stability criterion, reviewed in item 6, above, but where future intended use of OSR devices, upstream, may lead to watertable rise. This situation is possible in valley “bottoms” below hillside developments where significant water-retention is intended. Again, thorough geological exploration of locations is called for to assess the likely impact of (upstream) OSR devices on valley floor watertables : outcomes may limit or even preclude on-site water-retention in some cases.

Aquifer recharge/retrieval – **annual balance :** The regime of “flat” potentiometric gradients associated, normally, with flat and gentle-grade landscapes is where ASR (aquifer storage and recovery) technology is most likely to be feasible. It is recommended that recharge exceed retrieval of groundwater by about 20% on an annual basis (see Northern Adelaide and Barossa CWMB, 2000) : this ensures continued equilibrium of local potentiometric levels and, also, sustainability of the resource.

Water quality inflows to OSR devices : Don’t put uncleansed stormwater runoff directly into sub-surface OSR devices such as “leaky” wells and gravel-filled trenches. The **only** site drainage which can be directly accepted is roof runoff, and even this component should be treated by passage through a rainwater tank with ‘first flush’ filter, or the devices illustrated in Figure 1.5. All leaf-matter must be strained from roof runoff before entering these devices. “Leaky” wells are more robust than gravel-filled trenches in this regard, since they can be more easily de-silted. Any sediment entering a gravel-filled trench is, virtually, locked in for the life of the installation (see “Runoff categories and treatment”, Section 1.3.3).

These cautions do not preclude general surface runoff from on-site retention practice, but the treatment train required for a significant installation must be carefully designed using grassed surfaces, vegetated strips, swales, sand filters, gravel-based reed beds, ‘treatment train’ tanks, geotextile ‘final filters’, etc. reviewed in Chapter 2. Three installations using these elements are described in Section 3.9.

3.9 SOME EXAMPLES OF ON-SITE RETENTION PRACTICE

New Brompton Estate Stormwater Management Project : This project, constructed in 1991, is centred on a small reserve 50 m × 45 m in the Adelaide suburb of Brompton in the City of Charles Sturt. The reserve is surrounded by 15 townhouses, the roof runoff of which is directed, via PVC pipes, to a gravel-filled trench located around three sides of the reserve (see Figure 5.2). Storm runoff collected in winter in the trench is conveyed to a central point where it enters a bore and is transferred to storage in a *Tertiary* aquifer, 30 m below surface level. In summer the stored water can be retrieved from the aquifer and used for irrigating the reserve, thereby saving on mains water normally used for this purpose. This is an example of small-scale aquifer storage and recovery (ASR) technology (Hopkins and Argue, 1994). **Maintenance** of the system at New Brompton Estate involves retrieving the mesh bucket lined with geotextile located at the base of each inlet standpipe (see Figure 5.2) and cleaning it. This operation, for the set of nine standpipes, requires a two-hour visit by the technician four times per year (since 1992). About 2 kg of sediment is removed from each bucket on these occasions. No other maintenance directly connected with the stormwater management and recharge system is needed. [The water retrieval system required for irrigating the reserve has not been installed.] The standpipe system, although satisfactory, is **not** recommended. The approach illustrated in Figure 2.1 is preferred : this is a zero maintenance system with a 'lifespan' of 20 years receiving an uninterrupted supply of roof-originating sediment (laboratory study result). 'Lifespan' of indefinite length – without maintenance – is possible for a gravel-filled trench where it receives roof runoff subject to 'first-flush' treatment or is overflow from a rainwater tank.

Parfitt Square Stormwater Management Project : Opened in 1997, Parfitt Square is a 0.6 ha recreational reserve in the City of Charles Sturt, Adelaide. The reserve contains a 'treatment train' of elements which strip sediment passing to it from 1 ha of mixed housing – old and new – and carriageways (see Figure 3.9). Runoff entering the park via car park A encounters, at B, a long, sediment-holding trough (0.5 m × 0.5 m section, 30 m long), followed by a 300 m^2 gravel-based reed area where surface runoff is further cleansed. Next in line is a 100 m long, underground gravel trench with section 4 m × 1m (shown as C). This component terminates in four recharge bores (D) supplying the *Quaternary* aquifer at 12 m depth. Recharge takes place mainly during the winter rainy period; the stored water is recovered for summer irrigation of the reserve (irrigation bore, E). This is another example of small-scale ASR technology. The system was designed to control, without overflow, runoff in all events up to and including that corresponding to the "once in 100 years" event (Argue and Pezzaniti, 1999).

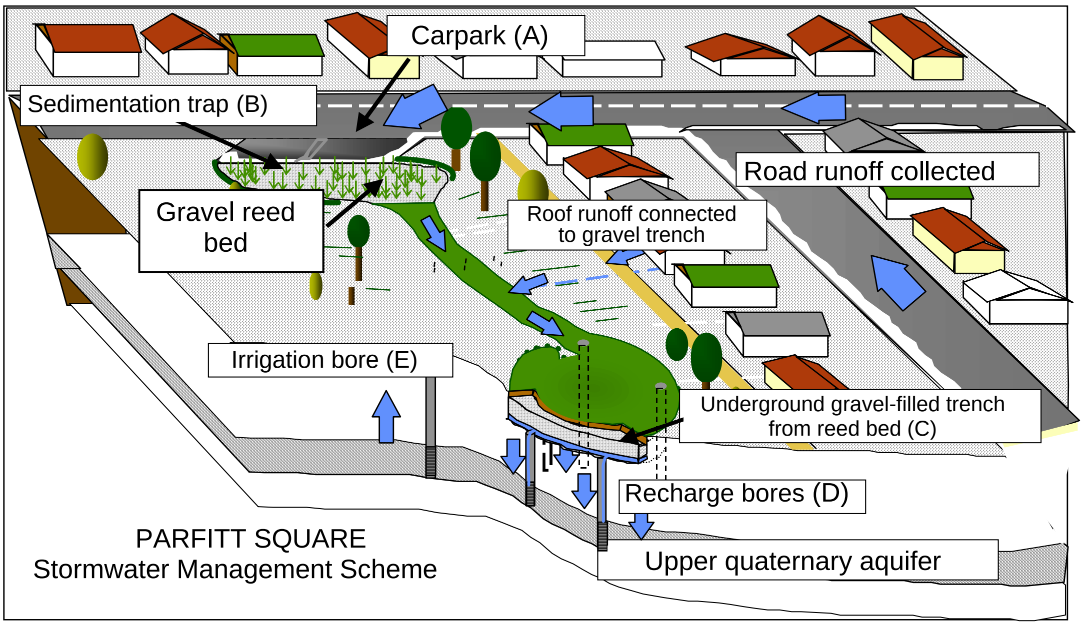


FIGURE 3.9 : "Parfitt Square" : general layout, looking North

Maintenance of the stormwater cleansing and recharge/retrieval system is concentrated on cleaning out the 30 m trough (B) where coarse sediment and other gross pollutants are deposited after each storm. A cleaning operation is mounted once in every two or three years. The gravel-based reed bed (see Section 7.8.5 for species) accumulates fine sediment which is retained within the spaces between the gravel – initial depth, 100 mm : when these spaces fill, another layer of gravel is introduced. The system has been constructed to retain, eventually, all of the fine sediment expected from the catchment over its 100 years ‘life’. Present indications (after six years service) are that the geotextile filter at the head of each bore (D) will need replacing about once in every 20 years.

St. Elizabeth Church Car Park : The car park at St. Elizabeth Church at Warradale, Adelaide (opened 1998), is part of a rainwater collection/treatment/irrigation system which includes tennis courts, the church building and landscaped area and the adjacent Council reserve. **All** surface runoff is retained on site: the design maximum storm is the “once in 100 years” event. In the case of the car park segment : all paved area runoff passes to a central, grassed ‘hard-standing’ area surfaced with “Grasspave”, a plastic ring-matrix product, where it is cleansed by the grass and filtered through 200 mm depth of sand/gravel mixture (see Figure 5.1 and Section 7.8.5). Beneath the filter is a gravel-filled “soakaway” which gathers the clean storm runoff and directs it to a bore where recharge is conveyed to a *Tertiary* aquifer 45 m below ground level. Stormwater generated on the tennis courts passes through a sedimentation tank before entering a gravel-filled trench which also conveys water to the bore; runoff from the church roof, after filtration, enters the same bore. In summer, water drawn from the aquifer through the recharge bore is used to irrigate the church landscaped area and portion of the Council reserve (Argue and Pezzaniti, 1999). **Maintenance** of the scheme at St. Elizabeth Church consists, almost entirely, of caring for the grassed hard-standing area – mowing, weeding, etc. – and providing adequate irrigation for the drought-tolerant grass (25 mm per month in summer). Apart from this, there is cleaning out of the commercial settling tank receiving runoff from the tennis courts. Sediment originating on the church and parish hall roofs is managed in the manner illustrated in Figure 2.1 and is expected to give at least 20 years service without reinstatement. Normal maintenance of the pumped retrieval system is experienced.

Plympton Church Outdoor Community Garden : The basis of the ‘passive’ irrigation facility at Plympton Anglican Church, Adelaide (completed 2001), is a “soakaway” (see Figure 5.3). Storm runoff passing to the “soakaway” is drawn from the main church buildings, an extensive paved area and the roofs of four church-associated residences. A sediment collection and ‘first flush’ filtration system for cleansing the runoff is located upstream of the sub-surface installation. The facility provides ‘passive’ irrigation of a 400 m^2 garden of native plants and drought-resistant grass planted on the backfill above the “soakaway”: water evaporating from the runoff stored among the gravel void spaces is applied directly to the roots of the native vegetation and grass (Argue and Pezzaniti, 1999). Evidence from this (trial) technology in the harsh South Australian climate suggests that the native vegetation and grass of the garden can be maintained indefinitely with minimal conventional (surface) irrigation. Further evidence of ‘passive’ irrigation with-standing severe drought conditions (spring/summer, 2002) was also observed in grass covering the “soakaways” constructed in Western Sydney (see Section 5.5). **Maintenance** of the Community Garden involves horticultural attention to the shrubs and grassed areas as required by any native garden, except for irrigation which is unnecessary. The scheme operates as a “waterless garden”, however, a sprinkling of water is sometimes applied to “green up” the grass ahead of community use of the site for weddings and other gatherings. The ‘first flush’ grassed area will in due course need to be replaced: its expected ‘lifespan’ is at least 15 years (see Section 7.8.6).

“Figtree Place”, Newcastle, N.S.W. : The Bus Station re-development in Central Newcastle (completed 1998) involves a 0.6 ha site which has been remediated after some decades of contamination by petrol hydrocarbons and other pollutants. The site is now home to 27 one-, two- and three-bedroom residential units called “Figtree Place” (see Figure 5.4). The Brief for the project called for a high level of water conservation to be incorporated into the ultimate design (Argue, 1997; Argue and Argue, 1998). The outcomes of the concept, planning and engineering phases of the project are :

- all roof runoff is collected and stored, temporarily, in underground tanks before being used as base supply for (gravity) hot water systems and toilet flushing;

- overflow from the rainwater tanks passes to gravel-filled trenches which temporarily store the excess before diverting it by percolation to the watertable (unconfined aquifer);

- all surface runoff is directed to a “dry” pond where it is cleansed prior to seeping downwards to also recharge the groundwater;

- groundwater beneath the site is used for open space and garden irrigation in summer, and to supply the entire water demand of the bus-washing facility in the adjacent Bus Station;

- the system contains all rainfall events up to and including the "once in 50 years" storm in 83% of the re-developed area.

The water conservation performance of the residential segment of the re-development shows around 50% reduction in domestic water consumption and overall (including the bus-washing facility) some 60%; there is virtually "zero runoff" from 83% of the site (see Coombes et al, 2000).

"Heritage Mews", Castle Hill, N.S.W. : This development (completed 2004) of a 3.2 ha "greenfields" site for townhouse accommodation in Western Sydney represents the first practical application of the **regime-in-balance** strategy (see Sections 4.2.1, 4.6.5 and 5.4). The terrain was characterised by shallow, heavy clay over rock and therefore unsuitable for infiltration as the primary mode of stormwater disposal. A Concept Design for the development which included 62 townhouses was prepared as a collaborative project between Dr. P. Coombes (University of Newcastle), Cardno BLH and UWRC staff (Argue et al, 2003; Coombes et al, 2003). Effective removal (of storm runoff) according to the time scale required – 24 hours – was achieved using 29 gravel-filled trenches and the 'slow-drainage' provisions of Procedure 4B (Section 5.1.5) : a special device was developed and incorporated with each trench to control outflow to the target value (see Figure 5.6). The inclusion of 62, 3.0 kL rainwater tanks – one with each dwelling – provided a significant component of retention within the development during storms, as well as replacement of some mains water. By these means, the development avoided using a 'bottom end' detention basin of plan area 2,500 m^2, the normal requirement of Council. The Developer was able to include six or seven housing lots which would otherwise have been allocated to the detention basin area. High standards of stormwater quality control were achieved using UniSA tanks (see Section 2.9.3).

4. GETTING STARTED WITH 'SOURCE CONTROL' IN WATER SENSITIVE URBAN DESIGN

4.1 INTRODUCTION : FOUR 'SOURCE CONTROL' CATEGORIES

'Source control' of stormwater embraces each of the aims listed in Section 1.3.1, involving :

- control of storm runoff quantity;
- control of pollution conveyed in storm runoff;
- harvesting of cleansed storm runoff.

Unlike conventional storm drainage systems which tend to be focussed on the single issue of peak flow conveyance/reduction, water-sensitive stormwater management usually reflects some level of involvement with each of the domains listed above. For example, a gravel-filled trench (see Figures 1.3, 1.4 and 1.5) required to retain roof runoff from a domestic building should be designed, primarily, to control flooding but its long-term effective operation depends to a large extent on the adequacy of provision made for sediment collection/disposal. Furthermore, the process of emptying the device between storms can involve recharge/percolation/evapotranspiration mechanisms that have significant groundwater or soil moisture enhancement and therefore harvesting implications. More complex, on-site stormwater retention systems such as those described in Section 3.9, illustrate this "interconnectedness" even more strikingly.

The various devices and systems available to the designer for achieving the three aims listed above may be divided into four broad categories :

Category 1 Systems : Those whose primary function is to manage the quantity of storm runoff generated in contributing catchments. This *primary function* has two sub-sets : quantity control in relation to flooding, and management of 'low flows' to maintain waterway bio-communities (fauna and flora). Management of pollution conveyed in the runoff in such cases should be recognised and addressed, but its influence on the dimensioning of the water-retaining devices and systems to achieve quantity control is secondary; stormwater harvesting has important links to both the flooding domain and that of environmental flows.

Examples of Category 1 installations include :

- infiltration or 'treatment' surfaces receiving paved area runoff;
- in-ground "leaky" devices receiving roof runoff;
- rainwater tanks dimensioned to achieve 'low flow' objectives.

Category 2 Systems : Those whose primary function is to achieve **high performance in containing or removing particulate matter** greater than a specified size from set percentages of annual average runoff flows, typically 90%, 80% 70%, 60%, generated in contributing urban catchments. Management of large storm runoff quantity must be recognised and addressed but its influence on detailed dimensioning of sediment control installations is secondary; similarly for stormwater harvesting.

Examples of Category 2 installations include :

- swales and sediment-retention tanks;
- 'first flush' pollution control tanks;
- systems involving treatment surfaces with underlying (temporary) storage, such as permeable paving.

Category 3 Systems : Those whose primary function is to **harvest runoff** from contributing catchments. Design of stormwater harvesting systems may involve analysis of the frequency of failure to supply demand as well as 'yield' considerations which differ greatly from the peak flow, volume and bypass/overflow-oriented analyses required by Category 1 and 2 systems. The flood control and water quality management aspects of such installations, while significant and must be taken into account, are of secondary importance.

Examples of Category 3 installations include :

- rainwater tanks;
- aquifer storage and recovery (ASR) schemes;
- basins designed for amenity or water recreational activities.

Category 4 Systems : Those with **multi-purpose functions (none dominant)** drawn from the fields of flood control, pollution containment, stormwater harvesting and amenity. Dimensions and detailing for such devices and systems require two or all of these aspects to be considered separately and designs developed which either embrace all demands or represent a compromise between them.

Examples of Category 4 installations include :

- ASR schemes with surface detention storages providing flood control capability;
- swales designed for sediment retention and flood conveyance;
- rainwater tanks used in conjunction with a flood control strategy.

4.2 STORM RUNOFF MANAGEMENT IN CATEGORY 1 SYSTEMS

4.2.1 Three basic stream management scenario/strategies

Category 1 systems should be understood as components of channel networks gathering runoff from groups of sub-areas into branch and main drainage paths, typically of dendritic form (see Figure 4.1). Such catchments or basins should be managed by municipal agencies responsible for planning policies and strategies governing the orderly development of 'greenfields' and old occupied sites within their jurisdictions (see Section 4.6). These administrative activities are aided by a classification approach which considers some point "P" on the relevant catchment mainstream (see Figure 4.1), as a **location of singular interest**. The interest may stem from three broad stream management scenarios and associated strategic objectives :

Scenario (a) : P is the site of an end-of-catchment harvesting scheme such as a wetland (on- or off-stream) where storm runoff, generated in the upstream urban landscape, is collected and cleansed prior to use. The objective of catchment management in this case is to maximise the quantity of water harvested in the catchment 'bottom lands' while, at the same time, ensuring that floodwaters (ARI = Y-years event) are contained within a defined flood plain : this objective is implemented at the site level in the **yield-maximum** strategy;

Scenario (b) : P is a recognised reference point in a catchment or sub-catchment in which urban development has occurred and/or is likely to occur under environmentally responsible local government or catchment-wide administrative control outside the borders of a defined flood plain (ARI = Y-years). The objective of catchment management in this case is to maintain the harmonious and synergistic relationship which exists between continuing urban development and 'acceptable' use of the flood plain for agricultural and amenity pursuits : this objective is implemented at the site level in the **regime-in-balance** strategy;

Scenario (c) : P is a recognised reference point in a catchment or sub-catchment in which poorly-controlled urban development has already intruded deeply into the flood plain with consequences of past serious flood damage and/or severe channel erosion with the potential for more trauma and environmental damage in the future. The objective of catchment management in this case is to establish and implement local government or catchment-wide strategies to improve the performance of the urban flood control infrastructure and to prevent further urbanisation of the flood plain (ARI = Y-years). This objective is achieved, in particular, by minimising the quantity of stormwater discharged from all new developments and re-developments under the **yield-minimum** strategy.

[**Environmental flows :** The bulk of text which follows under the Category 1 heading is devoted to the 'high flows' or *flooding* domain. The realm of 'low flow' hydrology contrasts sharply with this : indeed the two domains are, normally, mutually exclusive. 'Low flow' considerations lead to waterway environmental values (preservation of stream bio-communities (fauna and flora), etc). This topic is explored in Section 4.7, following, under the title "Mimicking of Natural Catchment 'low flows' in the Urban Landscape".]

These scenarios lead to three contrasting approaches to managing storm runoff in catchment components contributing runoff to and beyond point P. They are described as :

Yield-maximum strategy : This applies to case (a), above, where stormwater generated on all components of the upstream catchment is seen as a resource to be harvested at a central location (for example, a wetland). Retention of storm runoff using the 'source control' measures and devices introduced in Section 1.3 represents an approach which is in conflict with this objective and, therefore, to be avoided. The most effective way in which the aims of the **yield-maximum** strategy can be implemented is to use conventional storm drainage approaches – maximum paving, pipes, lined channels, etc. – to attract the greatest possible yield from the urban landscape. This has the potential to cause downstream flooding and degradation of waterways and receiving waters (see Section 1.1) : these consequences should and can be reduced through judicious use of detention techniques applied within the ARI = Y-years flood plain (UPRCT, 1999) together with wetland water quality improvement technology (see Sections 2.13 and 2.14). However, the warnings of Debo and Reese (1995) should not go unheeded (see also Nehrke and Roesner, 2004).

Regime-in-balance strategy : In this case, peak flow at point P (see Figure 4.1) for a specified average recurrence interval (ARI = Y-years), must be held at a target value as development/re-development takes place in the (upstream) contributing catchment. The 'defined flood plain' referred to in scenario (b), above, needs to be fixed by agreement between all interested stakeholders and should represent a realistic benchmark determined by consensus. [See the 'benchmark year' concept, Section 4.6.3.] A practical outcome from such consultation might be, for example, "… the 100-years flood level as it existed in 1980". The essence of the strategy is its insistence that surface runoff *volume* passing from each catchment component in the defined Y-years event **following development/re-development** is the same as it was (from that same catchment component) **at the time of the adopted 'benchmark'**. This approach was introduced in Section 1.4 : further explanation is offered in Section 4.6.5

It needs to be understood, in this context, that the Y-years defined flood plain **cannot,** for practical reasons, be the waterway that existed prior to European settlement. Only by removing all rural and urban development upstream and re-establishing indigenous species forests and ecosystems, could such an identification be possible. This harsh reality is not always well-received or understood by advocates of 'rejuvenation' who have been disappointed at times with the outcomes of contemporary stream restoration programmes (Chandler, 2001).

Yield-minimum strategy : This case is identical to that of the regime-in-balance strategy described above in terms of the Y-years peak flow target being maintained (not exceeded) as development/re-development takes place in the upstream catchment. However, the difference lies in recognition of the significantly smaller (flow) capacity of the 'defined flood plain' in the present case and, hence, the need for extreme measures to be instituted (for example, zero runoff from catchment components) to minimise runoff contributions from all or a high proportion of new development/re-development projects. Such action, suitably interpreted and implemented through catchment-wide planning instruments, can achieve the ultimate objective of matching the catchment's (Y-years storm) response to the available flood plain capacity (see Section 1.4). A procedure for applying the strategy is explained in Section 4.6.6.

All (urban) catchments and sub-catchments within a municipal jurisdiction rarely, if ever, fall into one, only, of the above three scenario-strategies. The context of each waterway – particularly its flood history – is likely to place it far more in one category than either of the other two. For example, a stream may show satisfactory containment of floodwaters within its floodplain in the majority of events, but pose the threat of significant property damage in rare floods. This puts the waterway predominantly in scenario (b), above, and therefore a candidate for primary application of the **regime-in-balance** strategy. However, the evidence of damage risk in rare floods carries with it the likelihood of a deteriorating situation if not well managed, so a modest-level application of the **yield-minimum** strategy would also be appropriate. The resulting planning policies applicable to each portion of catchment or land use change would have to be determined by careful and systematic hydrological modelling using a modern computer package. [This process is the basis of the strategic direction discussed in Section 4.6.]

[Scenarios (a), (b) and (c) and their associated strategies, above, may seem to provide, between them, the basis for the full range of policy/technical tools needed to manage catchments experiencing the impacts of contemporary urbanisation. While they (the scenarios/strategies) are readily recognisable in the Australian environment – and, possibly, other developed nation contexts – they are by no means universal. A quite different scenario/strategy system arises, for example, where managers of urban catchments set out to achieve correspondence of complete runoff hydrographs (**all** ARIs), not just peak flows and runoff volumes observed in pre-development catchments for a fixed critical storm duration and fixed ARI (as above). The need for this degree of streamflow control springs from a desire to re-establish stream conditions conducive to the annual cycle of spawning undertaken by salmon in Canadian and US Pacific North-west streams, seriously disturbed by catchment urbanisation. This approach is the goal of "Stormwater Planning : A Guidebook for British Columbia" (Stephens et al, 2002; see also Gilliard, 2004; Nehrke and Roesner, 2004; CIRIA, 2001)].

4.2.2 A fourth strategy for stormwater management

The three strategies for managing storm runoff described in Section 4.2.1 are proposed for use by municipal or catchment-wide water management agencies to control development/re-development requiring site-by-site (agency) approval as it occurs. A **fourth** strategy is also evident and widely employed in the Australian municipal context.

This is invoked where storm runoff from a general urban landscape is directed to an **end-of-pipe** stormwater quality improvement system prior to discharge into an environmentally-sensitive receiving water domain. Unlike the strategies associated with the listed scenarios [(a), (b) and (c), above], this (fourth) strategy cannot be readily interpreted into a site-by-site policy instrument in keeping with municipal needs to control progressive urbanisation (via the approval process) as it occurs.

4.2.3 The *design storm* critical duration : $T_{C(total)}$ and $T_{C(local)}$

The location P, defined above as a reference location of 'singular interest', may be part-way along a main drainage channel – natural or lined – as shown in Figure 4.1. The task of managing flow under any of the three stream scenarios [(a), (b) and (c), above] implies use – in the analytical process – of well-understood parameters such as contributing catchment area, runoff coefficient (or some other loss model), rainfall characteristics, etc. as well as **critical storm duration** at point P, introduced in Section 1.4.2. This latter parameter is defined, for the circumstances illustrated in Figure 4.1, as the (unique) storm duration which yields **the greatest peak discharge from the (upstream) catchment at point P for an adopted ARI = Y-years.** In the notes which follow, this critical duration is called $(T_C)_{total}$.

It should be noted that a different, larger, $(T_C)_{total}$ applies to catchment discharge point O, and smaller $(T_C)_{total}$ values at any points of singular interest upstream of P. Similar reasoning may be applied to point Q within a branch or 'local' sub-catchment (see Figure 4.1) leading to the use of $(T_C)_{local}$ in later notes.

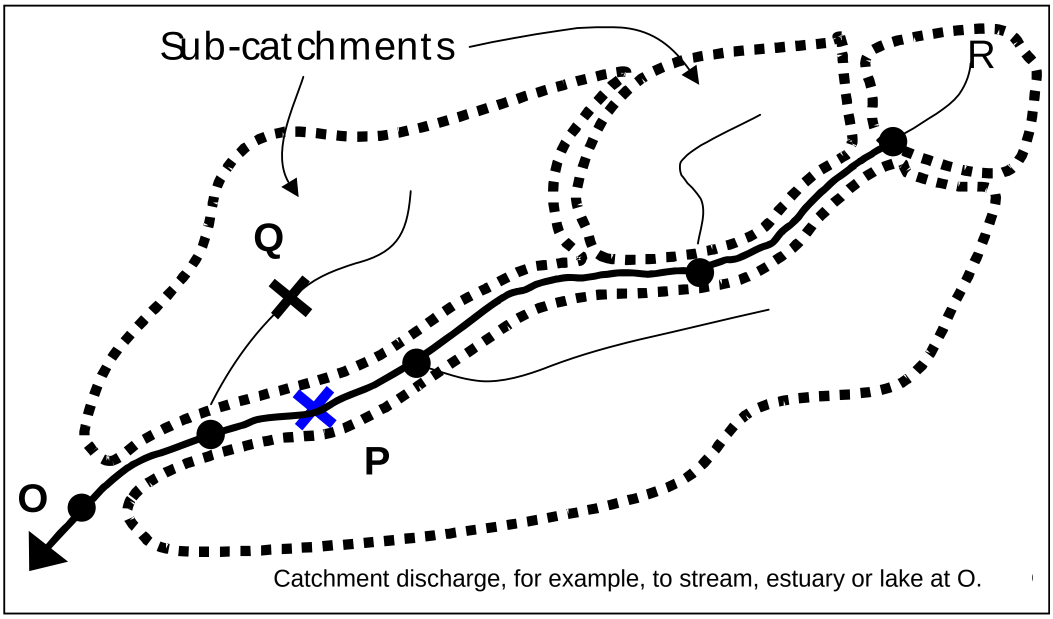


FIGURE 4.1 : Dendritic structure of a typical urban catchment

Values for $(T_C)_{total}$ and $(T_C)_{local}$, as described above, depend, of course, on the magnitudes of the respective catchments in which they are used. They are determined most effectively by applying a set of appropriate design storm patterns (ARI = Y-years) to mathematical models of their contributing catchments and finding the storm duration giving the greatest Q_{peak} at the location(s) of interest (see Section 3.4.5).

Satisfactory values of $(T_C)_{total}$ and $(T_C)_{local}$ in keeping with 'lag time' considerations (Sections 1.4.2 and 1.4.3) and the *modified* design storm approach (see Section 3.5) used in this document can be found, however, by reference to the familiar 'time of concentration' concept explained in Section 5.3 of AR&R – 1987 (IEAust, 1999). Where uncertainty exists concerning the exact location of a point of singular interest or where good stream management throughout the catchment is intended, it is recommended that the longest (catchment) travel time to discharge point O be used. This is recognised as the catchment-wide approach which leads to devices and installations whose volumes, in the short term, may seem excessive but which has the advantage of providing the basis for a comprehensive strategy covering all catchment eventualities. Practitioners with long experience in urban catchment management, particularly flood control, recommend novices to err on the side of overestimating critical storm duration : there are sound reasons behind this advice.

Applying the outcomes of this discussion to the type of problem reviewed in Section 1.2 (urban re-development) within Scenario (b), above, for example, leads to the use of the **regime-in-balance** strategy to eliminate all of the **additional runoff *volume*** generated on re-developed sites in storms of critical duration (ARI = Y-years). This opens the way for present development drainage capacity to serve the needs of re-developing urban landscapes with minimal, if any, expensive upgrades or duplication. Where re-development takes place within a catchment falling under Scenario (c), the **yield-minimum** strategy should be used to eliminate **all runoff *volume*** generated on re-developed sites, again, in storms of critical duration (ARI = Y-years). This will give relief to the overstressed (existing) drainage network; persistence with this strategy may lead, eventually, to the matching of runoff flows to present development drainage capacity. [These matters are revisited in greater depth in Section 4.6.]

It must be appreciated that design based on this approach does not guarantee zero outflow from retention-based systems (OSR) in all ARI = Y-years events. Y-years storms of duration **shorter** than 'critical' will (all) be fully contained, but **longer** (than 'critical') Y-years storms will produce overflow which must be catered for. However, such overflow, integrated with the general streamflow in these circumstances, produces channel peak flows at singular interest points such as P, Q, etc. which are **less** than Q_{peak} resulting from the Y-years storm of critical duration in the respective catchments.

4.2.4 The *design storm* in site drainage : on-site storage and the critical storm

The critical storm duration argument, above, can be extended "up" the catchment to discharge points of individual sub-areas enabling the potential for flooding at these points, as a consequence of storms of duration equal to **site time of concentration, t_C,** to be considered. Comparison of this potential threat against the economic and social consequences of flooding at, say, points P, Q, etc. in the complete catchment (see Figure 4.1), however, is likely to overwhelmingly favour the latter (i.e. P, Q, etc.) as the location of dominant concern in typical urban catchments. It follows that site drainage **storage devices** required under OSR (or OSD) policy applications, should be designed for $(T_C)_{total}$ or $(T_C)_{local}$ storm duration and **not** t_C. There is widespread misunderstanding about this, both nationally and internationally (Argue and Pezzaniti, 2004; Argue, 2005; Emerson et al, 2005).

While the use of $(T_C)_{total}$ or $(T_C)_{local}$, as appropriate, in the design of on-site storage devices satisfying catchment-wide flood control objectives is indisputable, use of these parameters to design **all** drainage elements on the site itself can lead to system failure and the potential for serious (site) flood damage. A clear distinction must therefore be drawn between design of site storages, as reviewed above, and design of the **entry** works leading to the storages and **exit** works leading from them (see Figures 1.3 and 1.5). Design of these components involves not only considerations of critical storm duration but also average recurrence interval, ARI. This issue is addressed in Section 4.2.6.

There are two unusual situations found in urban drainage systems which may be seen as departures from the theory presented above. They occur where surface runoff generated in isolated catchment elements passes out of the urban landscape without impacting in any way on neighbouring elements. Examples of the two circumstances are where controlled discharge from a relatively small catchment passes directly to :

- a waterway of great capacity – perhaps a tidal estuary, a lake or deeply incised channel,

OR

- a natural or purpose-built **sump** with zero overflow in all practical storm runoff circumstances. This is typically a groundwater recharge domain with large available storage and/or significant capability for into-ground water disposal.

Critical storm duration in the first case equals 'time of concentration', $\mathbf{t_C}$; marina developments are examples of this. But in the second case, it is incorrect to assume that the 'sump option' with critical storm duration, t_C , and ARI, Y = 100 years, covers "…all practical storm runoff circumstances", quoted above. A true 'sump' must be capable of disposing of, easily, runoff from the contributing micro-catchment **in the full range of storm events** – durations 10 minutes to 72 hours for ARI, Y = 100-years. Equations 5.7 to 5.9, inclusive, may be used to design such systems.

In summary, then, it is clear that where volume-dependent devices are to be designed as components of a catchment-wide stormwater management strategy, three values of critical storm duration have currency : $\mathbf{(T_C)_{total}}$ or $\mathbf{(T_C)_{local}}$ for the great bulk of on-site storage cases and $\mathbf{t_C}$ or site 'time of concentration' for the smaller number of applications reviewed above. The sump case requires the full range of storm durations to be considered at the level of ARI, Y = 100-years.

The aspects of critical storm duration explored in the present and previous sections cover the great majority of urban catchment storm runoff cases encountered in normal practice (see the final four paragraphs of Section 4.2.6). Special circumstances do occur, however, in 'top end' portions of large catchments which can justify the use of smaller and therefore less costly OSR installations. Theory explaining this departure from mainstream practice is presented in the following sub-section.

4.2.5 'Top end' site drainage

The considerations reviewed above relating to the determination of critical storm duration in 'total', 'local' and individual site situations are well-recognised and consistent with established hydrological design practice. The particular circumstances caused by the retention of runoff on **top end** sites in a design storm event, however, have important consequences which are additional to what has gone before. This novelty accounts for it being explored here in somewhat greater depth.

It is a frequent experience that basin-catchments of the type illustrated in Figure 4.1 are urbanised from their lower reaches (or 'bottom lands') upstream, leaving sub-catchments such as **R** till last. In the process, development encroaching close to or even into the floodplain occurs at low- or mid-catchment points, or a waterway crossing is constructed that is seriously restrictive when considered in the light of later, upstream development.

If the theory governing critical storm duration as explained above is applied, then design of components in sub-catchment R, where they are dominated by flooding concern at point P (see Figure 4.2), must be based on catchment travel time to P. This leads to OSR structures – "leaky" devices such as gravel-filled trenches, etc. – in sub-catchment R that must store the entire surface runoff in a critical design storm of, perhaps, long duration. The following theory shows this to be **unnecessary** and, further, that compliance with the criterion to preserve Q_{peak} at point P (or even lower it) with catchment development as it takes place, can be achieved with OSR structures of much lesser dimensions. The economic advantages of this approach are obvious.

It should be noted that the approach reviewed here is consistent with and useful in managing catchments falling into the (a) scenario-strategy, above (**yield-maximum** strategy). **However, it violates the principle of *before-and-after* runoff *volume* equality which is fundamental to the <u>regime-in-balance</u> strategy: examination of Figures 4.3(b) and (c) reveals why this is so. The approach is even less applicable in catchments where the <u>yield-minimum</u> strategy is employed to counteract the consequences of previous over-development.**

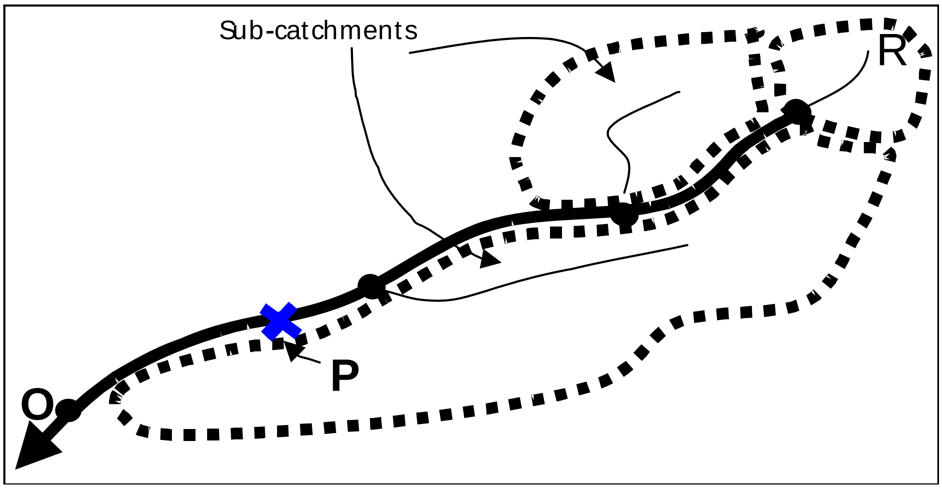


FIGURE 4.2 : Consideration of top-end sub-catchment R and those parts of the total catchment draining to P

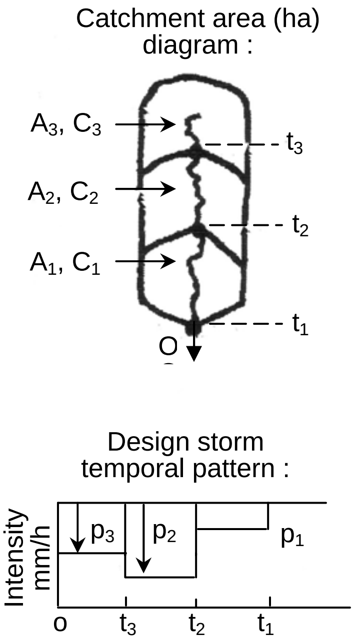


FIGURE 4.3(a)

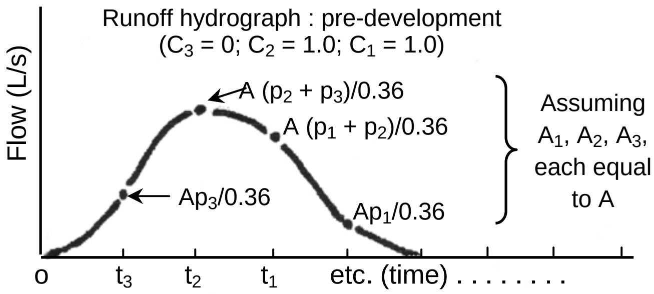


FIGURE 4.3(b)

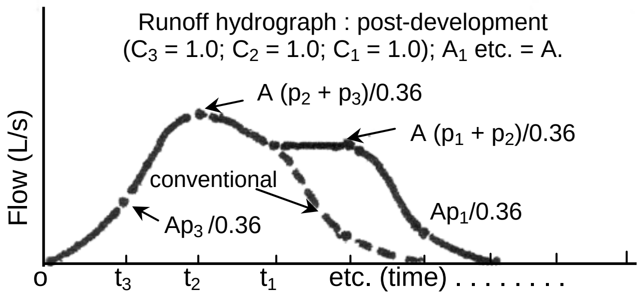


FIGURE 4.3(c)

FIGURE 4.3 : Illustrations for hydrological analysis of 'top-end' sub-catchments

Consider the sub-catchments shown in Figure 4.3a having catchment travel times as follows :

Sub-catchment 3 ('top end') :area A_3 ha : internal travel time t_3 hours

Sub-catchment 2 ('middle' sub-area) : area A_2 ha : internal travel time ($t_2 – t_3$) hours

Sub-catchment 1 ('bottom end') : area A_1 ha : internal travel time ($t_1 – t_2$) hours

Catchment **overall** travel time to O is t_1.

[Values for t_3 , t_2 and t_1 are determined by application (three times) of the methods referred to in Section 4.2.3, rather than – simply – flow travel time differences along the main drainage path.]

Runoff coefficients :

Area A_3, initially, $C_3 = 0$ (undeveloped);

Area A_2, fully developed, $C_2 = 1.0$;

Area A_1, fully developed, $C_1 = 1.0$.

Design storm temporal pattern :

$$\left.\begin{array}{l}\text{Time } 0 \rightarrow t_3 : \\ \text{Time } t_3 \rightarrow t_2 : \\ \text{Time } t_2 \rightarrow t_1 :\end{array}\right\} \begin{array}{l} p_3 \text{ mm/h} \\ p_2 \text{ mm/h typical pattern, } p_2 > p_3 > p_1 \\ p_1 \text{ mm/h}\end{array}$$

This leads to a runoff hydrograph (see Figure 4.3b) for which :

$$\begin{aligned} Q_{peak} &= (C_1\, A_1\, p_2 + C_2\, A_2\, p_3)/0.36 \text{ L/s} \\ &= A\,(p_2 + p_3)/0.36 \text{ L/s} \end{aligned} \qquad \textbf{(4.1)}$$

if areas A_1, A_2 and A_3 are each considered equal to A ha.

With development of area A_3 (new runoff coefficient $C_3 = 1.0$) and if **full retention of only the runoff occurring in the first segment of the storm on area A_3, p_3 mm/h for time t_3 hours, is assumed** (stored volume = 10 A_3 p_3 t_3 m³), then the resulting hydrograph also has a peak value equal to A ($p_2 + p_3$)/0.36 L/s. The hydrograph includes a plateau of flow equal to A ($p_1 + p_2$)/0.36 L/s (see Figure 4.3c).

The importance of this conclusion for design of OSR facilities in sub-catchment A_3 is that if normal theory (see the Procedure 2 cases, Section 5.1.3) were followed and containment of the entire critical design storm runoff (duration t_1 hours) were provided, the required volume would be 10 A_3 ($p_3 + p_2 + p_1$) t_3 m³. Provision of this storage volume in sub-catchment A_3 would, of course, give rise to significantly smaller runoff *volume* at O (see Figure 4.3c) than would occur with the 'first segment' approach described above, but the **peak flow rates** of both hydrographs (at O) would be, in fact, the same.

It should be noted that, in terms of the quantities introduced earlier in this Section : duration t_3 corresponds to 'time of concentration' of area A_3; and, duration t_1 corresponds to $(T_c)_{local}$ or $(T_c)_{total}$, depending on the extent, within the catchment, to which the 'first segment' approach is being applied. It should, further, be noted that the equality of segments assumed in Eqn. (4.1) represents a limiting case for satisfying equality of pre-development and post-development Q_{peak} flows at O. If $A_3 < A_2 < A_1$, then the post-development case shows a **lower** Q_{peak} which is a favourable outcome; if $A_3 > A_2 > A_1$ (an unusual catchment geometry), the opposite is true and the 'first segment' approach should **not** be used, because it leads to a **greater** Q_{peak} with development.

The same theory can be applied to 'second segment' cases but favourable outcomes in terms of pre- and post-development Q_{peak} flows at O are restricted to special circumstances requiring particular geometrical relationships between A_1, A_2 and A_3 as well as particular hyetograph temporal pattern distributions. The designer is advised, therefore, to either explore circumstances of 'second segment' development with great care to confirm peak flow equality/reduction, or employ conventional theory (Section 4.2.3) to base full retention storages on critical storm durations of $(T_C)_{total}$, $(T_C)_{local}$ or t_C, as apply.

Design procedures which apply to the basic Category 1 installations, are set out in Chapter 5 under four main headings :

- infiltration or 'treatment' surfaces;
- "leaky" devices;
- infiltration or 'dry' ponds;
- infiltration devices with slow-release capability.

4.2.6 Average recurrence interval (ARI) and installation components

The question : "what ARI should be employed in a given set of design circumstances?", is rarely, if ever, answered to the satisfaction of all concerned parties. Argue (1986) addressed this issue and offered a hierarchy of ARI and AEP (annual exceedance probability) values, applied to a wide range of land uses and community infrastructure components. This has been reproduced (modified) in Table 4.1.

In the earlier discussion, above, ARI = Y-years has been used in connection with the 'defined flood plain' and also in relation to the dimensions of on-site retention devices (Section 4.2.3). This should not be understood to mean equality of ARI value in the two cases quoted. Drawing together matters raised in the previous three sub-sections with those of ARI leads to the following summary (refer to Figures 1.3 and 1.5):

On-site entry works (Q_{peak}) : Most common applications (OSR devices as components of the minor drainage network) involve the use of design storm duration equal to t_C and ARI = Y-years, where "Y" is a value acceptable for minor system drainage in the range 2-years to 10-years inclusive. There are three vital elements of design that must be recognised with these (entry) systems :

- provision for bypass in events yielding runoff exceeding the entry works' capacity. In these circumstances, accommodation for bypass of the 'gap flow' – the **difference** between the rare event flow (ARI, Y = 50-years, etc.) and that from the minor storm event (ARI, Y = 2-years, etc.) – must be provided. Storm duration equal to t_C should be used exclusively in **both** of these calculations;
- provision for conveying runoff generated in rare storms (ARI, Y = 50- or 100-years, etc.) of long duration [critical storm duration – $(T_C)_{total}$, $(T_C)_{local}$ etc.] in some stormwater management strategies (see Section 4.6). This requirement is explained in more detail below;
- a value of ARI, Y = 0.25-years is recommended for 'entry' sediment control units or gross pollution traps receiving runoff from ground-level paved surfaces. This aspect is discussed under Category 2 installations, Section 4.3. Storm duration equal to t_c should be used exclusively in these calculations.

On-site storage devices and installations (runoff volume) : Use design storm of duration $(T_C)_{total}$, $(T_C)_{local}$ or t_C as appropriate, and ARI = Y-years, where "Y" is consistent with the strategic approach adopted as municipal agency policy (see Section 4.6). ARI may therefore be in the minor system range (2- to 10-years), or Y = 50- to 100-years. Design of these components is examined in considerable detail in Chapter 5. Take careful note of Sections 4.2.4 and 4.6.1.

On-site overflow systems (Q_{peak}) : The maximum possible flow which these components are ever required to discharge is that determined as $\mathbf{Q_{peak}}$ for the entry works. It is therefore recommended that the same value be also adopted for exit works. This is a conservative but, nevertheless, acceptable solution to the design problem.

The range of ARI values included in this outline covers the entire spectrum of Australian practice with a design difficulty arising, apparently, in relation to different ARIs and different design storm durations applied to entry works, above. How can an on-site storage device with entry flows restricted to minor system limits (ARI, Y = 5-years, for example) possibly receive major system runoff flows (ARI, Y = 100-years) where this is required by Council policy?

The answer to this question comes from recognition of the significant differences in rainfall intensities, and therefore runoff peak flows, which occur between storms of duration t_C – used in entry works design – and those of duration $(T_C)_{total}$ or $(T_C)_{local}$ which determine storage requirements. For example, in Newcastle NSW, the t_C = 10-minutes ARI = 5-years storm intensity which might be used to design entry works, is 109 mm/h giving Q_{peak} = 6.0 L/s for a 200 m^2 domestic roof. The peak flow entering a device receiving runoff from the same roof in the $(T_C)_{total}$ = 60-minutes, ARI = 100-years event is 4.2 L/s, based on rainfall intensity, i = 75 mm/h. So the minor system design governs.

However, this is not always the case and the **greater** of the two calculated peak flows should be used for entry system dimensioning. Note that in the Newcastle, NSW, area, the 10-minute ARI, Y = 5-years storm intensity is smaller than that of the 20-minute ARI, Y = 100-years event (see Appendix B). Thus, if site drainage works (t_C = 10 minutes) were required for a development in Newcastle in a sub-catchment for which $(T_C)_{local}$ = 20 minutes, then the entry system for the works should be designed for the **major** system Q_{peak} with ARI, Y = 100-years.

There is a further requirement of on-site drainage system behaviour which may sometimes be expected of OSR systems. This relates to performance **in all storms up to and including those of ARI = Y-years frequency**. At first sight this might appear to be covered by what has gone before, which is not the case. The difference lies in a change of focus from critical storm duration (ARI = Y-years) being central to the design process, to, frequency ARI = Y-years being central, untied to any particular storm duration.

The great majority of simple OSR systems such as those illustrated in Figures 1.3, 1.4 and 1.5, are unlikely to enter the domain under discussion here. But on-site retention installations which are integral with water features or have close interaction with the public, may well do so. Examples of such facilities include infiltration or 'dry' ponds which in normal times are used as open space recreation reserves, isolated roadway reserve swales, grassed hard-standing areas for motor vehicles, and sports fields integrated with OSR sub-structures.

Those responsible for financing and maintaining such installations – municipal, cultural, educational, commercial, etc. agencies – understand 'failure' with a recurrence interval of Y-years to mean "flooding, on average, once, **only**, in every Y-years." To achieve this standard of protection requires design to take account of **all** storm durations, not just those which are critical in certain catchment circumstances. [The **sump** discussed in Section 4.2.4 represents the limit of this case with ARI, Y = 100-years.]

Strictly, the class of problem discussed in these latter paragraphs lies not in Category 1 but, rather, in Category 4 because of its linking of two aspects of stormwater management – flood control and amenity. An illustration of this aspect of drainage design is presented in Example 5.5, Section 5.2.

TABLE 4.1
A LAND-USE/FLOOD FREQUENCY HIERARCHY FOR AN URBAN COMMUNITY
(after Argue, 1986)

FLOOD SECURITY LEVEL	CLASSIFICATION	DESCRIPTION OF COMPONENTS	DESIGN FLOOD FREQUENCY
VERY HIGH	Strategic I	Floor levels of hospitals, Civil Defence HQ	Design AEP 1 in 500
	Strategic II	Floor levels of police, ambulance and fire stations; water supply and wastewater treatment centres; electric power and gas supply stations. Floor levels of convalescent homes and community buildings which could operate as dormitory centres in great flood events.	Design AEP 1 in 200
HIGH	Dormitory I Dormitory II P/C/I* I	Floor levels of high density residential. Floor levels of low-medium density residential. Floor levels of essential food, pharmaceutical, retail and department stores; centres employing large labour force; community administration and education centres; centres for storage of rare artefacts; venues for entertainment, dining or popular indoor sports.	Design ARI 100-years
	P/C/I* II	Floor levels of factories and outlets supplying non-essential items; premises of businesses and institutions which involve small numbers of people; premises of sport or community activities infrequently used.	Design ARI 50-years
LOW	Open space I (and arterial roads)	Frontages of all units belonging to "very high" priority above; outdoor areas where rare artefacts are displayed or stored.	Design ARI 5 – 10 years
	Open space II (and local roads)	Frontages of all units belonging to "high" priority, above.	Design ARI 3 – 5 years
	Open space III	Other open space areas including general parks and outdoor recreation areas.	Design ARI 1 – 3 years

* Public/Commercial/Industrial

4.3 CATEGORY 2 SYSTEMS

4.3.1 Strategy A and strategy B

Pollution control in the urban landscape is normally implemented as a highly focussed activity using devices and systems ranging from trash racks and sedimentation basins in urban waterways to gross pollution traps (GPTs) used in pipe networks and minor drainage paths. These installations undoubtedly have a valid role to play in reducing gross pollution generated in medium to large catchments, but they do not – as stand-alone installations – provide the best option, necessarily, for stormwater 'source control' in urban design that is water-sensitive. There are two reasons for this :

1. The sediment-removing capabilities of GPTs do not extend, typically, to the fine sediments (size 100 μm and smaller) which cause greatest damage to aquatic (river and marine) ecosystems (Walker et al, 1997; Weisner et al, 1998). Further, they do not treat dissolved pollution.

2. GPTs play no part in retaining storm runoff and, thereby, contributing to flood control objectives.

It is possible – within the realm of 'source control' – to devise systems which satisfy **both** of these objections, however the issues raised by such design solutions are complex : they are reviewed in Sections 7.1 and 7.6. Suffice it to say at this point that systems which retain high levels of water-borne urban pollution, including fine sediments, and which also retain/dispose of cleansed storm runoff on site represent valuable elements of any WSUD strategy : such elements fall into **Category 2**. They invariably incorporate two components – some form of filter/treatment system as well as capacity for storing cleansed storm runoff and disposing of it without contributing materially to downstream flooding. The two components are illustrated in Figure 3.5a : the most satisfactory method for designing them – as explained in Section 3.4 – is through the medium of 'continuous simulation' modelling.

It is well-recognised (NSW Dept of Housing, 1998; CSIRO, 1999; Wong et al, 2000) that at least 95% of the total runoff (volume) generated annually – on average – in an urban catchment arises from flows equal to or smaller than the flood resulting from the storm of critical duration in a catchment and ARI, Y = 0.25-years. Now, if it were assumed that all storm runoff, regardless of flood magnitude, carries the same pollution concentration, and, if it were further assumed that an installation or device 'matched' to the above limit is in place at the discharge point of a catchment and that it is effective in treating all flow passing through it, then it would be possible to guarantee at least 95% pollution control for the total (annual average) flow passing from the contributing catchment.

The assumption, above, relating to pollution concentration uniformity is erroneous : the true situation is for pollution concentrations in the larger flood flows to be usually **lower** (there are exceptions!) than those in small-medium flows, leading to a generally better that 95% performance. This consequence is further strengthened when it is recognised that much of the 'first flush' of pollution conveyed in runoff generated in greater-than-0.25-year events, is also treated. It is quite possible, in certain circumstances, for the 'first flush' of the 20-years or 50-years or even 100-years flood runoff from a small site to be treated in this way.

These considerations turn the suggested target of 95% pollution removal using on-site filtration at the discharge point of a catchment into an entirely feasible possibility. The linking of this goal with that of a modest contribution to flood-control by on-site retention (see above) of cleansed stormwater, is an objective well aligned to the principles of WSUD.

An **alternative** approach, but one which delivers an equally acceptable outcome, is that presented in Table 3.2 ("leaky" devices). In this case, bypass and overflow, **together** account for 10%, 20%, 30% and 40%, etc respectively, of catchment flow entering the system. Treatment of this flow **at the 90% retention level**, that is 10% (bypass + overflow), corresponds approximately to the 95% pollution removal level discussed above. This is the preferred *approach* of current Australian practice (Engineers Australia, 2005)

Both of the approaches described above are referred to as **Strategy A** pollution control in later sections of the Handbook.

Of course, the resources needed to achieve these objectives, in terms of required storage volume, may exceed the space available, or the cost of the required works may be beyond the means of agencies wishing to use the technology. In such cases, a second best option should be considered which amounts to using a $Q_{0.25}$ entry filter matched to the site critical storm duration, t_C **,** followed by storage capacity sufficient to contain **the volume of the 'first flush' produced in the storm of critical duration, only.** This volume is much smaller than that required for Strategy A compliance. All runoff flows greater than $Q_{0.25}$ as well as all later flows (following the 'first flush') with their lesser pollution concentrations, would be bypassed.

The goal achieved in these circumstances is referred to as **Strategy B** in later sections of the Handbook and, although a significantly lower standard of pollution containment than Strategy A, is, nevertheless, an improvement on many current practices.

Aquifer access or slow-drainage provision

Storages required as the Strategy A outcome from the Table 3.2 "leaky" devices analysis, referred to above – and, to a much lesser extent, Strategy B cases – may be greater in plan area extent than the space available for them. [In this connection it should be noted that removal of **retained**, i.e. stored water, in the "leaky" devices analysis (Table 3.2), is entirely by 'natural' means (percolation) only.] In these circumstances, a steady release of some stored water throughout the filling stage, can lead to a decrease in the size (plan area) of the installation sufficient to achieve a satisfactory 'fit' in most cases.

The basis for modifying the initial design (given by the Table 3.2 analysis) is the same as that used to *modify* the 'design storm' method, namely, 'hydraulic' means of stored water release into an accessible aquifer, or by slow-drainage (see Section 3.5). This process is explained in Section 7.3.

Design procedures for Category 2 installations are set out in Chapter 7 under three main headings :

- Simple pollution control (Strategy A and Strategy B) systems.
- More complex pollution control (Strategy A and Strategy B) systems.
- 'Filter strip' swale systems.

CATEGORY 3 SYSTEMS

The primary difference between the analyses which are used in the design of Category 1 and Category 2 systems and those of Category 3, relates to their data bases. The design process which produces Category 3 installations typically involves the use of historical volumetric data which may take the form of rainfall records (design of rainwater tanks) or monthly streamflows (water amenity installations) or annual rainfall (ASR schemes). These data, in each case, provide information on the **resource available**.

How the resource is used provides the other main ingredient in the design process. In the case of domestic rainwater tanks, this may be determined as a daily quantity but its magnitude will vary according to household membership and the range of uses to which the resource is put : with these matters resolved, use is also observed to vary diurnally and, possibly, seasonally. Design, taking all these variables into account, therefore, requires a trial-and-error approach to be taken to arrive at a final 'match' between water availability and use. For example, roof areas of arid region domestic dwellings may only be capable of collecting enough rainwater for household toilet flushing, whereas in a tropical location, raintank water may meet all domestic demand for toilet flushing, hot water and clothes washing and, possibly, some outdoor irrigation (see Chapter 8).

The same principles may be employed in fixing storage volumes for commercial/industrial plant using rainwater for processes such as cooling, slurry formation, washing or irrigation. Again, the feasibility of matching the various demands to the available resource can only be established by trial-and-error procedures using long-term rainfall data.

Similar procedures to those referred to above are employed to establish storage volumes for water amenity/recreation ponds and basins. In these cases, unlike rainwater tanks, account must be taken of various losses to the natural environment, in particular, evaporation from the water surface, leakage and 'release policy' to provide downstream environmental flows. In these cases direct use may be small compared to losses : these analyses may involve the use of monthly or daily inflow data.

The third broad category of stormwater harvesting subsumed under Category 3 is that of aquifer storage and recovery (ASR) schemes. Compared with rainwater tank and amenity basin dimensioning, these systems are 'coarse' : the reason for this is their dependence on aquifer hydraulic processes which respond to water input/output stimuli much more slowly than surface water systems. The data base required for the design of ASR systems is little more than annual rainfall.

Harvesting which occurs as a by-product of on-site stormwater retention, for example "passive irrigation" in Example 5.3, Section 5.2, is **not** addressed in Chapter 8. The reasons for this are the secondary nature of such systems among retention practices, and the difficulties they pose in terms of analysis and quantification.

Design of 'pure' Category 3 systems is reviewed in Chapter 8, where consideration is given to harvesting of roof runoff : a secondary objective – but an important one – is the beneficial effect raintank presence in an urban catchment can have on local storm runoff peak flows.

CATEGORY 4 SYSTEMS

No additional types of analyses beyond those referred to in Sections 4.2, 4.3 and 4.4 are required to design devices and systems falling into Category 4 (see Section 4.1). This includes installations "….. with multi-purpose, approximately equal functions drawn from the fields of flood control, pollution containment, stormwater harvesting and amenity". This gives rise to four different combination possibilities :

1. flood control and pollution containment (Problem 1);
2. flood control and stormwater harvesting (Problem 2);
3. pollution containment and stormwater harvesting (Problem 3);
4. flood control, pollution containment and stormwater harvesting (Problem 4).

It may be concluded from Sections 4.2 – 4.4, inclusive, that solutions to these problems of overlap may be solved, typically, as follows :

Problem 1 : Determine a design for ARI = Y_{flood} using the flood control procedures in Section 5.1, hence volume $\forall_{\mathbf{f}}$; determine a pollution control storage volume, $\forall_{\mathbf{p}}$ (90% retention) from Chapter 7; compare the capacity of storage given by the flood control design, $\forall_{\mathbf{f}}$, with that required for pollution control, $\forall_{\mathbf{p}}$. If $\forall_{\mathbf{f}} > \forall_{\mathbf{p}}$ – which is usually the case (see Section 7.6.2) – accept design without storage modification; if $\forall_{\mathbf{p}} > \forall_{\mathbf{f}}$, increase capacity of flood control design to give storage required by $\forall_{\mathbf{p}}$. [The design must include a required treatment train to ensure that pollution content up to the level of ARI, Y = 0.25-year events, at least, can be contained including overflow.]

Problem 2 : The use of rain tank storage to achieve the joint objectives of *harvesting* and *flood control* raises complex issues which are reviewed in Section 8.7.

Problem 3 : Roof runoff is recognised as the cleanest stormwater source available in the urban landscape. Control of its pollution load – meeting the highest standards set in Chapter 7 – is achieved through the process of collection (including provision for bypass and 'first flush' treatment) and storage with use (refer to Chapter 8). The problem of pollution conveyed in general surface runoff is addressed in Chapter 2 and re-visited in Chapter 7.

Problem 4 : Solutions to problems involving all three domains of stormwater management are highly complex and require first contact management ('collection' as in Problem 3, above, or bypass and a treatment train) followed by storage (with use) and, possibly, provision for overflow. Each of the projects described in Section 3.9 represents a solution to 'Problem 4' situations. Designs of some elements of these projects can be found in Sections 5.2 and 7.7.

COUNCIL RESPONSIBILITIES AND STRATEGIC PLANNING

4.6.1 The four networks

Responsibility for policy decisions relating to the raft of issues reviewed in the foregoing sections rests firmly with councils or municipal agencies having oversight of the catchment-wide planning process. There are, broadly, four sets of catchment network conditions in which councils are required to make decisions about future development/re-development in their areas of jurisdiction. These are :

- ***High-capacity* major/minor networks :** These are associated with catchment-wide street drainage installations capable of satisfactorily controlling minor stormwater flows (2-years to 10-years, as appropriate), located within a matrix of well-defined overland flow paths providing acceptable freeboard to floors of residential and business premises in all flood flows up to and including the average recurrence interval (ARI), Y = 100-years critical storm event. The capabilities of both networks are such that they are unlikely to suffer loss of integrity in the face of future development/re-development provided it is well ordered and falls within the requirements of the governing local agency.

  These networks represent 'model' systems for designers of new developments and re-developments. It is suggested that stormwater flows in such catchments and sub-catchments could be well managed using the **yield-maximum** strategy in selected, small sub-catchments with potential for catchment-based stormwater harvesting if desired, along with the **regime-in-balance** strategy applied generally (see Section 4.2.1). The suggested ARI used for design of future on-site water-retaining devices in this case should be in the range 50-years to 100-years.

- ***Medium-capacity* major/minor networks :** These are associated with catchment-wide or sub-catchment-wide drainage installations incapable of satisfactorily controlling minor stormwater flows, located within a matrix of well-defined overland flow paths providing acceptable freeboard to floors of residential and business premises in all flood flows up to and including the ARI, Y = 100-years critical storm event. The minor system network is likely to provide an even lower level of flood protection as development/re-development proceeds unless it is well ordered and controlled by the municipal agency. The major system network is sufficiently robust to maintain a high standard of performance in the face of future development/re-development provided it is well ordered and falls within the municipal agency policies and requirements.

  It is suggested that stormwater flows in such catchments and sub-catchments could be well managed using, principally, the **yield-minimum** strategy focussed on the minor system. The suggested ARI used for design of on-site water-retaining devices in this case could be in the range 2-years to 10-years, as appropriate. [This process will lead to a reduction in the standard of major system flood protection towards ARI, Y = 50-years.] In time, it may be found that the performance of the minor system, following this strategy, improves significantly enabling the network to be upgraded to **high-capacity** status with corresponding strategy change to that of **regime-in-balance** with ARI, Y = 50-years to 100-years.

- ***Low-capacity* major/minor networks :** These are associated with catchment-wide or sub-catchment-wide installations capable of satisfactorily controlling minor stormwater flows, located within a matrix of poorly-defined overland flow paths resulting in progressively more serious flooding of floors of residential and business premises in all flood flows above the ARI, Y = 10-years critical storm event. The minor system network is sufficiently robust to maintain a satisfactory level of performance in the face of future development/re-development provided it is well ordered and falls within municipal agency requirements. The major system network is likely to deteriorate even further in performance as development/re-development proceeds leading to increased flooding of floors of residential and business premises in rare storm events unless it is significantly upgraded and controlled by the municipal agency policies and requirements.

  It is suggested that the local government agency, in these circumstances, should institute structural changes to the urban landscape including property resumption/purchase if necessary to provide space for overland flow paths, coupled with application of the **yield-minimum** strategy with suggested ARI, Y = 100-years for design of all on-site water-retaining devices.

[This process will lead to an increase in the standard of nuisance flood protection offered by the minor system towards or even above the ARI, Y = 10-years level.] In time, it may be found that the performance of the major system improves significantly enabling the network to be upgraded to **medium-capacity** status with corresponding **yield-minimum** ARI change from 100-years to 2 – 10-years for all development/re-development. Subsequent status change to that of **regime-in-balance** with ARI = 50-years to 100-years may follow.

- ***Poor-capacity* major/minor networks :** These are associated with catchment-wide or sub-catchment-wide installations incapable of satisfactorily controlling minor stormwater flows, located within a matrix of poorly-defined overland flow paths resulting in progressively more serious flooding of floors of residential and business premises in all flood flows above the ARI, Y = 10-years critical storm event. The capabilities of both networks are likely to deteriorate even further in the face of any future development/re-development resulting in increased nuisance flooding and increased flooding of floors of residential and business premises unless both networks are significantly upgraded and controlled by municipal agency requirements.

  It is suggested in this case that the local government agency undertakes a comprehensive review of the entire storm drainage system and takes action, including property resumption/purchase if necessary to provide space for overland flow paths. This action should be accompanied by application of the **yield-minimum** strategy with suggested ARI, Y = 100-years for design of all on-site water-retaining devices. In time, it may be found that the performances of both the major and minor networks, following this strategy, has improved significantly enabling the entire system to be upgraded to **medium-capacity** status with corresponding **yield-minimum** ARI change from 100-years to 2 – 10-years for all development/re-development. Subsequent status change to that of **regime-in-balance** with ARI = 50-years to 100-years may follow.

4.6.2 The four networks : management issues and ARIs

The four networks of flood management described above represent the full range of relationships possible between major and minor stormwater flow scenarios in the urban landscape. The history of flooding and/or the environmental performance (for example, with respect to channel erosion) of particular networks in significant storms of critical or near-critical duration will provide the data base needed by a municipal agency to divide catchments and sub-catchments into the four classifications.

With this determined, the local government agency can then institute a coherent strategy of management to maintain the status of well-performing networks and progressively upgrade those which are deficient. This (latter) action can be taken, not necessarily through expensive infrastructure upgrades using scarce community resources, but through policy instruments and the approval process. This should certainly be the case for networks described as “medium-capacity” and, possibly, for some classified as “low capacity”. Networks classified as “poor-capacity” (and some of “low-capacity”) are likely to need a concentration of available resources to pay for significant structural (including property resumption/purchase) improvement of the stormwater infrastructure. However, the classifications presented in Section 4.6.1 should aid this process.

Experienced practitioners may be surprised by the ARIs suggested for design of new installations (development and re-development) which appear to be outside minor system practice (typically, 2 – 10 years). For example, some **regime-in-balance** strategy applications may require on-site storages to be sized to ARI, Y = 100-years [**after** development *minus* **before** development (runoff *volumes*)]. These lead, usually, to storage volumes **less** than those given by the **yield-minimum** strategy with ARI, Y = 10-years.

4.6.3 The ‘benchmark year’ concept

A municipal agency responsible for managing the water resources and environmental qualities within a defined basin may employ the strategies outlined above as follows :

STEP 1 : **Classify** each catchment and sub-catchment within its jurisdiction according to the four drainage network categories described in Section 4.6.1. Acceptable average recurrence interval (ARI) and **critical storm duration** for each catchment or sub-catchment emerges from this process;

STEP 2 : **Identify** which of the three development/re-development strategies described in Section 4.2.1 should be applied to each catchment or sub-catchment. In cases where the overland flow path is grossly inadequate ('low-' and 'poor-capacity' networks), then separate strategies of resumption/purchase of properties should be applied.

It is simplest in terms of data acquisition and model validation etc., to conduct the analyses required under STEP 1 (see Section 4.6.4, following) to catchments and sub-catchments in their present states of development. But they (the analyses) need not be so constrained. It is quite reasonable – but more difficult – to adopt a stage of basin development earlier than 'present', at a time when the impact of development in terms of flooding or stream erosion in the critical (duration) storm is considered, retrospectively, to be 'acceptable'. The time of this 'acceptable performance' – decided by consensus among the stakeholders – is termed the **benchmark year**.

It is important to recognise that the decision to choose the 'present' as the benchmark year fixes the time of 'acceptable performance' just as rigidly as if 1980 or 1990 were chosen. Hence, in applying **the regime-in-balance** strategy to design (see Section 4.6.5, following), the status of the site **before** development or re-development must be literally "as it was in the benchmark year", **not** the time when the development/re-development happens to be taking place.

Comparison of the outcomes of hydrological analyses carried out for 'present' development and for the benchmark year will reveal, quite dramatically, how urbanisation has impacted, particularly where the time gap is a decade or more. For example, a major/minor network of interest may be classified (retrospectively) as having medium-capacity in the benchmark year; over the intervening period (to the present) this classification may have degraded to low-capacity. The correct strategic approach to be taken in such a situation would therefore be to assess present drainage networks and environmental qualities against the behaviour/performance of corresponding infrastructure in the benchmark year, and arrive at policy decisions accordingly.

4.6.4 Summary of tasks for a stormwater management agency

Primary task

Commission a comprehensive hydrological study as well as associated community action to achieve the following aims :

- determine a "point of singular interest" in each significant drainage unit (catchment or sub-catchment) where flood risk or environmental damage (channel scour) is readily assessable. This task involves gathering together the wealth of available information – formal records as well as anecdotal information – that is available in the community;
- determine peak flood flows (and hence peak stages) for a range of frequencies (ARIs) at each nominated point of singular interest;
- establish, through consensus among community stakeholders, acceptance of a limit of flooding and/or environmental damage together with an associated frequency (ARI), applicable to each point of singular interest. A decision involving 'retrospectivity' (the 'benchmark year') is permitted;
- review and classify the major/minor drainage networks in each component catchment and sub-catchment according to the four categories described in Section 4.6.1, together with appropriate ARIs, advised by the consensus decision, above;
- identify the critical storm duration appropriate to each component catchment and sub-catchment and its associated point of singular interest.

With these analyses, records and community decision-making in hand, the municipal agency can then proceed with the following tasks :

1. Determine which of the three scenarios/strategies (a, b or c) described in Section 4.2.1 most closely fits their situations and/or what balances between them should be observed. The **type** of hybrid policy which might emerge from such considerations is :

   "yield-minimum strategy to be applied to all new housing on "greenfields" and previously undeveloped sites; **regime-in-balance** strategy to be applied to all other cases of development and re-development (see Section 4.2.1)".

2. List values of **critical storm duration** which should be applied in each catchment, $(T_C)_{total}$, and/or sub-catchment, $(T_C)_{local}$, identified in the basic hydrological investigation. As an interim measure, the familiar 'time of concentration' concept may be used, but this tends to under-estimate the critical storm duration determined by comprehensive hydrological analysis. Use of the appropriate Bransby Williams formula as explained in Sections 5.3 and 5.4 of "AR&R – 1987" (IEAust, 1987) provides simple yet acceptable estimates of $(T_C)_{total}$ or $(T_C)_{local}$.

3. Identify the values of average recurrence interval (ARI) which should be applied to drainage networks, taking account of the types of development which have taken place and are likely to take place in the area of jurisdiction. Clear policy decisions identifying council-adopted levels for both the minor and major systems should be made (see Section 4.2.6);

4. Identify acceptable emptying time criteria for application to "leaky" devices and installations temporarily storing stormwater on-site (see Table 3.3);

5. Determine policy on the two pollution containment strategies – Strategy A and/or Strategy B (see Section 4.3) – and how they should be interpreted on development/re-development sites. For example, it may be appropriate for Strategy A to be required in all development/re-development sites greater than 1.0 ha, and Strategy B in all small developments;

6. Determine if a catchment or sub-catchment-wide stormwater harvesting strategy should be followed. This requires detailed analyses of catchment characteristics linked to policies governing storm runoff management – retention or detention – as well as investigation of potential constructed wetland sites and ASR opportunities (see **yield-maximum** strategy, Section 4.2.1).

4.6.5 The regime-in-balance strategy (see Section 4.2.1)

Interpretation of the **regime-in-balance** strategy – as specified by Council – in relation to a new development or re-development involves design of the three main elements of on-site devices and installations (see Section 4.2.6) :

- entry works peak flow – based on site t_C ;
- OSR device storage volume – based on $(T_C)_{total}$,$(T_C)_{local}$ or t_C;
- exit works peak flow : for simplicity, same as entry works peak flow.

The first and third of these elements may be determined by normal Rational Method procedures or more sophisticated techniques, as preferred, applied to the appropriate (site) contributing area, using council-advised ARI, local rainfall data, etc. The second element – on-site storage – must be designed to ensure that 'before and after' (development or re-development) runoff flows from the site are as close to equal as possible. This equality of flows should be interpreted in terms of **volume** of site runoff in the Y-years storm of critical duration as determined by council, **not** peak flow. Compliance involves the designer in obtaining the following information :

DEVELOPMENT (OF "GREENFIELDS" SITE) OR	**RE-DEVELOPMENT (OF DEVELOPED SITE)***
• area of "greenfields" site, A_o	• area of developed (present) site, A_{dev}
• runoff coefficient for "greenfields" site, C_g	• runoff coefficient for developed site, C_{dev}
• runoff coefficient for "greenfields" site, C_{dev}	• runoff coefficient for re-developed site, C_{red}
• developed site 'time of concentration', t_c	• re-developed site 'time of concentration', t_c
• critical storm duration, T_C	• critical storm duration, T_C
• design average recurrence interval, Y	• design average recurrence interval, Y
• rainfall information for site location	• rainfall information for location

* "Developed site" here means : the site as it was in the benchmark year (see Section 4.6.3).

These data are then applied into the following steps to determine the 'before and after' runoff volumes and their difference(s) :

STEP 1 : Identify the storm duration, T_C, which is **critical** for flooding at the location of singular interest, P, Q etc. in the ARI = Y-years event (see Sections 4.2.1, 4.2.3, 4.2.4 and Figure 4.1).

STEP 2a : Determine for any **development** application, the quantity (volume) of stormwater discharged from the undeveloped ("greenfields") site in the ARI = Y-years design storm of duration T_C using parameters A_o, C_g, etc.

STEP 2b : Determine for any **re-development** application, the quantity (volume) of stormwater discharged from the developed (benchmark year) site in the ARI = Y-years design storm of duration T_C using parameters A_{dev}, C_{dev} , etc.

STEP 3a : Determine for any **development** application, the quantity (volume) of stormwater discharged from the developed site in the ARI = Y-years design storm of duration T_C using parameters A_o, C_{dev} , etc.

STEP 3b : Determine for any **re-development** application, the quantity (volume) of stormwater discharged from the re-developed site in the ARI = Y-years design storm of duration T_C using parameters A_{dev}, C_{red} , etc.

STEP 4a : For any **development** application, determine the **difference** in the volumes required under 2a and 3a, above : this becomes the parameter ∀.

.**STEP 4b :** For any **re-development** application, determine the **difference** in the volumes required under 2b and 3b, above : this becomes the parameter ∀.

STEP 5 : Require the developer or re-developer to provide on-site stormwater retention facilities to fully retain* **without overflow**, the quantity ∀, determined in STEPS 4a or 4b, respectively, above. Delivery of the calculated volume to the OSR installation or device takes place over a time period $\boldsymbol{\tau = (T_C + t_C)}$ [see Section 3.3].

The volume ∀ and the runoff hydrograph time base τ are the primary parameters used to design OSR installations and devices for 'source control' stormwater management (Procedures 2, 3 and 4, Section 5.1). It is presumed in the procedure listed in the five steps, above, for the **re-development** case, that the (b) scenario described in Section 4.2.1 – that of a waterway in harmony with its urban environment – applies to development in the benchmark year.

* A small outflow downstream from a retention facility *during the filling process* may occur – by design – in cases where Procedure 4 is employed (see Chapter 5).

4.6.6 The yield-minimum strategy (see Section 4.2.1)

The procedure for interpreting the **yield-minimum** strategy into design of OSR installations or devices in a changing urban landscape follows the same pattern set out for the **regime-in-balance** strategy presented in Section 4.6.5 with two important differences. These are :

- Development (of "greenfields" site) case :

  STEP 2a : $C_g = 0$ leading to a zero outcome for all cases in STEP 2a.

- Re-development (of developed site) case :

  STEP 2b : $C_{dev} = 0$ leading to a zero outcome for all cases in STEP 2b.

4.7 MIMICKING NATURAL CATCHMENT 'LOW FLOWS' IN THE URBAN LANDSCAPE

The quest for mimicking of natural catchment behaviour in an urban landscape, with important implications for environmental flows, must recognise the primacy of the volume-equality principle (as reviewed in Section 4.6.5) as a necessary *but not sufficient* condition for its achievement. An additional, required condition is that on-site stormwater retention measures carried out to achieve volume-equality do not reduce surface runoff *below the needs of environmental flows*. This threat is of major importance since it is the 'low flow' regime of environmental flows which determines the fate of aquatic bio-communities - both fauna and flora (see Smakhtin, 2001; Lee et al, 2007). It is tempting to believe that satisfaction of the volume-equality principle (the first requirement, above) embraces the 'additional' required condition. But this is not so, because the **frequency bases of the two cases are significantly different:** *flood flow* frequencies are focussed on 5-year, 10-year, 20-year etc ARIs; the *low flow regime* corresponds to peaks in the ARI range "3-months", "2-months", etc associated with 90 – 70 percentile average annual flows.

Research aimed at resolving the *tension* between the two requirements is in its infancy, but the following brief analysis based on '**low flow' mimicking** considerations is offered as a pathfinding contribution to this domain of water-sensitive urban design.

The principal difference between natural and developed catchments is, of course, the replacement of pervious land surface with paving – roofs, carriageways, car parking areas, etc. On any developed site there is – in general – an area of pervious surface and an area of paving. We can assume that the hydrological behaviour of the *pervious* component is acceptably similar in both the natural and developed catchments. It therefore remains only for surface runoff from the *impervious* areas to be managed so that the **yield** (average annual runoff volume) and ***spell* characteristics** of each area are much the same in the developed catchment as they were (from the same area) in the pre-developed state. [A 'spell', in the present context, is a period of time during which flow or successions of flows (including 'no flow' episodes) passing from a catchment component, *falls below an identified 'threshold' value* (see below).]

If a developed catchment is to mimic natural (catchment) processes and behaviour, then it must actively simulate – as closely as practicable – **four rainfall destinies** of particular interest, illustrated in Figure 1.7 –

1. events which satisfy interception loss, only, and deliver *no input to the forest floor;*
2. events which satisfy interception loss and 'take-up' by tree roots, and deliver *no input below the root zone;*
3. events which satisfy interception loss and 'take-up' by tree roots, and also deliver *a significant seepage flow* ("base flow") to the catchment waterways and to deep aquifers; and,
4. events which satisfy (1), (2) and (3), above, and deliver, in addition, *a small surface runoff*.

It is impossible to apply universal magnitudes to these levels of events because they vary so greatly between catchment types (vegetation, terrain, soils, etc) and climates across Australia. However, 'notional' values can be assigned to them in keeping with the spirit of the Figure 1.7 model as follows :

- around 20% of annual rainfall input, for item 1;
- around 40% of annual rainfall input for the *tree roots 'take-up'* component of item 2;
- around 20% of annual rainfall input for the *seepage flow* component of Item 3; and,
- surface runoff up to a 'threshold' *low flow* peak value of, say, $Q_{0.25}$, Q_1, etc for Item 4.

The types of urban catchments in which the suggested 'mimicking' should take place are not universal. Inner-city and old suburbs with well-established, conventional stormwater infrastructures are specifically excluded on the ground that any presence of aquatic bio-communities requiring 'low flows' for their survival vanished decades ago. So the scope of the present initiative is 'greenfields' developments (see Section 1.4.2) and urban catchments where rejuvenation of drainage lines is a considered option (see Section 1.4.3). In both of these, residential development is likely to be the predominant if not the only land use. Furthermore, the residential roof represents a logical 'first option' for implementing a conscious strategy of low flow mimicking available in the urban landscape. How this might be accomplished is explained as follows :

Figure 1.1b presents the layout of a dual-occupancy allotment typical of those constructed in recent time in the suburbs and 'greenfield' estates of Australian main population concentrations. This layout has been used to explore its role in the mimicking process. The principal physical and occupancy characteristics of the layout – reduced to that of a **single** dwelling unit are :

- roof area, 250 m² for which *effective* (catchment) area is 80-90% or 210 m²;
- share of fronting carriageway/verge : 75 m² (equivalent impervious area);
- impervious area runoff coefficient (annual volumetric, incorporating initial loss) : C = 0.75;
- allotment (outdoor) **permeable** paving – driveway, outdoor living area, etc. : 150 m²;
- irrigated lawn/garden area : 110 m²;
- occupancy : 2.5 persons;
- daily (in-house) water use : 140 litres per person;

Application of the 'continuous simulation' modelling software for raintank sizing listed in Section 8.6.5 to these data for Australian climate zones produces the following results :

Tropical Northern Zone: It is impractical to use rainwater tanks in Darwin to achieve the 'low flow' mimicking objectives specified above because the required tanks would need to be immense. However, the goals can be achieved in Brisbane with tanks of capacity 25 – 30 kL where in-house demand *only* is met, and 13 kL where both in-house and outdoor irrigation is supplied. These storages are, also, greater than the typical householder is likely to tolerate within the confines of a medium-density residential allotment.

Intermediate Australia Zone: It is impractical (immense storages) to use rainwater tanks in Sydney to achieve 'low flow' mimicking objectives where in-house demand *only* is met from a rainwater tank. However, a tank of 22 kL capacity will deliver the goals of mimicking provided the harvested water is used for both in-house supply and outdoor irrigation. Full mimicking can be achieved in both Perth and Canberra with 16 kL and 2.8 kL tanks, respectively, meeting in-house *only* demand, and 12.5 kL and 2.7 kL storages, respectively, where both in-house and irrigation demands are catered for. Typical householders could not be expected to install rainwater tanks of the sizes indicated for Sydney or Perth, but 'low flow' mimicking based on tanks of acceptable size is feasible in Canberra.

Southern Australia: 'Low flow' mimicking in urban catchments, meeting the objectives listed above, is entirely feasible in all of the nation's southern cities – in particular, Adelaide, Melbourne and Hobart (in addition to Canberra) – where rainwater tanks of acceptable size (2.0 – 2.5 kL) are required.

Some important conclusions may be drawn from this brief exploration of the potential for 'low flow' mimicking in 'greenfields' catchments and urban landscapes with rejuvenated waterways :

1. Mimicking objectives **can** be achieved through judicious choice of rainwater tank storages in Canberra and the southern Australian population concentrations;

2. Mimicking objectives **cannot** be achieved with domestic rainwater tanks of acceptable size in Australia's high rainfall regions including Sydney and Perth; centralised facilities such as swales and mini-wetlands are needed to achieve the objectives in these locations;

3. Rainwater tanks with installed capacity greater than 2.5 kL in southern Australia (a not uncommon practice) are likely to impact adversely on environmental flows in local remnant waterways and rejuvenated streams.