5 RUNOFF QUANTITY CONTROL (Category 1 systems)

5.1 FOUR BASIC PROCEDURES

5.1.1 Introduction

There are three basic types of devices/installations used in runoff retention practice. These are :

1. **Infiltration or 'treatment' surfaces** (receiving runoff with no ponding) : design addressed under Procedure 1,

2. **"Leaky" devices and "soakaways"** with pre-treatment : design addressed under Procedure 2; and,

3. **Ponds with infiltration** : design addressed under Procedure 3.

A further element of runoff retention practice that deserves to be included among 'basic' procedures, is that relating to **slow-release** of water retained by the latter two types of devices/installations, introduced in Section 3.5.4. This component is included as Procedure 4.

Swales and wetlands are excluded from these basic, Category 1, procedures because their primary function is pollution control rather than quantity control. [Simple swales – called 'filter strip' swales in the Handbook – are considered in Chapter 7.] No consideration is given to constructed wetlands in the Handbook on the ground that they are, typically, beyond the scope of stormwater 'source control'.

The procedures, following, for designing the three categories of devices listed above owe much to Professor Wolfgang Geiger and the German ATV (Abwassertechnische Vereinigung EV), as explained in the Preface. They are subdivided into two groups : those based on $\mathbf{Q_{peak}}$ (infiltration or treatment surfaces without ponding), and those based on **volume of surface runoff,** $\forall$ ("leaky" devices and ponds with infiltration). In all cases where volume, $\forall$, dominates the design process, the analyses which follow lead to final formulations embracing the 'device full' assumption, namely :

*At the end of the period of surface runoff to the device in the design event (τ in Figure 3.4), the entire available storage is filled in readiness for the **emptying from full** process to commence.*

In fact, due to the combined effects of soil permeability, design rainfall intensity, critical storm duration, etc., this assumption is often invalid and the 'device full' stage is reached (and some emptying has taken place) **before** the end of the period of surface runoff, τ**.** Coincidence of the 'device full' condition with the end of the period of surface runoff used in the analyses is, therefore, **conservative** in such cases : more detailed mathematical modelling (runoff routing) is required to derive advantage from such departures from the simple approach presented below.

5.1.2 The Procedure 1 cases

The basis of the Procedure 1 cases is simple infiltration of storm runoff received from a paved or impervious area, such as a roof or bitumenised surface. They are included under Category 1 because quantity control (of peak flow rate) is identified as the primary objective; but, typically, pollution control is also involved. The widest potential use likely to be made of Procedure 1 is in the design of porous and permeable paving systems for car parks and roof areas draining directly to these spaces. The design process for such installations involves use of Guidelines 1 and 2 from Sections 3.1.2 and 3.1.3 respectively.

The principles which under-pin Procedure 1 are also employed in Chapter 7 of the Handbook to design 'filter strip' swales whose primary function is pollution control (Category 2 installations).

Two configurations of impermeable and porous/permeable paving need to be considered :

- those where the infiltration surface is *external* to the contributing impervious area; and,
- those where the infiltration (or treatment) surface is required to be *within* the total area.

Procedure 1A : Infiltration or treatment surface (without ponding) accepting runoff from roof or paved area, A m^2, and *external* to it

The basic data requirements are (see Sections 4.2 and 4.6) :

- peak flow, Q_{peak} m^3/s passing to the infiltration or treatment surface, A_s m^2; the analysis must take account of rainfall input to the surface A_s, itself;
- The choice of design storm duration, t_c, and ARI used to calculate Q_{peak} is the responsibility of the designer operating in consultation with Council;
- hydraulic conductivity of infiltration surface (porous or permeable paving), k_h m/s;
- average rainfall intensity, i mm/h for design storm of duration t_C;
- infiltration or treatment surface blockage factor, ψ;
- paved area runoff coefficient, C.

Total peak inflow into infiltration or treatment surface $= \dfrac{(CA)i}{10^4 \times 360} + \dfrac{A_s.i}{10^4 \times 360}$ m³/s

Flow (acceptance) capacity of surface $= (1-\psi)\ A_s.k_h$ m³/s

hence,

$$\frac{(CA)i}{10^4 \times 360} = (1-\psi)A_s.k_h - \frac{A_s.i}{10^4 \times 360}$$

$$= A_s\left[(1-\psi)k_h - \frac{i}{3.6\times 10^6}\right]$$

hence,

$$A_s = \frac{(CA)i}{3.6\times 10^6\left[(1-\psi)k_h - \frac{i}{3.6\times 10^6}\right]}$$

$$= \frac{Q_{peak}}{\left[(1-\psi)k_h.U^* - \frac{i}{3.6\times 10^6}\right]}\ m^2 \qquad \textbf{(5.1)}$$

where U = 1.0.

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* Soil hydraulic conductivity values are obtained from site tests on **small** test pits and boreholes, typically. When the results of these tests are applied to design infiltration surfaces or on-site stormwater retention devices, it is found that the systems or devices are “too big”, where site soil is clay, and “too small” where the soil is sandy (see Bressan, 1996). This observation has led to the introduction of a correction factor, **Moderation Factor, U**, which should be applied to hydraulic conductivity, k_h, in the formulae which follow :

In clay soils, U = 2.0 should be used; in sandy clay soils, U = 1.0; in sandy soils, U = 0.5;

U = 1.0 may also be used with **long-term** hydraulic conductivity, k_h, where this is known or estimated for the ‘lifespan’ of the system (see Sections 3.1.2 and 3.1.3).

Some additional points of explanation are :

- Q_{peak} in Eqn. (5.1) is calculated for the *total* contributing paved or roof surface area A m², **only**;
- Area A_s determined from Eqn. (5.1) is the (**total) pervious** surface area of the facility, including – where applicable – space taken up by lattice blockwork, plastic rings, trees, etc. (see Figure 3.1a) : the factor (1 – ψ) in Eqn. (5.1) takes account of the presence of these elements. In **porous or permeable paving** cases, area A_S is determined using an appropriate hydraulic conductivity, k_h, for example its long-term value (see Guideline 1, Section 3.1.2); systems designed according to this approach, use ψ = 0 in Eqn. (5.1).
- U = 1.0 is used for design, based on **long-term** hydraulic conductivity value, known or estimated (see Footnote, above);
- Every result given by Eqn. (5.1) should be followed by application of Guidelines 1 and 2 (Sections 3.1.2 and 3.1.3, respectively) to determine expected 'lifespan(s)'. **Two** estimates arise in the case of permeable paving (geotextile layer and surface), and **one** in vegetated porous systems (surface only);
- See also the application of Procedure 1A or 1B in Example 7.2, Section 7.7.5.

Procedure 1B : Infiltration or treatment surface (without ponding) accepting runoff from paved area and located *within* the defined site area, A m²

The basic data requirements and theory correspond to those given for case 1A, above.

Total peak inflow into treatment surface

$$= \frac{(CA)i}{10^4 \times 360} \text{ m}^3/\text{s}$$

Flow (acceptance) capacity of treatment surface

$$= (1-\psi)A_s.k_h \text{ m}^3/\text{s}$$

hence,

$$A_s = \frac{(CA)i}{3.6\times10^6(1-\psi).k_h}$$

$$= \frac{Q_{peak}}{(1-\psi).k_h.U} \text{ m}^2 \qquad \textbf{(5.2)}$$

where U = 1.0.

- Q_{peak} in Eqn. (5.2) is calculated for the defined (total) **site** area A m²;
- All other points of explanation included under Procedure 1A, above, with respect to the factor (1 – ψ), long-term value of k_h in porous/permeable paving systems, ψ in porous/permeable systems, U and use of Guidelines 1 and 2 (Sections 3.1.2 and 3.1.3), apply equally to Procedure 1B.

5.1.3 The Procedure 2 cases

The broad class of on-site retention installations described as "leaky" devices and "soakaways" includes two basic sub-categories :

- devices which provide unencumbered, well-type storage space; and,
- trench-shaped devices which provide storage space, part-occupied by materials such as gravel or plastic.

Each of these types of devices (see Figure 1.3) releases stored water into the surrounding soil medium by percolation; they also incorporate provision for overflows (see Figures 1.3 and 1.5) and, possibly, for slow-drainage (see Figure 1.4 and Procedure 4).

There is an important constraint or condition which must be observed in all Procedure 2 devices and installations, particularly those which involve gravel or sand as the water-retaining medium (see Section 1.3.3). This concerns the quality of water input, described as 'cleansed water'. To be acceptable, this water should have TSS ≤ 20 mg/L **and** maximum particle size ≤ 20 μm (see Section 7.1).

Procedure 2A : "Leaky" well with cleansed water inflow, typically, from a roof

The basic data requirements are (see Sections 4.2 and 4.6) :

- "Block" runoff volume, $\forall m^3$, passing to well from roof. Normal hydrological processes are used to estimate $\forall$; storm durations equal to $(T_c)_{total}$, $(T_c)_{local}$ and t_C should be considered (see "The Parameter, τ, and design storm duration", Table 5.1);
- The choice of design ARI used to calculate $\forall$ is the responsibility of the designer operating in consultation with Council;
- hydraulic conductivity of soil, k_h m/s.

Design Assumptions :

- well is empty at commencement of inflow;
- well fills over time τ minutes (see Figure 3.4);
- percolation through floor is at full rate of k_h for period of τ minutes;
- percolation through walls is distributed hydrostatically, hence this component of outflow is **half** saturated (outflow) rate for period of τ minutes;
- the perforated wall offers no restriction to outflow;
- groundwater level is significantly below the floor of the well.

For perforated well, diameter D m, height H m :

Inflow (volume) to well $= \forall \ m^3$

volume of well $= \dfrac{\pi D^2}{4} . H$

outflow from well floor $= \dfrac{\pi D^2}{4} . k_h \ .60\,\tau$

outflow from well wall $= \dfrac{1}{2}\left[\pi DH.\, k_h \,.\dfrac{60\,\tau}{2}\right] = 15\,\pi DH\,.\,k_h\ .\,\tau$

Practical cases (particularly in clay soils) usually result in D ≈ H : **this simplification is adopted**.

Hence, diameter of well, D :

$$D \approx \sqrt{\frac{\forall}{\frac{\pi}{4}\left(H + 120\,k_h\,.\,\tau\right)}} \ \ m$$

Incorporating the Moderation Factor, U, this formula becomes :

$$D = \sqrt{\frac{\forall}{\frac{\pi}{4}\left(H + 120 k_h . \tau . U\right)}} \ \text{m} \qquad \textbf{(5.3a)}$$

where U = 0.5 for sandy soil; U = 1.0 for sandy clay; U = 2.0 for clay soil.

There is another form of this equation which is useful in designing "leaky" well installations :

$$H = \frac{4(\forall - 30\pi\, k_h . \tau\, D^2 U)}{\pi\, D^2} \qquad \textbf{(5.3b)}$$

Important Note : Every design application of Procedure 2A must be followed by a check on emptying time, T, in accordance with the requirements of the *modified* design storm method (see Section 3.5.4).

Procedure 2B : Trench, part-occupied with impervious material, pipes, etc., receiving cleansed water inflow

The basic data requirements and design assumptions correspond to those given for case 2A, above. For trench of length L, width b, and depth H, part-occupied with impervious material (gravel, plastic, perforated pipes, etc.) and providing void space, e_S, where :

$$e_S = \frac{\text{void space available}}{L\, b\, H}$$; e_s is identical to porosity, n, used in geotechnical engineering.

Typical values of the ratio e_S are 0.35 for gravel and 0.95 for certain "milk crate" plastic units. Values of e_S for trenches part-occupied by perforated pipes range from 0.5 to 0.75 depending on pipe sizes and trench cross-section dimensions (see Dierkes et al, 2002).

Inflow volume to system = $\forall$ m³; storm durations $(T_C)_{total}$, $(T_C)_{local}$ and t_C should be considered (see "The Parameter, τ, and design storm duration", Table 5.1) :

volume of trench void space = $e_S.LbH$

outflow from trench floor = $60\ Lb.k_h.\tau$

outflow from trench walls = $\frac{1}{2}\left[2(L+b)H.k_h.\frac{60\,\tau}{2}\right] = 30\,(L+b)\,H.k_h.\tau$

Ignoring outflow from ends of trench :

$$L = \frac{\forall}{\left[e_s.bH + 60\,k_h.\tau\left(b + \frac{H}{2}\right)\right]} \ \text{m}$$

Incorporating the moderating parameter, U, this formula becomes :

$$L = \frac{\forall}{\left[e_s.bH + 60\,k_h.\tau\left(b + \frac{H}{2}\right)U\right]} \ \text{m} \qquad \textbf{(5.4a)}$$

where U = 0.5 for sandy soil; U = 1.0 for sandy clay; U = 2.0 for clay soil.

There are two other forms of this equation which are useful in designing trench systems :

$$H = \frac{\forall - 60k_h.L\tau bU}{e_s Lb + 30k_h.L\tau U} \qquad \textbf{(5.4b)}$$

$$b = \frac{\forall - 30k_h.L\tau HU}{e_s LH + 60k_h.L\tau U} \qquad \textbf{(5.4c)}$$

Important Note : Every design application of Procedure 2B must be followed by a check on emptying time, T, in accordance with the requirements of the *modified* design storm method (see Section 3.5.4).

Procedure 2C : "Soakaway" part-occupied with impervious material, pipes etc., receiving cleansed water inflow

Application of Procedure 2B in soils of very low permeability, notably heavy clays, yields trenches of impractical length. Such cases are better constructed as "soakaways", that is, trenches of various types with length, L, approximately equal to width, b. Otherwise, the basic data requirements and theory correspond to those given for cases 2A and 2B, above.

Inflow volume to system = $\forall$ m^3; storm durations $(T_C)_{total}$, $(T_C)_{local}$ and t_C should be considered (see "The Parameter τ, and design storm duration", Table 5.1).

Volume of "soakaway" void space = $e_S.aH$

where a = plan area with $L \approx b$

Outflow from "soakaway" floor = $60\ a.k_h.\tau$

Ignoring outflow from "soakaway" walls :

$$a = \frac{\forall}{(e_s.H + 60k_h.\tau)}\ m^2$$

Incorporating the moderation factor, U, this formula becomes :

$$a = \frac{\forall}{(e_s.H + 60k_h.\tau.U)}\ m^2 \qquad \textbf{(5.5a)}$$

where U = 0.5 for sandy soil; U = 1.0 for sandy clay; U = 2.0 for clay soil.

The magnitude of "a" can be satisfied with any combination of L.b = a, but length, L, and width, b, should be as close as possible to being equal. There is another form of Eqn. (5.5a) which is useful in designing "soakaways" :

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

Important Note : Every design application of Procedure 2C must be followed by a check on emptying time, T, in accordance with the requirement of the *modified* design storm method (see Section 3.5.4).

5.1.4 Infiltration or 'dry' ponds

Procedure 3 : Pond with infiltration, also called a 'dry' pond

The basic data requirements are (see Sections 4.2 and 4.6) :

- "Block" runoff volume, $\forall m^3$ passing to pond site; storm durations $(T_C)_{total}$, $(T_C)_{local}$ and t_C should be considered (see "The Parameter, τ, and design storm duration", Table 5.1);
- the choice of design ARI used to calculate $\forall$ is the responsibility of the designer operating in consultation with Council;
- **long-term** hydraulic conductivity of pond floor soil, k_h m/s;
- pond design depth, d m, usually determined by safety considerations;
- critical storm duration for this case is expressed as $(\tau - t_C)$ minutes; this term effectively identifies the three **alternative** storm durations $(T_C)_{total}$, $(T_C)_{local}$ and t_C which should be considered;
- average rainfall intensity, i mm/h, for design storm;

Design assumptions :

- pond is empty at commencement of inflow;
- pond fills over time equal to τ minutes (see Figure 3.4);
- percolation through floor of pond, area A_p m², is at full rate of k_h for period of τ minutes;
- percolation through wall of pond is negligible;
- pond design storage volume, $\forall_p$ m³ represents the maximum quantity of runoff which can be stored temporarily as 'open water' at the pond site;
- the area used to determine $\forall$ is the contributing catchment area, only, but the analysis of **total inflow to the pond must include rainfall on pond area A_p**, in addition to that on the contributing catchment area;
- groundwater level is significantly below the floor of the infiltration pond system.

For pond with surface area A_p m² and design storage volume $\forall_p$ m^3 :

Runoff volume entering pond	=	$\forall$
Volume of pond	=	$A_p.d$, where d = design depth in metres
Outflow from pond floor	=	$A_p.k_h.60\tau$
Rainfall volume entering pond	=	$A_p.\dfrac{i.(\tau - t_c)}{1000 \times 60}$

The term $(\tau - t_C)$ effectively identifies the three alternative storm durations which should be considered in the design process (see Table 5.1).

Hence,

$$\forall = A_p.d - \frac{A_p.i.(\tau - t_c)}{60 \times 1000} + A_p.k_h.60\tau \quad = \quad A_p\left[d + 60k_h.\tau - \frac{i.(\tau - t_c)}{6 \times 10^4}\right]$$

hence,

$$A_p = \frac{V\!\!\!\!-}{\left[d + 60k_h.\tau - \dfrac{i.(\tau - t_c)}{6 \times 10^4} \right]} \ \mathrm{m}^2$$

Incorporating the moderation factor, U, this formula becomes :

$$A_p = \frac{V\!\!\!\!-}{\left[d + 60k_h.\tau.U - \dfrac{i.(\tau - t_c)}{6 \times 10^4} \right]} \ \mathrm{m}^2 \qquad \textbf{(5.6a)}$$

where U = 1.0.

There is another form of this equation which is useful in designing “dry ponds” :

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

Important Note : Every design application of Procedure 3 must be followed by a check on emptying time, T, in accordance with the requirement of the *modified* design storm method (see Section 3.5.4).

This technology requires pond site soil permeability to be in the range medium to high, that is $k_h \nless 1 \times 10^{-5}$ m/s which is the limit for sandy clay. Infiltration ponds of the type analysed here are **not recommended** in medium and heavy clays and constructed clays (see Section 1.3.4) if ‘natural’ drainage (percolation) is the only mode of emptying. This does not preclude the use of infiltration ponds in soils of low permeability, but such installations require (introduced) sand-bed bases 200 mm – 300 mm thick over “leaky” storage sub-structures (Procedures 2B, 2C, above), often with drainage assistance (see below) to enable emptying time criteria to be met.

5.1.5 The Procedure 4 cases

Introduction : There are two sets of circumstances which call for the application of Procedure 4. The **first** is where aquifer injection of cleansed stormwater contained in a “leaky” sub-surface device is proposed in order to meet a water resources objective (see Sections 1.3.5 – 1.3.7) : flood mitigation and/or water quality goals may also be involved. The **second** circumstance – more frequent than the first – arises where application of Procedures 2A, 2B or 2C, above, leads to designs which are very difficult or impossible to drain, by ‘natural’ drainage (percolation) alone, and satisfy required emptying time criteria (see Section 3.5.3).

The first case, called “Procedure 4A” in the following note, is an essential element in the design of aquifer recharge/retrieval (ASR) ‘source control’ schemes of the types described in Section 3.9 (see also Figure 1.4a).

The second case – “Procedure 4B” – might be the consequence of soil permeability being extremely low or site space limitations being too restrictive, or an emptying time requirement which is, perhaps, very conservative. In such circumstances it may be necessary to assist the emptying process by providing **slow-drainage** of stored water (see Figure 1.4b). Such provision is, in essence, the basis of on-site detention (OSD) practice: however, application of this technology to on-site retention (OSR) devices differs significantly in the length of time during which the slow-drainage process takes place. It is normal practice for OSD installations to empty in periods of four hours or less; “slow-drainage” is calculated to take place, purposefully, over periods of 12 hours or longer (see Table 3.3) : the practice has been recognised in US literature where it is referred to a “extended detention” (US Dept of Transportation, 1996). See Section 5.4.

Procedure 4 should **only** be acted upon after a preliminary design for an installation, dimensioned under the provisions of Procedures 2A, 2B or 2C, has failed to meet a required emptying time criterion and other options, e.g. changing the configuration of the installation or removing/using the retained water, including

ASR where possible, have been exhausted. This guideline applies equally to trenches and "soakaways" constructed beneath infiltration or 'dry' ponds located in soils of low permeability.

Procedure 4A : Slow-drainage with aquifer-access

This procedure involves the following steps :

STEP 1 : Establish by geological investigation or from otherwise available geotechnical data bases, the presence of a readily-accessible aquifer where a scheme for aquifer storage of cleansed storm-water without environmental damage can be established (see Figure 1.4a).

STEP 2 : Determine by field trial or otherwise a value for recharge rate per bore, q_r m³/s and, hence, the number of bores, n, required to empty the previously-designed device (Procedure 2A, 2B or 2C) in the required time (emptying time criterion, Table 3.3). Note that "n" is an integer. A recharge flow rate $Q_r = n.q_r$ m³/s, should then be determined. [Recharge rate may be taken as 50% of bore yield (Pavlic et al, 1992)].

STEP 3 : Revisit the design of the "leaky" device (Procedures 2A, 2B or 2C), already prepared, using the recharge-corrected form of Eqns. 5.3a or b, 5.4a, b or c or 5.5a or b, as appropriate, listed below as Eqns. 5.7a or b, 5.8a, b or c; 5.9a or b, respectively. Application of these formulae leads to smaller installations with acceptable emptying times.

Procedure 4B : Slow-drainage with pipeline

The procedure involves the following steps :

STEP 1 : Determine by local terrain and property survey the potential for installing a slow-drainage pipeline with sufficient fall from the site to convey a small flow to a nearby storm drain or urban waterway (see Figure 1.4b).

STEP 2 : Determine the volume rate of flow needed to remove the stored water from the "leaky" device (already designed) in the required emptying time (emptying time criterion, Table 3.3). In the case of "leaky" devices this is water volume stored within the installation – taking account of gravel, etc. – divided by the requiring emptying time, T. This calculation leads, in each case, to a value for Q_r m³/s, the slow-drainage flow rate.

STEP 3 : Revisit the design of the "leaky" device (Procedures 2A, 2B or 2C) already prepared using the discharge-corrected form of Eqns. 5.3a or b, 5.4a, b or c; 5.5a or b, as appropriate, listed below as Eqns. 5.7a or b; 5.8a, b or c; 5.9a or b, respectively. Application of these formulae leads to smaller installations with acceptable emptying times.

Data requirements and design assumptions for the discharge-corrected equations :

- "Block" runoff volume, $\forall$ m³, as per Procedure 2A, 2B or 2C as applicable; storm duration and parameter τ (see "The Parameter, τ, and design storm duration, Table 5.1), as applicable;
- design ARI as applicable;
- hydraulic conductivity of soil, k_h m/s, as applicable;
- device fills over time τ minutes (see Figure 3.4);
- percolation through floor is at full rate of k_h for period of τ minutes;
- exfiltration through walls and other assumptions, simplifications, etc., are the same as for corresponding analyses in Procedures 2A, 2B and 2C, above;
- groundwater level is significantly below the floor of the device in each case.

“Leaky” well with cleansed water inflow and outflow rate of q_r m³/s per well (see Procedure 2A) :

$$D = \sqrt{\frac{\forall - 60q_r.\tau}{\frac{\pi}{4}(H + 120k_h.\tau.U)}} \text{ m} \qquad \text{(single bore case)} \qquad \textbf{(5.7a)}$$

$$H = \frac{4(\forall - 60q_r\tau - 30\pi D^2 k_h.\tau U)}{\pi D^2} \text{ m} \qquad \text{(single bore case)} \qquad \textbf{(5.7b)}$$

Note that q_r may be replaced by Q_r in both equations 5.7a and 5.7b where slow-drainage by pipeline is employed.

Trench, part-occupied with impervious material, cleansed water inflow and outflow rate of Q_r m³/s (see Procedure 2B); note that Q_r = n.q_r may be an option to be considered :

$$L = \frac{\forall - 60Q_r.\tau}{\left[e_s.bH + 60k_h.\tau(b + \frac{H}{2})U\right]} \text{ m} \qquad \textbf{(5.8a)}$$

$$H = \frac{\forall - 60Q_r.\tau}{L\left(be_s + 30k_h.\tau U\right)} - \frac{60k_h.\tau bU}{(be_s + 30k_h.\tau U} \text{ m} \qquad \textbf{(5.8b)}$$

$$b = \frac{\forall - 60Q_r.\tau}{Le_sH + 60Lk_h.\tau U} - \frac{30Lk_h.\tau HU}{Le_sH + 60Lk_h.\tau U} \text{ m} \qquad \textbf{(5.8c)}$$

“Soakaway” part-occupied with impervious material, cleansed water inflow and outflow rate of Q_r m³/s (see Procedure 2C); note that Q_r = n.q_r may be an option to be considered :

$$a = \frac{\forall - 60Q_r.\tau}{(e_s.H + 60k_h.\tau.U)} \text{ m}^2 \qquad \textbf{(5.9a)}$$

$$H = \frac{\forall - 60Q_r.\tau}{ae_s} - \frac{60k_h.\tau U}{e_s} \text{ m} \qquad \textbf{(5.9b)}$$

where U = 0.5 for sandy soil; U = 1.0 for sandy clay; U = 2.0 for clay soils.

Important note : The discussion, above, of slow-drainage is confined to components of Procedure 2 (‘leaky’ wells, trenches and “soakaways”) and avoids Procedure 3 (infiltration or ‘dry’ ponds). This is because stormwater passing to a pond is unlikely to be ‘cleansed’, as required for all Procedure 2 devices. It is unacceptable environmental practice for aquifer access, applying Procedure 4A, or piped drainage (Procedure 4B) to be permitted **directly** to receiving domains with untreated water from a pond.

However, if treatment takes place within the pond ensuring that satisfactorily cleansed water, only, is admitted to the outlet, then the same design approach reviewed above for the trench and “soakaway” cases, namely catchment runoff volume ∀ replaced by (∀ – 60 $Q_r.\tau$) in the corresponding equations [(5.6a) and (5.6b)], etc., etc., can be employed to yield appropriate outcomes.

A guideline on the standard of treatment required to achieve satisfactory cleansing is : subject to passage through two thicknesses of non-woven geotextile fabric before exit. This can be accomplished using a perforated collector/pipe system enclosed in a geotextile fabric sock. Regular inspection and maintenance is an essential ingredient of this solution.

TABLE 5.1
SUMMARY OF GUIDELINES FOR PROCEDURES 1A, 1B, 2A, 2B, 2C, 3 AND 4 : SOME COMBINATION CASES
[Section 4.2 should be read and thoroughly understood before attempting to use this Table]

FACILITY TYPE AND CALCULATION PROCEDURE	PERCOLATION OUTFLOW TIME BASE, τ, MINS	DESIGN FREQUENCY	PRE-TREATMENT REQUIRED	PROVISION FOR HANDLING FLOWS EXCEEDING DESIGN
Infiltration surface receiving runoff from roof or paved surface, e.g. carriageway, car park, etc. (Eqns. 5.1, 5.2).	Not applicable : design is based on Q_{peak} for site t_c; normal hydrological procedures apply.	Generally 2 – 10 years where focus of design is on **minor system**. ARIs normally associated with **major system** design – 50- to 100-years – are employed in some catchment cases (see Section 4.6.1).	NIL	Provide for overflow, that is, 'gap' between design Q_{peak} and peak flow for site t_c in ARI, Y = 50-years to 100-years event.
"Leaky" well, trench or "soak-away" receiving runoff from roof area or similar (Eqns. 5.3, 5.4 and 5.5).	**The Parameter, τ, and 'design storm' duration** τ is the time base of the runoff hydrograph which is taken to be the "period of filling"; this is considered also to define the period of percolation or drainage outflow from a water-retaining device or system **during filling**. The **minimum** value τ can take is $\tau = 2t_C$ where t_C = site time of concentration (see Figure 3.4b). However, concern for flooding in local drainage paths and in main streams where local flows enter, requires that site drainage be designed with a more global view in mind, explained in Section 4.2.3. Typically, $\tau = (T_C + t_C)$ where T_C is local OR total catchment travel time (two values possible, see Figure 3.4). Three possible basic cases arise : • $\tau = 2t_C$, site drainage case, or where t_C coincides with $(T_C)_{local}$; • $\tau = [(T_C)_{local} + t_C]$ where local sub-area flooding, only, is to be considered; $(T_C)_{local}$ is, typically, local sub-area critical storm duration as defined in Section 4.2.3; • $\tau = [(T_C)_{total} + t_C]$ where total catchment flooding, only, is to be considered; $(T_C)_{total}$ is critical storm duration as defined in Section 4.2.3. The corresponding **storm durations** are t_C, $(T_C)_{local}$ and $(T_C)_{total}$ respectively. The dominant flooding consideration – site, local or total catchment – determines which of the three alternatives should be used in the design process.		Requires 'first flush' or simple sediment removal/containment system as in Figs. 1.5 or 5.2.	It is important that provision be made for 'overflows' : these will occur in **ALL** cases except systems which have been designed to combat total catchment flooding in major, e.g. ARI, Y = 50-years to 100-years events with the **yield-minimum** strategy. Where site flooding, only, is the focus of design at, say, ARI, Y = 5-years, then a flow path with capacity equal to the 'gap flow' between Q_{100} and Q_5 must be provided.
Infiltration pond receiving runoff from small urban catchment, large paved area, etc. (Eqns. 5.6).			Filter strip, bio-retention swale or similar upstream of pond is advised.	
Sub-structure (gravel-filled device) beneath treatment surface, infiltration pond, etc. (Eqns. 5.4 and 5.5).			Not applicable. Pre-treatment to pond or the pond/filter system itself provides all required pre-treatment.	
Combined "leaky" well/gravel-filled systems. Design of these requires trial solutions until match is achieved (see Combination Structures, Section 5.3.3) for runoff from roof area or similar (Eqns. 5.3, 5.4 and 5.5).			Requires 'first flush' or simple sediment removal/containment system as in Figs. 1.5 or 5.2.	

5.2 EXAMPLES OF CATEGORY 1 INSTALLATIONS USING PROCEDURES 1A, 2A, 2B, 2C, 3, 1A/4A AND 2B/4B

Example 5.1 : Infiltration surface without ponding

The case study used to illustrate this aspect of 'source control' of stormwater is St. Elizabeth Church car park at Warradale in Adelaide (see Section 3.9) illustrated in Figure 5.1. Determine the size of the central grassed hard-standing area. Surface reinstatement to take place less frequently than every 30 years; flooding ARI = 3 years (client requirements).

Details of the catchment draining to the central section of the car park :

- **total** car park area (40 car park spaces + carriageway, $C_{10} = 0.9$) .... 1,220 m²
- catchment description : car park in average suburb with some trees;
- site time of concentration, t_c.. 10 minutes
- design storm intensity (ARI = 3-years, $F_y = 0.9$, Table 3.5) .... 50 mm/h

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

Procedure 1B : Infiltration surface (without ponding) accepting runoff from paved area, A m², passing to :

an *internal* area, A_S :

required area, $$A_S = \frac{Q_{peak}}{(1 - \psi).k_h.U} \text{ m}^2 \qquad \textbf{(5.2)}$$

where ψ = blockage factor ("Grasspave", $\psi = 0.1$);

k_h = 2.5×10^{-4} m/s : hydraulic conductivity of 'as constructed' soil for grassed area;

k_h = **long-term** value of infiltration soil by Guideline 1, Section 3.1.2, is 5×10^{-5} m/s,

Hence $A_S = 311$ m² : this corresponds to (I/P) = 2.9, and 'lifespan' ≅ (5.2 years × 5) = 26 years interpolated from Table 3.1 ("…average suburb with some trees").

This is unsatisfactory. To meet minimum 30 years required by client, choose I/P = 2.0,

hence 'lifespan' = (6.7 years × 5) = 33.5 years, OK.

Hence, vegetated porous area $= \frac{1{,}220}{3} = \mathbf{407\ m^2}$.

[It should be noted that if the same result for A_S had been determined for a **permeable** paving system, then the 'lifespan' consideration process would proceed as follows :

$A_S = 311$ m^2 : this corresponds to (I/P) = 2.9;

Hence, 'lifespan' (Figure 3.1b) ≈ 27 years (geotextile layer) at 20% 'as constructed', and,

'lifespan' (Table 3.1) ≈ 5.2 years (surface blockage).

In the event of these estimates being unsatisfactory, a review and reconfiguration process similar to that outlined above would be needed to produce an acceptable outcome.]

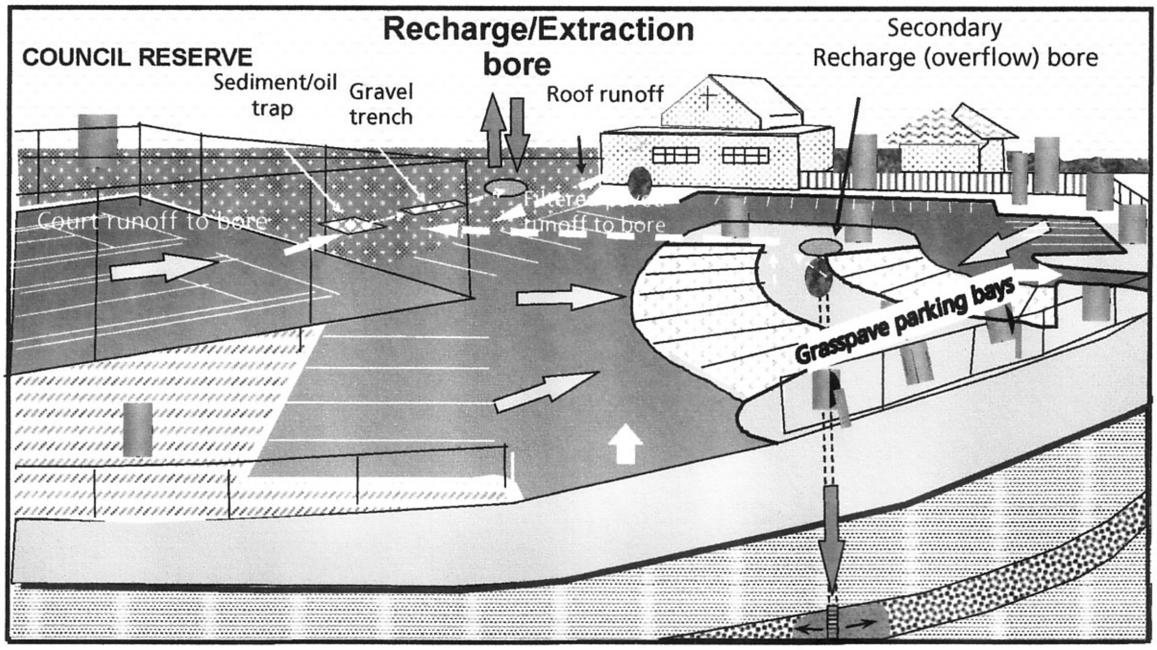

FIGURE 5.1 : General layout of St. Elizabeth Church car park

Example 5.2 : Gravel-filled trench receiving roof runoff

The New Brompton Estate, Adelaide, central reserve is the site of a "greenfields" development (modified project) whose aim is to retain roof runoff on site. The Council specification requires that the development be designed under the provisions of the **yield-minimum** strategy (see Section 4.2). The actual scheme is described in Section 3.9 and illustrated in Figure 5.2.

Details of the catchment (roof areas) draining to the gravel-filled trench bordering the reserve are as follows :

- total connected roof areas (and "greenfields" area) .... 1,200 m²
- site time of concentration, t_C .... 10 minutes
- critical storm duration, $(T_C)_{total}$, Council specification.................................... 60 minutes
- design storm intensity (ARI, Y = 10-years, Council specification) .... 26 mm/h

Hence, design runoff volume, $\forall$ = 28.1 m³ (**yield-minimum** strategy, Council specification).

Procedure 2B : Trench, part occupied with impervious material, pipes, etc., with clear water inflow :

Required length,

$$L = \frac{\forall}{\left[e_S \, . bH + 60 \, k_h \, . \tau \left(b + \frac{H}{2} \right) U \right]} \text{ m} \qquad \textbf{(5.4a)}$$

where e_S = void space (e_S = 0.35 for gravel)

b = trench width (b = 0.9 m in this case)

H = trench height (H = 1.2 m in this case)

k_h = soil hydraulic conductivity (1×10^{-6} m/s)

τ = time base of design storm runoff hydrograph (τ = 70 minutes in this case)

U = 2.0 for clay soil.

Hence L = 72.0 m. This must be followed by emptying time calculation :

Emptying time,

$$T = -\frac{4.6 L b e_S}{2 k_h (L + b)} \log_{10} \left[\frac{Lb}{Lb + 2H(L + b)} \right] \text{, s} \qquad \textbf{(3.31)}$$

Hence, emptying time, T = 4.7 days (unsatisfactory, see Table 3.3).

In reality, the infill soil at New Brompton Estate, site of a former brick (clay) pit, was heavy clay laid in 150 mm layers and compacted with heavy equipment to a total depth of seven metres. Its measured hydraulic conductivity was 3×10^{-9} m/s, a very low value. In its second year of operation a recharge bore receiving water from the trench (low level overflow) was installed: this provided the elements of an aquifer storage and recovery (ASR) scheme which ensured satisfactory emptying time; the project retains the high-level overflow to street drainage as originally planned.

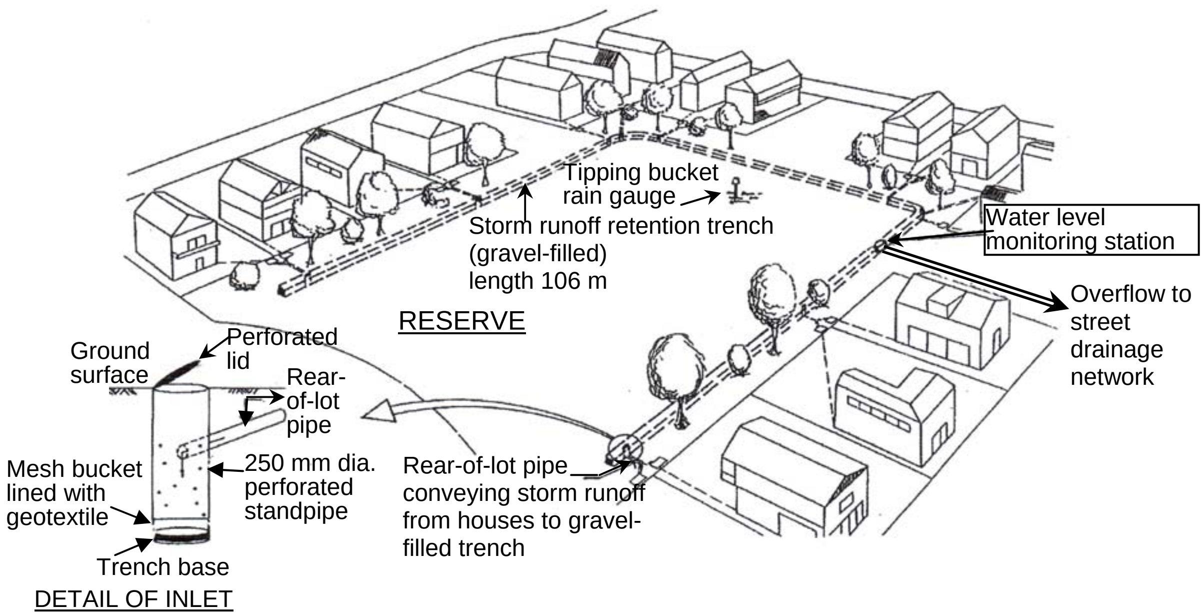

FIGURE 5.2 : Development at New Brompton Estate, showing gravel-filled trench and sediment retention standpipe

Example 5.3 : Gravel-filled "soakaway" and passive irrigation facility

The basis of the passive irrigation facility at Plympton Anglican Church is a perforated pipe and gravel "soakaway" (see Section 3.9) illustrated in Figure 5.3. The following design is prepared for the project considered as occupying a "greenfields" site. The Council specification requires that the development be designed under provisions of the **regime-in-balance** strategy (see Sections 4.2 and 4.6).

Original "greenfields" site :

- site area (C_{10} = 0.10) .... 2,640 m²
- site time of concentration, t_C .... 10 minutes
- critical storm duration, $(T_C)_{total}$, Council specification........................ 30 minutes
- design storm intensity
  [ARI = 100-years, (F_Y = 1.2, Table 3.5), Council spec.] .... 73 mm/h

Hence, design runoff volume = 11.6 m^3 from "greenfields" site in 100-years design storm.

Developed site :

- site area .... 2,640 m²
- total connected roof areas.. 1,600 m²
- total connected paved area .... 360 m²
- total connected pervious areas (C_{10} = 0.10)…..…...………...……............... 680 m²
- site time of concentration, t_C .... 10 minutes
- critical storm duration, $(T_C)_{total}$, Council specification........................ 30 minutes
- design storm intensity
  [ARI = 100-years, (F_Y = 1.2, Table 3.5), Council spec.] .... 73 mm/h

Hence, design runoff volume = 74.5 m^3 from developed site in 100-years design storm.

Hence, design runoff volume, $\forall$ = (74.5 – 11.6) = 62.9 m³ (**regime-in-balance** strategy, Council specification).

Procedure 2C : "Soakaway" part-occupied with impervious material, pipes, etc., with cleansed water inflow :

required area, $$a = \frac{\forall}{\left[e_s \cdot H + 60 k_h \cdot \tau \cdot U\right]} \ m^2 \qquad \textbf{(5.5a)}$$

where e_S = void space (e_S = 0.50 for perforated pipe/gravel system, see Procedure 2B, Section 5.1)

H = height (thickness) of soakaway (0.3 m in this case)

k_h = soil hydraulic conductivity (1×10^{-6} m/s); U = 2.0 for clay soil

τ = time base of design storm runoff hydrograph (τ = 40 minutes in this case)

Hence **a** **=** **406 m²**. This must be followed by emptying time calculation :

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

Hence emptying time, T = 3.5 days (satisfactory, see Table 3.3).

The installation at Plympton is enclosed in geotextile and overlayed by a 0.3 m thickness of loam supporting a garden of native plants and shrubs and drought-resistant grass. Surface watering was needed to establish the garden; it now receives normal rainfall and 'passive' irrigation (from below). The grassed area may receive a light "sprinkle" of mains water prior to use for outdoor ceremonies, otherwise no irrigation.

It is of interest to note how transfer of this illustration from Adelaide in Southern Australia to Brisbane in Northern Australia affects the design outcome. In this case – using the same basic data except for "greenfields" pervious area runoff coefficient, C_{10} = 0.7 (for Northern Australia, see Section 3.6), and Brisbane rainfall – the design runoff volume which must be accommodated is :

$\forall$ = **24.9 m³** (**regime-in-balance** strategy, Council specification)

leading to "soakaway" area, **a = 161 m²** and emptying time of 3.5 days.

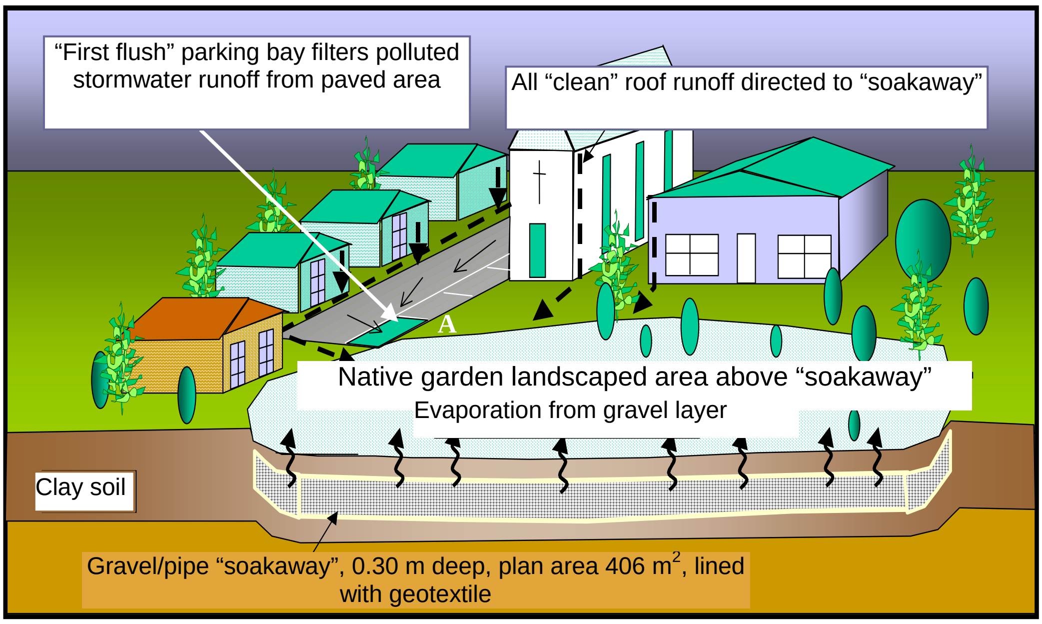

FIGURE 5.3 : Layout of Community Garden, Plympton Anglican Church, Adelaide

Example 5.4 : Infiltration or 'dry' pond

"Figtree Place" is an inner-city residential (27 units) re-development project completed in Newcastle, NSW, in 1998: its layout is illustrated in Figure 5.4 (see Section 3.9). The Council Specification required that the development be designed under the provisions of the **yield-minimum** strategy (see Sections 4.2 and 4.6). [The infiltration pond at "Figtree Place" is **not** a 'sump' referred to in Section 4.2.4, because it overflows in all storms greater than the 50-years critical storm event]

Details of the catchment (paved area only) draining to the recharge facility are as follows :

- total connected paved areas.. 1,900 m²
- site time of concentration, t_C .... 10 minutes
- critical storm duration, $(T_C)_{total}$, Council specification.................................. 60 minutes
- design storm intensity
  [ARI = 50-years (F_Y = 1.15, Table 3.5) Council spec.] .... 67 mm/h

Hence, design runoff volume, ∀ = 127 m³ (**yield-minimum** strategy, Council specification).

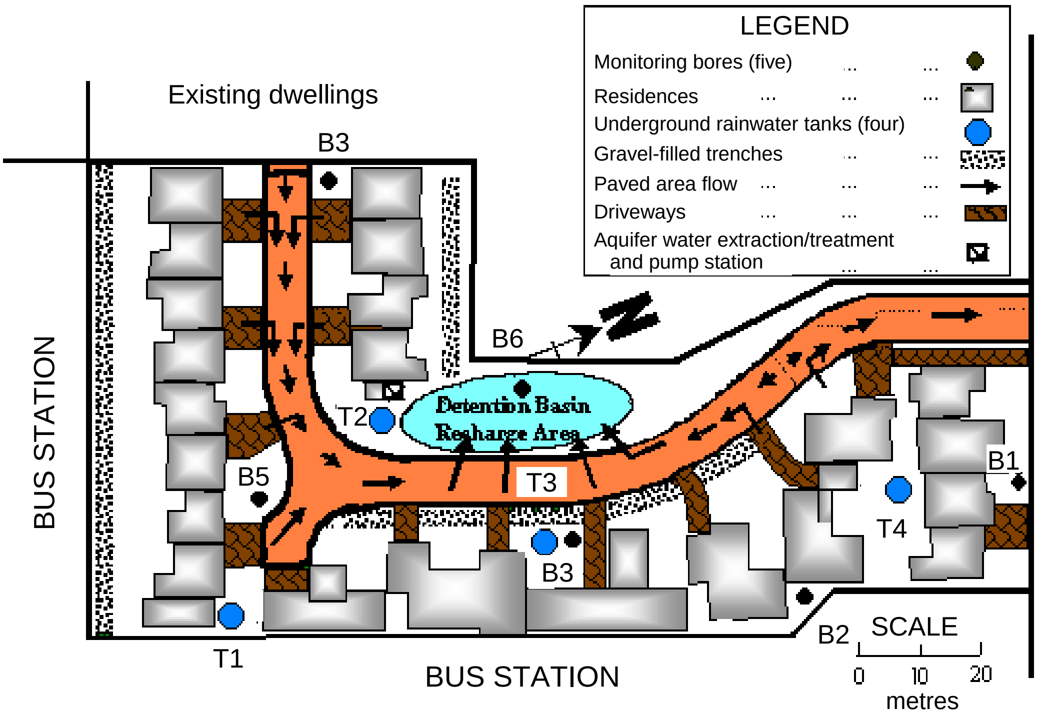

FIGURE 5.4 : "Figtree Place", Newcastle : general plan and water-sensitive features

Procedure 3 : Pond with infiltration :

required area,

$$A_p = \frac{\forall}{\left[d + 60 k_h . \tau . U - \frac{i.(\tau - t_c)}{6 \times 10^4} \right]} \text{ m}^2 \qquad \textbf{(5.6a)}$$

where d = design depth of pond (0.5 m in this case)

k_h = long-term hydraulic conductivity of pond surface soil (5×10^{-5} m/s) : U = 1.0

τ = time base of design storm runoff hydrograph (τ = 70 minutes in this case)

i = rainfall intensity for design storm, duration 60 minutes, ARI = 50-years (67 mm/h).

Hence A_p = 198 m². This must be followed by emptying time calculation :

Emptying time, $$T = \frac{2d.e_S}{k_h}, s \quad \text{with } e_S = 1.0, \qquad \textbf{(3.32)}$$

$$= \frac{2d}{k_h}, \text{ seconds for open water body}$$

Hence emptying time = 0.23 days (satisfactory, see Table 3.3).

The hydraulic conductivity of the general site soil at "Figtree Place" is less than that of the introduced soil used on the floor of the 'dry' pond. A sub-surface "soakaway" is therefore provided to ensure that the pond clears quickly following rainfall, enabling its other use as a recreation facility to be fulfilled (see Example 5.5, following).

Example 5.5 : Sub-structure "soakaway" beneath infiltration surface

This example re-visits Example 5.1: its aim is to dimension the gravel-filled sub-structure beneath the 407 m² car park infiltration or 'treatment' surface at St. Elizabeth Church. This is a prime example of the type of water-retaining facility whose overflow is understood by the public to occur, literally, "once only on average in 3-years", not tied to site critical storm duration, t_C (see the final four paragraphs of Section 4.2.6). The design must therefore consider **all** storm durations with frequency ARI = 3-years to determine the optimum case.

Details of the catchment draining to the treatment surface and sub-structure are as follows :

- paved area (40 car park spaces + carriageway) .... 1220 m²
- grassed hard-standing area (15 car park spaces) .... 407 m²
- site time of concentration, t_C .... 10 minutes
- storm duration range considered
  (ARI, Y = 3-years, F_Y = 0.9, Table 3.5) .... 10 mins to 3 days.

Procedure 4A : Establish the presence of a readily accessible aquifer; determine recharge rate per bore, q_r m³/s; use Eqn. (5.9a) :

$$a = \frac{\cancel{V} - 60\,q_r\,.\tau}{(e_S\,.H + 60\,k_h\,.\tau.U)} \text{ m}^2 \quad \text{(single bore case)} \qquad \textbf{(5.9a)}$$

where e_S = void space (e_S = 0.35 for gravel)

H = "soakaway" height (use H = 0.5 m in this case)

k_h = soil hydraulic conductivity (1×10^{-6} m/s); U = 2.0, clay soil;

q_r = 0.0005 m³/s (given)

τ = time base of design storm runoff hydrograph [τ = (t + 10 mins),

where t = storm duration].

Tabulation to determine the optimum value of gravel-filled "soakaway" needed to give overflow literally once, only, in 3-years ($F_Y = 0.90$) for **all** storm durations :

Storm duration, t	**Intensity, (mm/h)**	**Rainfall depth, d (mm)**	**$\tau = (t + t_C)$ (mins)**	**$\cancel{V}$ = $(0.81 \times 814 + 407)d$ (m^3)**	**$\cancel{V} - 60\, q_r.\, \tau$ (m^3)**	**$e_sH + 60k_h.\, \tau.U$ (m)**	**hence, a (m^2)**
10 mins	50	8.3	20	8.85	8.25	0.177	46.5
…	…	…	…	…	…	…	…
…	…	…	…	…	…	…	…
3 hours	9.2	27.6	190	29.4	23.7	0.198	120
4 hours	**7.6**	**30.4**	**250**	**32.4**	**24.9**	**0.205**	**121**
6 hours	**5.9**	**35.4**	**370**	**37.7**	**26.6**	**0.219**	**121**
8 hours	4.9	39.2	490	41.8	27.1	0.234	116

The optimum area of "soakaway" required is 121 m², resulting from storm duration between 4 and 6 hours. This implies a gravel-filled trench, 0.5 m deep, under approximately 1/3 of the 407m² infiltration surface: its storage volume is 21.2 m^3; the "soakaway" includes a single bore (see Figure 1.4a for sketch details). This must be followed by emptying time calculation :

Emptying time, $\quad T = \dfrac{\text{stored volume}}{q_r}$

$$= \frac{121 \times 0.5 \times 0.35}{0.0005}$$

$$= 41{,}650 \text{ s}$$

= 11.8 hours (see Table 3.3, satisfactory).

This design has been determined assuming the sub-structure to be a "soakaway", not a gravel-filled trench. Strictly, the design should be reviewed using the trench formula, Eqn. (5.8a), in place of Eqn. (5.9a), inserting H = 0.50 m and b = 2.0 m, say, to determine a value for length L; the **final** design should then be re-checked for emptying time. This process leads to L = 59.6 m (or plan area, a = 119 m²) and emptying time of 11.6 hours, very close to the "soakaway" solution.

The car park at St. Elizabeth Church has been designed with sufficient above-ground storage capacity to manage/dispose of all storm runoff generated on the site up to and including 100-years events covering the full range of storm durations (10 minutes to three days). It is therefore, for all practical purposes, a 'sump' (see Section 4.2.4). Its design required additional computations to those given above.

However, were this **not** the case and if overflow from the grassed hard-standing area were to find its way directly into the local (surface) drainage path *without ponding*, then an alternative design of the infiltration surface and the "soakaway" would have to be carried out. This (design) would have to consider the car park as a component of the catchment-wide stormwater management plan for, say, the **yield-minimum** strategy and ARI, Y = 100-years required (see Sections 4.2 and 4.6). The infiltration surface (redesign of Example 5.1) in this case would be $A_S = 1024\ m^2$ (Adelaide, 10 minutes, i_{100} = 136 mm/h): this configuration of the car park leads to an impervious-to-pervious ratio of 0.19 which easily satisfies Guidelines 1 and 2, Sections 3.1.2 and 3.1.3) and suggests service without maintenance for a long period of time. The revised "soakaway" calculations, under these altered circumstances, would be as follows :

Developed site :

- paved area (carriageway) .... 196 m²
- grassed hard-standing area (40 car parking spaces)………… .... 1,024 m²
- site time of concentration, t_C .... 10 minutes
- critical storm duration, $(T_C)_{local}$ (Council specification)…………… .... 30 minutes
- design storm intensity [ARI = 100-years (F_Y= 1.2) Council spec.] .... 73 mm/h

Hence, total (stormwater) volume which must be managed on site *without overflow* :

$\forall = 44.5\ m^3$ from the site in the 100-years design storm.

The gravel-filled "soakaway" has aquifer access, so Procedure 4A may be applied :

Procedure 4A : establish the presence of a readily accessible aquifer; determine recharge rate per bore, q_r m^3/s; use Eqn. (5.9a) :

$$a = \frac{\forall - 60\, q_r\, .\tau}{(e_S\, .H + 60\, k_h\, .\tau .U)}\ m^2 \qquad \text{(single bore case)} \qquad \textbf{(5.9a)}$$

where	e_S	=	void space (e_S = 0.35 for gravel)
	H	=	"soakaway" height (use H = 0.5 m in this case)
	k_h	=	soil hydraulic conductivity (1×10^{-6} m/s); U = 2.0 for clay;
	q_r	=	0.0005 m^3/s (given)
	τ	=	time base of design storm runoff hydrograph (τ = 40 mins in this case).

Application of this formula to the case yields a "soakaway" plan area :

$a = 241\ m^2.$

This area **exceeds** that determined previously, therefore it would over-ride it *but only in the circumstances proposed in this addendum to the earlier example.* Note that emptying time for the alternative design is also satisfactory at 23.4 hours (see Table 3.3).

Important additional note :

Both analyses reviewed above provide dimensions – plan area or length, and depth – of the gravel-filled sub-structure required beneath the porous car park surface based on *hydrological*, only, considerations. In terms of total design of such installations, this represents half of the task. The other 'half' is that determined by the need for structural integrity of the facility under its expected vehicle loads : this activity should be carried out by someone competent in the design of road and highway pavements. The consequences of the two design directions, almost inevitably, are :

- two significantly different sets of basic dimensions for the installation (plan area and gravel depth); and
- two contrasting specifications for the gravel sub-structure.

Study of Example 5.5, above, reveals the hydrological requirements being satisfied by a gravel-filled sub-structure with depth H = 0.50 m located under less than half of the assigned 'hard standing' area. While the 0.50 m thickness of sub-structure may be harmonious with, or even exceed the needs of, structural integrity beneath the porous paved car park surface, it is certain that treatment of the *remaining* area (either case) without support from a gravel-filled sub-structure, would not. Competent structural design of a porous or permeable paved car park calls for substantial sub-structure support under the entire area of loading.

Then there is the issue of gravel Specification. The urban hydrologist may prefer a single-size gravel – to maximise void space; the pavement structural designer is likely to specify a graded mix of gravels for the sub-structure.

The apparent design conflict outlined here can be readily resolved by early consultation between the (design) parties. If the urban hydrologist is advised of the depth of (gravel) sub-structure needed to meet the structural requirements of the project, then this dimension becomes the value of "H" ("soakaway" height) used in the appropriate design procedure (Procedure 4A in the above illustrations). The hydro-logical design then becomes an exercise in checking whether the runoff volume, $\forall$, can be accommodated by a sub-structure system extending beneath the entire (car park) area. The issue of gravel specification is, also, not a cause of conflict. Graded gravel has a void space ratio which is little different from single-size gravel : e_S = 0.30 in this case is an adequate substitution for 0.35 normally used with single-size gravels.

Introduction to Examples 5.6 and 5.7 : OSR Devices for Developed/Re-developed Sites

The next two examples relate to the tasks of, firstly, providing a "leaky" well on-site stormwater retention device for a new, quarter-acre (1,000 m^2) residential development and, secondly, designing a gravel-filled trench (OSR device) for the same site re-developed for dual (site) occupancy. The two examples use different soil types in order to extend the range of case study illustrations. Figure 1.1 is repeated here for ease of access.

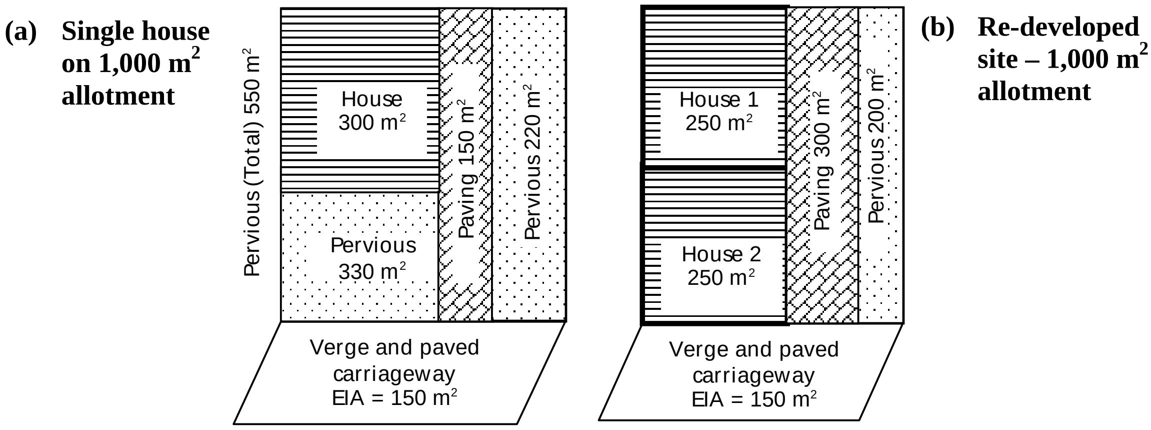

FIGURE 5.5 : Layout of typical development and re-development cases used in Examples 5.6 and 5.7

Example 5.6 : "Leaky" well for residence in new sub-division

The primary stormwater management device to be used on the new domestic site illustrated in Figure 5.5a is a "leaky" well receiving **all roof runoff** (only) from the residence. The allotment size is 1,000 m^2 and the site soil is sandy clay; the site is located in the Parramatta, NSW, region (Toongabbie rainfall). The complete site layout includes paved and pervious areas as well as footpath/nature strip and a segment of the fronting carriageway: the latter were used in the modelling reviewed in Section 1.2. The Council specification requires that the development be designed under the **yield-minimum** strategy (see Sections 4.2 and 4.6) with ARI, Y = 50 years.

Details of the roof area (only) draining to the "leaky" well are as follows :

- total connected roof area .... 300 m²
- site time of concentration, t_C .... 10 minutes
- critical storm duration, $(T_C)_{total}$, Council specification................................. 90 minutes
- design storm intensity (ARI, Y = 50-years, Council specification) .... 50 mm/h

Hence, design runoff volume, $\forall$ = 22.5 m^3 (**yield-minimum** strategy, Council specification).

Procedure 2A : "Leaky" well with cleansed water inflow :

The runoff collected and retained on site is sourced from the roof area only.

$$D = \sqrt{\frac{\forall}{\frac{\pi}{4}\left(H + 120k_h \, . \tau . U\right)}} \text{ m} \qquad \textbf{(5.3a)}$$

[This formula includes the assumption, D ≈ H]

where D = diam of "leaky" well.

H = height of well = 2.0 m

k_h = soil hydraulic conductivity (sandy clay)…3×10^{-5} m/s

τ = $(T_C + t_C)$ = 100 minutes

U = 1.0 for sandy clay.

Hence, D = 3.48 m (too big!).

Re-design, using $\forall$ = 22.5/3 = 7.5 m^3 each

giving 3 wells, each 2.01 m diam.………**USE 3 × 2.10 m diam.**

Check emptying time :

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

= **15.3 hours** (satisfactory, see Table 3.3).

The three "leaky" wells should be located where they can receive **all** roof runoff; they should be placed at least 4 m (clear distance) apart and should be located at least 2.0 m from footings and property boundaries (see Section 1.3.4); they should have overflow provision to street drainage, if possible or, if not, be remote from down-slope properties.

Example 5.7 : Gravel-filled trench for re-developed residential site

A developed residential site identical to that shown in Figure 5.5a, except for soil type and council-specified conditions, is re-developed to provide two residential units as illustrated in Figure 5.5b. It is proposed that a gravel-filled trench be installed receiving storm runoff from the roof area only. The allotment size is 1,000 m^2 and the site soil is heavy clay, characteristic of the Parramatta region (Toongabbie rainfall). The Council specification requires that the re-development be designed under the **regime-in-balance** strategy (see Sections 4.2 and 4.6) with ARI, Y = 100 years. Note that OSR installations in heavy clay soils pose major challenges for the designer (see Section 1.3.4).

Original developed site :

- total connected roof area .... 300 m²
- total connected paved area………………………………………….............. 150 m^2
- total connected pervious area (C_{10} = 0.39)…………………….……….. .... 550 m^2
- verge and paved carriageway (EIA)………………………………............. 150 m^2
- site time of concentration, t_C .... 10 minutes
- critical storm duration, $(T_C)_{total}$, Council specification……………................. 60 minutes
- design storm intensity [ARI = 100-years (F_Y = 1.2) Council spec.]... .... 70 mm/h

Hence, design runoff volume = 60.0 m^3 from **developed** site in 100-years design storm.

Re-developed site :

- total connected roof area……………………………………….. .... 500 m²
- total connected paved area………………………………………............. 300 m^2
- total connected pervious area (C_{10} = 0.39)………………….……..…............ 200 m^2
- verge and paved carriageway (EIA)……………………..…………........... 150 m^2
- site time of concentration, t_C .... 10 minutes
- critical storm duration, $(T_C)_{total}$, Council specification…………….................. 60 minutes
- design storm intensity [ARI = 100-years (F_Y = 1.2) Council spec.] .... 70 mm/h

Hence, design runoff volume = 73.0 m^3 from re-developed site in 100-years design storm.

Hence, design runoff volume, $\forall$ = (73.0 – 60.0) m^3 = 13.0 m^3 (**regime-in-balance** strategy, Council specification).

Procedure 2B : Trench, gravel filled, with cleansed water inflow :

Required length,

$$L = \frac{\forall}{\left[e_S .bH + 60\, k_h .\tau \left(b + \frac{H}{2} \right) U \right]} \text{ m} \qquad \textbf{(5.4a)}$$

The runoff collected and retained on site is sourced *from the roof areas only*.

L	=	length of trench
e_S	=	0.35
b	=	2.0 m
H	=	1.2 m
k_h	=	soil hydraulic conductivity (heavy clay)…1 × 10^{-7} m/s
τ	=	(T_C + t_C) = 70 minutes
U	=	2.0 for heavy clay.

This leads to L = 15.5 m and emptying time, T = 61 days (very unsatisfactory, see Table 3.3).

Various options may be followed to arrive at a satisfactory solution. For example using plastic 'milk crate' units, say 0.4 m deep × 2.0 m wide (see Figure 1.4b) and e_S = 0.95, will give much the same plan area (34.2 m^2 , 17.1 m trench) as the gravel-filled trench, but emptying time will be more than 70 days, which is still very unsatisfactory. The 'limiting case' for use of OSR technology in circumstances such as this, in fact, becomes a form of OSD technology with relatively long emptying time compared with those normally expected of OSD systems. Use is made of Procedures 4A or 4B to achieve a satisfactory outcome -

Procedure 4B : applied to the 'milk crate' device :

STEP 1 : Volume of water held in 17.1 m trench = 17.1 × 2.0 × 0.4 × 0.95 m^3 = 13.0 m^3; Calculate drainage flow, Q_r , to meet emptying time, T = 24 hours (very conservative criterion required by Council which is not bound to accept Table 3.3 suggestions!) :

$$Q_r = \frac{13.0}{24 \times 3600} = 0.00015 \text{ m}^3/\text{s}$$

STEP 2 : Determine L from :

$$L = \frac{\cancel{V} - 60Q_r.\tau}{\left[e_s.bH + 60k_h.\tau(b + \frac{H}{2})U\right]} \text{ m} \qquad \textbf{(5.8a)}$$

with Q_r = 0.00015 m^3/s, and U = 2.0.

STEP 3 : L = 16.2 m, and emptying time = 1.0 day (OK).

The trench should be located where it can receive **all** roof runoff; it may be placed **half** the recommended clearance distance (5.0m) from footings and property boundaries (see "Water-reactivity and 'clearance'" in Section 1.3.4). It should have provision for overflow – meeting Council requirements – and a small diam pipe (e.g. 50 mm PVC) laid from floor level to street drainage or to an urban waterway (see Figure 1.4b, and Sections 3.5.4 and 5.4). If these are not available, a 'final option' is offered in Section 5.5.

Important comment : The re-development scenario reviewed here is common in the older residential suburbs of Australian cities. There are some important lessons which can be learned from a more detailed exploration of the proposed solution, above, as well as from different strategy circumstances which might be applied to the same case :

1. The (**regime-in-balance**) strategy employed in the illustration requires 13.0 m^3 of runoff to be retained on site in the design storm. Part, only, of the *total* storm runoff occurring on the (re-developed) roof area in the design storm (35 m^3) is collected to meet this target, yet this provision takes account of the OSR needs of the **entire** (re-developed) site, not just those of the enlarged roof area. This is not an unusual outcome and is repeated wherever the runoff volume generated on the (re-developed) roof area in the design storm is significantly greater than the "after-minus-before" stormwater volumes [(73.0 – 60.0) m^3 in this case] determined for an altered site.

2. The interpretation of **regime-in-balance** strategy (Council requirement) employed in the illustration treats the 'developed' situation – the single house on the 1,000m^2 allotment and the drainage infrastructure associated with it – as 'acceptable' development dating from the **benchmark year** (see Section 4.6.3). All instances of re-development in the region must therefore incorporate on-site works, such as the 'milk crate' trench used, above, to ensure that the existing (stormwater) infrastructure is not overloaded. Another Council, facing the same re-development scenario, might apply a more fundamental approach to its planning and insist that the benchmark year pre-dates **any** urban development in the region. [Council policy matching this approach may see the developed catchment main drainage line being returned to its pre-development alignment and (natural) channel form, as a long-term goal.] This interpretation of the **regime-in-balance** strategy leads to a requirement in the above illustration for 35 m^3 of on-site retention, not the 13.0 m^3 required in Example 5.7. It is interesting to note that this, also, can be achieved (just!), by collecting all of the roof, only, runoff from the re-development in the design storm.

3. In the event that Council strategy in relation to stormwater management were to favour the **yield-minimum** approach, then the quantity of runoff which would have to be retained on site in the design storm would, of course, be 73.0 m^3 (see above). It is not possible, in these circumstances, to solve the problem of on-site retention requirement sourcing the stormwater from the (re-developed) roof areas alone : these can only provide 35 m^3 of runoff in the design storm (see above). Designers must be vigilant to ensure that in circumstances such as these, they do not 'oversize' installations, that is, **provide greater storage capacity than the catchment component is able to deliver in the design storm** (see Section 1.4.4). Example 5.7 could only be solved as a yield-minimum problem by providing two elements of on-site storage – one receiving runoff from the roof areas (35 m^3 capacity), and, perhaps, two others retaining ground-level runoff, with joint holding capacity of 38 m^3.

5.3 SOME ADDITIONAL DESIGN CONSIDERATIONS

The Procedures detailed above, together with the accompanying notes, guidelines and examples, should settle many of the issues relating to design storm duration, design average recurrence interval (ARI) and emptying time which need to be understood to calculate sizes for 'source control' water-retaining installations. There are a number of practical issues, however, which need to be clarified before the practitioner can proceed with confidence to design installations for any and every situation.

While the following considerations are not comprehensive, they offer some help in finding solutions to commonly encountered problems.

5.3.1 Design modification to meet emptying time criteria

5.3.2 Impact of site constraints and/or economics on design

5.3.3 Combination structures

5.3.4 High groundwater environments

5.4 SLOW-DRAINAGE PIPES : SOME ISSUES AND OPTIONS

5.5 ON-SITE RETENTION (OSR) IN CLAY SOIL SITES : THE FINAL OPTION

5.6 DETENTION-RETENTION : A LAST WORD

[These sections omitted from the 2-Day Workshop Edition]

6. ASPECTS OF STORMWATER QUALITY

6.1 URBAN STORMWATER PARAMETERS AND OTHER FACTORS

6.1.1 Chapter overview

The quality of stormwater present in the urban environment varies from components which are potable – runoff from clean roof surfaces, for example – to runoff from major city traffic lanes and from heavy industry sites, which is likely to contain a mix of urban litter, vegetable matter, sediment and heavy metal contaminants – all in high concentrations. The task of managing this array of components to achieve the goals of WSUD related to water quality issues, requires that :

1. all important parameters and factors used to characterise water quality must be recognised;
2. the characteristics of the full range of urban stormwaters must be identified; and,
3. the question of limits or criteria for acceptance of stormwater elements into downstream uses or environments – with or without pre-treatment – must be answered.

This chapter is divided, broadly, between these three domains with a review of untreated water quality parameters, particularly those relevant to stormwater – physical, chemical and biological properties – examined first. This is followed by a review of the literature on urban stormwater pollution characteristics. The chapter concludes with a wealth of information on standards and guidelines – mainly in tabular form – which may be used to guide practitioners in their endeavours to achieve high standards of performance in urban design which is water-sensitive.

6.1.2 Physical water quality parameters

Temperature : The temperature of a water sample varies regionally, seasonally, diurnally (with night and day) and depending on shade-sunlight availability. In the context of water sensitive urban design, this is particularly relevant to the selection of underground or aboveground rainwater tanks. It affects the dissolved oxygen saturation levels of water, solubility of gases, aquatic fauna and algal growth rates and biological degradation rates (different bacteria species participate in degradation at different temperature ranges).

A phenomenon that may occur in rainwater tanks is thermal stratification whereby heat is absorbed throughout the surface layer of the water, with a cool, stratified layer below that does not allow mixing. This may result in an anoxic layer that permits the mobilisation of nutrients from the benthos layer.

Suspended solids : Suspended solids are defined as "the material that can be removed from a water sample by filtration under standard conditions" (Duncan, 1999). The solids may be organic (for example in the form of plant matter or algae), or inorganic (in the form of clay and silt emanating from increased erosion).

In urban environments, suspended solids are typically between 1 – 50 µm (Duncan, 1999) and may cause blockage to drainage pipes and culverts, for example. Increased turbidity may also result, reducing the light penetration through water and consequently decreasing the amount of light available for aquatic photosynthesis. Of equal concern are the other associated pollutants (such as hydrocarbons, heavy metals and phosphorous) that adsorb onto the surface of the suspended solids.

Turbidity : Turbidity is a measure of the quantity of light either absorbed or scattered by suspended material in water (Horner, 1999). The particle sizes rather than the quantity of sediment determines water sample turbidity. A water sample showing high turbidity may be characterised by having a small concentration of fine particles while another sample with lower turbidity has a higher concentrations of coarse suspended solids. High turbidity reduces the amount of light available for aquatic algae and flora photosynthesis.

Colour : A water sample that has temporary discolouration due to suspended particles has *apparent* colour, while one that has permanent discolouration due to dissolved solids has *true* colour. Many Australian waterways have a yellowish-brown hue due to the humic acid from decaying eucalyptus trees, for example. Water colour pollution may be of particular concern when the interaction of some dissolved solids with chlorine reduces its effectiveness as a disinfectant or facilitates the formation of by-products that may have an objectionable taste or are carcinogenic (e.g. Trihalomethane –THM).

Odour and taste : Biological by-products, metals and minerals may produce an odour or taste in water (Horner, 1999). In the case of inorganic substances, the result is usually the production of taste without odour, while organic substances may cause both. For example, the biological decay of organisms sometimes causes a sulphuric ("rotten egg") odour and/or taste to the water, alkaline water may have a bitter taste or metal and metallic salts may produce a bitter or salty taste.

6.2 CHEMICAL WATER QUALITY PARAMETERS

Oxygen : Dissolved oxygen (DO) is defined as the amount of free oxygen, or oxygen present excluding that which is bound by compounds such as H_2O, CO_3 and NO_3, in a water sample. It is an essential component required for metabolic biological processes (DLWC, 1998).

The dissolved oxygen concentration in a sample is limited to its saturation point which is dependent on a number of factors including :

- temperature – as temperature increases, the DO saturation point decreases;
- atmospheric pressure - as atmospheric pressure increases, the DO saturation point increases;
- salinity – as total dissolved solids increases, the DO saturation point decreases.

The difference between the actual dissolved oxygen concentration in a water sample and the dissolved oxygen saturation point is referred to as the 'dissolved oxygen deficit'.

The sources of dissolved oxygen in a water sample include :

- Algal photosynthesis : The rate of which is affected by the amount of light available for photosynthetic processes. Factors affecting the available light include the turbidity of the water, the position of the sun due to geographic location or change in seasons and the position of structures blocking sunlight penetration such as trees, buildings and fences.
- Atmospheric re-aeration : The dissolution of oxygen in the atmosphere to the water surface. This is influenced by the dissolved oxygen deficit (and thus water temperature) and the relative turbulence of the water surface (affected by the water depth and velocity as well as the presence of wind and hydraulic structures). Dissolved oxygen contained in incoming flows is also a source of dissolved oxygen in water bodies.

The sinks of dissolved oxygen include biochemical oxygen demand (BOD), sediment oxygen demand (SOD) and the dissolved oxygen deficit of incoming flows. Biochemical oxygen demand is the measure of the oxygen requirement of a mixed population of aerobic bacteria in oxidising the biodegradable organic matter in water (Horner, 1999). This is sometimes separated into carbonaceous BOD for all carbon oxidisation and nitrogenous BOD for the oxygen required for nitrification. It is a process that ultimately increases the dissolved oxygen deficit. Sediment oxygen demand is the measure of oxygen required to oxidise sediments and organisms.

The testing of oxygen demand in waters may be unreliable if high concentrations of heavy metals exist as they inhibit the growth of bacteria (Schueler, 1987).

The combined effects of the oxygen sources and sinks described above are competing processes that are simultaneously adding and removing oxygen from water. Initially, oxygen-demanding wastes are oxidising usually at a faster rate than the combined actions of re-aeration. As the oxygen-demanding wastes are oxidised and the available dissolved oxygen is reduced, the oxidisation rate gradually slows so that eventually re-aeration rate is greater than the oxidisation (see Figure 6.1).

It is, however important to understand the limitations of the DO sag model. The model, for example does not take into account factors such as differential temperatures and light conditions between day, night and times of the year. It also does not take into account the effects of pollutants that are toxic to biota, such as heavy metals, that may affect oxidisation or photosynthesis rates in water.

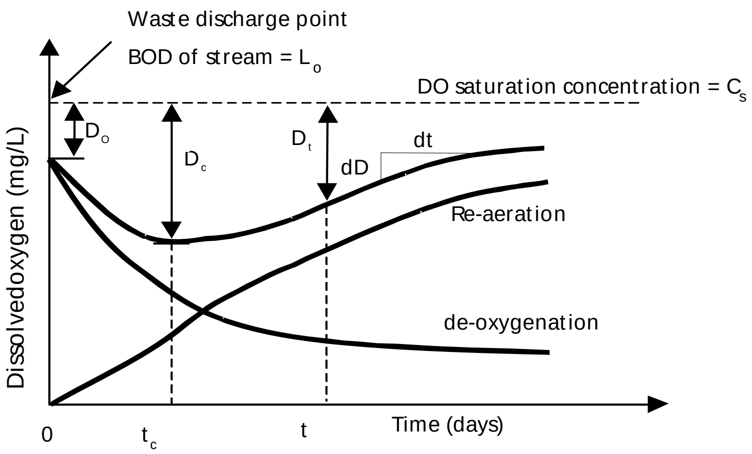

FIGURE 6.1 : Dissolved oxygen sag curve

Nitrogen : Nitrogen is the most common atmospheric gas and a major constituent of living organisms. In the aquatic environment, however, very little nitrogen is utilised (DLWC, 1998).

Nitrogen occurs in many forms, some of which are highly toxic with varying stabilities and oxidisation rates. Some of the more important forms of nitrogen are :

- Organic nitrogen (–N) : often in the form of urea, amino acids, or amines.
- Dissolved ammonia (NH_4^+) : the prevalent form of ammonia at pH7 (DLWC, 1998).
- Ammonia gas (NH_3) : the prevalent form of ammonia at pH12 (DLWC, 1998). At concentrations greater than 0.2 mg/L ammonia is toxic to aquatic life.
- Nitrite (NO_2^-) : an intermediate form in the nitrogen cycle that is toxic.
- Nitrate (NO_3^-) : essential nutrient for aquatic plant growth. In high concentrations (>10 mg/L), nitrates may cause the potentially fatal methemoglobenemia or "blue baby" syndrome.
- Nitrogen gas (N_2) : approximately 80% of atmosphere is nitrogen gas.

6.2.1 The nitrogen cycle

Organic Nitrogen : Amines are separated from particulate organic nitrogen (Org-N) that is produced from the expiration of biomass by heterotrophic bacteria, releasing ammonia nitrogen ($NH4^+$). Particulate organic nitrogen that is not converted to ammonia in this manner settles to the benthos layer and may later be converted to ammonia by 'benthic uptake', which is the oxidisation of ammonia from the benthos layer.

Ammonia Nitrogen : Sources of ammonia nitrogen include hydrolysis of organic nitrogen, benthic uptake and other external inputs including animal faeces and sewer overflows.

Nitrification : Once the above processes have formed ammonia, nitrification occurs in two steps. The first is the oxidisation of ammonia to nitrite in an aerobic process by nitrosomonas bacteria.

$$NH_4^+ + O_2 \xrightarrow{\text{nitrosomonas}} H^+ + NO_2^- + H_2O$$

An important aspect of nitrification is that the nitrosomonas bacteria that perform the oxidisation must compete with heterotrophic bacteria for the oxygen required for these reactions. Thus dissolved oxygen availability is a dependent variable in the process. The other major dependent variables in the process are temperature and retention time (DLWC, 1998). Reduced alkalinity of the water may occur as the hydrogen ions (H^+) produced in this process react with naturally occurring carbonates.

The nitrite produced is then in turn oxidised by nitrobacter bacteria to nitrate form.

$$NO_2\text{-} + O_2 \xrightarrow{\text{nitrobacter}} NO_3\text{-}$$

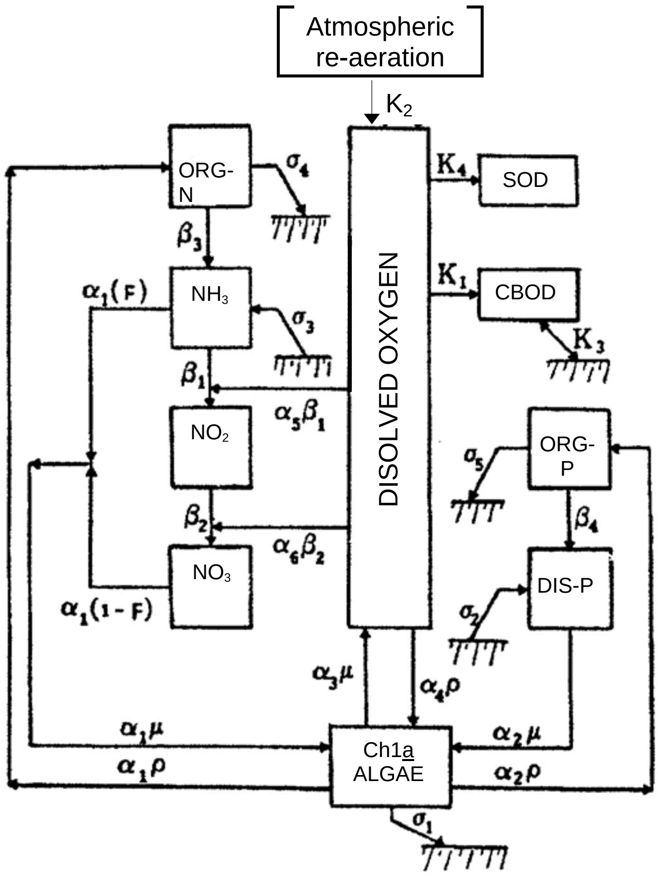

FIGURE 6.2 : Schematic view of the interrelations and oxygen processes used in QUAL2E (US EPA, 1987)

where	α_1	=	Fraction of algal biomass that is nitrogen (mg-N/mg-A)
	α_2	=	Fraction of algal biomass that is phosphorous (mg-P/mg-A)
	α_3	=	Benthos source rate for ammonia nitrogen (mg-N/ft^2/day)
	α_4	=	O_2 production per unit of algal growth (mg-O/mg-A)
	α_5	=	O_2 uptake per unit of NH_3 oxidation (mg-O/mg-N)
	α_6	=	O_2 uptake per unit of NO_2 oxidation (mg-O/mg-N)
	β_1	=	Rate constant for the biological oxidation of NH_3 to NO_2 (day^{-1})
	β_2	=	Rate constant for the biological oxidation of NO_2 to NO_3 (day^{-1})
	β_3	=	Rate constant for the hydrolysis of organic-N to NH_3 (day^{-1})
	β_4	=	Rate constant for the decay of organic-P to dissolved-P (day^{-1})
	μ	=	Algal growth rate (day^{-1})
	ρ	=	Algal respiration rate (day^{-1})
	σ_1	=	Algal settling rate (ft/day)
	σ_2	=	Benthos source rate for dissolved phosphorous (mg-P/ft^2/day)
	σ_3	=	Benthos source rate for ammonia nitrogen (mg-N/ft^2/day)
	σ_4	=	Organic nitrogen settling rate (ft/day)
	σ_5	=	Organic phosphorous settling rate (ft/day)
	F	=	Fraction of algal nitrogen uptake from the ammonia pool
	K_1	=	Carbonaceous deoxygenation rate constant (day^{-1})
	K_2	=	Re-aeration rate constant (day^{-1})
	K_3	=	Rate of loss of BOD due to settling (day^{-1})
	K_4	=	Benthic oxygen uptake (mg-O/ft^2/day).

Denitrification : Denitrification is the anoxic process of transformation of nitrate into gaseous forms in the benthos. It is utilised by denitrifying bacteria including *pseudomonas sp.* and is dependent on the presence of nitrate, anoxic conditions and a readily biodegradable carbon source.

$$NO_3^- + Org - C \xrightarrow{\textbf{denitrifying}} N_2\ (NO\ \&\ N_2)_{(G)} + CO_{2(G)} + H_2O$$

where $_{(G)}$ refers to a substance in its gaseous state.

Due to its dependence on anoxic conditions, denitrification generally occurs after sedimentation of the nitrate in the benthic layer. It also increases the acidity of the water.

Phosphorous : Phosphorous occurs in water in two forms – organic (e.g. nucleic acids, proteins) and in-organic dissolved phosphorous (polyphosphates often found in domestic detergents and orthophosphates that participate in the metabolism of organisms including algae).

Some phosphorous originates from natural sources such as biomass expiration and geological weathering and dissolution while most in the urban environment comes from sewer overflows, animal faeces, agricultural fertilisers and other effluent discharges including those from industry.

6.2.2 The phosphorous cycle

Organic Phosphorous : Organic phosphorous, from the sources listed above, may be transformed to inorganic dissolved phosphorous by bacterial decomposition, or may be adsorbed onto the surface of sediments that settle to the benthos layer.

Inorganic Dissolved Phosphorous : Aquatic biota including bacteria, algae, fungi and aquatic plants subsequently take up the phosphorous and assimilate it into their cells. In acidic conditions, insoluble phosphates of aluminium, calcium, iron or magnesium may be formed and precipitate with sediment to the benthos layer (DLWC, 1998). The phosphorous cycle is completed with the expiration of biota. This again provides organic phosphorous as a nutrient source.

pH : The pH parameter refers to the concentration of hydrogen ions in solution, with a value of 7 being neutral at normal temperatures. The pH quoted is actually the negative base 10 logarithm of the hydrogen ion activity ($-\log_{10} [H^+]$), with most raw water sources having a pH of between 6.5 to 8.5 (Duncan, 1999). A water body's pH level does not directly affect water quality, but it is a determining factor in almost every natural treatment process (Horner, 1999). Nitrifying bacteria, for example, operate at maximum efficiency in the range $7.5 < pH < 8.5$.

Levels of pH are usually determined using a pH meter that measures the voltage produced by a glass electrode in the specimen (the voltage produced in the electrode varies linearly with pH).

Alkalinity : The alkalinity parameter refers to the concentration of anions present that neutralise hydrogen ions in solution (acidity). Some of the anions present in natural waters include carbonates, bicarbonates, silicates, phosphates, sulphides and ammonia. The main water quality problem with alkalinity is that the reactions with cations can form precipitates. Alkalinity is expressed in milligrams per litre of $CaCO_4$ with typical concentrations in drinking water ranging from 5 to 125 mg/L.

The alkalinity of a water sample is determined by titriting the specimen using strong acids and measuring its pH as outlined above.

Heavy Metals : Heavy metals are all metals above calcium (Ca) in the periodic table of elements and have a specific weight higher than 5. They are often very toxic and, studies have shown (DePinto et al, 1980) that the density of benthos macroinvertibrates are related to sediment concentrations of several heavy metals. Because heavy metals are totally non-degradable, they accumulate in the aquatic environment over very long periods of time.

Some of the main heavy metals encountered in stormwater are described in more detail below.

Lead (Pb) : Occurs both in a particulate and dissolved form [but is mainly associated in the particulate form, adsorbed onto suspended solids in runoff (Duncan, 1999)] from sources such as petrol additives, motor vehicle tyre wear, industrial waste, soldered pipe joints and roof flashing. Lead bio-accumulates in the tissue of animals, plants and bacteria and if consumed in high concentrations by humans can cause developmental retardation in infants, brain damage in adults, convulsions, behavioural problems or in extreme cases, death.

Zinc (Zn) : An essential element in human growth, zinc is readily bio-accumulated in the tissue of aquatic animals and plants. Usually in the from of dissolved solids in runoff (Duncan, 1999) zinc will sometimes adsorb onto suspended solids and colloidal particles. Typical sources of zinc in the urban environment include motor vehicle tyre and brake pad wear, combustion of oils and the corrosion of various galvanised metals.

Copper (Cu) : Copper is an essential element aiding in human metabolism that in very large doses may lead to widespread irritation. Dissolved copper imparts undesirable taste and colour on drinking water and is also easily bio-accumulated in the aquatic environment. In the urban environment copper originates from motor vehicle tyre wear, corrosion of rooves, water pipes and from fungicides and pesticides.

Cadmium (Cd) : A highly toxic heavy metal, cadmium has been associated with food poisoning, cardio-vascular disease and some cancers affecting humans. It accumulates mainly in the liver and kidneys of animals (including humans) and in the aquatic environment is found mainly in shellfish. In stormwater, cadmium is mostly present in dissolved form (Duncan, 1999). Sources of cadmium include motor vehicle tyre wear, combustion of oils, industrial waste, fertilisers, pesticides and the corrosion of galvanised metals.

Chromium (Cr) : Chromium occurs in two forms, trivalent (an essential mineral for human metabolism) and hexavalent (the predominant form in aerated waters that is associated with kidney and liver damage, gastrointestinal irritations and an increased risk of cancer). Chromium emanates from the wear of moving parts in engines, paints, ceramics, corrosion inhibitors, fertilisers and pesticides.

Iron (Fe) : Iron is an essential mineral for human nutrition that occurs in runoff in two main forms. The divalent (ferrous) state that is highly soluble and the trivalent (ferric) state that occurs in oxidising conditions that is only soluble at low pH levels. Iron is usually present adsorbed onto suspended solids with sources including the corrosion of motor vehicles, landfill leachate and the corrosion of service pipelines. In large doses, iron may be toxic to some fish and invertebrate species.

Hydrocarbons : Hydrocarbons are organic compounds of carbon and hydrogen that are typically present in roads and car park runoff in the petrochemical form originating from lubricating oils, petrols and hydraulic liquids. Some hydrocarbons, such as bitumen become more dense than water when affected microbially, but most remain less dense than water and as such form sheens on the surface of receiving waters. Some hydrocarbons have been shown to be toxic to aquatic fauna.

Approximately 70 - 75% of hydrocarbon oils display a strong attraction to suspended solids with suspended solids removal resulting in hydrocarbon concentration reduction. Methyl-tertiary-butyl-ether (MTBE), a common additive in unleaded petrol, is more soluble than most hydrocarbons while polynuclear aromatic hydrocarbons (PAHs) display extremely strong attractions to sediment.

Hydrocarbon degradation is facilitated by microbial and oxidative processes that have a biochemical oxygen demand.

6.2.3 Biological water quality parameters

Pathogens : A pathogen is a micro-organism (bacteria, virus, protozoa or helminth) that causes waterborne disease (DLWC, 1999). Typical sources of pathogens in the urban environment are animal (including human) faeces and other decaying biological matter. They may be suspended in the water or adsorbed onto the surface of particulate matter. Some of the typical pathogens found in the urban environment are listed in Table 6.1.

Bacteria : Due to the large variety of species of bacteria that require different procedures to identify, specific species are chosen as indicator organisms. Faecal pollution, for example may be monitored by detecting Escherichia coli (E. Coli) bacteria for long-term faecal pollution and faecal streptococci for short-term.

TABLE 6.1
TYPICAL PATHOGENS EXCRETED IN HUMAN FAECES
[Masters, 1998]

PATHOGEN GROUP AND NAME	ASSOCIATED DISEASE
Virus	
Adenoviruses	Respiratory, eye infections
Enteroviruses :	
▪ Polioviruses	Aseptic meningitis, poliomyelitis
▪ Echoviruses	Aseptic meningitis, diarrhoea, respiratory infection
▪ Coxsackie viruses	Aseptic meningitis, herpangina, myocarditis
Hepatitis A viruses	Infectious hepatitis
Bacteria	
Salmonella typhi	Typhoid fever
Salmonella paratyphi	Paratyphoid fever
Other salmonellae	Gastroenteritis
Shigella sp.	Bacillary dysentery
Vibrio cholerae	Cholera
Other vibrios	Diarrhoea
Yersinia enterocolitica	Gastroenteritis
Protozoa	
Entamoeba histolytica	Amoebic dysentery
Giardia lamblia	Diarrhoea
Cryptosporidium	Diarrhoea
Helminth	
Ancylostoma duodenale	Hookworm
Ascaris lumbricoides	Ascariasis
Nymenolypis nana	Hymenolepiases
Necator americanus	Hookworm
Strongyloides stercoralis	Strongyloidiasis
Trichuris trichiura	Trichuriasis

Protozoa : There are over 50,000 species of protozoa, approximately one fifth of which are parasitic. They infect vertebrates and invertebrates and some are even parasitic in plants. Parasitic protozoa are, in general, small, have short generation times, high rates of reproduction and a tendency to induce immunity to reinfection in those hosts that survive. Protozoa include flagellates, amoeba and ciliates.

The most notorious protozoa in Australian waterways are giardia and cryptosporidium. *Giardia intestinalis*, which is common in the small intestine, is a parasite that has a pair of adhesive suckers which gives it a characteristic appearance. It attaches to the cells of the gut using the suckers and divides by binary fission, in this way huge numbers can build up in the intestine. Giardia are spread by resistant cysts, each of which contains a pair of parasites that causes disease symptoms such as diarrhoea, vomiting and loss of weight.

Cryptosporidium parvum is a parasite found in a range of vertebrates including cows, sheep, rodents, cats, dogs and humans. Many people are infected as children and develop a life-long immunity. Cryptosporidium cysts are so small and resistant that they are not removed by conventional water treatment. Cryptosporidium is a major problem in immuno-suppressed people, including AIDS sufferers and transplant patients.

Viruses : Viruses are parasitic organisms that are dependent on a living host for reproduction and long-term survival. Although they have a relatively short life span in water systems, they are capable of causing significant health risks in recently contaminated waters, while protozoa are unicellular organisms that can exist as parasites or independently. Few protozoan species are pathogenic and those that are may cause gastrointestinal infections in milder forms than that caused by bacteria.

Helminths : Commonly called parasitic worms, helminths originate from human or other animal waste and multiply under aerobic conditions.

6.3 CHARACTERISTICS OF URBAN STORMWATER POLLUTION

6.3.1 Effect of urbanisation on runoff quality

Urban stormwater is an extremely variable resource in terms of both quantity and chemical characteristics. It is defined as “pure rainwater runoff from urban areas including anything the rain carries along with it”. (NSW EPA, 2001)

Urbanisation compromises runoff water quality by making a wider range of pollutants more immediately available to surface runoff. Changes to the natural water balance as a result of urban activity exacerbate this effect.

In an area of undeveloped woodland the natural water cycle processes involve significant proportions of rainfall infiltrating the soil. Some is released back into the atmosphere by evaporation and transpiration and generally a small amount (10% – 20% of annual rainfall input) runs off into creeks or streams.

The activities of urbanisation turn this water balance around. Much less water finds its way into the soil, and the amount of water which runs off is significantly increased. Differences between undisturbed watershed and urban characteristics affect both the speed and volume of runoff events. Even small events of low intensity can result in runoff when natural landscape is replaced by high proportions of impervious area. Depending upon the land use, urbanisation will increase the runoff coefficient by up to 600% [based on undeveloped runoff coefficient of 0.15 increasing to 0.9, representing commercial or industrial development (Argue, 1986)]. As a result, runoff responds much more directly to rainfall. Discharges reach higher peaks more rapidly, resulting in higher flow/velocity runoff. While there is wide agreement that developed watersheds show an increase in peak discharge rate, the extent of this increase is less certain.

In the “Watershed Restoration Sourcebook”, prepared by the Department of Environmental Programs in Washington (Schueler, 1992b), it is claimed that the post-development peak discharge rate may increase by a factor of five times from the pre-development rate, while the National Water Quality Management Strategy (ANZECC 2000) and the Commonwealth EPA (1993) quote ten- and twenty-fold increases respectively.

The effects of these ‘quantitative’ changes have serious impacts on consequent water quality. Higher runoff velocity tends to increase mobilisation of accumulated pollution/sediment on road surfaces and parking areas, and intensifies erosion of soil from land surfaces and from streambeds and banks. Less rain is infiltrated where it falls and instead, it is conveyed to pipes and then on to urban creeks and waterways. This leads to accumulation and concentration of pollution, seriously degrading receiving waters and sensitive water-associated environments.

6.3.2 Sources of pollution

The potential exists for rainwater falling in urban areas to accumulate pollutants throughout the journey from atmosphere to final receiving waters. Even before it falls as rain, water vapour will “wash out” contaminants from the atmosphere. It has been observed that the deposition rate of atmospheric pollutants during dry periods is considerably less than wash out rates during precipitation events (Gutteridge, Haskins & Davey, 1981). Early Swedish studies (Malmquist, 1978; Goettle, 1978) reported that the atmospheric contribution to total pollutant concentrations in stormwater are considerable, for example, 20% organic matter, 70% total nitrogen, 25% total phosphorous and 7 – 40% heavy metals. In fact, in the case of nitrogen, rainfall concentrations have been reported to be higher than those in the stormwater flow itself, indicating that urban areas may be acting as a ‘sink’ for nitrogen (Randall and Grizzard, 1981). Clearly, this is a site specific phenomenon, related to the agricultural and industrial activity in the surrounding regions.

Most runoff from urban areas will contain pollution which has accumulated on roads and other impervious surfaces. Vehicular traffic is the most significant contributor to accumulated road contaminants. Old, poorly maintained vehicles are a particularly high source, as they will, in general, be characterised by higher oil and fluid leaks, leaded fuel usage and generally inefficient combustion. The following table (Table 6.2) from U.S. data (Maestri et al, 1985; US EPA, 1993) describes a number of highway-associated runoff contaminants and their particular sources. Some of these, such as de-icing salts and PCB spraying of highways, are not significant sources in Australian conditions.

TABLE 6.2

HIGHWAY RUNOFF CONSTITUENTS AND THEIR PRIMARY SOURCES

CONSTITUENTS	PRIMARY SOURCES
Particulate	Pavement wear, vehicles, atmosphere, maintenance
Nitrogen, phosphorous	Atmosphere, roadside fertiliser application
Lead	Leaded gasoline (auto exhaust), tyre wear (lead oxide filler material), lubricating oil and grease, bearing wear
Zinc	Tyre wear (filler material), motor oil (stabilising additive), grease
Iron	Auto body rust, steel highway structures (guard rails, etc.), moving engine parts
Copper	Metal plating, bearing and bushing wear, moving engine parts, brake lining wear, fungicides and insecticides
Cadmium	Tyre wear (filler materials), insecticide application
Chromium	Metal plating, moving parts, brake lining wear
Nickel	Diesel fuel and gasoline (exhaust), lubricating oil, metal plating, bushing wear, brake lining wear, asphalt paving
Manganese	Moving engine parts
Cyanide	Anti-caking compounds to keep de-icing salts granular
Sodium/Calcium chloride	De-icing salts
Sulphate	Roadway beds, fuel, de-icing salts
Petroleum	Spills, leaks or blow-by of motor lubricants, anti-freeze and hydraulic fluids, asphalt surface leachate
PCB	Spraying of highway rights of way, background atmospheric deposition, PCB catalyst in synthetic tyres.

Source : Maestri et al, 1985; US EPA, 1993

Sewer overflows and septic tank seepage are sources of faecal bacteria in urban stormwater runoff, as are animal and pet wastes: fines imposed on irresponsible pet owners are aimed at reducing the impact of this latter source. Tree-lined streets contribute significant loads of organic, oxygen demanding material by way of decaying vegetation. Gross litter accumulates on most urban pervious and impervious surfaces and is readily mobilised and added to the stormwater pollutant load.

Construction activity is a major source of pollution in developing areas contributing dust and a variety of sediment and suspended solids. Improper storage or spillage of toxic chemicals from industrial sites can contaminate stormwater runoff from these areas. Other sources of contamination include lawns, gardens and parks which may contribute pesticides, herbicides and fertilisers, and illegal dumping of waste.

6.3.3 Factors influencing the degree of contamination

The composition of stormwater runoff generated from an urban area will vary significantly from catchment to catchment. The processes involved in the generation of pollutant loads are complex and large spatial and temporal variations persist down to small scales. One factor in determining the quality of stormwater is clearly the presence or absence of various sources, as listed previously.

Given that at least some of these pollution sources are present, a number of other factors also contribute to the level of contamination contained in runoff during any particular event. Also, the proportion of impervious surfaces and the general topography will affect the velocity of stormwater conveyance. This, in turn, affects levels of pollutant mobilisation through erosion and re-suspension (or ‘wash-off’). Similarly, the duration and intensity of the rain plays a role in determining what percentages of those pollutants present will be suspended and hence transported. The time between storm events is significant in relation to some pollutants : for example, longer antecedent dry periods will allow more time for pollutant build-up on runoff surfaces.

Other considerations when assessing the degree of contamination include seasonal factors (e.g. the rate at which leaf litter accumulates on catchment surfaces), the presence of any pollution control measures already in place and the extent of local community interest in environmental and catchment issues.

Even within a given storm event there will be variation in the runoff water quality. The most recognised effect is that of the 'first flush', a feature of wash-off related to both storm and catchment characteristics. The first flush effect refers to a pollutant concentration peak which precedes the flow peak, and is most recognisable in small catchments. First flush effects have been observed for suspended solids, hydrocarbons and heavy metals as well as dissolved pollutants (Peterson and Batley, 1992). Results from a study of 13 sites in and around Melbourne confirmed higher concentrations in initial runoff flows. In heavy storms, up to 48% of pollutants by mass were found in the first 37% of the runoff (Gutteridge, Haskins & Davey, 1981). A first flush effect is not always significant, and in certain hydrological or catchment conditions may not be recognisable. For example, climatic regions characterised by frequent, low intensity storms which continually wash surfaces may present little evidence of first flush (Stockdale, 1991).

6.3.4 Contaminants and some of their impacts

Contaminants in urban stormwater can be categorised as :

- Gross litter
- Suspended solids
- Oil and grease
- Nutrients
- Oxygen demanding materials (organic matter)
- Bacteria/micro-organisms
- Heavy metals and other toxicants.

Gross litter comprises a significant portion of stormwater pollution. By definition, gross pollution as it is referred to here is everything greater than 4 mm in diameter, including paper, plastics, organic litter, packaging materials, glass and metal. The main impact of gross pollution is loss of aesthetic qualities and potential for injury when in contact with the runoff. Gross pollution may serve as a visual indicator for the many invisible contaminants most likely present when gross litter is a problem.

Suspended solid levels in urban runoff are predominantly due to inorganic soil particles. Suspended sediment causes a variety of adverse consequences ranging from increased turbidity and reduced light penetration to direct interference with aquatic and sediment dwelling (benthic) life. It is widely recognised that many other stormwater pollutants are associated with suspended material, particularly heavy metals. Contaminants become bound to the smaller sediments in particular and are thus transported in the stormwater from catchment areas to waterways.

Oil and grease derive mainly from motor vehicles, and hence are associated with roads, parking bays and service stations. Some of these hydrocarbon compounds are known to be toxic to aquatic life at relatively low concentrations. They may also exert a detrimental effect on aquatic life by interrupting the entry of oxygen as they form a film on the water surface. Aromatic hydrocarbons are present in most vehicle exhaust emissions, and have been identified as being carcinogenic. Once bound to sediments, these aromatic hydrocarbons present long term dangers due to their high toxicity.

Stormwater nitrogen and phosphorous levels, when in excess, can have significantly damaging impacts on receiving waters. Because the majority of nutrient load is present in a soluble form it is easily taken up by algae. When favourable conditions exist, algal blooms can result, severely depressing oxygen levels (during decomposition) and releasing toxins resulting in fish kills and potential human hazard (Schueler, 1987).

Bacteria levels in stormwater generally exist far in excess of water quality standards for drinking, irrigation or contact beneficial uses (Schueler, 1987; Dillon and Pavelic, 1996). Originating from human and animal faecal matter, the main indicator is faecal coliform bacteria. Levels are exacerbated by warm conditions and sewer overflows during large events.

A wide range of heavy metals can be present in urban runoff. Most are highly toxic at small concentrations and can accumulate in the body to threshold levels. Within stormwater they pose a threat to humans both via direct contact and via accumulation in the food chain. Metals found in stormwater include Arsenic, Cadmium, Chromium, Copper, Cyanide, Lead, Nickel and Zinc. Heavy metals must be chemically available to present a toxic hazard, though all metals, whether bound to sediments or not, are potentially toxic, and slight changes in sediment chemistry could mobilise previously harmless contaminants.

6.3.5 Data on pollution concentrations in urban runoff

Relatively little data are available for use in the prediction of stormwater pollution concentrations based on catchment characteristics. From existing data it is apparent that pollution concentrations are extremely variable in the urban setting (Dillon and Pavelic, 1996; Schueler, 1987; Commonwealth EPA, 1993). A number of studies have been carried out throughout Australia and the world which attempt to correlate stormwater pollution with land use and management factors. What follows is by no means a comprehensive review of existing studies in the area, but rather a small number of examples demonstrating the variety of results and hopefully highlighting the variability of conclusions.

Schueler (1987) writing for the Washington Metropolitan Water Resources Planning Board in 1987 prepared a manual in which he makes reference to a "Simple Method" for estimating pollutant export from urban development sites. The empirical approach is based on data coming from studies in Washington and throughout the U.S.A. True to its name, the method is used to determine pollutant export (mass) based on rainfall, runoff coefficients and a mean concentration of pollutant in urban environments (averaged from empirical studies). These mean concentrations of various pollutants, named "C" values, are based on statistical analysis of over 300 runoff events in Washington and over 2,300 events at 22 sites drawn from a much larger national database (Schueler, 1987). An interesting conclusion from the study was that there was no significant difference in average pollutant concentrations between the eight widely different urban sites measured within Washington D.C.: also, no consistent correlations between pollution concentrations and storm volume or intensity were found (Schueler, 1987). However, there were significant differences between pollutant levels in relatively recent urban development sites and those in older, poorly maintained urban neighbourhoods. Table 6.3 summarises results from these studies, categorised into broad watershed conditions.

A brief review of other studies carried out in America can be found in New Techniques for Modelling the Management of Stormwater Quality Impacts (Liesko et al, 1993).

Due to specific regional hydrology and climatic characteristics it may be inappropriate to simply apply international data concerning stormwater pollution for use in Australia. A review of stormwater quality studies performed by Dillon and Pavelic (1996) examined 14 published studies from 18 sites, two thirds of which were from Australia. They acknowledge that "concentrations across the sites vary significantly", though still consider it useful to make some general comments in relation to existing guidelines.

Parameters which did not meet **drinking water** guidelines on all occasions include Ammonia, Aluminium, Cadmium, Chromium, Iron, Lead, Turbidity and Faecal Coliforms. Parameters which did not always meet **irrigation** guidelines include Cadmium, Copper, Iron, Lead, Zinc and Faecal Coliforms. A summary table of results for various parameters is reproduced in Table 6.4, detailing ranges and mean concentrations with comparison to recommended guidelines for drinking, irrigation and livestock. However, rather than providing firm conclusions, the greatest value of such a review is to "point out what we do not know about the quality of urban stormwater" (Dillon and Pavelic, 1996).

Another thorough review of Australian and international (mainly US) studies was provided by CSIRO in 1992, focusing primarily on roadway runoff quality. It was concluded that lead, zinc and copper are the pollutants of major concern, comprising 90 - 95% of all metals observed (Peterson and Batley, 1992). Once again however, there are large variations in pollutants, particularly heavy metals, apparent from analyses at different sites around the world, as emphasised in 6.5 showing comparative metal concentrations. Peterson and Batley suggest that these variations are strongly related to the range of catchment characteristics, site activities and traffic flows.

Cadmium levels are virtually invariant across the range of sites, which include light industrial, residential, commercial and agricultural catchments. This suggests the possibility of a more remote source, perhaps entering the stormwater via atmospheric wash-out. Zinc is generally present in the highest concentrations, and in particular is associated with commercial centres and busy intersections. Lead and copper also follow this trend, indicating motor vehicles as the major source. The two catchments with housing only presented comparatively low levels of all heavy metals.

TABLE 6.3

MEAN CONCENTRATIONS OF POLLUTANTS IN URBAN ENVIRONMENTS
(Source : Schueler, 1987)

POLLUTANT	NEW SUBURBAN NURP SITES (WASH., DC) mg/L	OLDER URBAN AREAS (BALTIMORE) mg/L	CENTRAL BUSINESS DISTRICT (WASH., DC) mg/L	NATIONAL NURP STUDY AVERAGE mg/L	HARDWOOD FOREST (NORTHERN VIRGINIA) mg/L	NATIONAL URBAN HIGHWAY RUNOFF mg/L
Phosphorous : Total	0.26	1.08	-	0.46	0.15	-
Ortho	0.12	0.26	1.01	-	0.02	-
Soluble	0.16	-	-	0.16	0.04	0.59
Organic	0.10	0.82	-	0.13	0.11	-
Nitrogen : Total	2.00	13.6	2.17	3.31	0.78	-
Nitrate	0.48	8.9	0.84	0.96	0.17	-
Ammonia	0.26	1.1	-	-	0.07	-
Organic	1.25	-	-	-	0.54	-
TKN	1.51	7.2	1.49	2.35	0.61	2.72
COD	35.6	163.0	-	90.8	>40.0	124.0
BOD (5 day)	5.1	-	36.0	11.9	-	-
Metals : Zinc	0.037	0.397	0.250	0.176	-	0.380
Lead	0.018	0.389	0.370	0.180	-	0.550
Copper	-	0.105	-	0.047	-	-

The critical difficulty in using data such as these to establish and validate runoff and pollutant load models is the high number of model parameters required. In their summary of Australian research in this field, Allison and Chiew (1995) come to the following conclusion :

> *"It is conceivable that there may already be sufficient data to provide rough estimates of pollutant loads generated from large urban areas. However, because of the large variability in pollution characteristics, specific monitoring in the area of interest may always have to be carried out if detailed information on the pollution characteristics for the area is required."*

6.3.6 Conclusion

Urban areas inevitably generate considerable amounts of undesirable waste products, or pollutants, which contaminate runoff. Due to the nature of Australian conventional stormwater management systems being efficient at collection and local disposal, with little or no pre-treatment, these contaminants pose a growing threat to our immediate environment. Understanding the sources of pollution and the mechanisms which drive pollutant build-up, wash-off and transport will help focus attempts to alleviate the problem. To date, research has revealed the variable nature of urban stormwater contaminants. Heavy metals are a focal point, due to their relatively high concentrations and potential toxicity. Further research must pinpoint where critical pollutants are located within the 'pollution column' and hence suggest strategies for control. This will almost inevitably lead to changes at all levels of collection, transport and subsequent disposal or use of stormwater.

TABLE 6.4
TYPICAL CONCENTRATIONS OF CONTAMINANTS IN STORMWATER AND WASTEWATER AND ASSOCIATED GUIDELINES FOR VARIOUS BENEFICIAL USES

PARAMETER	NWQMS GUIDELINES				STORMWATER		WASTEWATER		STORMWATER		WASTEWATER	
									% ANALYSES THAT MEET GUIDELINES			
	DRINKING	RAW DRINKING	IRRIGATION	LIVESTOCK	RANGE	MEAN	RANGE	MEAN	DRINKING	IRRIGATION	DRINKING	IRRIGATION
pH	6.5 – 8.5	6.5 – 8.5	6 - 8.5 groundwater 6 – 9 surface water		6.7 – 8.5[10]	7.7	6.9 – 8.7[10]	7.6	100	100	90	100
EC (μS/cm)	mg/L	mg/L	mg/L	mg/L	mg/L 197 – 8140[7]	mg/L 3250	mg/L	mg/L				
TDS	500[a]	1000	Refer to Section 9.2.3 ANZECC 2000	Refer to Table 9.3.3 ANZECC 2000	44 – 208[10]	118	520 – 4940[9]	1950	100		0	
Calcium				1000	20 – 38.6[3]	26.8	21 – 236[5]	114				
Magnesium					3 – 15[3]	8.7	2.8 – 157[5]	59.2				
Sodium	180[a]				12 – 116[3]	54	41 – 1540[5]	581	100		65	
Potassium					4.6 – 5.8[3]	5	8.1 – 37[5]	18.6				
Sulphate	250[a]			1000	8 – 505[4]	146	5.5 – 152[4]	67	100		100	
Chloride		400	175 – 750		15 – 173[5]	62	38 – 494[3]	223	100		65	
Ammonia-N		0.01			0.03 – 8.8[14]	0.9	1.6 – 27.7[11]	11.5	55		0	
Total-N					0.6 – 5.6[15]	2.1	6.1 – 44.2[11]	20.3				
Nitrate-N		50 infants under 3 months, 100 otherwise		400	0.1 – 6.2[13]	1.5	0.1 – 19.5[4]	6.4	100		75	
Nitrite-N		3.0		30	0.1 – 0.3[3]	0.1	0.5 – 1.1[3]	0.9	100		65	
Phosphate-P			Refer to Table 9.2.19 ANZECC 2000		0 – 4.2[10]	0.5	2.3 - 9[4]	5.5				

TABLE 6.4 (cont.)

PARAMETER	NWQMS GUIDELINES				STORMWATER		WASTEWATER		STORMWATER		WASTEWATER	
									% ANALYSES THAT MEET GUIDELINES			
	DRINKING	RAW DRINKING	IRRIGATION	LIVESTOCK	RANGE	MEAN	RANGE	MEAN	DRINKING	IRRIGATION	DRINKING	IRRIGATION
Aluminium	mg/L	0.2	5.0	0.5	0.3 – 1.4[2]	0.9	0.1 – 2.6[8]	0.5	0	100	50	100
Arsenic		0.007	0.5	5.0	0 – 0.003[3]	0.002	0 004 – 0.007[2]	0.006	100	100	100	100
Boron		4.0	B	5.0	0.08 – 0.17[2]	0.13	0.6 – 2.7[2]	1.7	100	100	0	
Cadmium		0.002	0.01 – 2 kg/ha[b]	0.01	0 – 0.011[9]	0.003	0 – 0.002[11]	0.001	80	90	100	100
Chromium		0.05	0.01 – 1.0[b]	1.0	0 – 0.58[10]	0.07	0 – 0.1[11]	0.02	90	100	80	100
Copper		1.0	0.2 – 140 kg/ha[b]	0.5	0 – 0.48[11]	0.08	0.001 – 0.12[11]	0.04	100	90	100	100
Iron		0.3	0.2 – 10[b]		2.4 – 7.3[3]	4.0	0.03 – 1.6[9]	0.5	0	0	80	80
Lead		0.01	2.0 – 260 kg/ha[b]	0.1	0 – 0.53[11]	0.25	0 – 0.03[11]	0.01	10	45	90	100
Manganese	0.1[a]	0.5	0.2 – 10[b]		0.04 – 0.11[4]	0.08	0.02 – 0.08[10]	0.04	100	100	100	100
Zinc	3[a]		1.0 – 300 kg/ha[b]	20.0	0.02 – 5.8[10]	0.91	0.0 – 0.26[11]	0.09	90	90	100	100
Suspended solids					6 – 476[13]	164	11 – 250[10]	65				
Turbidity (NTU)	5[a]				12 – 34[4]	18			0			
Total organic carbon					5.8 – 61[8]	23.6	14 – 61[8]	37.5				
BOD					3 – 73[10]	23	8 – 80[10]	25				
?					0.2 – 0.45[3]	0.35	0 – 0.5[8]	0.3				
Coliforms	Per 100 mL 0	<10		100	4000 – 11700000[3]	3900000	1000 - 16500[3]	8200	0		0	
Faecal coliforms	0	0	1000	1000	0 - 600000[10]	150000			0	0		

[a] aesthetic guideline [b] specific to vegetation species [f – 15] number of analyses

Source : ANZACC, 2000; Dillon and Pavelic, 1996; NHMRC/ARMCANZ, 2001.

TABLE 6.5

COMPARATIVE MEAN HEAVY METAL CONCENTRATIONS IN ROAD RUNOFF OVERSEAS AND LOCALLY

SITE / METAL	SEATTLE AREA, USA	ORLANDO, FLORIDA	GERMAN HIGHWAYS	MELBOURNE, VICTORIA	SYDNEY, NSW	YARRA RIVER, VICTORIA
	MEAN CONCENTRATION, *MICROGRAMS PER LITRE*					
Arsenic	13	94	–	–	–	–
Cadmium	0.7	25	5	10	0.2	.33 - .49
Chromium	7	46	12	580	–	–
Copper	20	513	91	480	14	2.9 - 6.6
Lead	210	1580	203	690	20	2.2 - 16.1
Nickel	12	30	–	70	–	–
Zinc	120	423	430	5800	150	30 - 55

Note : "–" implies *not analysed*. Source : Peterson and Batley, 1992

A summary of metal concentration data from thirteen Melbourne sites shown in Table 6 indicates a number of possible trends.

TABLE 6.6

MEAN CONCENTRATIONS OF METALS IN RUNOFF AT DIFFERENT SITES NEAR MELBOURNE

SITE / METAL	1	2	3	4	5	6	7	10	13
	MEAN CONCENTRATION, *MICROGRAMS PER LITRE*								
Lead	490	690	330	70	510	530	360	120	160
Zinc	5800	1100	1000	320	310	1230	710	340	210
Cadmium	11	9	9	10	9	10	10	9	10
Copper	480	120	45	50	59	91	18	27	26
Chromium	580	24	63	130	120	24	20	20	23
Nickel	20	31	69	10	30	22	17	28	22

Source : Gutteridge, Haskins & Davey, 1981

Site Descriptions :

1. Light Industrial
2. Major Traffic Junction
3. Mixed Urban - includes industrial and service stations
4. Mixed residential, light and heavy industry
5. Mixed industrial, commercial and residential
6. Residential and two shopping centres
7. Housing
10. Housing
13. Orchards and light grazing.

6.4 QUALITY STANDARDS FOR RECEIVING WATERS

6.5 QUALITY STANDARDS FOR AQUIFER RECHARGE

[These sections omitted from the 2-Day Workshop Edition]