CHAPTER 8

CHAPTER 8

DETENTION, RETENTION AND INFILTRATION METHODS

8.1 INTRODUCTION

Stormwater detention or retarding basins and retention basins are commonly used in conjunction with urban development of a catchment to reduce the effect of catchment urbanisation on peak flow rates generated from the catchment. A distinction is made here regarding stormwater detention and stormwater retention. Stormwater detention is defined as the temporary storage of stormwater (ie. detention) for subsequent discharge, at a lower rate, to the receiving waters. Stormwater retention however is the removal, normally by infiltration in the basin, of stormwater and thereby preventing their discharge to the receiving waters.

Detention basins can either have a permanent water storage component (ie. a wet detention basin) or are completely dry during non-flood periods (ie. a dry detention basin). Both types of basins have the potential to serve multiple objectives in addition to their primary function of flood mitigation. Wet detention basins are commonly utilised to provide water pollution control, ecological and conservation functions as well as being public passive recreational amenities as shown in Figure 8.1. Common wet detention basins are ponds and wetlands. Naturally occurring lakes can be developed as detention, and to a certain extent, retention basins. These basins are becoming widely used to treat urban catchment runoff for removal of sediment and sediment-bound trace metals prior to their discharge to receiving waters. Dry detention basins often serve as playing fields and recreation parks in addition to its flood mitigation function as shown in Figure 8.2.

Figure 8.1 An urban pond used as a retarding basin (Wet Detention Basin)

Retention basins or infiltration basins are also used to control runoff volume and stormwater quality generated from urbanised catchments. They are very effective in catchments with sandy soil and often do not require elaborate outlet structures.

They are commonly used for stormwater management in arid and semi-arid regions and are based on the operation concept of groundwater recharge during most periods and overflow release of stormwater to surface receiving waters only in large storm events. A schematic diagram of an infiltration basin is shown in Figure 8.3. The design of retention basins is not covered in detailed in this manual and the reader is referred to a manual for infiltration systems which is currently being prepared by consultants for the Drainage and Irrigation Department of Malaysia.

Figure 8.2 Playing field used as a Retarding Basin (Dry Detention Basin)
Ref: Inst.of Engrs., Aust, 1987

Stormwater detention and retention systems can be utilised as part of the drainage systems of an urban catchment. Stormwater retention systems are perhaps best suited for such source areas as roofs although there is currently much research being undertaken to examine their effectiveness and sustainable operation in treating runoff from highways and main roads. It should however be recognised that the provision of detention and retention basins is only one method in a number of techniques available to manage stormwater runoff and their utilisation should be tested against other drainage strategies to arrive at a holistic and integrated strategy for the catchment. Generally, the detention and retention basins are utilised to serve four main functions:

- To restrict the peak discharge from a development area to a level no greater than that discharging from the area prior to the development, ie. by attenuating the runoff hydrograph;

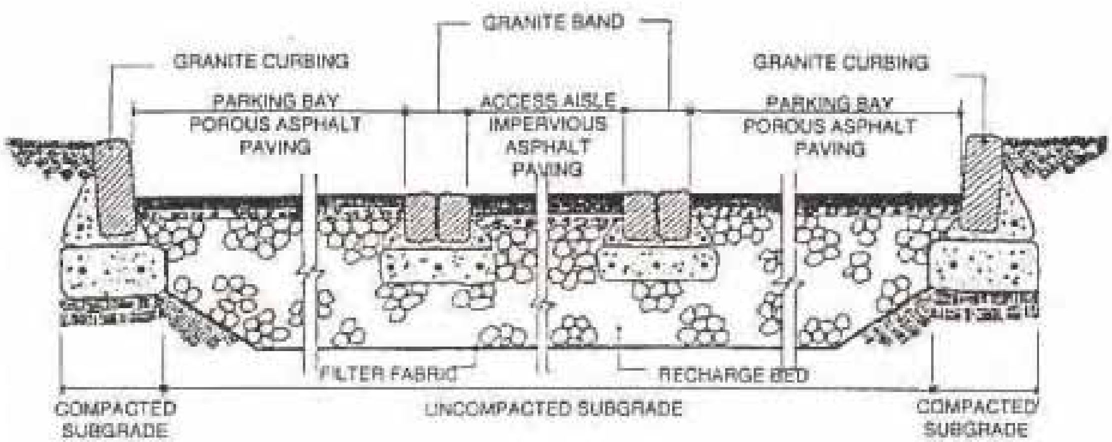

Figure 8.3 Schematic illustration of a Stormwater Infiltration System beneath a car park

- To reduce peak discharges in downstream major flow paths by attenuating and delaying upstream runoff contributions;
- To reduce the potential for scour and sediment transportation in downstream flow paths;
- To reduce downstream pollutant transport.

Retarding basins can be designed to provide beneficial outcomes in terms of protecting aquatic habitat in urban creeks (Wong *et al.*, 1998). Current design objectives have however often focused on reducing the peak urban discharge for events of low Average Recurrence Intervals (ARI) to pre-development conditions. It is only in recent times that many of these flow detention systems have been designed and retrofitted for use close to runoff sources for both stormwater quantity and quality management. Opportunities for retrofitting retarding basins to provide water quality enhancement functions include the development of wetland systems within retarding basins.

Notwithstanding the above, care needs to be taken in the design of these basins to ensure that downstream flood conditions are not exacerbated by their installation. Problems may include:

i. the creation of co-incident peaks with downstream tributaries or where parallel basins are installed;
ii. extended periods of inundation especially for more frequent flood events.

8.2 APPROPRIATE UTILISATION OF ON-SITE RETENTION SYSTEMS

It should be noted that not all catchments are suited to on-site retention systems. Careful consideration of the type of runoff source area from which runoff is to be directed to infiltration systems is important to ensure the continued effective operation of these schemes. Infiltration systems for treating runoff from more general areas such as streets and car parks are often integrated into landscaping features in urban design in Europe. Australian experiences have highlighted the importance of proper design of these systems and the position of these systems in the stormwater treatment train. Poor consideration of catchment pollutant types and characteristics and site conditions is often the main cause for their deteriorating effectiveness over time due to clogging and lack of appropriate maintenance. Pre-screening is a vital component in the treatment train and wetlands are one possible pre-treatment of stormwater runoff before discharge to infiltration systems and swale drains.

Soils with low hydraulic conductivities do not necessarily preclude them from being suitable for on-site retention system even though the required infiltration area may become uneconomical. However, these soils are likely to render them more susceptible to clogging if the stormwater inflow has not undergone some degree of pre-treatment to remove litter and sediment.

On-site retention systems should not be placed near building footings to remove the influence of continually wet subsurface on the structural integrity of these structures.

Identification of suitable sites for on-site retention systems should also include avoidance of steep terrain and area of shallow soil cover over rock. An understanding of the seasonal variation of the groundwater table is also an essential element in the design of these systems.

Camp Dresser and McKee (1993) suggest a point system for evaluating the suitability of sites for stormwater infiltration using retention basins. The point system is summarised in Table 8.1 and sites which score less than 20 points are generally considered to be unsuited to this form of stormwater management measures.

Table 8.1
Site Evaluation System for Stormwater Infiltration
(Camp Dresser & McKee, 1993)

Item	Conditions	Points
Ratio between the directly connected impervious area (DCIA) and the infiltration area (IA)	• IA > 2 DCIA • DCIA < IA < 2 DCIA • 0.5 DCIA < IA < DCIA	20 10 5
Nature of the surface soil	• Coarse soil and low organic material fraction • Normal humus soil • Fine grained soils and high organic matter fraction	7 5 0
Underlying soil (if finer than surface soils, otherwise use surface soil classification)	• Gravel or sand • Silty sand or loam • Fine silt or clay	7 5 0
Slope of the infiltration surface	• S < 7% • 7% < S < 20% • s > 20%	5 3 0
Catchment vegetation cover	• Healthy natural vegetation • Well established lawn • New lawn • No vegetation (bare soil)	5 3 0 -5
Degree of traffic on infiltration surface	• Light foot traffic • Average foot traffic (eg. Parks and lawn) • Considerable foot traffic (eg. Playing fields)	5 3 0

8.3 PERFORMANCE CHARACTERISTICS

Detention and retention basins are designed principally to attenuate the runoff generated from the upstream catchment to a pre-specified design level. The main features affecting the performance characteristics of these systems are the storage volume and the outflow control. The hydraulic characteristics of the outlet structure in the case of a detention, system or the infiltration capacity of the underlying soil in the case of retention systems, define the outflow control of the basin. A low-level pipe or culvert for normal operation up to the design event and a high-level overflow spillway for above design events control outflows from a typical detention basin. A combination of excavation and construction of embankments create the provision of flood storage.

Performance specifications for these basins can include the peak outflow from the basin for a given design event, the probability of exceedence of a specified water level in the basin or the frequency of basin spillway operation. Once the desired performance of the basin is specified, the design process involves selecting the appropriate combination of basin embankment height, basin area and outlet configuration to meet the design specification.

It is often necessary to recognise site constraints in defining the performance characteristics of these basins. For example, the specification of a very low peak outflow from a detention basin for a given probabilistic flood event will result in a high storage requirement and consequently high embankment (in the case of a site in steep terrain) or large area (in flat terrain) requirements. If this peak outflow specification is to be met in conjunction with a further specification on the maximum water level in the basin, the solution may become unattainable in steep terrain.

In the case of a retention basin, soil hydraulic conductivity and groundwater levels determine the required area of the infiltration system.

The storage characteristics of the detention or retention basin defines the relationship between the available storage in the basin and the surface area and depth of water in the basin. This information is then combined with the outflow characteristics for routing the inflow hydrograph through the storage. The outflow characteristics of detention and retention basins reflect the relationship between the depth of water in the detention or retention basin and the rate of discharge from the basin. In the case of detention basins, this characteristic is simply the combined water elevation-discharge relationship of the low flow culvert and the overflow weir as discussed in Section 8.4. For infiltration systems, the discharge characteristics can often be expressed by Darcy's equation with the principal parameter being the soil hydraulic conductivity.

8.4 DETENTION BASINS

8.4.1 Discharge Characteristics

Generally, the flow characteristics of the respective components of outflow conditions can be derived from standard textbook equations corresponding to calculation of critical depth at the entrance to the culvert, orifice flow, closed conduit flow and weir flow. Typical equations are presented by Laurenson and Mein (1995) for computing discharge characteristics of a detention basin as follows:-

$$Q = 1.50 N_p (S/40)^{0.05} (H - H_o)^{1.9} D^{0.6} \quad \text{for } H < 0.8D \qquad - \qquad 8.1$$

$$Q = 1.38 N_p (S/40)^{0.05} (H - H_o)^{1.5} D \quad \text{for } H > 0.8D \qquad - \qquad 8.2$$

$$Q = 0.785 N_p D^2 \sqrt{\frac{19.6(H - H_T)}{k_e + k_b + 1.0 + f(L_p / D)}} \qquad - \qquad 8.3$$

where
- Q is the discharge (m^3/s)
- N_p is the number of pipes
- S is the average slope of the pipe outlet (%)
- D is the diameter of the outlet pipe (m)
- H is the elevation of the water in the retarding basin (m)
- H_o is the elevation of the pipe entrance invert (m)
- H_T is the tailwater elevation (m)
- k_e is the entrance head loss coefficient

k_b	is the bend loss coefficient
L_p	is the length of the pipe (m)
f	is the Colebrook-White friction factor

The spillway discharge equation is given as follows:-

$$Q_s = cL(H - H_s)^{1.5} \qquad - \qquad 8.4$$

where

c	is the weir coefficient for the spillway (ranging from 1.45 $m^{0.5}/s$ for a broad crested weir to 2.15 $m^{0.5}/s$ for an ogee crested weir)
L	is the effective length of the spillway (m)
H_s	is the spillway crest elevation (m)

For any water level, the lower of the inlet and outlet control discharges (as defined by Equations 8.1 to 8.3) is to be adopted. However, once outlet control is established on a rising water level, it is assumed to persist for all higher water levels regardless of which conditions gives the lower capacity. Figure 8.4 shows the discharge-depth characteristics of detention basins, which combine the hydraulic characteristics of the low flow culvert and spillway as defined by Equations 8.1 to 8.4.

8.4.2 Designing for Multiple Discharge Objectives

Designing retarding basins to meet more than one discharge criterion can provide beneficial outcomes for ecological management of the downstream aquatic environment without compromising drainage and flood protection requirements. This is demonstrated in Figure 8.5 which shows the flood frequency curve resulting from a retarding basin design to match the 1.5 year ARI and 100 year ARI rural conditions peak discharges. This flood frequency curve is compared against the flood frequency curve of the catchment in rural conditions and in an urbanised condition without a retarding basin. The comparison clearly

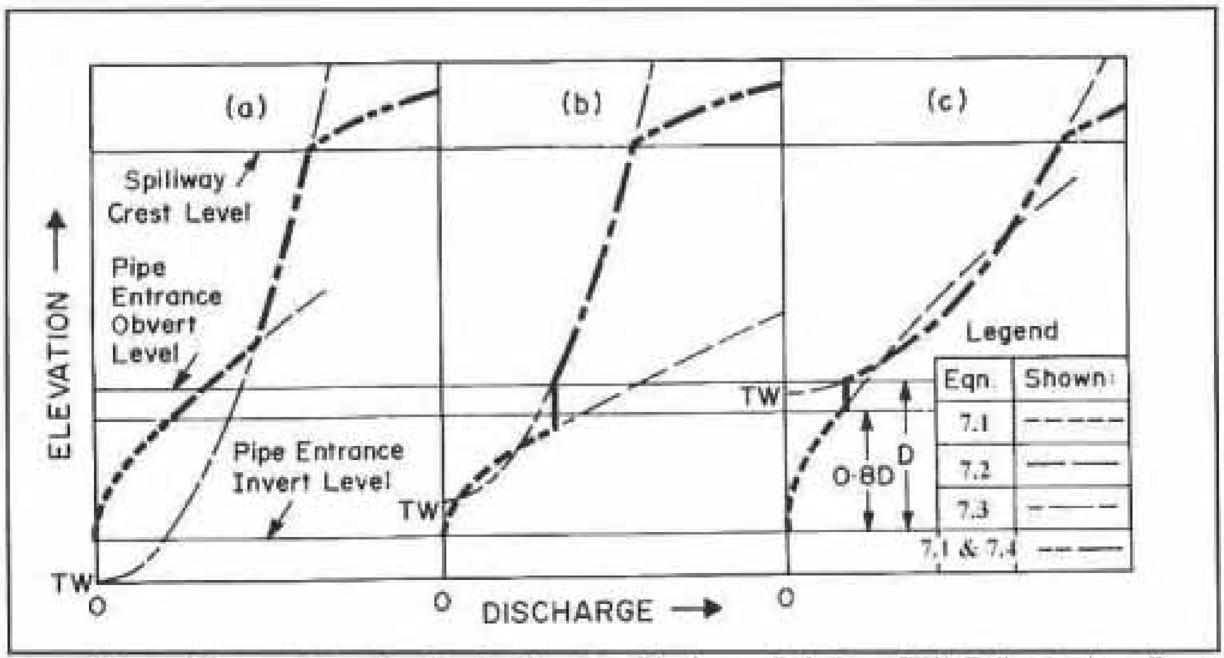

Figure 8.4 **Retarding Basin Elevation-Discharge Relations [TW=Tailwater Level] Adopted curve shown in bold line in each case (Laurenson & Mein, 1995)**

demonstrates that retarding basins can be designed to match a range of rural conditions peak discharges. In this case, the flood frequency curve under urbanised condition (with the retarding basin) is almost identical to the rural condition curve for events larger than the 1.5 year ARI event. Drainage systems design initiatives (such as grass swales and distributed storages) can be used to further steepen the flood frequency curve for events more frequent that the 1.5 year ARI event.

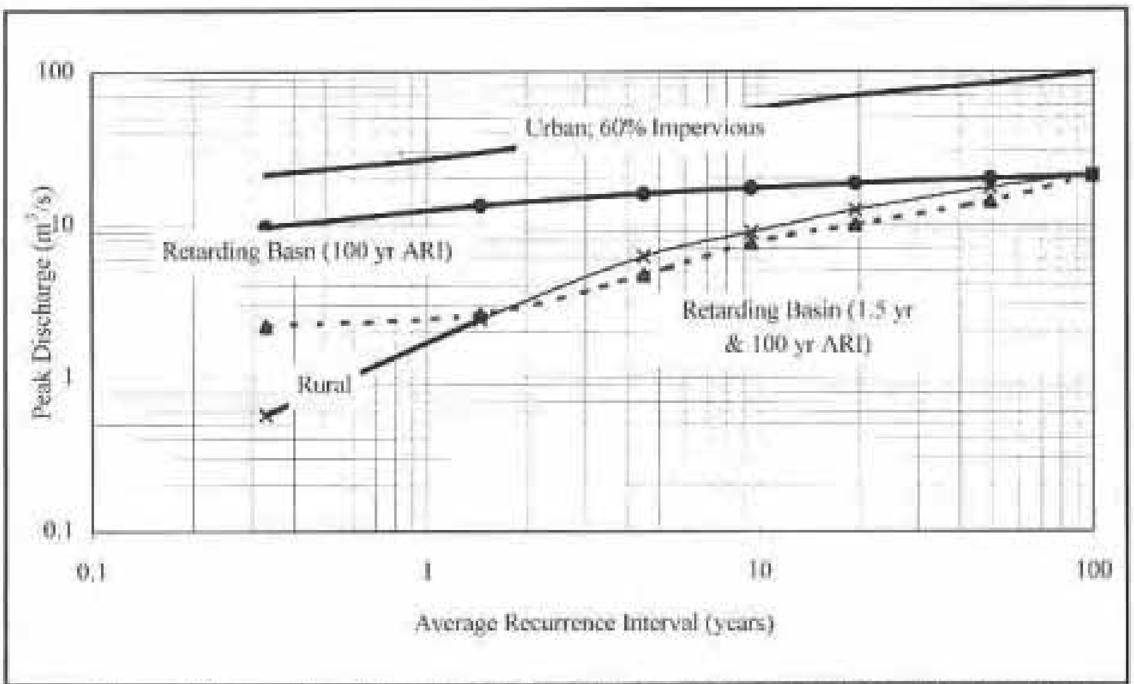

Figure 8.5 Flood Frequency Resulting from Retarding Basin Designed for Two Rural Condition Discharge Criteria ($Q_{1.5}$ & Q_{100})

8.4.3 On-site Detention Basins and Storage Tank Systems

On-site detention basins are used in urbanised catchments to facilitate urban consolidation in a climate of aging and under-capacity stormwater infrastructure. They are essentially individually sized tanks for mitigating the effects of increasing area impervious resulting from urban consolidation and re-development of residential and commercial buildings. These devices are becoming more common as devices that may be constructed in lieu of a drainage levy (Ribbons et al., 1995) associated with re-development in built-up catchments. They are often used as a last resort to facilitate urban re-development in built-up catchments owing to the lack of available space for other measures.

Current experiences on the success of the use of on-site detention tanks in Australia are mixed and the main disadvantage of this approach is the possible fragmented manner in which stormwater management is implemented in the catchment. Private ownership of detention basins can often lead to varying levels of maintenance of these structures. Common problems associated with this type of runoff control devices are siltation of the basins and poor construction. These problems have raised questions on the long-term sustainability of such devices as effective runoff control measures.

8.5 INFILTRATION SYSTEMS

8.5.1 Darcy's Equation

Retention basins used for infiltration of stormwater are viable alternatives to on-site detention tanks. In their design, the outflow characteristics reflect the hydraulic conductivity of the underlying soil through which stormwater is to infiltrate. Typically, Darcy's Equation is used to define the relationship between the depth of water in the basin and the rate of infiltration, ie.

$$Q = kA\frac{h}{L} \qquad - \qquad 8.5$$

where
- Q is the discharge (m^3/s)
- k is the hydraulic conductivity of the underlying soil (m/s)
- A is the basin area (m^2)
- Δh is the depth of water overlying the infiltration area (m)
- ΔL is the depth of soil layer to the underground aquifer (m)

Fundamental to the utilisation of the Darcy's Equation is the determination of the soil permeability or hydraulic conductivity. The field hydraulic conductivity can be determined using the falling head augerhole method of Jonasson (1984). Four broad soil permeability classifications generally apply as follows:-

Sandy soil:	k_{60}	=	5×10^{-5} m/s
Sandy clay:	k_{60}	=	between 1×10^{-5} and 5×10^{-5} m/s
Medium clay:	k_{60}	=	between 1×10^{-6} and 1×10^{-5} m/s
Heavy clay:	k_{60}	=	between 1×10^{-8} and 1×10^{-6} m/s

where k_{60} is the 60-minute value of hydraulic conductivity.

8.5.2 Porous Pavements

Porous pavements are commonly used in open car parks and driveways. They are constructed from modular or lattice paving as typically shown in Figure 8.6. These paving blocks are used to provide structural support while retaining a large proportion of the "paved-area" pervious for infiltration of rainfall and ponded stormwater. For porous pavement infiltration systems, the "fully drained" conditions is assumed and the term $\Delta h/\Delta L$ approaches unity. As reported by Argue (1997), measurements made at many field sites and retention installations show this assumption to be generally satisfactory. For $\Delta h/\Delta L$ to be unity represents the limiting conditions to which the hydraulic gradient tends during the wetting-up process. Thus most design of porous pavement systems under fully drained conditions is simply based on determining the required area to match the design discharge Q_{peak}. The design of retention basins where the system operates by the temporary storage of inflow for subsequent discharge by infiltration is discussed further in the next section.

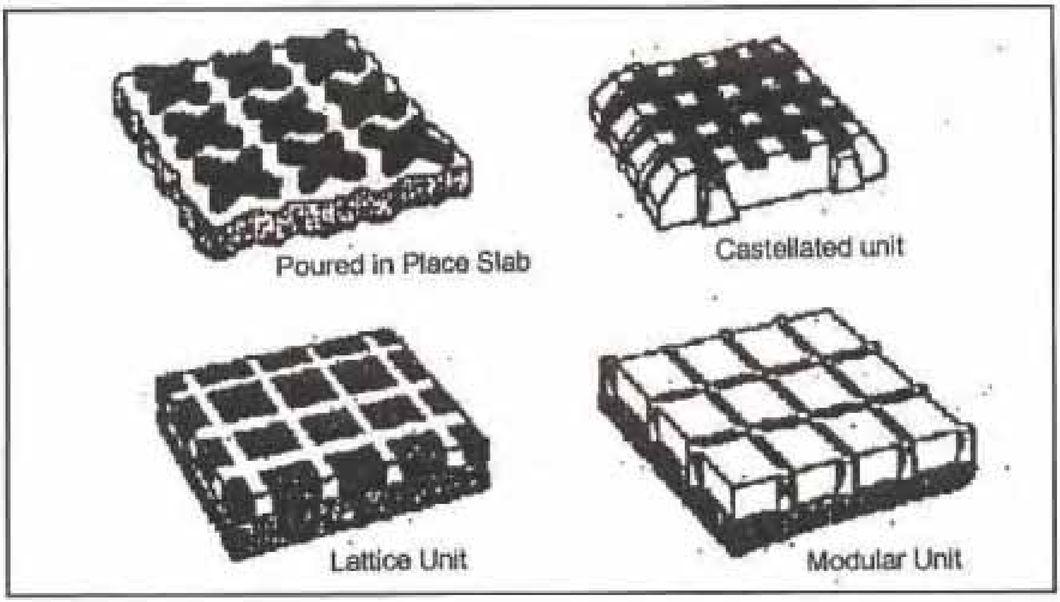

Figure 8.6 Porous pavements are commonly used in car parks and driveways to allow infiltration of rainfall

8.5.3 Infiltration Basins

Infiltration basins are open water systems, which are essentially terminal systems where stormwater are held in temporary storage for subsequent infiltration. They are often designed to notional retarding basin standards with no overflow for events up to the 100 year ARI event. Analyses of the discharge characteristics differ depending on the relative position of the watertable and the effect of clogging, over the long term, on the infiltration capacity of the basin. Three types of analysis are common, ie.

1. Shallow Watertable Model
2. Clogged Base Model
3. Deep Watertable Model

Shallow Watertable Model

When water infiltrate through a basin towards a shallow watertable, the resulting mound of water is the mirror image of the core of depression resulting from a groundwater well withdrawal. Various analytical methods can be used to describe the mounding associated with recharge, the simplest being based on steady state radial flow in an infinite aquifer as expressed in equation 8.6.

$$q = K\pi \frac{\left(h_1^2 - h_2^2\right)}{\ln\left(\frac{R}{r}\right)} \qquad - \qquad 8.6$$

where q is the rate of outflow from the basin by infiltration
K is the soil permeability
R is the radius of influence from the centre of the basin (Figure 8.7)
r is the extend of the water surface from the centre of the basin (Figure 8.7)
h_1 is the vertical distance from the water surface in the basin to the impervious

layer (Figure 8.7)

h_2 is the vertical distance from the groundwater table to the impervious layer (Figure 8.7)

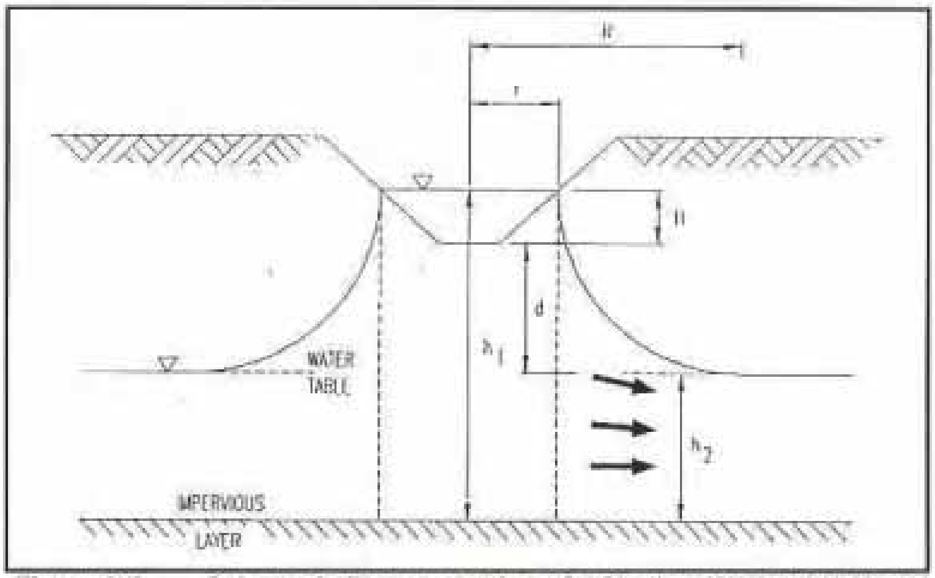

Figure 8.7 Schematic Representation of a Shallow Watertable Model

For most applications in retention basin design, equation (8.6) is conservatively approximated as follows:-

$$q = K\pi \frac{(H+d)^2}{\ln\left(\frac{R}{r}\right)} \qquad - \qquad 8.7$$

where H is the depth of water in the retention basin (Figure 8.7)

d is the vertical distance from the base of the retention basin to the watertable (Figure 8.7)

Analysis with steady state flow solutions is limited because of its inherent simplifying assumptions which include the following:-

- the soil is isotropic and homogeneous;
- the flow is radial and saturated;
- the watertable is horizontal and acts like an impermeable layer;
- the basin has vertical sides.

The choice of the radius of influence needs to be made in applying the above equations. In practice, infiltration of stored water is not a steady state process and the radius of influence will vary with time. Bear (1968) derived an equation for approximation of the radius of influence as follows:-

$$R = r + 50(H+d)K^{0.5} \qquad - \qquad 8.8$$

The cumulative outflow volume at time t_1 from the commencement of outflow can be calculated as follows:-

$$V_{out} = \left[K\pi t \frac{(H' + d)^2}{\ln\left(\frac{R}{r}\right)} \right]_0^t \qquad - \qquad 8.9$$

A simplifying assumption that the basin is instantaneously filled to a constant depth H' equal to half the maximum depth is applied when using the above equation.

Clogged Base Model

Over time the base and sides of the basin will become progressively clogged with oils, silt and trash. In such circumstances, the infiltration rate is controlled by the permeability and thickness of the clogged layer and is independent of the permeability of the surrounding soil. Bouwer (1989), suggested that the zone between the base of the basin and the groundwater table is unsaturated under such circumstances and water will move essentially vertically downward due to gravity as shown in Figure 8.8. Application of Darcy's Equation to the flow through the clogging layer gives an essentially linear relation between water depth in the basin and the infiltration rate.

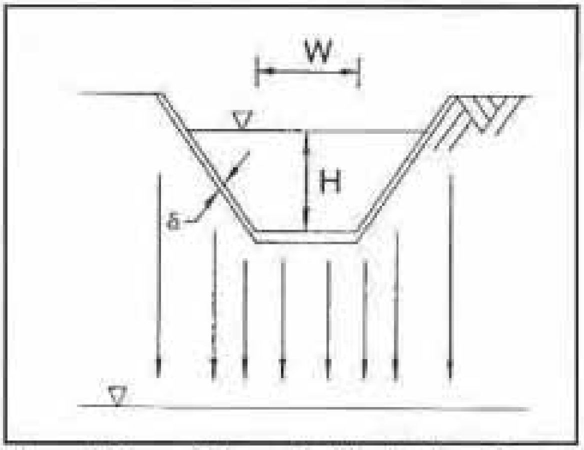

Figure 8.8 Schematic Illustration of a Clogged Base Model

The flow rate per unit length through a clogged layer of uniform thickness can be expressed as follows:-

$$q = K_c W\left(\frac{H + \delta}{\delta}\right) \qquad - \qquad 8.10$$

where W is the width of the base of the basin (m)

δ is the thickness of the clogged layer (m)

H is the maximum depth of water in the basin relative to the base (m).

Applied to a rectangular basin with a clogged layer of uniform thickness and integrating with time gives the volume of outflow as follows:-

$$V_{out} = \left[K_c t \left(WL\left\{\frac{H' + \delta}{\delta}\right\} + 2(W + L)\sqrt{1+S}H'\left\{\frac{0.5H' + \delta}{\delta}\right\} + 4(H')^2 S\sqrt{1+S}\left\{\frac{0.33H' + \delta}{\delta}\right\} \right) \right]_0^t \qquad - \qquad 8.11$$

Again, a simplifying assumption that the basin is instantaneously filled to a constant depth H' equal to half the maximum depth is applied when using the above equation. Analysis of typical clogged layer permeability K_c suggests a value of 1.5×10^{-6} m/s.

Deep Watertable Model

A rigorous solution for steady state saturated flow from a trapezoidal canal was published by Harr (1962). For the normal range of values of side slope, depth and width, the expression for the infiltration rate as depicted in Figure 8.9 may be approximated as follows:-

$$q = K(W + H'\sqrt{1+S^2}) \qquad - \qquad 8.12$$

Extending the above equation to three dimensions with a depth of H' and integrating with time gives the following expression for the volume of outflow from the basin:-

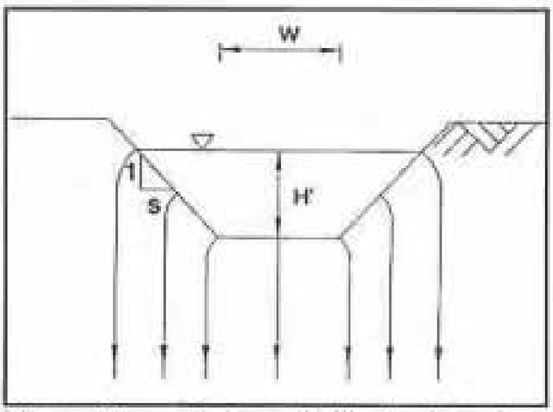

Figure 8.9 **Schematic Illustration of the Deep Watertable Model**

$$V_{out} = Kt\left(WL + 2WH'\sqrt{1+S^2} + 2LH'\sqrt{1+S^2} + 4(H')^2S\sqrt{1+S^2}\right) \qquad - \qquad 8.13$$

8.5.4 Retention/overflow Wells and Trenches

There has been much development in the use of on-site stormwater retention systems involving non-terminal devices incorporating overflow pipes to the minor drainage network. These devices are often used to retain roof runoff generated for storm events up to the design ARI and overflow to the street drainage network during medium and large storm events. Two typical types of on-site retention systems are the use of perforated soak wells (often referred to as "leaky wells" and gravel filled infiltration trenches as illustrated in Figures 8.10 and 8.11 (Argue, 1994).

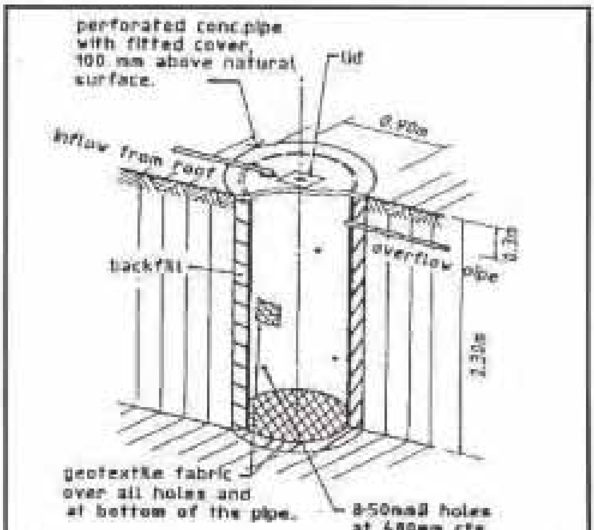

Figure 8.10 Illustration of a Perforated Retention/Overflow Well

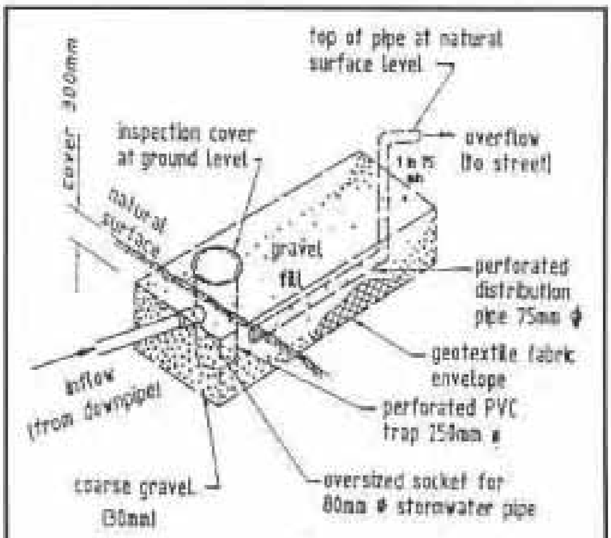

Figure 8.11 Illustration of a Shallow Retention/Overflow Trench

The principal design task associated with the use of these retention/overflow wells and trenches is the computation of the "emptying time". This has to match typical inter-event dry periods to avoid the possible reduction in available retention storage owing to a preceding events filling the basin prior to the occurrence of the design storm.

The emptying time (T) of a perforated well is given by Argue (1997) as

$$T = -\frac{2.3D}{4K}\log\left[\frac{0.25D}{H + 0.25D}\right] \qquad - \qquad 8.14$$

where D is the diameter of the perforated well (m)
H is the depth of water in the well (m)

The corresponding expression for the infiltration trench is expressed as follows:-

$$T = -\frac{2.3LBe}{2K(L+B)}\log\left[\frac{LB}{LB + 2H(L+B)}\right] \qquad - \qquad 8.15$$

where L is the length of the infiltration trench (m)
B is the width of the infiltration trench (m)
H is the depth of water in the trench (m)
e is the void space ratio (often taken as 0.35 for typical trenches)

8.6 STORAGE ROUTING

8.6.1 General

In evaluating the performance of detention and retention basins, it is necessary to carry out a flood routing analysis to determine the degree of flood peak attenuation and the shift in the time of peak (in the case of a detention basin) and the frequency of spillway overflow. A typical illustration of the inflow and outflow hydrographs of a detention basin is shown in Figure 8.12.

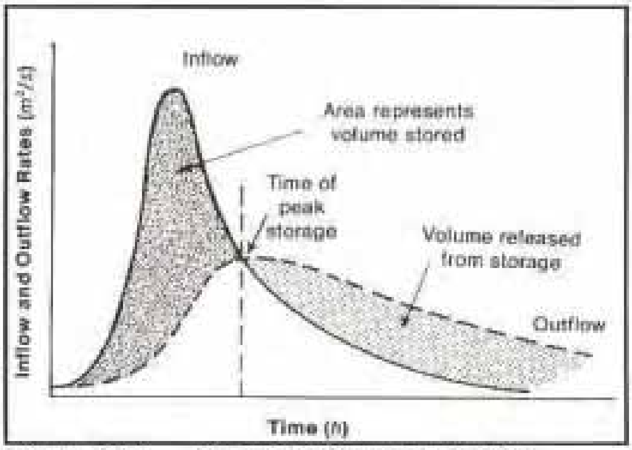

Figure 8.12 Typical Inflow and Outflow Hydrographs of a Detention Basin (ref. Inst. of Engrs.,Aust,1987)

Routing of the inflow hydrograph through the storage is undertaken numerically using the continuity equation and the storage-discharge relationship for the basin as described in Section 8.6.3.

8.6.2 Small Basins

Flood-routing calculations for small basins having less than 250 m^3 of storage volume or preliminary design of larger basin may be undertaken by simplified manual techniques, using either volume or hydrograph methods.

Volume Methods

Volume methods use a volumetric runoff coefficient to directly convert a rainfall hyetograph to a mass curve of inflow volume as shown in Figure 8.13. These methods are most appropriate for small fully urbanised catchments and where outflow from the basin is by means of pumps. As shown in Figure 8.13, mass curves of the inflow and outflow volumes are plotted and the required storage volume is determined graphically by subtracting the outflow volume from the inflow volume at the point where the outflow rate matches the inflow rate. The discharge characteristic of the outlet will have to be approximated by a constant or linearly increasing discharge rate in order to facilitate the graphical subtraction and the method is most suited to a pumped scheme.

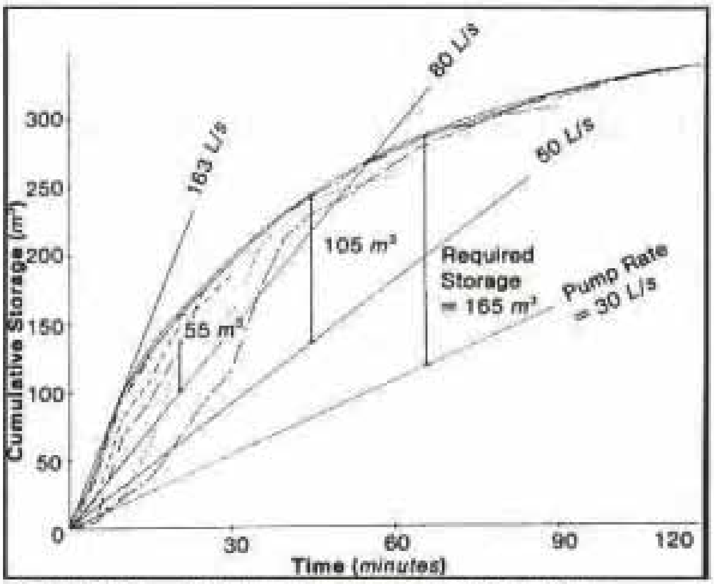

Figure 8.13 Determining Required Detention Storage using Mass Curves for Rainfall Patterns and Pumping Rates (ref: Inst. of Engrs., Aust., 1987)

Hydrograph Methods

Hydrograph methods rely on simplifying assumptions regarding the shape of the inflow hydrograph in order that the storage volume and the outflow capacity can be related by the single equation. Boyd (1980) suggested that an estimate of basin storage could be made using the following equation:

$$V_s / V_i = 1 - Q_o / Q_i \qquad - \qquad 8.16$$

where

- V_s is the required storage volume
- V_i is the inflow volume, determined as the product of the volumetric runoff coefficient, the average rainfall intensity, the storm duration and the catchment area.
- Q_o is the peak outflow capacity or desired peak outflow
- Q_i is the peak inflow rate, determined by the Rational Method

Sizing the storage volume according to Equation 8.16 was found to generally result in the peak discharge being on average 6% higher than the desired peak discharge for the design event.

An alternative equation is that of Culp (1948) and Carroll (1990), expressed respectively in equations (8.17) and (8.18). *Note that these procedures may give widely different answers and should be used with care.*

$$V_s / V_i = .67(1.25 - Q_o / Q_i)^2 - 0.04 \qquad \text{(After Culp 1948)} \qquad - \qquad 8.17$$

$$V_s = 1.33\, t_c.\, Q_i.\, [(Q_i - Q_o)/Q_i]^2 \qquad \text{(After Carroll 1990)} \qquad - \qquad 8.18$$

8.6.3 Large Basins

The sizing of large detention basins should normally be completed with the aid of computer models, which compute storage routing by solving the continuity equation and the storage function. The continuity equation or the equation of conservation of mass simply expresses the condition that the rate of inflow less the rate of outflow at any instance in time is equal to the rate of change in storage in the basin, ie.-

$$I - Q = \frac{\partial S}{\partial t} \qquad - \qquad 8.19$$

where
- I is the instantaneous inflow rate of discharge to the basin (m^3/s)
- Q is the instantaneous outflow rate of discharge from the basin (m^3/s)
- S is the volume of temporary storage in the basin (m^3)

The above equation may be expressed in finite difference form as follows:-

$$\frac{(I_t + I_{t+1})}{2} - \frac{(Q_t + Q_{t+1})}{2} = \frac{S_{t+1} - S_t}{\Delta t} \qquad - \qquad 8.20$$

where
- $t, t+1$ are instances in time
- Δt is the time interval defining the finite difference approximation of the continuity equation.

Equation 8.20 can be re-arranged such that all known variables are placed on the left side of the equation and all unknown variables on the right, ie.:-

$$\frac{(I_t + I_{t+1})}{2} - \frac{Q_t}{2} - \frac{S_t}{\Delta t} = \frac{Q_{t+1}}{2} + \frac{S_{t+1}}{\Delta t} \qquad - \qquad 8.21$$

It is evident from equation 8.21 that a second equation is necessary to solve the two unknown variables of Q_{t+1} and S_{t+1}. This second equation is referred to as the storage equation, which expresses the relationship between the storage in the basin and the discharge from the basin ie.

$$S = f(Q) \qquad - \qquad 8.22$$

The storage function represents the combined effect of the topography of the site (ie. the geometric properties) and the discharge characteristics (as discussed in Sections 8.4 for detention systems) in expressing the storage-discharge relationship for the basin.

The storage function can often be expressed in tabulated form and equation 8.21 can be solved at each time step sequentially. The formation of the storage function table to solve equation 8.21 requires the time step (Δt) to be first selected. A table relating the outflow discharge (Q) to the functions (0.5Q+S/Δt) and (0.5Q-S/Δt) can then be established. As is evident from equation 8.21, the terms on the left side of the equation are variables related to the present time step and are known. The term (0.5Q-S/Δt) for the time (t) can be determined from the known discharge Q_t using the tabulated relationship between Q and (0.5Q-S/Δt). The term (0.5Q+S/Δt) for the time (t+1) is equal to the sum of the terms on the left side of the equation. Hence the value of Q_{i+1} can be determined from the tabulated relationship between Q and $(0.5Q+S/\Delta t)$.

8.6.4 Retention Basins

Retention basins typically would have a significantly lower rate of outflow (infiltration) than typical detention basin and require special design considerations to be applied to examine the time series inflow of stormwater. The typical storm sequence is characterised by a sequence of storm events and inter-event dry period. The antecedent water level immediately prior to the occurrence of stormwater inflows to the temporary storage volume is dependent on the available retention storage, the emptying rate of the system and the period between storm events. Therefore the typical inter-event dry period of the catchment has a significant bearing on the required basin area for infiltration as the effectiveness of the system is dependent on the antecedent water level in the basin. In the case of Malaysian climatic conditions, the regular occurrence of late afternoon thunderstorms suggest a typical inter-event dry period to be of the order of 24 to 36 hours. It is often not sufficient, particularly in regions of relatively short inter-event dry period, such as in Malaysian catchments, for the design of retention basins to be based on event-modelling. A continuous modelling approach will often be necessary in a rigorous approach in quantifying the inter-relationship between the rate of outflow (defined by the infiltration rate multiplied by the basin area) and the sequence of stormwater inflow and inter-event dry period. This latter approach is not yet universally adopted.

In adopting a continuous simulation approach, the designer will find that a number of design criteria are available, each producing a different storage area and wetted area requirement. These include the following:-

- the design ARI event which must be retained by the system;
- the frequency of overflow;
- the overall long-term percentage of runoff volume which will overflow.

These issues will not be covered in detail in this document. It is sufficient to note that three key factors are highly inter-related in the sizing of retention basins, ie.

1. the emptying rate which is influenced by the product of the hydraulic conductivity and the wetted area;
2. the volume of the retention storage;
3. the frequency of overflow.

Design decisions and site conditions affecting any two of the above factors will fix the third.

8.7 DESIGN PROCEDURE

8.7.1 Calculations for Detention Basins

Maximum Discharge Criterion

Where the outflow from a basin must be limited to a certain flow-rate, the following design procedure is recommended:

a) Calculate inflow hydrographs of the required design ARIs for a range of storm durations.

b) Perform routing calculations for each of the inflow hydrographs for a variety of outlet arrangements and sizes and with the basin generally shaped to maximise its storage area and minimise its depth. Identify the storm duration that requires the most storage volume. This determines the critical storm duration for the detention basin.

c) Identify the outlet configuration which requires the smallest storage volume whilst limiting the outflow to the required value, for that duration, and then check this configuration for other storm durations and for intermediate ARIs.

d) Design of the high-level outlet and the embankment by routing through the basin designed in (b) floods of ARI equal to the Extreme Flood for a range of storm durations and alternative temporal patterns. Select the worst case.

e) Check the effect of the basin on flow-rates further downstream and on upstream flood levels and hydraulic grade levels, where appropriate.

Storage Volume Criterion
Where the storage volume of the basin is limited by site constraints and the objective is to achieve only the maximum practical attenuation of the inflow, a similar procedure may be used:

a) Calculate inflow hydrographs of the required design ARIs for a range of storm durations.

b) Perform routing calculations for each of the inflow hydrographs, for a variety of outlet arrangements and sizes. Select the arrangement that provides the greatest attenuation of the inflow while meeting the relevant freeboard requirements.

c) Design of the high-level outlet and the embankment by routing through the basin designed in (b) floods of ARI equal to the Extreme Flood for a range of storm durations and alternative temporal patterns. Select the worst case.

d) Check the effects of the basin on upstream flood levels and hydraulic grade levels for a range of flood events.

The routing timestep or increment must be short enough relative to the storm duration to ensure that the peak storage requirements will be accurately determined. The design of the basin and its outlet structures must also be based on a range of storm durations and appropriate temporal patterns in order to identify the critical hydraulic dimensions.

8.7.2 Calculations for Retention Basins

Argue (1997) lists the basic steps in calculating the required dimensions of on-site retention systems based on a combination of well-established theory and adjustments for soil heterogeneity and other non-ideal conditions. The steps are listed as follows:-

Treatment Surface (porous pavements)
In sizing the area of the treatment surface, the basic data requirements are as follows:-

(i) peak flow, Q_{peak};
(ii) surface area, A_s;
(iii) catchment area, A;

(iv) hydraulic conductivity of soil, k_{60}

The required surface area is given by the equation

$$A_s = \frac{Q_{pre}}{K_{60}U} \qquad - \qquad 8.23$$

where U is a moderating factor which range from 0.5 for sandy soil to 2.0 for clay soils. The moderating factor was found to be necessary as soil hydraulic conductivity values are typically obtained from site tests on small boreholes. When the results of these tests are applied to design on-site detention devices, experience found the size of these devices were to be often too big, where site soil is clay, and too small where the soil is sandy (Argue, 1997).

It is important to note that the surface area required A_s represents the open soil surface area and does not include the area of paving which is impervious. In calculating the catchment area, the designer must remember to include the porous pavement area as well.

Perforated Wells

In determinig the diameter of the perforated well, the following information is required:-

(i) the design runoff volume
(ii) the hydraulic conductivity of the soil, K

With the above information, the diameter of the perforated soak well may be computed as follows:-

$$D = \sqrt{\frac{V_{des}}{\frac{\pi}{4}\left(H + 300K_{60}t_cU\right)}} \qquad - \qquad 8.24$$

where t_c is the time of concentration of the catchment or source area (minutes)

For infiltration trenches, the length of the trench is related to the width (W) and depth (H) of the trench, the void ratio (e), the hydraulic conductivity and the time of concentration of the catchment by the following expression:-

$$L = \frac{V_{des}}{\left[eBH + 120K_{60}t_c(B + 0.5H)U\right]} \qquad - \qquad 8.25$$

Infiltration Basins

The design of infiltration basins is based on routing the inflow hydrographs corresponding to a range of storm duration using one of the three models discussed in Section 8.5.3 to define the hydraulic performance of the basin outlet. It is necessary to route design hydrographs corresponding to a range of storm duration to determine the maximum storage requirement for the design storm average recurrence interval.

8.7.3 Spillway and Embankment

The spillway and embankment should be designed both hydraulically and structurally to permit the safe discharge of floods in excess of the Design Flood. The ARI of the Extreme Flood, for which the performance of the basin should be checked, should be determined by consideration of the likely consequences of failure, in consultation with the relevant government authorities. For example, ANCOLD (1986) provides a basis for determining the ARI of the Extreme Flood based upon consideration of the incremental hazard associated with failure. Table 8.2 shows the range of ARIs applicable. Engineers should however confer with the relevant authorities to determine the actual ARI to be adopted, giving consideration to the strategic plan for the upstream and downstream areas, the risk of failure and the hazards associated with failure.

Table 8.2
Recommendations for Extreme Flood in Australia (ANCOLD, 1986)

Incremental Flood Hazard Category	Consequences of Failure	Extreme Flood ARI (years)
High	Identifiable loss of life	10 000 to P.M.F. (1)
Significant	Loss of life possible but not expected	1 000 to 10 000
Low	Lesser situations	100 to 1 000

(1) P.M.F. refers to Probable Maximum Flood

8.8 OTHER DESIGN CONSIDERATIONS

8.8.1 Multiple Outlets

Low-level outlet structures for large detention basins will more often be required to limit the outflows over a range of intermediate ARIs up to the ARI for the Design Flood. In such cases, the low level outlet structure may comprise either a single-level outlet sometimes preceded by a weir, or a multi-level outlet. A weir located immediately upstream of a single-level outlet may have an orifice of smaller diameter than the outlet to attenuate the outflows for smaller ARIs and to provide free drainage for the ponded water. During higher inflows the weir will overtop. A multi-level outlet will have a range of pipes or culverts set at different levels, possibly of different sizes to achieve the required attenuation throughout the ARI range.

The design of retarding basins to match more than one discharge will result in an increase in storage requirements and consequently higher embankments. This increase is however not likely to be excessive. The case study depicted in Figure 8.5 resulted in an increase in the embankment height of 10% as a result of the additional design criterion. The frequency at which the retarding basin will be inundated to some substantial level will however be increased as a result of the additional design criterion and may thus require a review of its appropriate landuse. Often the retrofit will be carried out in association with the establishment of a pond or wetland in part of the retarding basin. Some conventional usage of retarding basins such as sporting fields may need to be reviewed and the appropriate usage will vary considerably depending on site specific topography. Figure 8.14 shows a the possible combination of a stormwater quality treatment wetland incorporated into a retarding basin (Breen et al, 1992).

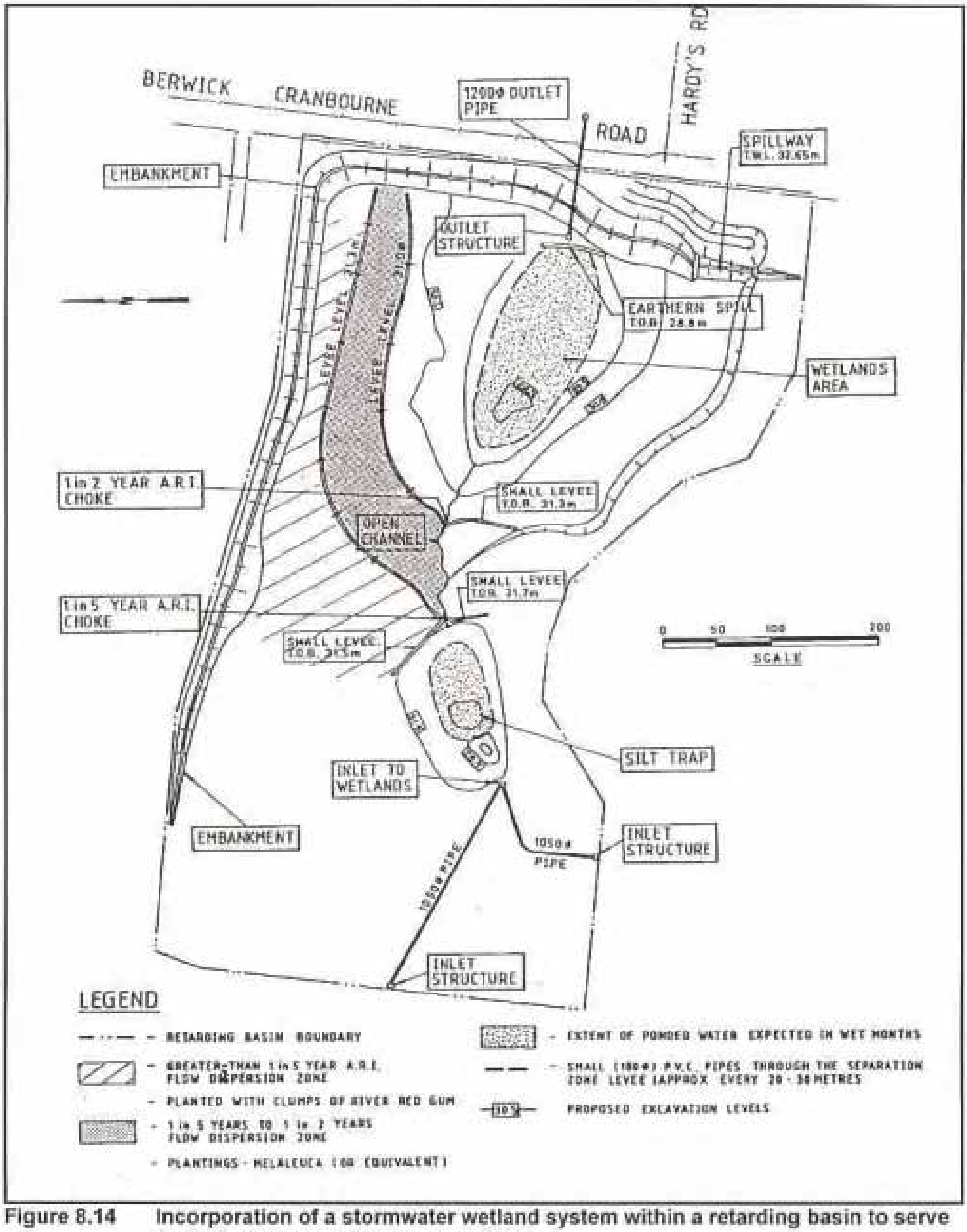

Figure 8.14 **Incorporation of a stormwater wetland system within a retarding basin to serve multiple objectives (Breen et al, 1992)**

8.8.2 Outlet Protection

The entry to a detention basin outlet structure should be protected against blockage and to reduce hazard for persons trapped in the basin during a storm. The level of protection will vary depending on the consequences of failure caused by blockage of the entry to the outlet structure and the potential frequency of blockage. It may be necessary for the Design Engineer to give consideration to the effects of a fully blocked low level outlet.

Protection can be achieved by the installation of a trash rack, bar screen or a fence. These should be designed to shed debris and to assist egress by persons trapped in the basin generally in accordance with the recommendations of Weissman (1989) as listed in Table 8.3. Trash racks comprising inclined vertical bars (inclined in the direction of flow) and spaced horizontal support bars are preferred.

Table 8.3
Design Criteria for Entry to Basin Outlet Structures

Item	Criterion
Spacing of Vertical Bars	125mm (max)
Spacing of Horizontal Supports	1m (min)
Nett Clear Opening Area	≥ 3 times the calculated outlet area[1]
Limiting Velocity through trash rack[2]	0.6m/s (not readily accessible) 1.5m/s (accessible)

Notes:

1. The calculated outlet area may depend upon the level of the outlet relative to the water surface. Where the outlet is contained in a drop structure the outlet area used for determination of the nett clear
   opening for the intake may need to be adjusted to account for the level difference.
2. The limiting velocity through the trash rack should be related to the accessibility of the intake structure for cleaning purposes.

8.8.3 Pipe Protection

Outlet pipes should have spigot and socket rubber-ring joints and lifting holes should be securely sealed. Pipe and culvert bedding should be carefully specified to minimise its permeability, and cutoff walls or seepage collars must be installed where appropriate, to control seepage and prevent piping failure adjacent to the outlet pipe. Appropriate measures, such as internal sealing of pipe joints and lifting holes, and bolting down of manhole lids, should be applied to any existing downstream systems which could be pressurised by the discharge from the outlet. Alternatively, surcharge chambers may need to be incorporated into the outlet pipe to limit the internal pressure.

8.8.4 Outflow Protection

Where the outlet from a basin is to be a free outfall, this should be located, where possible, within a well-defined natural depression or watercourse. Adequate protection must be provided both downstream and immediately upstream of the outlet, where appropriate, to prevent scour.

8.8.5 High-Level Outlet Structures

The high-level outlet, usually formed by a spillway, must be designed to safely convey extreme outflows from the basin. Where possible, the spillway should be cut into the abutment at the side of the basin or be located at the lowest section of the embankment to limit the possibility of embankment failure by scour.

In some circumstances the high-level outlet may be constructed as a gloryhole inlet (with a bar screen and an anti-vortex device if necessary) leading to a pipe or a culvert through the embankment.

The crest and the area downstream of the spillway may be protected by rip-rap, concrete, paving, or other suitable coverings, although grass cover may be adequate where spillway slopes are flatter than 1 in 6 (1V in 6H). Care should be taken to maintain good cover on grass spillways, and trees or shrubs should not be planted on the spillway chute. Design information for grassed spillways is described by the U.S. Soil Conservation Service (1979).

8.8.6 Embankments

Dry detention basins are intermittent water-retaining storages for which the embankments do not need to be as rigorously designed as dams unless they are particularly high or have special soil problems. Wet basins having a permanent or semi-permanent detention storage, may need particular design measures if the detention depth is significant. Nevertheless, the design of the embankment should be undertaken by a person experienced in the geotechnical aspects of such work.

The sides of the grassed embankment (or basin) should generally be flatter than 1 in 6 and never steeper than 1 in 4. The top-width should be at least three (3) metres. Steeper slopes may be used on embankments or basins lined with structural facings or low-maintenance ground-covers, but steps must be provided at appropriate intervals if the steepness of the slope could impede the egress of a person from the basin during a flood.

8.8.7 Public Safety Issues

While detention basins are generally less hazardous than channels because of the slower movement of water, the public may be caught unawares by the filling of a basin where the basin is used for either active or passive recreation. Preferably, the side slopes of basins should not be steeper than 1 in 6 to allow easy egress. Areas with slopes steeper than 1 in 4 may require steps and a handrail to assist egress.

Where suitable land is available, designers should aim to restrict basin depths to 1.2m at the 20 year ARI level and, if possible, for a greater recurrence interval. In cases where this is neither practical nor economical and the provision of a detention basin is considered to be better on safety grounds than other alternatives, greater depths are acceptable.

Suitable safety provisions (such as raised refuge mounds within large basins, fences and warning signs) should be provided for deeper basins.

Depth indicators should be installed within the basin and in the channel downstream of the embankment for basins with a storage depth of greater than one (1) metre. The indicator within the basin should have its zero level relative to the lowest point in the basin floor.

Special attention should be paid to basin outlets to ensure that persons are not drawn down into the intake or into the outlet. Rails, fences, anti-vortex devices, trash racks or grates should be provided where necessary.