

# Chapter 3 Sediment Basins

## Definition:

A sediment basin is a pond like structure designed to remove coarse sediments from stormwater by reducing flow velocities to the settling velocity of the target sediment size.

## Purpose:

- To retain coarse sediments from runoff.
- Are typically the first elements in a treatment train and play an important role by protecting downstream elements from becoming overloaded or smothered with sediments.
- Provide some minor flood attenuation by providing storage.
- Where the target sediment size is below 0.125mm, low levels of hydrocarbons (that have adhered to the particulates) will be removed.

## Implementation considerations:

- Sediment basins can require considerable area to achieve desired treatment levels of the target particle size.
- When used for trapping sediment in runoff from construction sites, maintenance frequency can increase dramatically from the high sediment yields.
- Sediment basins can have various configurations including hard edges and base (e.g. concrete) or a more natural form with edge vegetation creating an attractive urban element. They are, however, typically turbid and maintenance usually requires significant disturbance of the system.

Maintenance involves dewatering and dredging collected sediments. This is required approximately every five years.



*Sedimentation basins can be installed into hard or soft landscapes*

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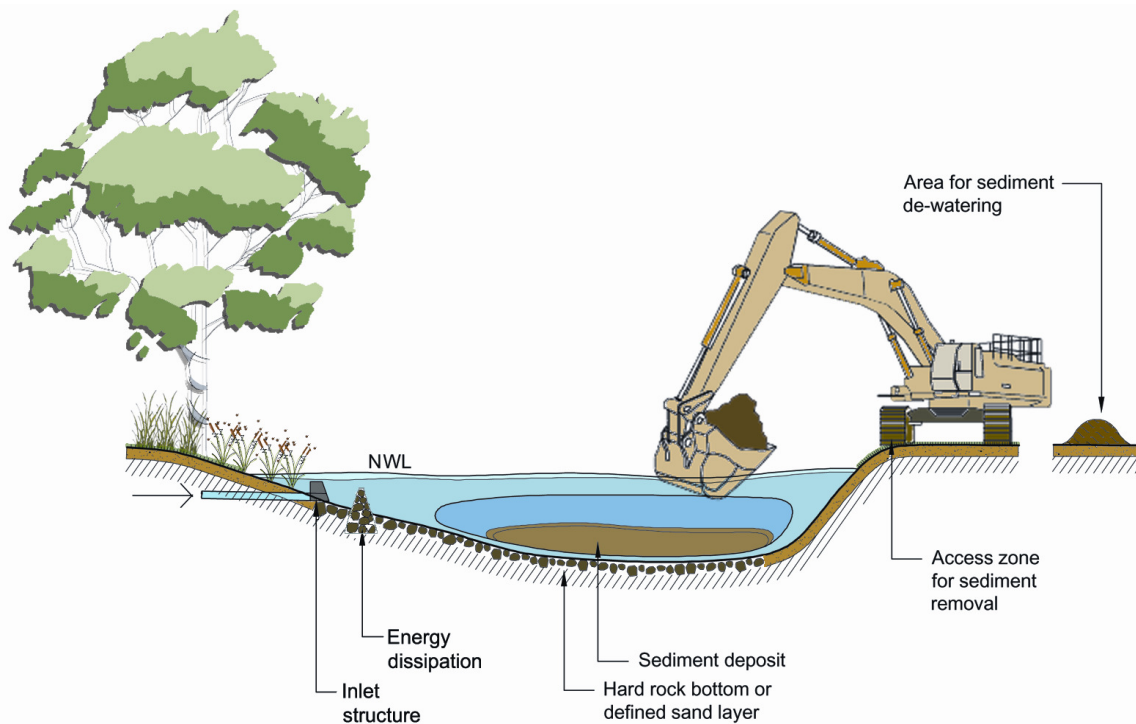
## 3.1 Introduction

The reduction of sediment loads is a key process in protecting downstream waterways ensuring the effective operation and long term efficiency of downstream stormwater treatment measures (known as treatment trains). Sedimentation basins are specifically employed to remove coarse to medium-sized sediments (generally 0.125mm and greater) by reducing flow velocities and providing retention of these particulates.

Sedimentation basins can take various forms (generally ponds) and can be classified as any treatment system that primarily functions to promote settling of sediments through processes of temporary detention and reduction of flow velocities

They can be made permanent structures and incorporated into urban designs (parkland or recreational area water features) or temporary structures to control sediment runoff during the development of new areas.

Figure 3.1 shows the layout of a typical permanent sedimentation basin.



**Figure 3.1. Sedimentation basin layout**

The required treatment size of a sedimentation basin is calculated primarily on two main factors:

1. The settling velocity of the target sediment size, and
2. The design flow at the required design rainfall intensity.

Analysis of typical urban catchment sediment loads suggest that between 50% to 80% of suspended solids conveyed in urban stormwater are 125  $\mu\text{m}$  or larger. Basins sized to target these size particulates are expected to capture sediment that has low levels of contamination (because of the larger sediment sizes) and is unlikely to require special handling and disposal.

Particular care needs to be undertaken when sizing sedimentation systems because a basin that is under sized may have limited effectiveness (insufficient retention of particulates) and cause smothering of downstream treatment measures reducing their treatment efficiency. Conversely, basins that are oversized run an increased risk of excess accumulation of fine particulates resulting in higher contaminant concentrations that could require specialist handling facilities for maintenance (de-watering and disposal).

A further consideration in sizing a sedimentation basin is the provision of adequate storage for settled sediment to prevent the need for frequent de-silting. A desirable maintenance frequency for permanent facilities is once every five years (normally triggered when sediment accumulates to half the basin depth). Temporary systems will require more frequent maintenance depending on the catchment area and likely sediment loads.

Apart from the issues associated with the appropriate sizing of a sedimentation basin for effective capture and retention of sediment, design considerations are similar to those for ponds and constructed wetlands.

### 3.2 Verifying size for treatment

Figure 3.2 shows relationships between a required basin area and design discharge for 125  $\mu\text{m}$  sediment capture efficiencies of 70%, 80% and 90% using a typical shape and configuration ( $\lambda = 0.5$ , see Section 4.3.2).

The influence of a permanent pool reduces flow velocities in the sedimentation basin and thus increases detention times in the basin (and hence removal efficiency). A typical two metre depth permanent pool was used to define the lower limit of the required sedimentation basin thus forming three shaded areas in Figure 3.2.

The performance of typical designs of sedimentation basins can be expected to fall within the shaded curves shown and they can be used to verify the selected size of a proposed sedimentation basin. As the design charts relate the size of a required sedimentation basin to a design flow, they are applicable in all regions and do not require any adjustments for the different hydrologic regions.

The volume of a permanent pool in a sedimentation basin should have sufficient capacity to ensure that desilting of the basin is not more frequent than once every 5 years (unless it is to be used for temporary sediment control when cleaning every 6-months may be appropriate).

A developing catchment can be expected to discharge between 50  $\text{m}^3$  and 200  $\text{m}^3$  of sediment per hectare each year. In a developed catchment, the annual sediment export is generally one to two orders of magnitude lower with an expected mean annual rate of 1.60  $\text{m}^3/\text{ha}$ . There are different methods used to estimate sediment loads and some authorities have produced charts of sediment loading rates (ACT Government, 1994; NSW Department of

Housing, 1998). Desilting is required when the permanent pool is half full with deposited sediment.

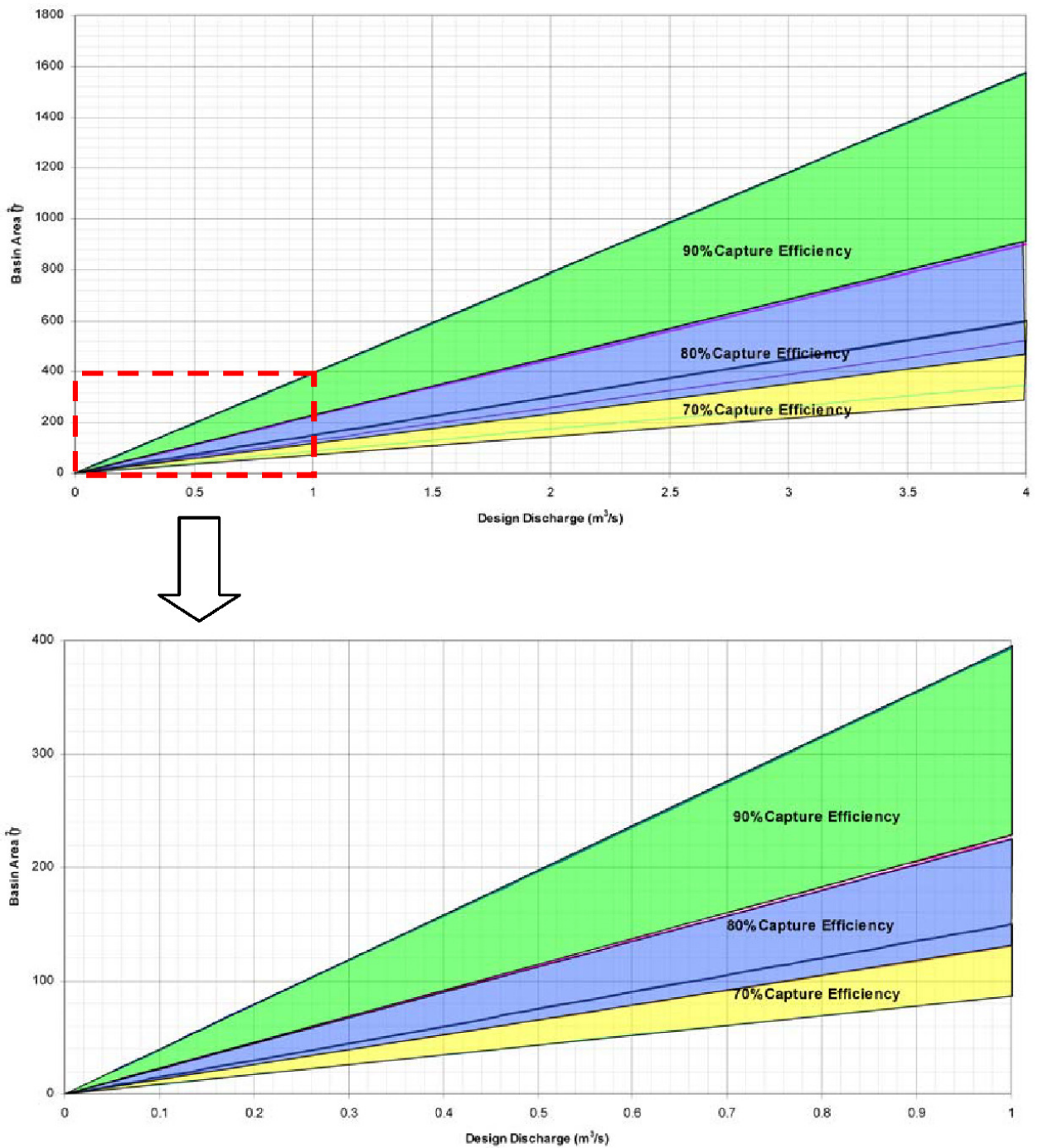


Figure 3.2. Sedimentation Basin Area Vs Design Discharges for varying capture efficiencies of 125 µm sediment

### 3.3 Design procedure: sedimentation basins

#### 3.3.1 Estimating design flows

##### 3.3.1.1 *Design Discharges*

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

- Design operation flow – required to enable calculation of the size of the basin to allow settling and retention of the design particulate size (Local councils and regional catchment management authorities may stipulate the design operation flow).
- Minor system design flow – for the design of the inlet and outlet structures to ensure system flows can be accommodated.
- Major flood flows – for the design of the basin overflow structure.

Typical design operation flows are the 1–year ARI (Average Recurrence Interval) or 2–year ARI peak discharge, and for permanent sedimentation basins used as pre–treatments for downstream stormwater treatment measures, this is normally the 1 year ARI peak design.

The design of inflow structures are typically based on higher ARI rainfall events as they need to have capacity to convey the design discharge of the upstream stormwater drainage system. In Tasmania the general design ARI for stormwater drainage systems (other than specific flood–way or conveyance structures) is the 10 year ARI. High risk areas such as commercial, industrial or dense residential may be designed for a 20 year ARI. Local authorities must be consulted prior to design to confirm the desired design ARI.

All flows should be directed through a sedimentation basin so that some level of sedimentation is achieved even during high flow conditions. In situations where the basin forms part or a major drainage system (through flow) the capacity of the inlet and outlet structures must be equal to or greater than the design ARI of the upstream and downstream infrastructure. Local authorities must be consulted prior to design to confirm the design ARI of these components.

##### 3.3.1.2 *Minor and major flood estimation*

Catchment peak discharge rates can be calculated using a range of hydrologic methods. The Rational Method Design Procedure is suitable for most, however for large catchments (greater than 100Ha), the Kinematic Wave Equation may be employed. These methods are described in detail in the document titled Australian Rainfall and Runoff.

Flood estimation in larger or complex catchments with multiple catchments and flow routing requires specialist knowledge of flood estimation methods and normally involves detailed topographical, hydrologic and hydraulic study of the catchment area. The Rational Method or Kinematic Wave Equation should not be relied upon singly for estimation in these situations.

## 3.3.2 Size and shape of sedimentation basins

Estimating the required area (A) of a sedimentation basin may be based on the mathematical expression derived by Fair and Geyer (1954), formulated for wastewater sedimentation basin design:

$$R = 1 - \left( 1 + \frac{1}{n} \frac{v_s}{Q/A} \right)^{-n}$$

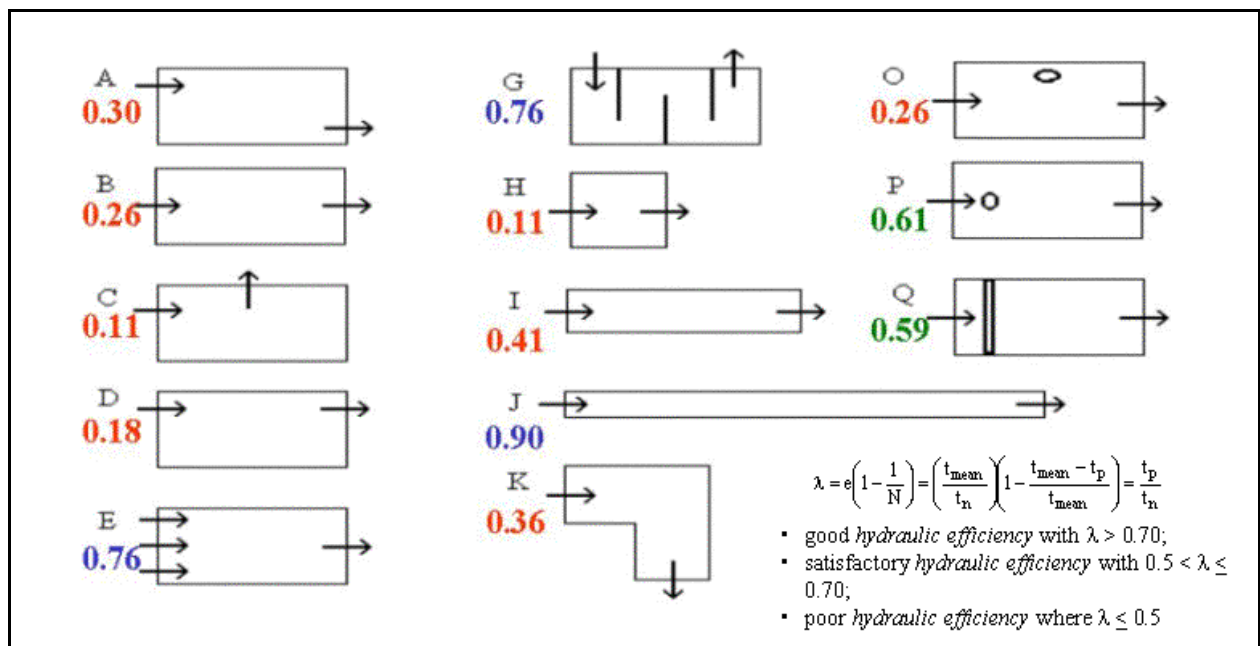
**Equation 3.1**

where

R	=	fraction of target sediment removed
$v_s$	=	settling velocity of target sediment
Q/A	=	rate of applied flow divided by basin surface area
n	=	turbulence or short-circuiting parameter

Note that Equation 3.1 is strictly applicable for systems with no permanent pool, and will generally over-estimate the required area of a sedimentation basin. This equation is thus often considered to provide an upper limit estimate of the required size for sedimentation basins.

In Equation 3.1 the key factor for design is the 'n' value which is known as the turbulence parameter. This factor takes into account the configuration of the basin (location of inlet and outlet structures, flow spreaders, flow diversion structure for mixing etc) and the effects of the different configurations on settling efficiency directly relating to the ability of the system to short circuit. In order to calculate 'n' a hydrodynamic adjustment value ( $\lambda$ ) must be selected that best represents the configuration of the basin and is described in Figure 3.3.



**Figure 3.3. Hydraulic Efficiency -  $\lambda$  - A measure of Flow Hydrodynamic Conditions in Constructed Wetlands and Ponds; Range is from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment (Persson et al., 1999)**

Figure 3.3 provides some guidance on what is considered to be good basin design with the higher values (of  $\lambda$ ) representing basins with good sediment retention properties, where a value of  $\lambda$  greater than 0.5 should be a design objective. If the basin configuration yields a lower value, modification to the basin configuration should be explored to increase the  $\lambda$  value (e.g. inclusion of baffles, islands or flow spreaders).

The shape of the basin has a large impact on the effectiveness of the basin to retain sediments and generally a length to width ratio of at least 3:1 should be aimed for. In addition, the location of the inlet and outlet, flow spreaders and internal baffles impact the hydraulic efficiency of the basin for stormwater treatment. These types of elements are noted in Figure 3.3 as the figure “o” in diagrams O and P (which represent islands in a waterbody) and the double line in diagram Q which represents a structure to distribute flows evenly.

### **DESIGN ADVICE –**

Consideration of maintenance access to a basin is also required when developing the shape of a basin as this can impact the allowable width (if access is from the banks) or the shape if access ramps into a basin are required. An area for sediment de-watering should also be accommodated and it is required to drain back into the basin. This too may impact on the footprint area required for a sedimentation basin system.

Once a design layout has been derived and an appropriate value of  $\lambda$  has been selected, a value for ‘n’ is then calculated using the following relationship:

$$\lambda = 1 - 1/n; \quad n = \frac{1}{1-\lambda}$$

**Equation 3.2**

### **DESIGN ADVICE –**

Good practice in the design of sedimentation basins will include a permanent pool of water to reduce flow velocities and provide storage of settled sediment. The presence of a permanent pool reduces flow velocities in the sedimentation basin and thus increases detention times.

With the outlet structure being located above the bed of a sedimentation basin, it is also not necessary for sediment particles to settle all the way to the bottom of the basin to be effectively retained. A reasonable depth is considered to be approximately 1 m below the permanent pool level.



If a permanent pool is to be employed, Equation 3.1 can be re-derived to account for the effect of the permanent pool storage as follows:

$$R = 1 - \left[ 1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)} \right]^{-n}$$

**Equation 3.3**

- where  $d_e$  is the extended detention depth (m) above the permanent pool level  
 $d_p$  is the depth (m) of the permanent pool  
 $d^*$  is the depth below the permanent pool level that is sufficient to retain the target sediment (m) – adopt 1.0 or  $d_p$  whichever is lower  
 $R$  is the fraction of target sediment removed  
 $v_s$  is the settling velocity of target sediment  
 $Q/A$  is the rate of applied flow divided by basin surface area  
 $n$  is the turbulence or short-circuiting parameter

The table below lists the typical settling velocities of sediments.

**Table 3-1. Settling velocities under ideal conditions**

Classification of particle size	Particle diameter (µm)	Settling velocities (mm/s)
Very coarse sand	2000	200
Coarse sand	1000	100
Medium sand	500	53
Fine sand	250	26
Very fine sand	125	11
Coarse silt	62	2.3
Medium silt	31	0.66
Fine silt	16	0.18
Very fine silt	8	0.04
Clay	4	0.011

A further check to confirm the size of a sedimentation basin is the required volume for storage of accumulated sediments and the impact of this volume on required cleaning frequencies.

Loading rates (estimates of loading rates are described in Section 3.2) can then be used to estimate the required storage volume for each clean out and this volume checked against the allowable sediment accumulation volume given the basin configuration estimated using Equation 3.1 or Equation 3.3. The storage volume should be estimated using half the permanent pool volume as this level of accumulation is recommended to be adopted as the trigger for a clean out.

The required volume of sediment storage (S) can be estimated using Equation 3.4:

$$S_t = C_a \times R \times L_o \times F_r$$

**Equation 3.4**

- Where:
- $S_t$  = volume of storage required (m<sup>3</sup>)
  - $C_a$  = contributing catchment area (ha)
  - R = capture efficiency (%), estimated from Equation 4.1 or 4.3
  - $L_o$  = sediment loading rate (m<sup>3</sup>/ha/year) \*\*
  - $F_r$  = desired cleanout frequency (years)

The fraction of sediment removed for the target pollutant (R) is assumed to represent the fraction of the total sediment load removed. In fact, a higher fraction of coarser particles than the target pollutant and a lower fraction for finer particles will be retained than the R-value. R however provides a reasonable estimate of the overall sediment capture efficiency.

\*\* – This figure can be obtained from widely available sediment yield tables. A table should be selected that matches the proposed development type, local soil conditions and topography.

### 3.3.3 Cross Sections

With the exception of temporary sedimentation basins for construction site management, batter slopes on approaches and immediately under the water line of a basin should be configured with the utmost consideration to public safety.

Generally there are two approaches to construction: either hard or soft edge treatments. These can be applied individually or combined to compliment the landscape of a surrounding area.

Soft edge treatments involve using gentle slopes to the waters edge, extending below the water line for a distance before batter slopes steepen into deeper areas. This is illustrated in Figure 3.4.

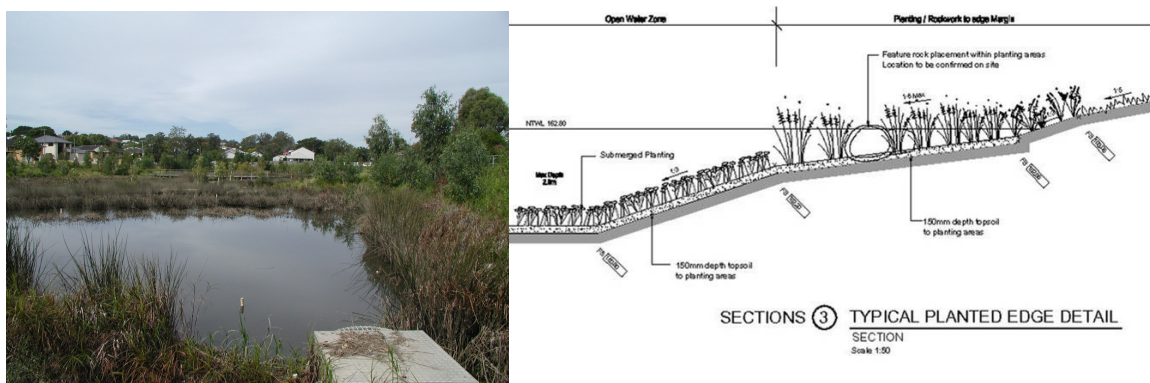


Figure 3.4. Illustration of a soft edge treatment for open waterbodies (GblA, 2004)

An alternative to the adoption of a flat batter slope beneath the water line is to provide a 3 m “safety bench” that is less than 0.2 m deep below the permanent pool level around the waterbody.

Figure 3.5 shows two options for hard edge details. One has a larger vertical wall and associated handrail for public safety and the other is a low vertical wall. In both hard edge details, it is proposed to line the bottom of the waterbody with rock to prevent vegetation (particularly weed) growth.

The safety requirements for individual basins may vary from site to site, and it is recommended that an independent safety audit be conducted of each design.

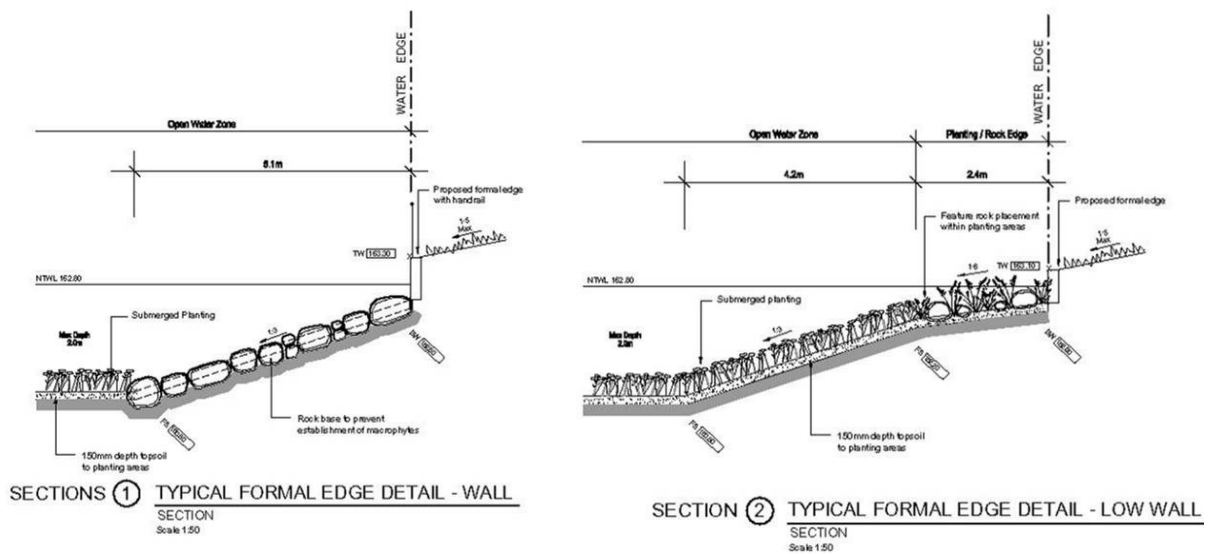


Figure 3.5. Illustration of Hard Edge Treatment for Open Waterbodies (GbLA, 2004)

### 3.3.4 Hydraulic Structures

Hydraulic structures are required at the inlet and outlet of a sedimentation basin. Their function is essentially one of conveyance of flow with provisions for (i) energy dissipation at the inlet structure(s), (ii) extended detention (if appropriate) at the outlet, and (iii) overflow pathway for above design conditions.

### 3.3.4.1 Inlet Structure

Sedimentation basins are generally configured so that flows from the upstream reticulated system discharge directly into the basin. The inlet structure must therefore have the capacity to adequately convey these flows. It is necessary to ensure that at the point of discharge, energy is adequately dissipated to minimise localised scouring.

There are many methods and designs of energy dissipating and scour reducing structures and as such they will not be covered in detail in this document. Local authorities may have standard drawings of preferred methods of velocity reduction structures and scour protection systems. Additionally see the resource appendices for further references.

Litter control is also normally required at an inlet structure and it is generally recommended that some form of gross pollutant trap (GPT) be installed as part of an inlet structure. The provision of a GPT will depend on catchment activities as well as any upstream measures in place. There are a number of proprietary products for removing gross pollutants and these are discussed in Chapter 7 in *Australian Runoff Quality* (Engineers Australia, 2006) and additional references are provided in the resource appendices.

### 3.3.4.2 Outlet Structure

An outlet structure of a sedimentation basin can be configured in many ways and is dependent on the specified operation of the system (ie. whether designed as a “stand-alone” sedimentation basin for managing construction site runoff or as part of a wetland system).

The outlet structure generally consists of an outlet pit and discharge control structure to control the rate of discharge from the basin under normal operation. During events that exceed the normal design flow, the discharge control structure must have adequate capacity to convey the design operation flow.

#### **Structures for construction sites –**

As a sedimentation basin for managing runoff from a construction site, landscape amenity is not an important design outcome. Therefore, floating discharge control structures, as shown in Figure 3.6, are considered the most effective outlets for construction sedimentation basins. They draw flows from the surface, which generally have the lowest suspended sediment concentrations. The discharge control structure consists of one or more slotted pipes mounted with floats to enable them to rise with the progressive filling of the basin as shown in Figure 3.6. Discharge from the basin is maintained at a relatively constant rate independent of the depth of water in the basin.

#### **Permanent applications –**

A weir is a more appropriate outlet structure for permanent sedimentation basins, and those that also serve as a landscape element. Where possible, a narrow weir structure (see Figure 3.6) should be adopted to promote a larger extended detention depth range while ensuring adequate capacity to convey the design discharge.



Figure 3.6. Sedimentation Basin Outlet Structures (L) a floating skimmer and (R) a narrow weir

### Design considerations –

An outlet pit and associated discharge control structure should be designed with the following considerations in mind:

- Ensure that the crest of the pit is set at or above the permanent pool level of the sedimentation basin
- Ensure that the dimension of the pit provides discharge capacity that is greater than or equal to the discharge capacity of the inlet structure
- Discharge capacity does not exceed that of the downstream infrastructure
- Protection is provided against blockage by flood debris
- Maintenance is simple to undertake and suitable provisions are made for access

Figure 3.7 summarises the design elements of the various components of a sedimentation basin.

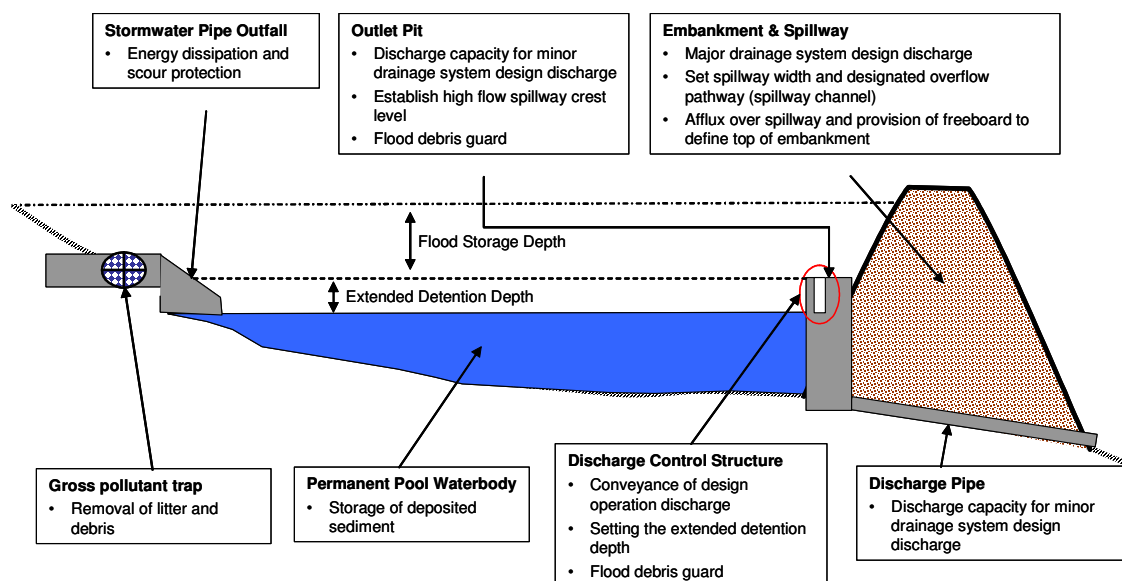


Figure 3.7. Overview of Design Elements of a Sedimentation Basin and Main Design Considerations

### Hydraulic Design of the Outlet Structure

An outlet structure is sized with a discharge capacity equal to or greater than the minor drainage system (e.g. 5 year ARI). The two flow conditions that must be considered when calculating the dimensions of an outlet pit are weir and orifice flow (Equations 3.5 and 3.6)

A blockage factor is also employed to account for any blockage of the structure by debris. It is generally considered that a blockage factor of 50% reasonably represents real world conditions.

1. **Weir flow condition** – *when free overall conditions occur over the pit (usually when the extended detention storage of the retarding basin is not fully engaged), ie.*

$$P = \frac{Q_{des}}{B \cdot C_w \cdot H^{1.5}}$$

Equation 3.5

- P = Perimeter of the outlet pit  
B = Blockage factor (0.5)  
H = Depth of water above the crest of the outlet pit  
Q<sub>des</sub> = Design discharge (m<sup>3</sup>/s)  
C<sub>w</sub> = weir coefficient (1.7)

2. **Orifice flow conditions** – *when the inlet pit is completely submerged (corresponding to conditions associated with larger flood events), ie.*

$$A_o = \frac{Q_{des}}{B \cdot C_d \sqrt{2gH}}$$

Equation 3.6

- C<sub>d</sub> = Orifice Discharge Coefficient (0.6)  
B = Blockage factor (0.5)  
H = Depth of water above the centroid of the orifice (m)  
A<sub>o</sub> = Orifice area (m<sup>2</sup>)  
Q<sub>des</sub> = Design discharge (m<sup>3</sup>/s)

It is important that an outlet pit is prevented from blockage by debris. Design consideration needs to include means of preventing blockage of the outlet structure.

Examples of this are shown below.



### Discharge Control Structure

Three types of discharge control structures can be used, ie.

1. Overflow weir – the length of the weir computed with the weir flow equation (Equation 3.5) but checked to ensure that there is adequate capacity when the structure operates under submerged conditions using the orifice flow equation (Equation 3.6).
2. Riser outlet – a vertical pipe with orifices located along the length of the pipe. The placement of outlet orifices and determining their appropriate diameters is designed iteratively by varying outlet diameters and levels, using the orifice equation (Equation 3.6) applied over discrete depths along the length of a riser up to the maximum detention depth. This can be performed with a spreadsheet as illustrated in the worked example. (see Chapter 8 for more discussion.)
3. Floating slotted pipe – the size and number of slots required to pass the operation design flow can be computed using the orifice flow equation (Equation 3.6).

With riser-type discharge control structures, an outlet orifice is likely to be small and it is important that these are prevented from clogging by debris. Some form of debris guard is recommended as illustrated in the images below.



### 3.3.5 Overflow Structure

The provision of a high-flow overflow structure is an essential design element. An overflow structure is normally a weir spillway structure. The required width of the spillway can be computed using the weir flow equation (Equation 3.5) with the design discharge being selected according to discussion in Section 3.3.1.1.



**Figure 3.8. Overflow Structure of a Sedimentation Basin**

### 3.3.6 Vegetation specification

Vegetation planted along the littoral zone of a sedimentation basin serve the primary function of inhibiting public access to the open waterbody and preventing edge erosion. Terrestrial planting beyond the littoral zone may also be recommended to screen areas and provide an access barrier to uncontrolled areas of the stormwater treatment system.

Appendix 'B' provides a list of suggested plant species suitable for sedimentation basin littoral zones in Tasmania.

### 3.3.7 Design calculation summary

The table on the following page provides a design calculation summary sheet for the key design elements of a sedimentation basin to aid the design process.



## Sedimentation Basin

## CALCULATION CHECKSHEET

CALCULATION TASK	OUTCOME	CHECK
<b>1 Identify design criteria</b>		
Design ARI Flow for inlet hydraulic structures		year
Design ARI Flow for outlet hydraulic structures		year
Design ARI for overflow hydraulic structures		year
		<input type="text"/>
<b>2 Catchment characteristics</b>		
	Residential	Ha
	Commercial	Ha
	Roads	Ha
<b>Fraction impervious</b>		
	Residential	
	Commercial	
	Roads	
		<input type="text"/>
<b>3 Estimate design flow rates</b>		
<b>Time of concentration</b>		
estimate from flow path length and velocities		minutes
		<input type="text"/>
<b>Identify rainfall intensities</b>		
	station used for IFD data:	
	Design Rainfall Intensity for inlet structure(s)	mm/hr
		<input type="text"/>
<b>Design runoff coefficient</b>		
	inlet structure(s)	
		<input type="text"/>
<b>Peak design flows</b>		
	Inlet structure(s)	m <sup>3</sup> /s
	Outlet structure(s)	m <sup>3</sup> /s
	Overflow structure(s)	m <sup>3</sup> /s
		<input type="text"/>
<b>4 Basin Dimension and Layout</b>		
	Area of sedimentation basin	m <sup>2</sup>
	Aspect Ratio	L:W
	Hydraulic Efficiency	
	Depth of permanent pool	m
	Permanent Pool Volume	m <sup>3</sup>
	Cross Section Batter Slope	V:H
		<input type="text"/>
<b>5 Basin Performance</b>		
	Capture efficiency (of 125 µm sediment)	%
	Sediment Cleanout Frequency	years
		<input type="text"/>
<b>6 Hydraulic Structures</b>		
<b>Inlet Structure</b>		
	Provision of energy dissipation	
<b>Outlet Structure</b>		
	Pit dimension	L x B
	or	mm diam
	Discharge capacity of outlet	m <sup>3</sup> /s
	Provision of debris trap	
		<input type="text"/>
<b>Discharge Pipe</b>		
	Discharge Capacity of Discharge Pipe	m <sup>3</sup> /s
		<input type="text"/>
<b>7 Spillway</b>		
	Discharge Capacity of Spillway	m <sup>3</sup> /s
	Afflux	m
	Freeboard to top of embankment	m
		<input type="text"/>

### 3.4 Checking tools

This section provides a number of checking aids for designers and referral authorities. In addition, advice on construction techniques and lessons learnt from building sediment basins are provided.

Checklists are provided for:

- Design assessments
- Construction (during and post)
- Operation and maintenance inspections
- Asset transfer (following defects period).

#### 3.4.1 Design assessment checklist

The checklist below presents the key design features that should be reviewed when assessing a design of a sediment basin either for temporary or permanent use. These considerations include configuration, safety, maintenance and operational issues that should be addressed during the design phase.

Where an item results in an “N” when reviewing the design, referral should be made back to the design procedure to determine the impact of the omission or error.

In addition to the checklist, a proposed design should have all necessary permits for its installations. The referral agency should ensure that all relevant permits are in place. These can include permits to clear vegetation, to dredge, create a waterbody, divert flows or disturb fish or platypus habitat.

Land ownership and asset ownership are key considerations prior to construction of a stormwater treatment device. A proposed design should clearly identify the asset owner and who is responsible for its maintenance. The proposed owner should be responsible for performing the asset transfer checklist (see 3.4.4).

Sediment Basin Design Assessment Checklist				
<b>Basin Location:</b>				
<b>Hydraulics</b>	Minor Flood: (m <sup>3</sup> /s)	Major Flood: (m <sup>3</sup> /s)		
<b>Area</b>	Catchment Area (ha):		Basin Area (ha)	
<b>Treatment</b>			<b>Y</b>	<b>N</b>
Treatment performance verified from curves?				
<b>Basin Configuration</b>			<b>Y</b>	<b>N</b>
Inlet pipe/structure sufficient for maximum design flow (minor and/or major flood event)?				
Scour protection provided at inlet?				
Basin capacity sufficient for maintenance period >=5 years?				
Configuration of basin (aspect, depth and flows) allows settling of particles >125µm?				
Maintenance access allowed for into base of sediment basin?				
Public access to inlet zone prevented through vegetation or other means?				
Gross pollutant protection measures provided on inlet structures?				
Freeboard provided above extended detention depth?				
Batter slopes shallow or safety bench provided in case of accidental entry into basin?				
<b>Hydraulic Structures</b>			<b>Y</b>	<b>N</b>
Outlet perimeter >= design discharge of outlet pipe?				
Outlet configuration suitable for basin type (eg riser for construction sediment, weir for wetland pretreatment etc)?				
Riser diameter sufficient to convey Q <sub>1</sub> flows when operating as a “glory hole” spillway?				
Maintenance drain provided?				
Discharge pipe from has sufficient capacity to convey the maintenance drain flows or Q <sub>1</sub> flows (whichever is higher)?				
Protection against clogging of orifice provided on outlet structure?				

### 3.4.2 Construction advice

This section provides general advice for the construction of sedimentation basins. It is based on observations from actual construction projects around Australia.

#### Building phase damage

It is important to have protection from upstream flows during construction of a sediment basin. A mechanism to divert flows around a construction site and to provide protection from litter and debris is required.

#### High flow contingencies

Contingencies to manage risks associated with flood events during construction are required. All machinery should be stored above acceptable flood levels and the site stabilised as best as possible at the end of each day as well as plans for de-watering following storms made.

#### Maintenance access

An important component of a sediment basin is accessibility for maintenance. Should excavators be capable of reaching all parts of the basin an access track may not be required to the base of the inlet zone, however an access track around the perimeter of the basin would be required. If sediment collection is by using earthmoving equipment, then a stable ramp will be required into the base of the inlet zone (maximum slope 1:10).

#### Solid base

To aid maintenance it is recommended to construct the inlet zone either with a hard (ie rock or concrete) bottom or a distinct sand layer. These serve an important role for determining the levels that excavation should extend to during sediment removal (i.e. how deep to dig) for either systems cleaned from the banks or directly accessed. Hard bases are also important if maintenance is by driving into the basin.

#### De-watering removed sediments

An area should be constructed that allows for de-watering of removed sediments from a sediment basin. This allows the removed sediments to be transported as 'dry' material and can greatly reduce disposal costs compared to liquid wastes. This area should be located such that water from the material drains back into the basin. Material should be allowed to drain for a minimum of overnight before disposal.

#### Inlet checks

It is good practice to check the operation of inlet erosion protection measures following the first few rainfall events. It is important to check for these early in the systems life, to avoid continuing problems. Should problems occur in these events the erosion protection should be enhanced.

## 3.4.3 Construction checklist

### CONSTRUCTION INSPECTION CHECKLIST Sediment basin

INSPECTED BY:
DATE:
TIME:
WEATHER:
CONTACT DURING VISIT:

SITE: \_\_\_\_\_  
 CONSTRUCTED BY: \_\_\_\_\_

<b>DURING CONSTRUCTION</b>									
Items inspected	Checked		Satisfactory	Unsatisfactory		Checked		Satisfactory	Unsatisfactory
<b>Preliminary works</b>	Y	N			<b>Structural components</b>	Y	N		
1. Erosion and sediment control plan adopted					16. Location and levels of outlet as designed				
2. Limit public access					17. Safety protection provided				
3. Location same as plans					18. Pipe joints and connections as designed				
4. Site protection from existing flows					19. Concrete and reinforcement as designed				
<b>Earthworks</b>					20. Inlets appropriately installed				
5. Integrity of banks					21. Inlet energy dissipation installed				
6. Batter slopes as plans					22. No seepage through banks				
7. Impermeable (solid) base installed					23. Ensure spillway is level				
8. Maintenance access (eg. ramp) installed					24. Provision of maintenance drain				
9. Compaction process as designed					25. Collar installed on pipes				
10. Levels of base, banks and spillway as designed					<b>Vegetation</b>				
					25. Stabilisation immediately following earthworks				
					26. Planting as designed (species and densities)				
					27. Weed removal before stabilisation				

<b>FINAL INSPECTION</b>									
1. Confirm levels of inlets and outlets					6. Check for uneven settling of banks				
2. Confirm structural element sizes					7. Inlet erosion protection working				
3. Check batter slopes					8. Maintenance access provided				
4. Vegetation as designed					9. Construction generated sediment removed				
5. Draining area for maintenance provided									

**COMMENTS ON INSPECTION**


**ACTIONS REQUIRED**

1.
2.
3.
4.
5.
6.

3.4.4 Asset transfer checklist

<b>Asset Handover Checklist</b>		
<i>Asset Location:</i>		
<i>Construction by:</i>		
<i>Defects and Liability Period</i>		
<b>Treatment</b>	<b>Y</b>	<b>N</b>
System appears to be working as designed visually?		
No obvious signs of under-performance?		
<b>Maintenance</b>	<b>Y</b>	<b>N</b>
Maintenance plans provided for each asset?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
Asset inspected for defects?		
<b>Asset Information</b>	<b>Y</b>	<b>N</b>
Design Assessment Checklist provided?		
As constructed plans provided?		
Copies of all required permits (both construction and operational) submitted?		
Proprietary information provided (if applicable)?		
Digital files (eg drawings, survey, models) provided?		
Asset listed on asset register or database?		

### 3.5 Maintenance requirements

Sediment basins treat runoff by slowing flow velocities and promoting settlement of coarse to medium sized sediments. Maintenance revolves around ensuring inlet erosion protection is operating as designed, monitoring sediment accumulation and ensuring that the outlet is not blocked with debris.

Inspections of the inlet configuration following storm events should be made soon after construction to check for erosion. In addition, regular checks of sediment build up will be required as sediment loads from developing catchments or construction sites vary enormously. ***The basins should be cleaned out if more than half full of accumulated sediment.***

Similar to other types of practices, debris removal is an ongoing maintenance function. Debris, if not removed, can block inlets or outlets, and can be unsightly if located in a visible location. Inspection and removal of debris should be done regularly, but debris should be removed whenever it is observed on the site.

#### 3.5.1 Operation & maintenance inspection form

The form below should be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

Sediment Basin Maintenance Checklist			
<b>Inspection Frequency:</b>	3 monthly	<b>Date of Visit:</b>	
<i>Location:</i>			
<i>Description:</i>			
<i>Site Visit by:</i>			
Inspection Items	Y	N	Action Required (details)
Litter within inlet or open water zones?			
Sediment within inlet zone requires removal (record depth, remove if >50%)?			
Overflow structure integrity satisfactory?			
Evidence of dumping (building waste, oils etc)?			
Terrestrial vegetation condition satisfactory (density, weeds etc)?			
Weeds require removal from within basin?			
Settling or erosion of bunds/batters present?			
Damage/vandalism to structures present?			
Outlet structure free of debris?			
Maintenance drain operational (check)?			
Comments:			



### 3.6 Sedimentation basin worked example

#### 3.6.1 Worked example introduction

A sedimentation basin and wetland system is proposed to treat runoff from a highway. This worked example focuses on the sediment basin (inlet zone) component of the system.

The site is triangular in shape with a surface area of 500 m<sup>2</sup> as shown in Figure 3.9. Road runoff is conveyed by stormwater pipes (design capacity up to the 100 year ARI event) and there are two highway outfall pipes that discharge to the two top apexes of the site. Each outfall services about 1km of the highway with the total contributing area of 4Ha (90% impervious) to each outfall. The site of the site has a fall of approximately 2m (from 5m to 3m AHD) towards a degraded watercourse.

Site constraints limit the size available for the stormwater treatment system therefore it is important to ensure that the size of the inlet zones (ie. sedimentation basins) are not compromised, to ensure that larger sediments are effectively trapped and prevented from smothering the planted zone (thereby creating future maintenance problems).

The consequence of this action is the reduction in the overall hydrologic effectiveness of the system (i.e. the proportion of mean annual runoff subjected to the full wetland treatment), but not its functional integrity.

All stormwater runoff will be subjected to primary treatment, by sedimentation of coarse to medium size sediment. The inlet zone will operate under by-pass conditions more often owing to a smaller macrophyte zone in this case.

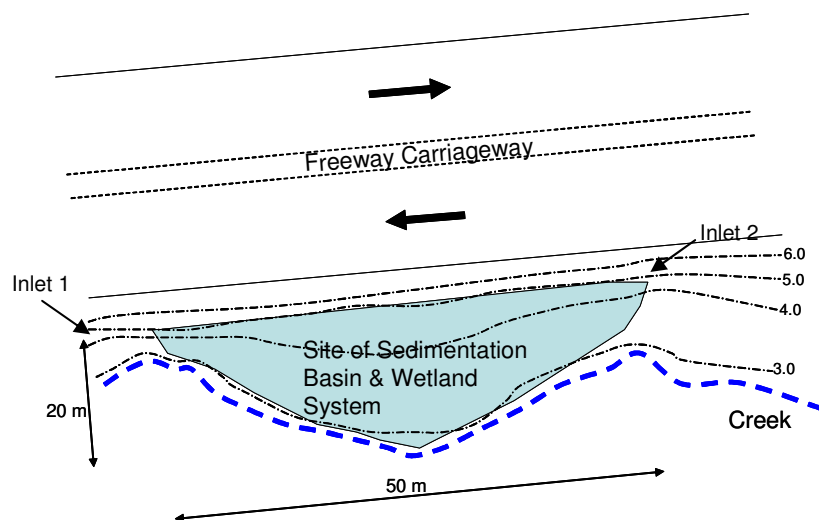


Figure 3.9 Layout of Proposed site for Sedimentation Basin

### 3.6.1.1 Design Objectives

As the sedimentation basins form part of a treatment train (with a small macrophyte wetland), sizing to meet the overall best practice stormwater quality objectives does not apply. Instead, the design requirements of the sedimentation basin system are to:

- Promote sedimentation of particles larger than 125 $\mu$ m (0.125mm) with a 90% capture efficiency for flows up to the 1-year ARI (un-attenuated) peak discharge.
- Provide for connection to the downstream macrophyte wetland zone with discharge capacity corresponding to the 1-year ARI (un-attenuated) peak discharge.
- Provide for by-pass operation when the inundation of the downstream macrophyte wetland zone reaches the design maximum extended detention depth with a discharge capacity corresponding to the 100-year ARI peak discharge.

Analyses to be undertaken during the detailed design phase include the following:

- Sizing the sedimentation basin (depth and area) using sedimentation theory (an extended detention depth of **0.25m** above the permanent pool level has been nominated to match the proposed maximum water level of the downstream macrophyte zone)
- Configure the layout of the basin such that the system *hydraulic efficiency* (see figure 3.3) can be optimised
- Design of the inlet structure to provide for energy dissipation of inflows up to the 100 year ARI peak discharge
- Design of by-pass structure to provide for flow by-pass of downstream wetland for events up to the 100 year ARI event
- Design of the basin outlet structure connecting to the macrophyte wetland zone, including debris trap.

In addition, landscape design will be required and this will include the following:

- Littoral zone vegetation
- Terrestrial vegetation

### 3.6.2 Estimating Design Flows

The procedures in Australian Rainfall and Runoff (ARR) are used to estimate the design flows. The site has two contributing catchments, each catchment is 4 Ha in area, 1km long (along the freeway) and is drained by culverts.

See Appendix E Design Flows –  $t_c$  for a discussion on methodology for calculation of time of concentration.

#### **Step 1 – Calculate the time of concentration.**

Velocity within the pipes is assumed to be 1m/s for the purposes of estimating the time of concentration ( $t_c$ ).

$$t_c = 1000\text{m} / 1\text{m/s} = 1000\text{s} = 17\text{ minutes}$$

Rainfall Intensities for the area of study (for the 1, 10 and 100 year average recurrence intervals) are estimated using ARR (1998) with a time of concentration of = 17 minutes and are:

$$I_1 = 27\text{ mm/hr}^*$$

$$I_{10} = 56\text{ mm/hr}^*$$

$$I_{100} = 95\text{ mm/hr}^*$$

\* These figures are for the worked example only. The appropriate region and corresponding rainfall intensities must be selected for each individual project.

**Step 2 – Calculate design runoff coefficients** (using the method outlined in Australian Rainfall and Runoff Book VIII (Engineers Australia, 2003)).

Where – Fraction impervious ( $f$ ) = 0.9

Rainfall intensity ( $^{10}I_1$ ) = 26.4mm/hr (from the relevant IFD chart)

Calculate  $C^{10}$  (pervious runoff coefficient)

$$C^{10} = 0.1 + 0.0133 (^{10}I_1 - 25) = 0.12$$

Calculate  $C_{10}$  (10 year ARI runoff coefficient)

$$C_{10} = 0.9f + C^{10} (1-f)$$

$$C_{10} = 0.82$$

**Step 3 – Convert  $C_{10}$  to values for  $C_1$  and  $C_{100}$**

Where –  $C_y = F_y \times C_{10}$

Runoff coefficients as per Table 1.6 Book VIII ARR 1998

ARI (years)	Frequency Factor, $F_y$	Runoff Coefficient, $C_y$
1	0.8	0.66
10	1.0	0.82
100	1.2	0.98

**Step 4 - Calculate peak design flow (calculated using the Rational Method).**

$$Q = \frac{CIA}{360}$$

Where –  
 C is the runoff coefficient ( $C_1$ ,  $C_{10}$  and  $C_{100}$ )  
 I is the design rainfall intensity mm/hr ( $I_1$ ,  $I_{10}$  and  $I_{100}$ )  
 A is the catchment area (Ha)

$$\begin{aligned} Q_1 &= 0.20 \text{ m}^3/\text{s} \\ Q_{10} &= 0.51 \text{ m}^3/\text{s} \\ Q_{100} &= 1.0 \text{ m}^3/\text{s} \end{aligned}$$

Operation Design Discharge = 0.20 m<sup>3</sup>/s  
 Design discharge for connection to microphyte zone = 0.20 m<sup>3</sup>/s  
 Spillway Design Discharge = 1.0 m<sup>3</sup>/s

### 3.6.3 Size and Shape of Sedimentation Basin

The inlet zone is to be sized to remove at least 90% of 125µm particles for the peak 1-year flow.

Pollutant removal is estimated using the following:

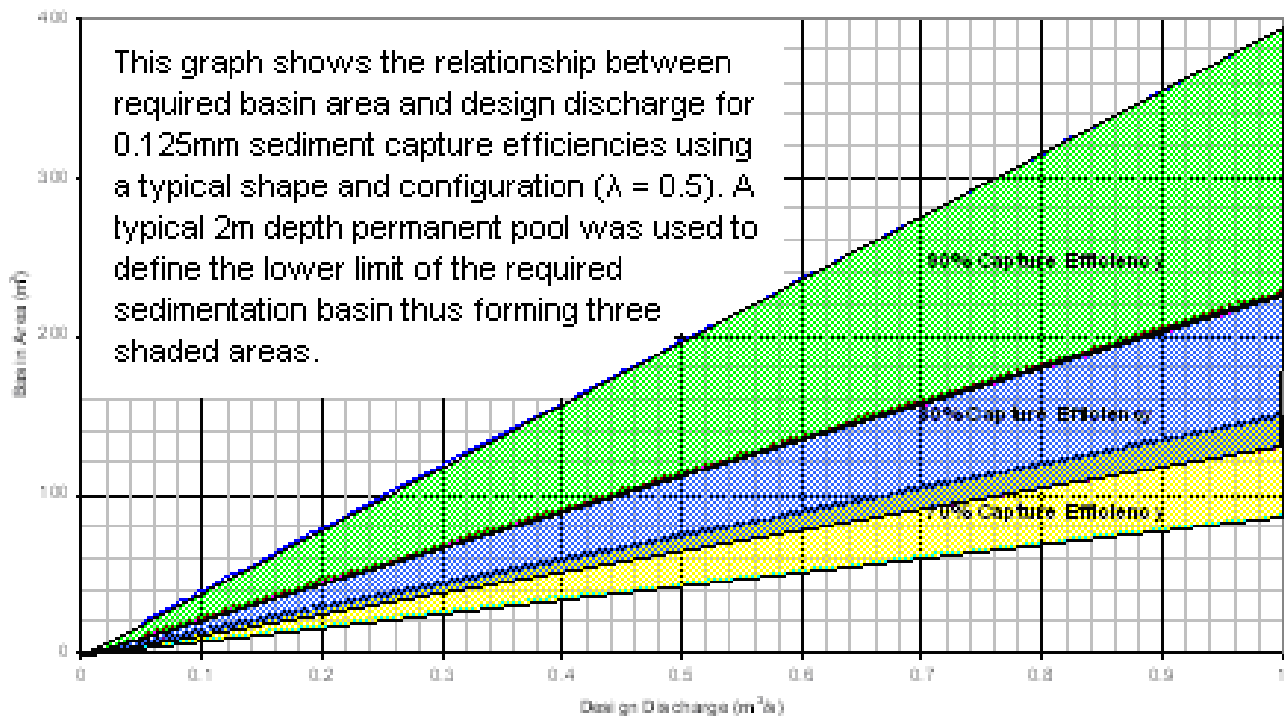
$$R = 1 - \left[ 1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)} \right]^{-n}$$

**An aspect ratio of 1 (w) to 4 (L) is adopted based on the available space (Figure 3.9). Using Equation 3.2, the hydraulic efficiency ( $\lambda$ ) is estimated to be 0.4. This value is less than desirable, however, site constraints prevent any other configuration. The turbulence factor (n) is computed from Equation 3.2**

Hydraulic efficiency ( $\lambda$ ) = 0.4  
 Turbulence factor (n) = 1.67

The proposed extended detention depth of the basin is 0.25 (as outlined in Section 3.6.1) and a notional permanent pool depth of 2 m has been adopted, ie.

$$\begin{aligned} d_p &= 2.0 \text{ m} & V_s &= 0.011 \text{ m/s for } 125\mu\text{m particles} \\ d^* &= 1.0 \text{ m} & Q &= \text{Design operation flow rate} = 0.20 \text{ m}^3/\text{s} \\ d_e &= 0.25 \text{ m} \end{aligned}$$



From above, the required sedimentation basin area to achieve target sediment (125 μm) capture efficiency of 90% is 50 m<sup>2</sup>. With a W to L ratio of 1:4, the notional dimensions of the basin are 3.5 m x 14 m. This figure correlates with the graph above.

The available sediment storage is 50 x 2 = 100 m<sup>3</sup>. Cleanout is to be scheduled when the storage is half full, therefore the available sediment storage prior to clean out is 50m<sup>3</sup>.

The required volume of sediment storage to ensure cleaning is not required more frequently than every five years is estimated using Equation 3.4 (using a sediment discharge rate of 1.6 m<sup>3</sup>/Ha/yr).

$$\begin{aligned} \text{Require storage } (S_0) &= C_a \times R \times L_o \times F_r \\ &= 4 \times 0.9 \times 1.6 \times 5 = 29 \text{ m}^3 \end{aligned}$$

Available storage volume is 50m<sup>3</sup>, therefore it is OK.

The required clean out frequency is estimated to be (by rearranging Equation 3.4):

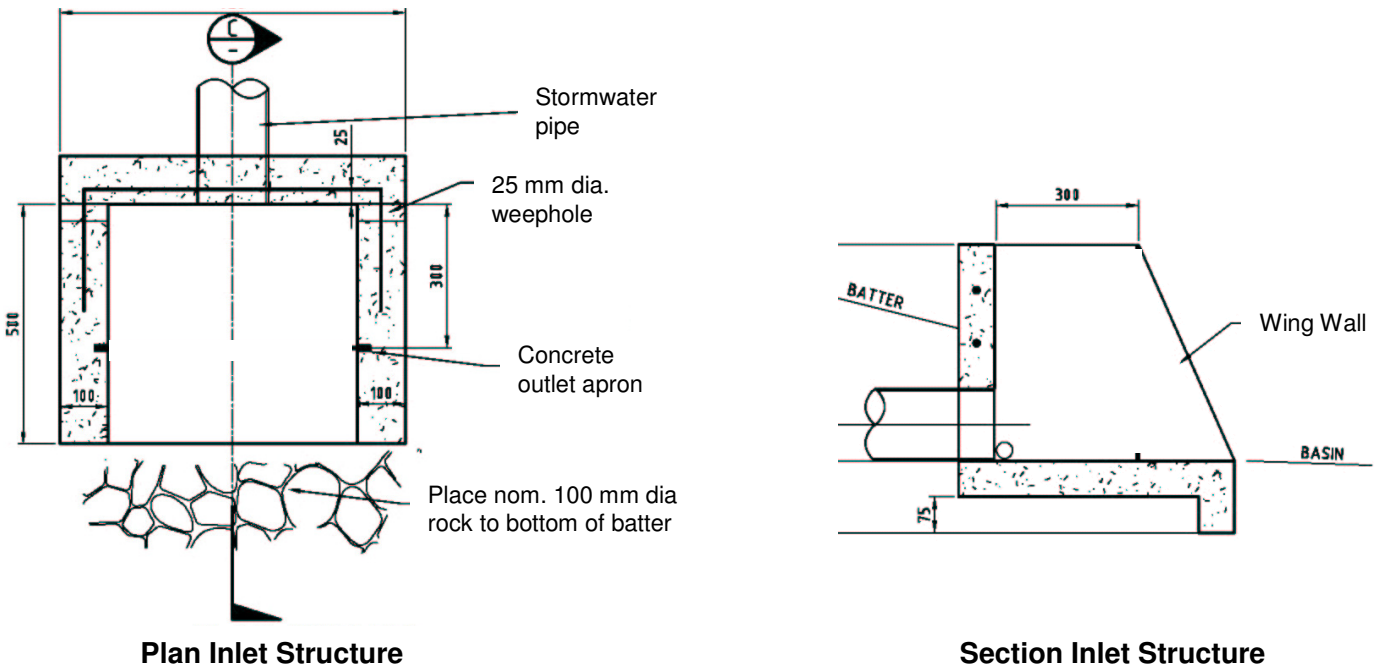
$$\text{Frequency of basin desilting} = \frac{0.5 \times 100}{1.6 \times 4 \times 0.9} = 8.6 \text{ years} > 5 \text{ years} \rightarrow \text{OK}$$

Open Water Area = 50 m<sup>2</sup>  
 Width = 3.5 m; Length = 12 m  
 Depth of Permanent Pool (d<sub>p</sub>) = 2.0 m  
 Depth of Extended Detention (d<sub>e</sub>) = 0.25m

## 3.6.4 Hydraulic Structure design

### 3.6.4.1 Inlet Structure

To prevent scour of deposited sediments from flows in the inlet pipes, it is necessary to limit velocities adjacent to the inlet to below 1 m/s. Culvert invert assumed to be RL 3.5m AHD  
 Rock beaching will be required in this area to ensure that excessive scour does not occur.



Provide energy dissipation and erosion protection in the form of rock beaching at the inlet structure;  $Q_{des} = 0.51 \text{ m}^3/\text{s}$ .

### 3.6.4.2 Outlet Structure

The outlet structure is to consist of an outlet pit with the top of the pit set at the permanent pool level, creating a permanent pool depth of 2 m. The dimension of the pit should ensure adequate discharge capacity to discharge the design flow for the connection to the macrophyte zone (ie. 1 year ARI peak discharge of  $0.2 \text{ m}^3/\text{s}$ ).

According to section 3.3.4.2, two possible flow conditions need to be checked, i.e. weir flow conditions (with extended detention of 0.25 m) and orifice flow conditions.

### *Weir Flow Conditions*

From Equation 3.5

, the required perimeter of the outlet pit to pass 0.2 m<sup>3</sup>/s with an afflux of 0.25 m can be calculated:-

$$P = \frac{Q_{des}}{B \cdot C_w \cdot H^{1.5}} = \frac{0.2}{0.5 \cdot 1.7 \cdot 0.25^{1.5}} = 1.88 \text{ m}$$

### *Orifice Flow Conditions*

From Equation 3.6, the required area of the outlet pit can be calculated as follows:-

$$A_o = \frac{Q_{des}}{B \cdot C_d \sqrt{2gH}} = \frac{0.2}{0.5 \cdot 0.6 \sqrt{2g(0.25)}} = 0.30 \text{ m}^2$$

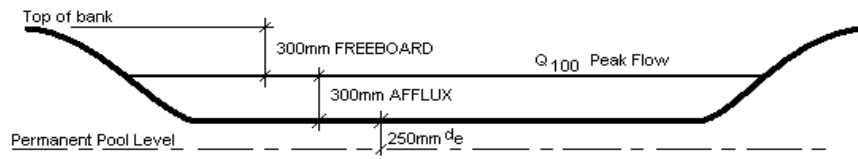
Adopt 600 x 600 mm pit: Area = 0.36 m<sup>2</sup>; Perimeter = 2.4 m; Q<sub>cap</sub> = 0.24 m<sup>3</sup>/s → OK

The top of the pit should be fitted with a standard grating to prevent flood debris from blocking the outlet pit.

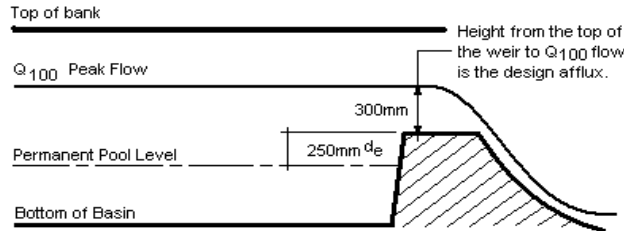
Outlet Pit = 600 x 600 mm diameter with standard grating

#### 3.6.4.3 Overflow Structure

The overflow structure must be capable of conveying the Q<sub>100</sub> peak flow. A weir with a crest elevation set at 0.25 m (ie. d<sub>e</sub>) above the permanent pool level, and an afflux of 300mm has been adopted in order to calculate the required length of the spillway weir.



LONG SECTION OF WEIR



CROSS SECTION OF WEIR

It is common practice to allow for 300 mm of freeboard above the afflux level when setting the top of embankment elevation. This value was adopted as a trade off between the bank height and the width of the weir. A bank height of 600mm (300mm afflux and 300mm freeboard) above the normal water level was deemed acceptable. The length is calculated using the weir flow equation with a weir coefficient of 1.7, i.e.

$$L = \frac{Q_{des}}{C_w \cdot H^{1.5}} = \frac{1.0}{1.7 \cdot 0.3^{1.5}} = 3.6 \text{ m}$$

Bypass weir is located adjacent to inflow culvert to minimise risk of sediment scour.

Spillway length = 3.6 m set at 0.25 m above permanent pool level  
 Top of embankment set at 0.6 m above the permanent pool level.

### 3.6.4.4 Discharge to Macrophyte Zone

A culvert connection between the sedimentation basin (inlet zone) and macrophyte zone will also need to be designed with the design criterion that the culvert will need to have adequate capacity to pass the 1 year ARI peak discharge when the water level in macrophyte zone is at its permanent pool level. This will also provide the flow control into the wetland.

The design calculation and configuration of this connection is described in Chapter 8 on constructed wetland design.

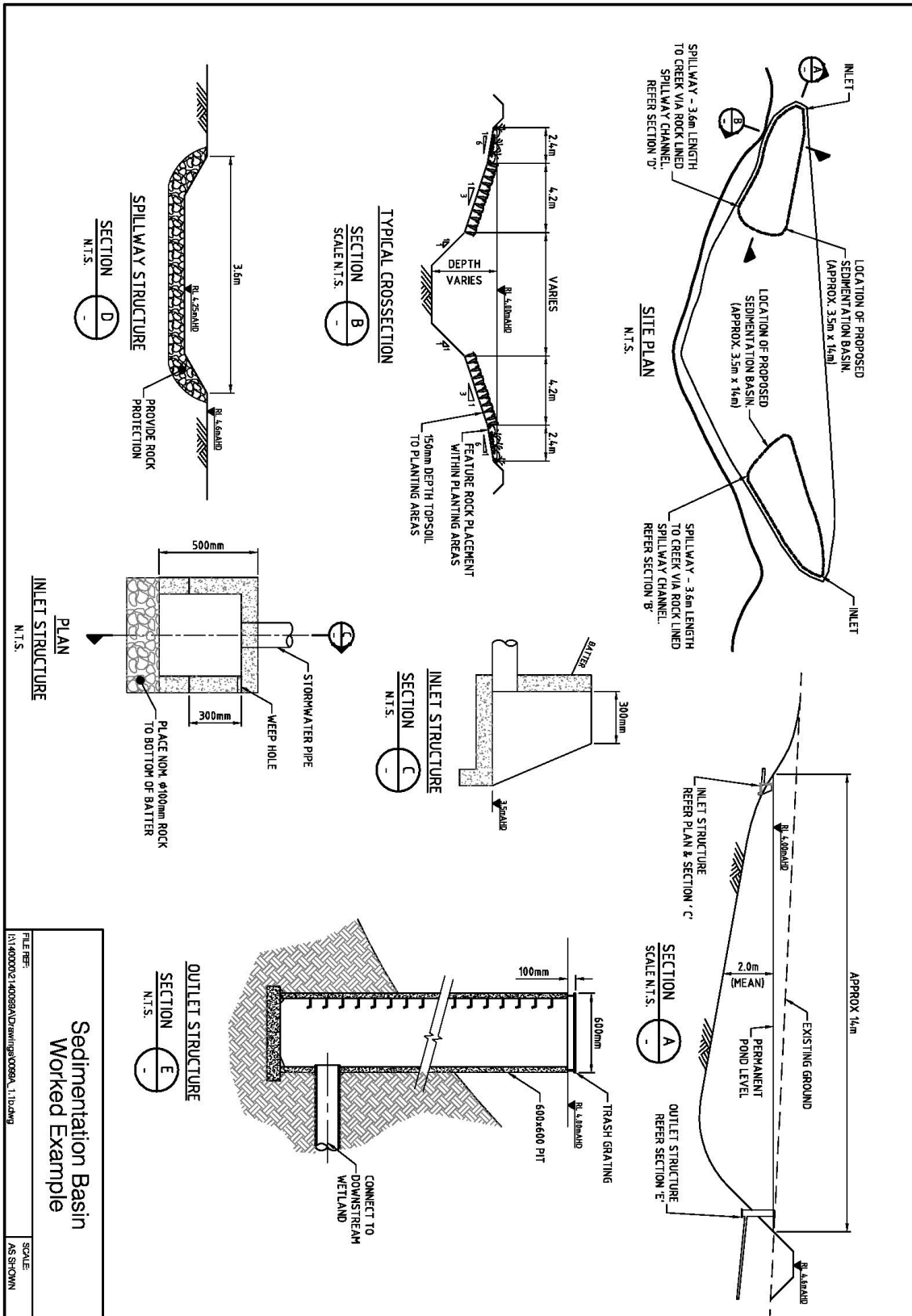


## Sedimentation Basin

## CALCULATION CHECKSHEET

CALCULATION TASK	OUTCOME	CHECK
<b>1 Identify design criteria</b>		<input checked="" type="checkbox"/>
Design ARI Flow for inlet hydraulic structures	10	year
Design ARI Flow for outlet hydraulic structures	1	year
Design ARI for overflow hydraulic structures	100	year
<b>2 Catchment characteristics</b>		<input checked="" type="checkbox"/>
Residential	0	Ha
Commercial	0	Ha
Roads	4	Ha
<b>Fraction impervious</b>		<input checked="" type="checkbox"/>
Residential	N/A	
Commercial	N/A	
Roads	0.9	
<b>3 Estimate design flow rates</b>		
<b>Time of concentration</b>		
estimate from flow path length and velocities	17	minutes
<b>Identify rainfall intensities</b>		
station used for IFD data:	Hobart	
Design Rainfall Intensity for inlet structure(s)	27 to 56	mm/hr
<b>Design runoff coefficient</b>		
inlet structure(s)	0.66 to 0.98	
<b>Peak design flows</b>		
Inlet structure(s)	0.51	m <sup>3</sup> /s
Outlet structure(s)	0.20	m <sup>3</sup> /s
Overflow structure(s)	1.00	m <sup>3</sup> /s
<b>4 Basin Dimension and Layout</b>		<input checked="" type="checkbox"/>
Area of sedimentation basin	50	m <sup>2</sup>
Aspect Ratio	4(L):1(W)	L:W
Hydraulic Efficiency	0.4	
Depth of permanent pool	2	m
Permanent Pool Volume	100	m <sup>3</sup>
Cross Section Batter Slope	1(V):8(H)	V:H
<b>5 Basin Performance</b>		<input checked="" type="checkbox"/>
Capture efficiency (of 125 µm sediment)	90	%
Sediment Cleanout Frequency	8.6	years
<b>6 Hydraulic Structures</b>		
<b>Inlet Structure</b>		<input checked="" type="checkbox"/>
Provision of energy dissipation	Y	
<b>Outlet Structure</b>		<input checked="" type="checkbox"/>
Pit dimension	600 x 600	L x B
or		mm diam
Discharge capacity of outlet	0.21	m <sup>3</sup> /s
Provision of debris trap	Y	
<b>Discharge Pipe</b>		<input checked="" type="checkbox"/>
Discharge Capacity of Discharge Pipe	0.2	m <sup>3</sup> /s
<b>7 Spillway</b>		<input checked="" type="checkbox"/>
Discharge Capacity of Spillway	1	m <sup>3</sup> /s

3.6.5 Construction drawings



### 3.7 References

ACT Government, 1994, *Standard Engineering Practices: Urban Stormwater*, Edition 1

Chow, V.T., 1959, *Open-Channel Hydraulics*, McGraw-Hill Book Co.

Engineers Australia, 2006, *Australian Runoff Quality Guidelines: A Guide to Water Sensitive Urban Design*, Editor in Chief, Wong, T.H.F., National Committee for Water Engineering

Fair G.M. and Geyer J.C., 1954, *Water Supply and Waste Disposal*, John Wiley and Sons, New York, Vol. 2

Graeme Bentley Landscape Architects (GbLA), 2004, *Preliminary drawings for Mernda Wetland*, Report for Stockland

Henderson, F.M., 1966, *Open Channel Flow*, Macmillan Publishing, New York

Institution of Engineers Australia, 1997, *Australian Rainfall and Runoff - A guide to flood estimation*, Editor in Chief - Pilgram, D.H.

NSW Department of Housing, 1998, *Managing Urban Stormwater: Soils and Construction*, 3<sup>rd</sup> Edition

Persson, J., Somes, N.L.G. and Wong T.H.F., 1999, *Hydraulic efficiency and constructed wetland and ponds*, Water Science and Technology Vol 40 No 3 pp 291-289