Chapter 4 **Sedimentation basins**

Sedimentation basin as an inlet zone for a constructed wetland.

4.1 Introduction

Reducing sediment loads is an important way to improve **stormwater** quality. **Sedimentation** basins are an integral component in a stormwater **treatment train** and are specifically employed to remove (by settling) coarse to medium-sized sediments. Sedimentation basins can take various forms and can be used as permanent systems integrated into an urban design or temporary measures to control sediment **discharge** during construction. They include all forms of stormwater detention systems that function primarily through sedimentation to promote settling of sediments through processes of temporary detention and reduction of flow velocities. Key design parameters are selecting a target sediment size, design discharge and sediment storage volume. Figure 4.1 shows the layout of a typical permanent sedimentation basin.

The required size of a sedimentation basin is calculated to match the settling velocity of a target sediment size with a design flow. Selecting a target sediment size is an important design consideration. As a pretreatment facility, selecting a sediment particle size of 125 μm is recommended as the target size.

Analysis of typical **catchment** sediment loads suggests that between 50% and 80% of suspended solids conveyed in urban stormwater are 125 μm or larger. Almost all sediment bed loads are larger than this target sediment size. However, coarse to medium-sized sediments have low concentrations of contaminant association compared to finer sediment and **colloidal particles**.

Analysis of the characteristics of particulate nutrients and metals indicate that these contaminants are mostly smaller than 50 μm and effective removal is best undertaken by treatment measures (e.g. **constructed wetlands**) other than sedimentation basins.

A sedimentation basin that is too small could have limited effectiveness and cause smothering of downstream treatment measures, thereby reducing their effectiveness in removing finer particulates and increasing maintenance.

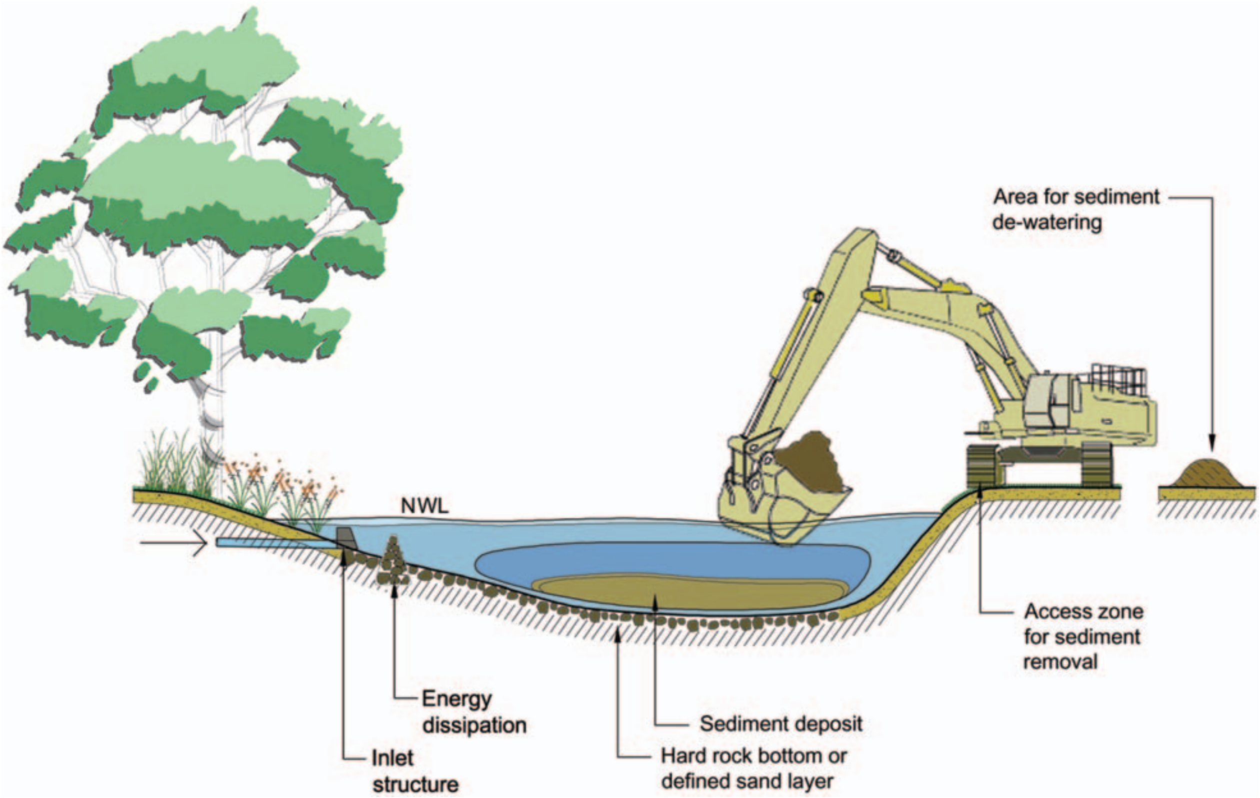


Figure 4.1 Sedimentation basin layout.

Basins that are sized to target coarse to medium-size sediment (e.g. 125 μm) 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. However, if a basin is oversized, there is an increased risk of much finer sediment accumulating and potentially having higher contaminant concentrations that could require specialist handling facilities for maintenance. Therefore, while a basin must have adequate size for capturing the target particles, they should not be grossly oversized.

A further consideration in sizing a sedimentation basin is providing adequate storage for settled sediment to prevent the need for frequent desilting. A desirable frequency of basin desilting for permanent facilities (not temporary basins for construction sites) is once every five years (triggered when sediment accumulates to half the basin depth).

Apart from needing to size a sedimentation basin appropriately for effective capture and retention of sediment, design considerations are similar to those for **ponds** and constructed wetlands.

4.2 Verifying size for treatment

Figure 4.2 shows relationships between a required basin area and design discharge for 125 μm 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 permanent pool of 2 m depth was used to define the lower limit of the required sedimentation basin, thus forming three shaded areas in Figure 4.2 with the upper limit being defined using no permanent pool.

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 design regions** in Victoria.

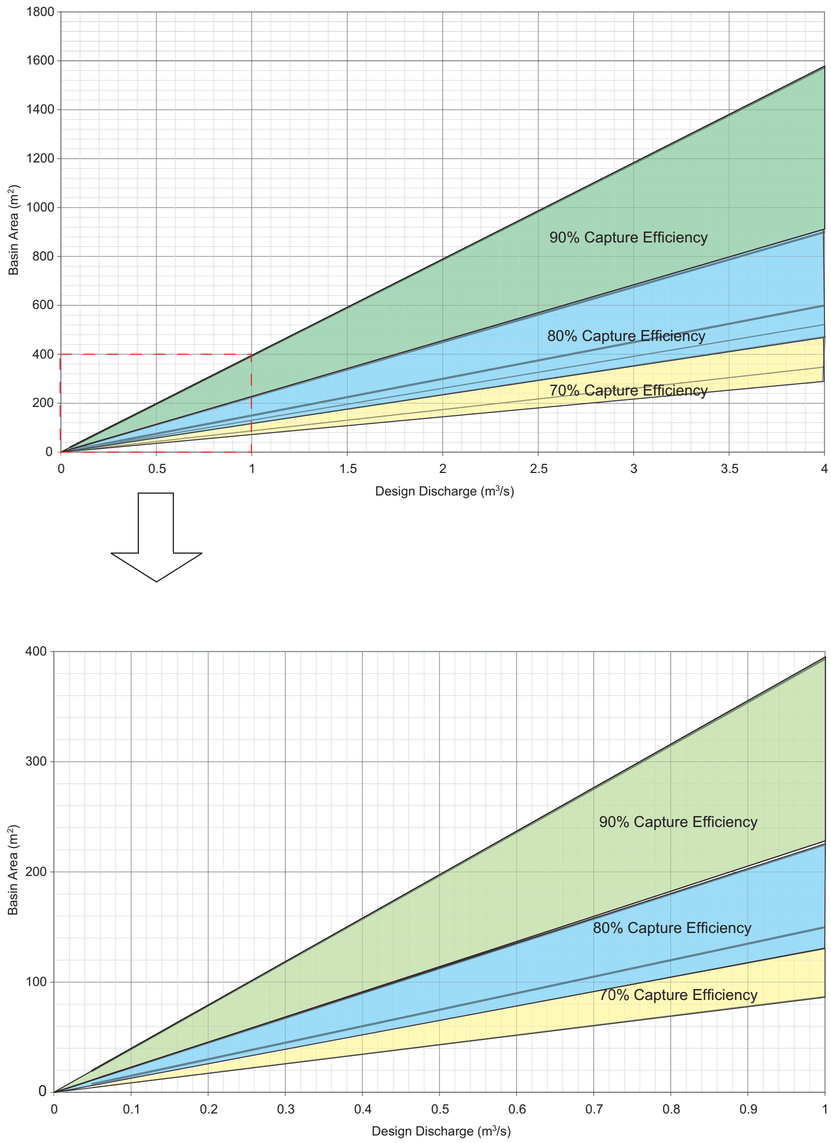


Figure 4.2 Sedimentation basin area versus design discharges for varying capture efficiencies of 125 μm sediment.

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 five years (unless it is to be used for temporary sediment control when cleaning every six-months may be appropriate). A developing catchment can be expected to discharge between 50 m^3/ha and 200 m^3/ha of sediment 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 m^3/ha. There are different methods used to estimate sediment loads and some authorities have produced charts of sediment loading rates (ACT Department of Urban Services 1994; NSW Department of Housing 1998). Desilting should be required when the permanent pool is half full with deposited sediment.

4.3 Design procedure: sedimentation basins

4.3.1 Estimating design flows

4.3.1.1 Design discharges

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

- design flow for sizing the required basin size
- minor system design flow for the design of the inlet structure
- major flood flows for the design of the basin overflow structure.

Local councils and regional catchment management authorities may stipulate the design operation discharge for sedimentation basins, especially for temporary basins used to manage sediment discharge from construction sites. Normally the design discharge stipulated would be either the one-year Average Recurrance Interval (ARI) or two-year ARI peak discharge. The design operation flow for permanent sedimentation basins used as pretreatments for downstream stormwater treatment measures is normally the one-year ARI peak design.

An inflow structure of a sedimentation basin needs to have capacity to convey the design discharge of the minor stormwater drainage system. The design discharge varies according to location and requirements of local councils or regional drainage authorities (e.g. a 5-year or 10-year ARI peak discharge).

Sedimentation basins should not be designed to a have high flow bypass. All flows should be directed through a sedimentation basin such that some level of sedimentation is achieved even during high flow conditions. The design discharge for an overflow structure could be the same as that for the inflow structure if overland flow is not directed to the basin. In most drainage systems, a sedimentation basin forms part of the major drainage system, in which case, the design discharge for the overflow structure should correspond to the 100-year ARI event.

4.3.1.2 Minor and major flood estimation

A range of hydrologic methods can be applied to estimate design flows. With typical catchment areas being relatively small, the **Rational Method** Design Procedure is considered to be a suitable method for estimating design flows.

4.3.2 Size and shape of sedimentation basins

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

$$R = 1 - \left(1 + \frac{1}{n}\frac{v_s}{Q/A}\right)^{-n} \qquad \text{(Equation 4.1)}$$

where R represents the fraction of target sediment removed;
v_S = settling velocity of target sediment (m/s);
Q/A = rate of applied flow (m^3/s) divided by basin surface area (m^2);
n = turbulence or short-circuiting parameter.

The above expression for sedimentation is applied with ***n*** being a turbulence parameter. Figure 4.3 provides guidance on estimating a λ value that is then used to calculate an appropriate ***n*** value (according to the configuration of the basin). The shape of a basin has a large impact on the effectiveness of the basin to retain sediments. Generally a length to width ratio of at least 3:1 should be achieved. In addition, the location of the inlet and outlet, flow spreaders and internal baffles affectct the hydraulic efficiency of the basin for stormwater treatment as the range of values in Figure 4.3 demonstrates. Figure 4.3 provides some guidance on what is considered to be good basin design with the higher values (of λ) representing basins with good sediment retention properties. Values of λ 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 λ value (e.g. inclusion of baffles, islands or flow spreaders).

The maintenance access to a basin also needs to be considered when developing the shape of a basin as this can affect the allowable width (if access is from the banks) or the shape if access ramps into a basin are required. An area for sediment dewatering should also be accommodated which should drain back into the basin. This too may affect the footprint area required for a sedimentation basin system.

The value for λ is estimated from the configuration of the basin according to Figure 4.3. A value of ***n*** is estimated using the following relationship:

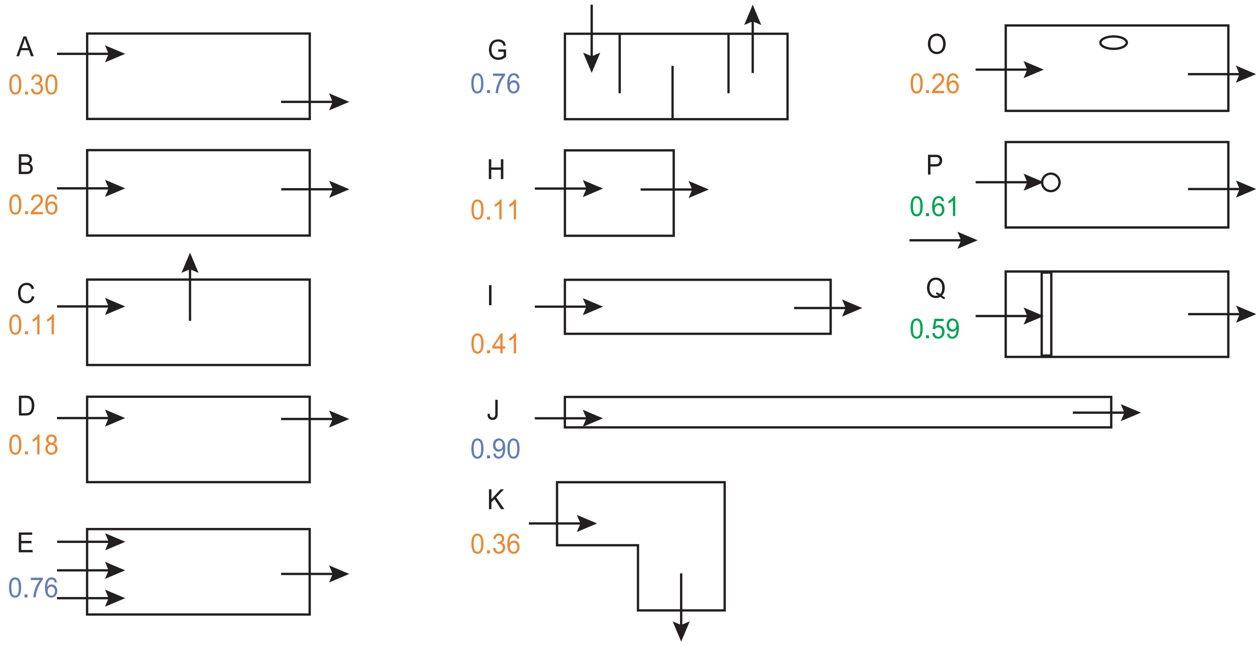


Figure 4.3 Hydraulic efficiency: λ is a measure of flow hydrodynamic conditions in constructed wetlands and ponds; range of λ is from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment (Persson et al. 1999).

$$(\lambda = 1 - 1/n);\quad n = \frac{1}{1-\lambda} \qquad \text{(Equation 4.2)}$$

The numbers in Figure 4.3 represent the values of λ that are used to estimate the turbulence parameter $\boldsymbol{n}$ for Equation 4.2. In Figure 4.3, 'o' in diagrams O and P represent islands in a waterbody and the double line in diagram Q represents a structure to distribute flows evenly.

Equation 4.1 is strictly applicable for systems with no permanent pool, and will generally overestimate 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.

Good practice in the design of sedimentation basins will include a permanent pool 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 some distance above the bed of a sedimentation basin, it is also not necessary for sediment particles to settle all the way to the bed of the basin to be effectively retained. It is envisaged that sediments need only settle to an effective depth which is less than the depth to the bed of the sediment. This depth is considered to be about 1 m below the permanent pool level. Equation 4.1 can thus be re-derived to account for the effect of the permanent pool storage as follows:

$$R = 1 - \left[1 + \frac{1}{n} \times \frac{v_s}{Q/A} \times \frac{(d_e + d_p)}{(d_e + d^\star)}\right]^{-n} \qquad \text{(Equation 4.3)}$$

where d_e represents the **extended detention** depth (m) above the permanent pool level;
d_p is the depth (m) of the permanent pool;
$d^\star$ 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.

Table 4.1 lists the typical settling velocities of sediments.

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. Estimates of the loading rates are required (depending on whether the basin is for sediment control during construction work or post development) (see Section 4.2).

Table 4.1 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

Loading rates (e.g. 1.6 m^3/ha per year for developed catchments) can then be used to estimate the required storage volume for each clean-out and this volume checked against the volume of allowable sediment accumulation given the basin configuration (estimated using Equation 4.1 or 4.3). The allowable sediment storage volume should be estimated using half the permanent pool volume (as this level of accumulation should trigger a clean-out).

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

The required volume of sediment storage (S) can be estimated using Equation 4.4:

$$S_t = C_a \times R \times L_0 \times F_r \qquad \text{(Equation 4.4)}$$

where S_t represents the volume of storage required (m^3);
C_a = contributing catchment area (ha);
R = capture efficiency (%), estimated from Equation 4.1 or 4.3;
L_o = sediment loading rate (m^3/ha per year);
F_r = desired clean-out frequency (years).

A calculation spreadsheet which calculates sedimentation basin areas is included on the attached CD.

4.3.3 Cross sections

With the exception of temporary sedimentation basins used for construction sites, **batter slopes** on approaches and immediately under the water line of a basin should be configured with consideration of public safety. Both hard and soft edge treatments can be applied to complement the landscape of a surrounding area. Soft edge treatments involve using gentle slopes to the water's edge, extending below the water line for a distance before batter slopes steepen into deeper areas (Figure 4.4).

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

Figure 4.5 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 between sites, and it is recommended that an independent safety audit be conducted of each design.

4.3.4 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.

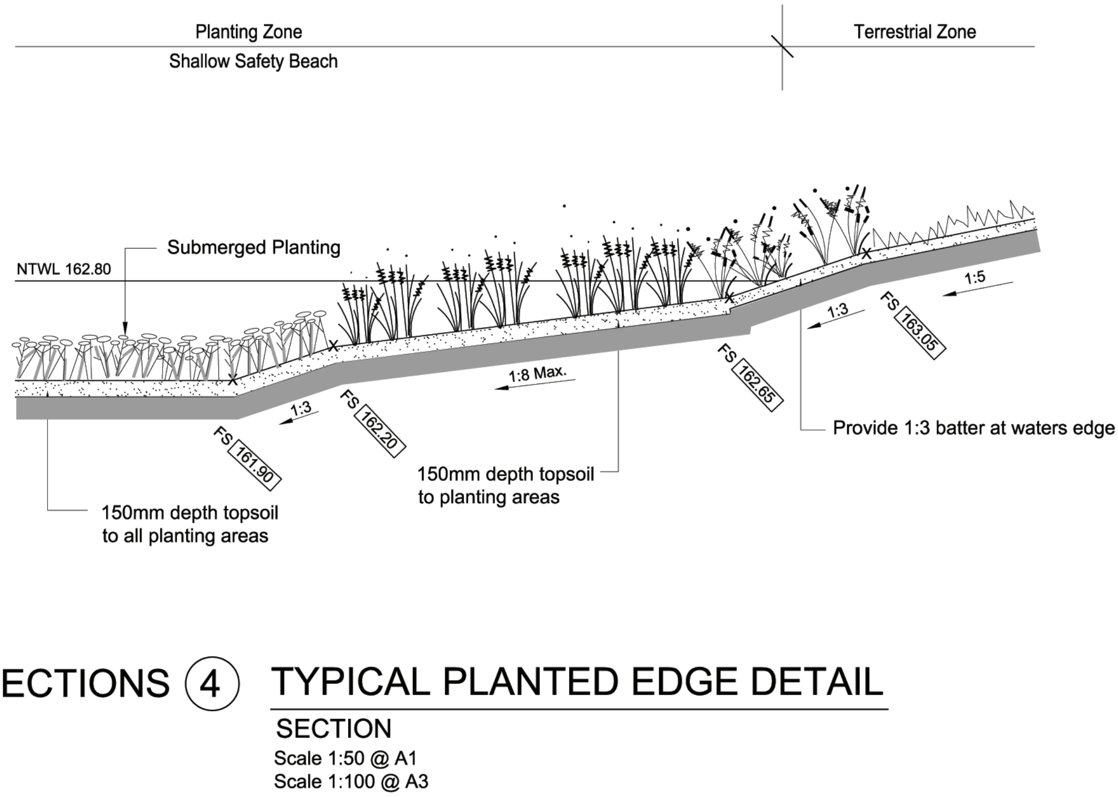


Figure 4.4 A soft edge treatment for open waterbodies (Graeme Bentley Landscape Architects 2004).

4.3.4.1 Inlet structure

Stormwater conveyed by a pipe or open channel would normally discharge directly into a sedimentation basin as this is often the first element of a stormwater treatment train. Inflow energy needs to be adequately dissipated so as not to cause localised scour near a pipe or channel outfall. Design of the inlet structures for adequate protection against scour is common hydraulic engineering practice (see e.g. Chow 1959; Henderson 1966).

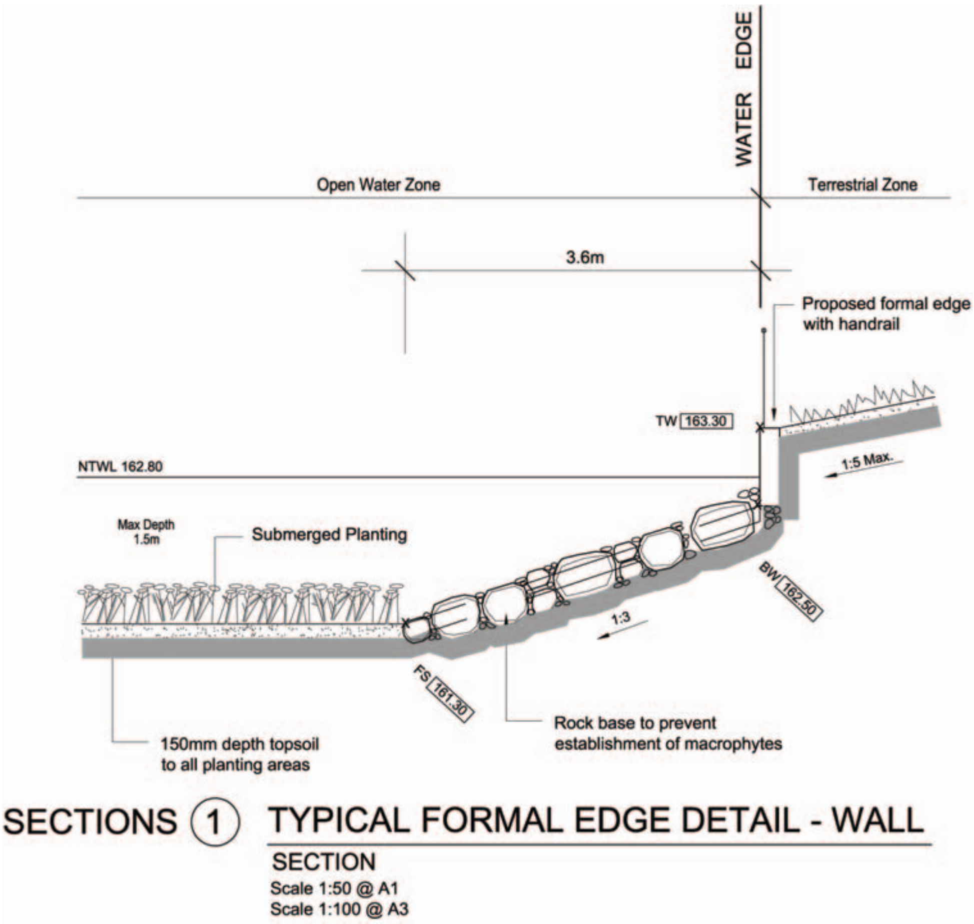
Litter control is also normally required at an inlet structure and it is generally recommended that some form of **gross pollutant trap (GPT)** be installed as part of an inlet structure. The provision of a GPT will depend on catchment activities as well as any upstream measures in place. Several proprietary products are available for removing gross pollutants (see Engineers Australia 2003, Chapter 7). The storage capacity of GPTs should be sized to ensure that maintenance (clean-out) frequency is not greater than once every three months.

4.3.4.2 Outlet structure

An outlet structure of a sedimentation basin can be configured in many ways and depends on the specified operation of the system (e.g. whether as a 'stand-alone' sedimentation basin for managing construction site runoff or as part of a wetland). The outlet structure generally consists of an outlet pit and a discharge control structure to control the rate of discharge from the basin under normal operation. The discharge control structure should have adequate capacity to convey the design operation flow.

Landscape amenity is not an important design outcome for a sedimentation basin used for managing runoff from a construction site.. Therefore, floating discharge control structures are considered to be the most effective outlets for sedimentation basins for construction sites (Figure 4.6). 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 (Figure 4.6). Discharge from the basin is maintained at a relatively constant rate independent of the depth of water in the basin.

With sedimentation basins that also serve as a landscape element, a more appropriate discharge control structure is a **weir**. Where possible, a narrow weir (Figure 4.6) should be



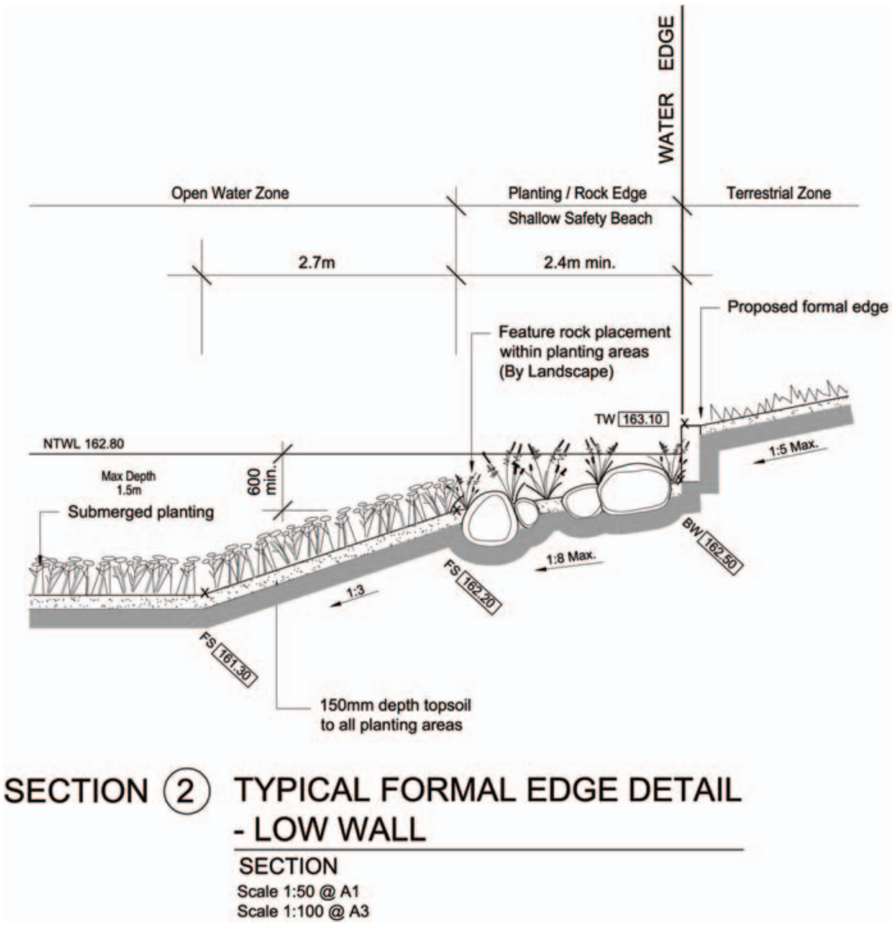


Figure 4.5 Hard edge treatment for open waterbodies (Graeme Bentley Landscape Architects 2004).

Figure 4.5 (Continued)

b

Figure 4.6 Sedimentation basin outlet structures: (a) a floating skimmer and (b) a narrow weir.

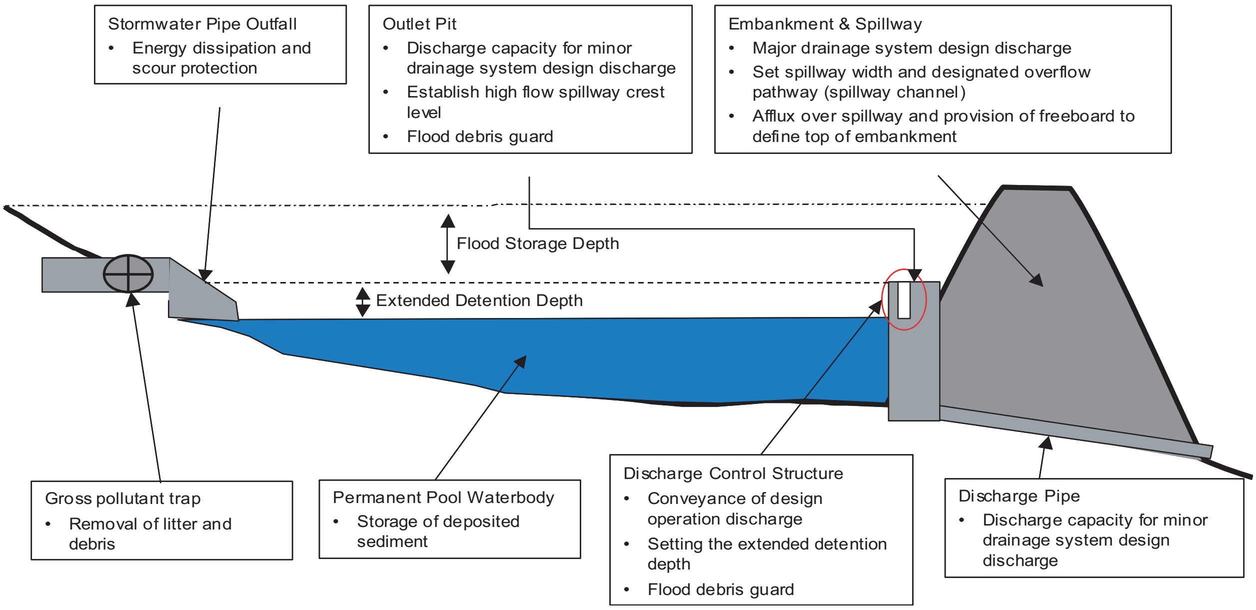


Figure 4.7 Overview of design elements of a sedimentation basin and main design considerations.

adopted to promote a larger range of extended detention depths while ensuring adequate capacity to convey the design discharge.

Dimensions of an outlet should ensure that the perimeter is sufficiently long to pass the design discharge into the connecting pipe, to either (i) a downstream treatment measure or (ii) receiving waters or downstream stormwater drainage infrastructure. In (i), the one-year ARI peak discharge is normally adopted as the design discharge whereas in (ii), the peak discharge corresponding to the design discharge for the minor stormwater drainage system should be adopted.

Design of an outlet pit and associated discharge control structure include the following:

- placement of the crest of the pit at or above the permanent pool level of the sedimentation basin
- sizing the pit to provide discharge capacity that is greater than the discharge capacity of the outlet culvert
- protection against blockage by flood debris.

Figure 4.7 summarises the design elements of the various components of a sedimentation basin.

Outlet pit

An outlet pit is sized with a discharge capacity of the minor drainage system (e.g. five-year ARI). The dimension of an outlet pit is determined by considering two flow conditions, (i) weir and (ii) orifice flow (Equations 4.5 and 4.6).

A blockage factor (*B*) is also used to account for any debris blockage. An assumption that the outlet is 50% blocked is recommended (i.e. $B = 0.5$). Generally it will be the discharge pipe from the sediment basin (and downstream water levels) that controls the maximum flow rate from the basin; it is therefore less critical if the outlet pit is oversized to allow for blockage.

1. Weir flow condition – usually when the extended detention storage of the retarding basin is not fully engaged, that is:

$$P = \frac{Q_{des}}{B \times C_w \times H^{1.5}} \qquad \text{(Equation 4.5)}$$

where P = perimeter of the outlet pit;
B = blockage factor (0.5);
H = depth of water above the crest of the outlet pit (m);

Q_{des} = Design discharge (m^3/s);
C_w = weir coefficient (1.7).

2. Orifice flow conditions – these occur when the inlet pit is completely submerged (corresponding to conditions associated with larger flood events), that is:

$$A_o = \frac{Q_{des}}{B \times C_d \sqrt{2gH}} \qquad \text{(Equation 4.6)}$$

where C_d = Orifice Discharge Coefficient (0.6);
H = Depth of water above the centroid of the orifice (m);
A_o = Orifice area (m^2);
g = Acceleration due to gravity (9.81 m/s^2)

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 (Figure 4.8).

Figure 4.8 Debris protection for outlet pits.

Discharge control structure

Three types of discharge control structures can be used.

1. Overflow weir – the length of the weir is computed with the weir flow equation (Equation 4.5) but checked to ensure that there is adequate discharge capacity when the structure operates under submerged conditions using the orifice flow equation (Equation 4.6).
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 4.6) 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 Section 4.6 and Chapter 9).
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 4.6).

With riser-type structures to control discharge , 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 (Figure 4.9).

Figure 4.9 Debris guards for outlet structures.

4.3.5 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 length of the spillway can be computed using the weir flow equation (Equation 4.5) with the design discharge being selected according to discussion in Section 4.3.1.1.

4.3.6 Vegetation specification

Vegetation planted along the **littoral zone** of a sedimentation basin serves the primary function of inhibiting public access to the open waterbody and preventing edge erosion. Terrestrial

Figure 4.10 Overflow structure of a sedimentation basin.

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. A list of suggested plant species suitable for a sedimentation basin littoral zone in Victoria is provided in Appendix A.

4.3.7 Design calculation summary

The *Sedimentation Basin Calculation Checklist* is a design calculation summary sheet for the key design elements of a sedimentation basin. It has been included to aid the design process.

Sedimentation Basin **CALCULATION CHECKLIST**

CALCULATION TASK	OUTCOME		CHECK
1 Identify design criteria			☐
Design ARI flow for inlet hydraulic structures		year	
Design ARI flow for outlet hydraulic structures		year	
Design ARI for overflow hydraulic structures		year	
2 Catchment characteristics			☐
Residential		ha	
Commercial		ha	
Roads		ha	
Fraction impervious			☐
Residential			
Commercial			
Roads			
3 Estimate design flow rates			
Time of concentration			
Estimate from flow path length and velocities		minutes	☐
Identify rainfall intensities			
station used for IFD data:			
Design rainfall intensity for inlet structure(s)		mm/hr	☐
Design runoff coefficient			
Inlet structure(s)			☐
Peak design flows			☐
Inlet structure(s)		m^3/s	
Outlet structure(s)		m^3/s	
Overflow structure(s)		m^3/s	
4 Basin dimension and layout			☐
Area of sedimentation basin		m^2	
Aspect ratio		L:W	
Hydraulic efficiency			
Depth of permanent pool		m	
Permanent pool volume		m^3	
Cross section batter slope		V:H	
5 Basin performance			☐
Capture efficiency (of 125 μm sediment)		%	
Sediment cleanout frequency		years	
6 Hydraulic structures			
Inlet structure			☐
Provision of energy dissipation			
Outlet structure			☐
Pit dimension		L x B	
or		mm diam	
Discharge capacity of outlet		m^3/s	
Provision of debris trap			
Discharge pipe			☐
Discharge capacity of discharge pipe		m^3/s	
7 Spillway			☐
Discharge capacity of spillway		m^3/s	

4.4 Checking tools

Checking aids are included 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).

4.4.1 Design assessment checklist

The *Sedimentation Basin Design Assessment Checklist* 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.

Sediment Basin Design Assessment Checklist					
Basin location:					
Hydraulics	Minor flood: (m³/s)		Major flood: (m³/s)		
Area	Catchment area (ha):		Basin area (ha)		
Treatment				**Y**	**N**
Treatment performance verified from curves?					
Basin configuration				**Y**	**N**
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 >125 μm?					
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?					
Hydraulic structures				**Y**	**N**
Outlet perimeter > = design discharge of outlet pipe?					
Outlet configuration suitable for basin type (e.g. riser for construction sediment, weir for wetland pretreatment)?					
Riser diameter sufficient to convey Q_1 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_1 flows (whichever is higher)?					
Protection against clogging of orifice provided on outlet structure?					

Where an item results in an 'N' when reviewing the design, the design procedure should be assessed to determine the effect 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 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 Handover Checklist* (see Section 4.4.4).

4.4.2 Construction advice

General advice is provided for the construction of sedimentation basins. It is based on observations from construction projects around Australia.

Building phase damage

It is important to protect a sediment basin from upstream flows during its construction. A mechanism to divert flows around a construction site, 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 well as possible at the end of each day. Plans for dewatering following storms should also be 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**. An access track around the perimeter of the basin would, however, be required. If sediment collection is by using earthmoving equipment, then a stable ramp will be required lesding 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 (i.e. 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.

Dewatering removed sediments

An area should be constructed that allows for dewatering 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. These need to be checked early in the system's life, to avoid continuing problems. If problems occur in these events, then erosion protection should be enhanced.

4.4.3 Construction checklist

CONSTRUCTION INSPECTION CHECKLIST

Sediment basin

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

SITE: ____________________

CONSTRUCTED BY: ____________________

DURING CONSTRUCTION									
'tems inspected	Checked		Satisfactory	Unsatisfactory		Checked		Satisfactory	Unsatisfactory
Preliminary works	Y	N			**Structural components**	Y	N		
1. Erosion and sediment control plan adopted					11. Location and levels of outlet as designed				
					12. Safety protection provided				
2. Limit public access					13. Pipe joints and connections as designed				
3. Location same as plans					14. Concrete and reinforcement as designed				
4. Site protection from existing flows					15. Inlets appropriately installed				
Earthworks					16. Inlet energy dissipation installed				
5. Integrity of banks					17. No seepage through banks				
6. Batter slopes as plans					18. Ensure spillway is level				
7. Impermeable (solid) base installed					19. Provision of maintenance drain				
8. Maintenance access (e.g. ramp) installed					20. Collar installed on pipes				
9. Compaction process as designed					**Vegetation**				
10. Levels of base, banks and spillway as designed					21. Stablisation immediately following earthworks				
					22. Planting as designed (species and densities)				
					23. Weed removal before stabilisation				
FINAL INSPECTION									
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.

4.4.4 Asset handover checklist

Asset Handover Checklist		
Asset location:		
Construction by:		
Defects and liability period		
Treatment	**Y**	**N**
System appears to be working as designed visually?		
No obvious signs of under-performance?		
Maintenance	**Y**	**N**
Maintenance plans provided for each asset?		
Inspection and maintenance undertaken as per maintenance plan?		
Inspection and maintenance forms provided?		
Asset inspected for defects?		
Asset information	**Y**	**N**
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 (e.g. drawings, survey, models) provided?		
Asset listed on asset register or database?		

4.5 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.

4.5.1 Operation and maintenance inspection form

The *Sediment Basin Maintenance Checklist* is designed to be used whenever an inspection is conducted and kept as a record on the asset condition and quantity of removed pollutants over time.

4.6 Sedimentation basin worked example

4.6.1 Worked example introduction

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

Sediment Basin Maintenance Checklist			
Inspection frequency: 3 monthly		Date of visit:	
Location:			
Description:			
Site visit by:			
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:			

system. The site is triangular with a surface area of 500 m^2 (Figure 4.11). Road runoff is conveyed by conventional stormwater pipes (up to the 100-year ARI event) and there are two freeway outfall pipes that discharge to the two top apexes of the site. Each outfall services about 1 km of the freeway with the total contributing area of 4 ha (90% impervious) to each outfall. The site of the sedimentation basin has a fall of about 2 m (from 5 m-AHD (Australian Height Datum) to 3 m-AHD) towards a degraded watercourse.

Site constraints limit the size available for the stormwater treatment system. In principle, when available space is constrained, the size of the inlet zones (i.e. sedimentation basins) should not be compromised, to ensure that larger sediments are effectively trapped and prevented from smothering the **macrophyte zone** (thereby creating future maintenance problems).

Therefore, if the site constrains the total size of the treatment system, the macrophyte zone should be reduced accordingly. This will reduce the overall **hydrologic effectiveness** of the system (i.e. the proportion of Mean Annual Runoff, MAR, 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-sized sediment. The inlet zone will operate under bypass conditions more often owing to a smaller macrophyte zone in this case.

4.6.1.1 Design objectives

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

- promote sedimentation of particles larger than 125 μm with a 90% capture efficiency for flows up to the one-year ARI (unattenuated) peak discharge
- provide for connection to the downstream macrophyte zone with discharge capacity corresponding to the one-year ARI (unattenuated) peak discharge
- provide for a bypass operation when the inundation of the downstream macrophyte 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:

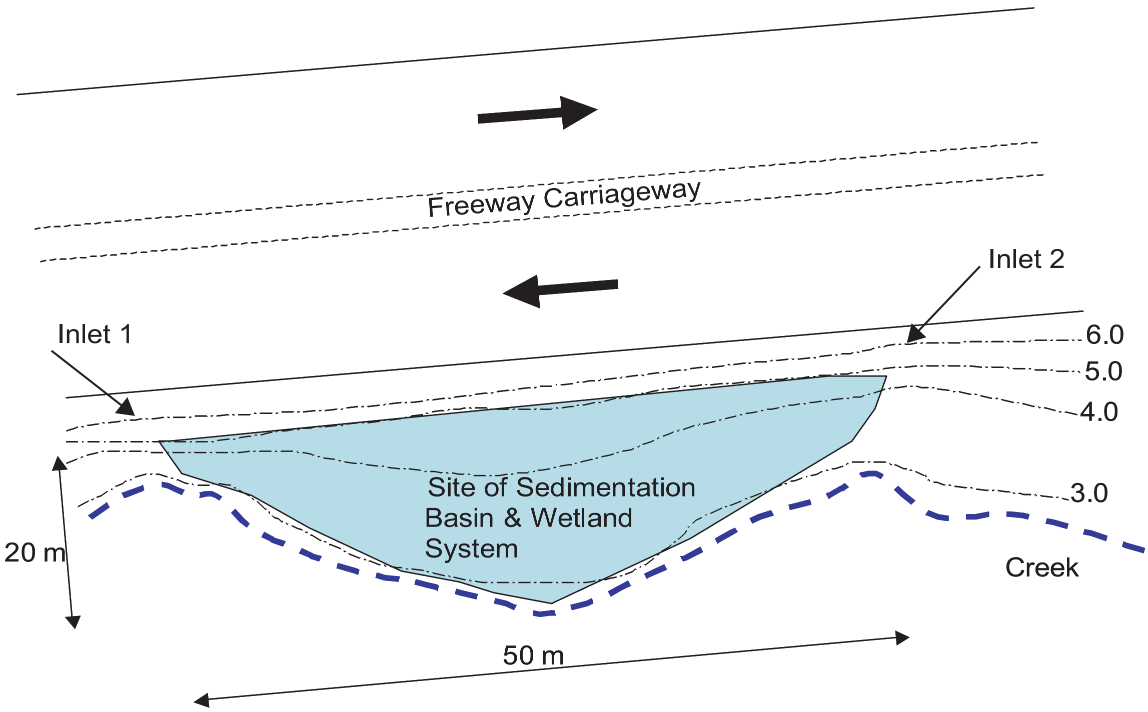


Figure 4.11 Layout of proposed site for sedimentation basin.

- sizing the sedimentation basin (depth and area) using sedimentation theory (an extended detention depth of 0.25 m above the permanent pool level has been nominated to match the proposed maximum water level of the downstream macrophyte zone)
- configuring the layout of the basin such that the system hydraulic efficiency 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 bypass structure to provide for flow bypass of downstream wetland for events up to the 100-year ARI event
- design of the basin outlet structure connecting to the macrophyte zone, including the debris trap.

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

- littoral zone vegetation
- terrestrial vegetation.

4.6.2 Estimating design flows

The procedures in Australian Rainfall and Runoff (ARR) (Institution of Engineers 1987) are used to estimate the design flows. The site has two contributing catchments, each catchment is 4 ha, 1 km long (along the freeway) and is drained by culverts. Velocity within the pipes is assumed to be 1 m/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 Geelong (for the 1, 10 and 100-year average recurrence intervals) are estimated using ARR (Institution of Engineers 2001) with a time of concentration of 17 minutes and are:

$$I_1 = 27 \text{ mm/hr}$$
$$I_{10} = 56 \text{ mm/hr}$$
$$I_{100} = 95 \text{ mm/hr}$$

Runoff coefficients as per ARR (Institution of Engineers 2001):

$$^{10}I_1 = 26.4 \text{ mm/hr}$$

Fraction impervious, $F_{imp} = 0.9$:

$$C_{10}^{1} = 0.12$$
$$C_{10} = 0.82$$

Runoff coefficients are as per Table 1.6 Institution of Engineers, Book VIII of ARR (2001)

ARI (years)	**Frequency factor, *Fy***	**Runoff coefficient, *Cy***
1	0.8	0.66
10	1.0	0.82
100	1.2	0.98

From the Rational Method Design Procedure:

$$Q = CIA/360;\ Q_1 = 0.20 \text{ m}^3/\text{s};\ Q_{10} = 0.51 \text{ m}^3/\text{s; and } Q_{100} = 1.0 \text{ m}^3/\text{s}.$$

Operation design discharge = $0.20 \text{ m}^3/\text{s}$
Design discharge for connection to macrophyte zone = $0.20 \text{ m}^3/\text{s}$
Spillway design discharge = $1.0 \text{ m}^3/\text{s}$

4.6.3 Size and shape of sedimentation basin

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

Pollutant removal is estimated using Equation 4.3:

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

An aspect ratio of 1 (w) to 4 (L) is adopted based on the available space (Figure 4.11). Using Figure 4.3, the hydraulic efficiency λ 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 4.2 to be 1.67. Thus,

Hydraulic efficiency $\lambda = 0.4$

Turbulence factor ($\boldsymbol{n}$) = 1.67.

The proposed extended detention depth of the basin is 0.25 m (as outlined in Section 4.6.1.1) and a notional permanent pool depth of 2 m has been adopted, that is:

$$d_p = 2.0 \text{ m}$$
$$d^* = 1.0 \text{ m}$$
$$d_e = 0.25 \text{ m}$$

$v_s = 0.011$ m/s for 125 μm particles
Q = design operation flow rate = $0.20 \text{ m}^3/\text{s}$.

From Equation 4.3, the required sedimentation basin area to achieve target sediment (125 μm) capture efficiency of 90% is 50 m^2. With a W to L ratio of 1:4, the notional dimensions of the basin are 3.5 m × 14 m. This size is validated against the curves presented in Figure 4.2.

The available sediment storage is 50 × 2 = 100 m^3. Clean-out is to be scheduled when the storage is half full, therefore the available sediment storage prior to clean-out is 50 m^3.

The required volume of sediment storage to ensure cleaning is not required more frequently than every five years is estimated using Equation 4.4 (using a sediment discharge rate of 1.6 m^3/ha per year).

$$\text{Required storage } (S_t) = C_\text{a} \times R \times L_0 \times F_\text{r}$$
$$= 4 \times 0.9 \times 1.6 \times 5 = 29 \text{ m}^3$$

Available storage volume is 50 m^3, and therefore it is OK.

The required clean-out frequency is estimated to be (by rearranging Equation 4.4):

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

Open water area = 50 m^2
Width = 3.5 m; Length = 12 m
Depth of permanent pool (d_p) = 2.0 m
Depth of extended detention (d_e) = 0.25 m.

4.6.4 Hydraulic structure design

4.6.4.1 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 is assumed to be RL 3.5 m AHD.

$$(Q_{10} = 0.51 \text{ m}^3)/\text{s}$$

Rock beaching will be required in this area to ensure that excessive scour does not occur.

Energy dissipation and erosion protection will need to be provided in the form of rock beaching at the inlet structure; Q_des = 0.51 m^3/s (see Section 4.6.2).

4.6.4.2 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 (i.e. one-year ARI peak discharge of 0.2 m^3/s).

According to Section 4.4.2, 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 4.5, the required perimeter of the outlet pit to pass 0.2 m^3/s with an **afflux** of 0.25 m can be calculated:

$$P = \frac{Q}{B \times C_\text{w} \times H^{1.5}} = \frac{0.2}{0.5 \times 1.7 \times 0.25^{1.5}} = 1.88 \text{ m}$$

Orifice flow conditions

From Equation 4.6, the required area of the outlet pit can be calculated as follows:

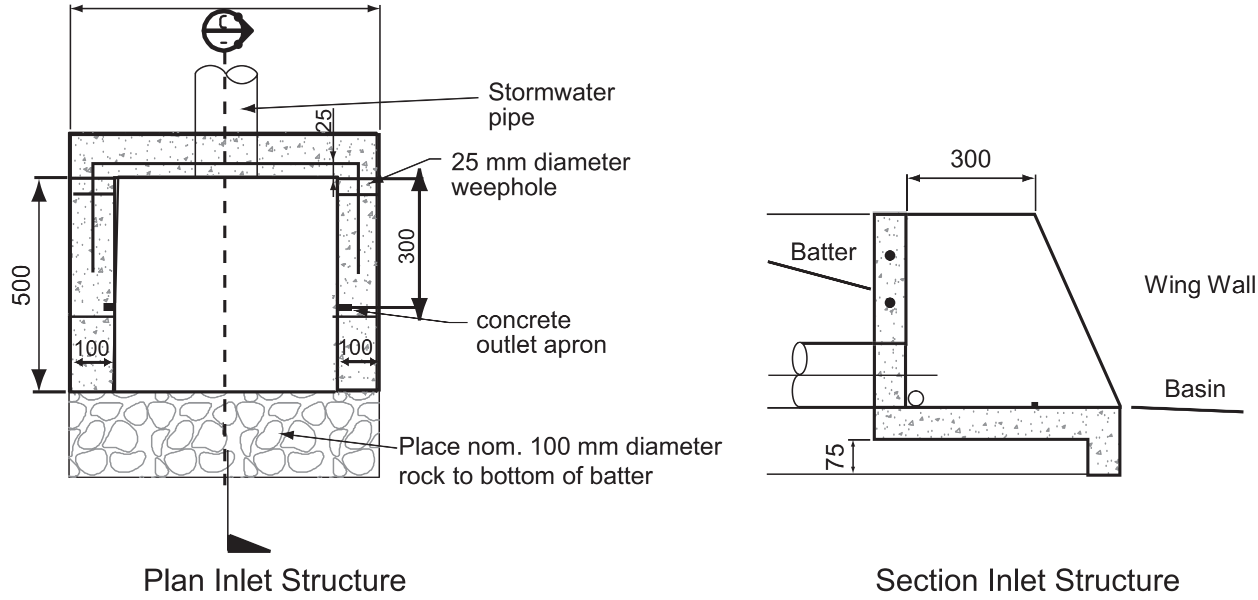


Figure 4.12 Inlet structure.

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

Adopt 600 × 600 mm pit: area = 0.36 m^2; perimeter = 2.4 m; Q_{cap} = 0.24 m^3/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.

4.6.4.3 Overflow structure

The overflow structure is to discharge Q_{100} peak flow. The overflow structure is an overflow weir with a crest elevation set at 0.25 m (i.e. d_e) above the permanent pool level. The length of this weir determines the afflux for the 100-year ARI peak discharge and sets the top of embankment of the sedimentation basin. It is common practice to allow for 300 mm of freeboard above the afflux level when setting the top of embankment elevation. An afflux of 0.3 m has been adopted in defining the length of the spillway weir. This value was adopted as a trade off between the bank height and the width of the weir. A bank height of 600 mm (300 mm afflux and 300 mm 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, that is:

$$L = \frac{Q_{des}}{C_w \times H^{1.5}} = \frac{1.0}{1.7 \times 0.3^{1.5}} = 3.6 \text{ m} \qquad \text{(Equation 4.7)}$$

where L represents the length of the weir.

A 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.

4.6.4.4 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 one-year ARI peak discharge when the water level in the 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 9 on constructed wetland design.

4.6.5 Design calculation summary

Sedimentation Basin	CALCULATION SUMMARY		
CALCULATION TASK	**OUTCOME**		**CHECK**
1 Identify design criteria			✓
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	
2 Catchment characteristics			✓
Residential	0	ha	
Commercial	0	ha	
Roads	4	ha	
Fraction impervious			✓
Residential	N/A		
Commercial	N/A		
Roads	0.9		
3 Estimate design flow rates			
Time of concentration			
Estimate from flow path length and velocities	17	minutes	✓
Identify rainfall intensities			
station used for IFD data:	Geelong		
Design rainfall intensity for inlet structure(s)	27 to 56	mm/hr	✓
Design runoff coefficient			
Inlet structure(s)	0.66 to 0.98		✓
Peak design flows			✓
Inlet structure(s)	0.51	m^3/s	
Outlet structure(s)	0.20	m^3/s	
Overflow structure(s)	1.00	m^3/s	
4 Basin dimension and layout			✓
Area of sedimentation basin	50	m^2	
Aspect ratio	4(L):1(W)	L:W	
Hydraulic efficiency	0.4		
Depth of permanent pool	2	m	
Permanent pool volume	100	m^3	
Cross section batter slope	1(V):8(H)	V:H	
5 Basin performance			✓
Capture efficiency (of 125 μm sediment)	90	%	
Sediment cleanout frequency	8.6	years	
6 Hydraulic structures			
Inlet structure			✓
Provision of energy dissipation	Y		
Outlet structure			✓
Pit dimension	600 x 600	L x B	
or		mm diam	
Discharge capacity of outlet	0.21	m^3/s	
Provision of debris trap	Y		
Discharge pipe			✓
Discharge capacity of discharge pipe	0.2	m^3/s	
7 Spillway			✓
Discharge capacity of spillway	1	m^3/s	

4.6.6 Construction drawings

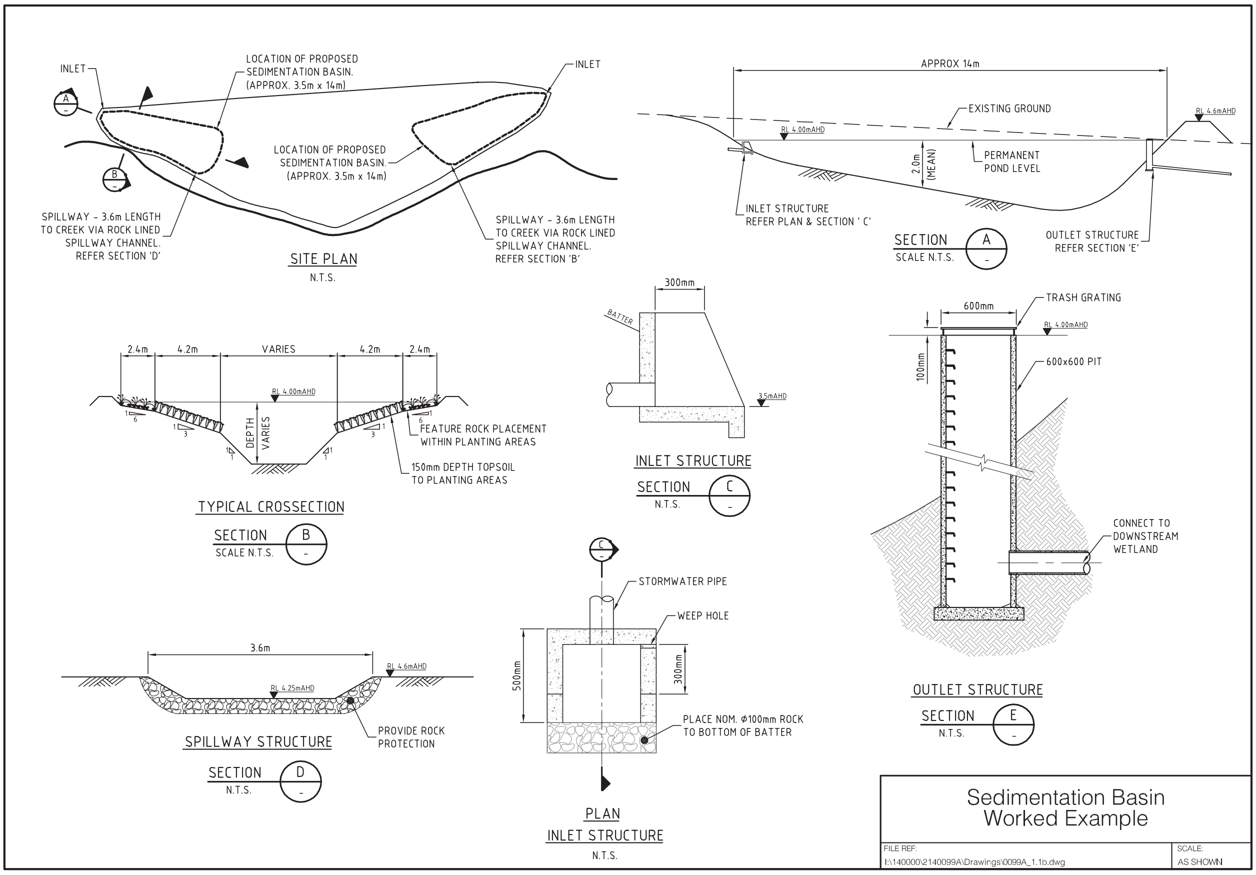


Figure 4.13 Construction drawing of the sedimentation basin worked example.

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