Construction Site Sediment Basins

Version 1, 2022

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Prepared by: Grant Witheridge, Catchments & Creeks Pty Ltd

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Cover image: Type A sediment basin, Sunshine Coast, Queensland supplied by Scott Paten

Disclaimer

Significant effort has been taken to ensure that this document is representative of current best practice sediment control; however, the author cannot and does not claim that the document is without error, or that the recommendations presented within this document will not be subject to future amendment.

To be effective, sediment control measures must be investigated, planned, and designed in a manner appropriate for the given work activity and site conditions.

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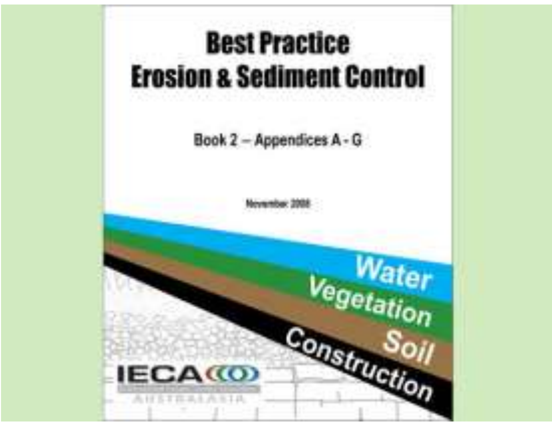
Specifically, adoption of the recommendations and procedures presented within this field guide will not guarantee:

(i) compliance with any statutory obligations
(ii) compliance with Appendix B of IECA (Australasia) *Best Practice ESC*
(iii) compliance with specific water quality objectives
(iv) avoidance of environmental harm or nuisance.

Principal reference documents:



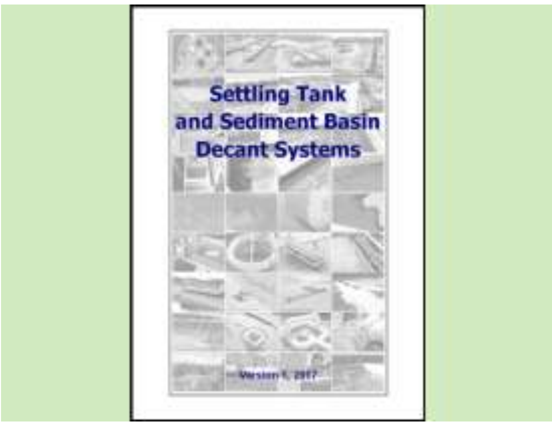
IECA (2008) – Book 2

Best Practice Erosion and Sediment Control – Book 2

IECA (Australasia), Picton, NSW

A. Construction site hydrology and hydraulics
B. Sediment basin design and operation
C. Soils and revegetation
D. Example plans
E. Soil loss estimation
F. Erosion hazard assessment
G. Model code of practice

Appendix B was updated in 2018 and released as a free PDF



Settling Tank & Sediment Basin Decant

Settling Tank and Sediment Basin Decant Systems

Witheridge, G., 2017, Catchments and Creeks Pty. Ltd., Brisbane, Queensland.

This publication describes in more detail the fluid mechanics associated with decanting water from a stratified (multi-density layered) settling tank. The publication looks at both wastewater settling tanks and construction site sediment basins.

Use of Rock in Stormwater Engineering

Use of Rock in Stormwater Engineering

Witheridge, G., 2021, Catchments and Creeks Pty. Ltd., Brisbane, Queensland.

First released in 2014, with several updates

Journal of Hydraulic Engineering, 1989

Density Measurement of Particle and Floc Suspensions

Witheridge, G.M. and Wilkinson, D.L. 1989, Journal of Hydraulic Engineering, Vol. 115, No. 3, March 1989, American Society of Civil Engineers, pp. 403–408

Contents

Purpose of this field guide

This field guide has been prepared specifically to:

- educate readers on the design of construction site sediment basins
- to assist readers in the hydraulic analysis of sediment basin spillways
- to supplement the design information provided in Appendix B of the IECA (Australasia) *Best Practice Erosion and Sediment Control* (ESC) publication.

It is not the intention of this field guide to replace the use of Appendix B (IECA, 2018), but instead to supplement its use. Consequently, many of the tables and equations presented in this field guide are of such low reproduction quality that readers will be required to refer back to the original text of Appendix B (2018, or later version).

The photos presented within this document are intended to represent the current topic of discussion. These photos are presented for the purpose of depicting either a preferred or discouraged outcome (as the case may be). In some cases the photos may not represent current best practice, but are simply the best photos available to the author at the time.

The caption and/or associated discussion should **not** imply that the site shown within the photograph represents either good or bad ESC practice. The circumstances, site conditions and history of each site are not known, and may not be directly relevant to the current discussion. This means that there may be a completely valid site-specific reason why the designer chose the sediment basin design depicted in the photo.

About the author

Grant Witheridge is a civil engineer with both Bachelor and Masters degrees from the University of NSW (UNSW). He has over 40 years experience in the fields of hydraulics, creek engineering and erosion & sediment control, during which time he has worked for a variety of Federal, state, local government, and private organisations.

He commenced his career at the UNSW Water Research Laboratory operating physical flood models of river floodplains. He later worked for Brisbane City Council on creek engineering and stormwater management issues. He currently works through his own company Catchments and Creeks Pty Ltd.

Grant has authored over 40 documents, including three editions of the Queensland Urban Drainage Manual (2007, 2013 & 2016); Brisbane City Council's Natural Channel Design and Creek Erosion guidelines (1997 & 2000); the IECA's Best Practice Erosion & Sediment Control documents (2008), and various fish passage guidelines.

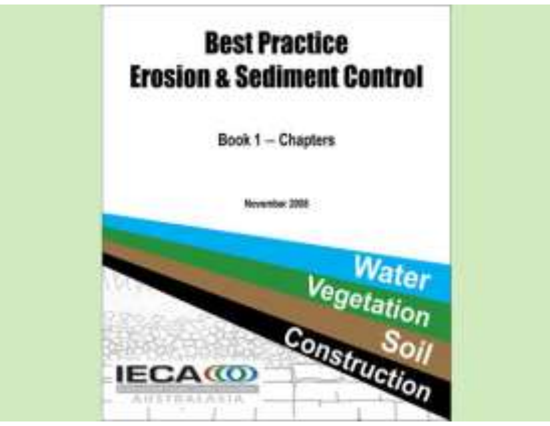
Introduction

Hydraulics plays an important role in the design and operation of sediment basins, including:

- mixing zones, where introduced coagulants or flocculants are encouraged to mix with the sediment-laden water
- forebays (stilling ponds), where turbulence is removed from the inflow in order to allow this inflow to enter the main settling pond in a uniform manner in order to improve the hydraulic efficiency of the settling pond
- the length-to-width ratio of the main settling pond, which can be used to reduce the risk of flow short circuiting during normal operational flows
- the low-flow decant system
- the high-flow decant system (if any)
- the emergency spillway weir, utilised when inflows exceed the design storm
- the design of the spillway chute
- the design of the energy dissipater.

This field guide follows the sediment basin classification system presented in Appendix B of the IECA (Australasia) *Best Practice Erosion and Sediment Control* publication, which means sediment basins are grouped into Type A, Type B, Type C and Type D basins.

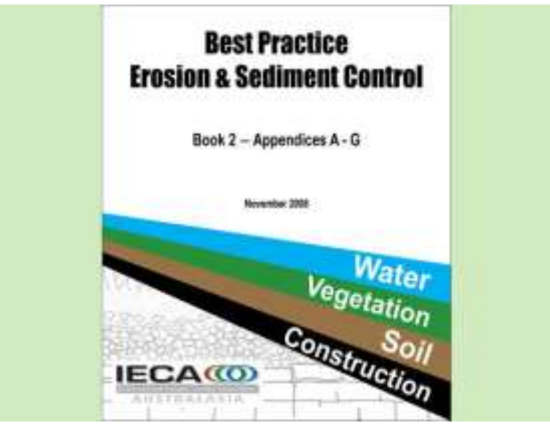
Design steps



IECA (2008) – Book 1

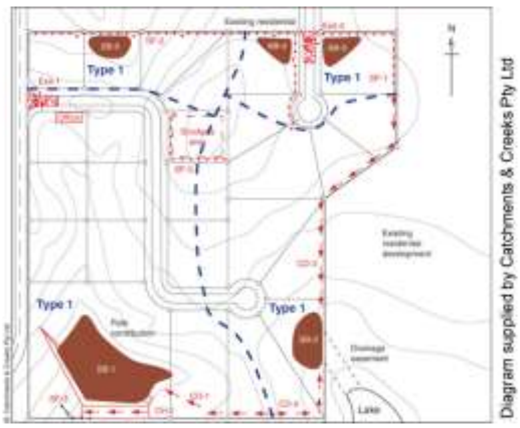
Introduction

- Best Practice Erosion and Sediment Control – IECA (Australasia), Picton, NSW.
- This field guide closely follows the recommendations of Appendix B in the IECA Best Practice guidelines, but the two publications are independent, and variations in the publications can and do occur.



IECA (2008) – Book 2

Design meeting



Erosion and Sediment Control Plan

Sediment basin design steps

The design steps presented in Appendix B of the IECA (Australasia) *Best Practice Erosion and Sediment Control* guidelines are:

1. Assess the need for a sediment basin
2. Select the type of sediment basin
3. Determine basin location
4. Divert up-slope 'clean' water
5. Select internal and external bank gradients
6. (a) Sizing Type A basins
   (b) Sizing Type B basins
   (c) Sizing Type C basins
   (d) Sizing Type D basins
7. Determine the sediment storage volume
8. Design of flow control baffles
9. Design the basin's inflow system
   (i) Forebay – Type A and B basins
   (ii) Inlet chamber – Type C and D basins
10. Design the primary outlet system
    (i) Floating decant – Type A basins
    (ii) Pumped decant – Type B & D basins
    (iii) Riser pipe outlet – Type C basins
11. Design the emergency spillway
12. Determine the overall dimensions of the basin
13. Locate maintenance access (de-silting)
14. Define the sediment disposal method
15. Assess need for safety fencing
16. Define the rehabilitation process for the basin area
17. Define the basin's operational procedures
18. Complete the *Standard Basin Data* forms issued by the regulating authority.

Sediment basins on civil construction projects

Type A sediment basin (Auckland, NZ)

Type B sediment basin (Qld)

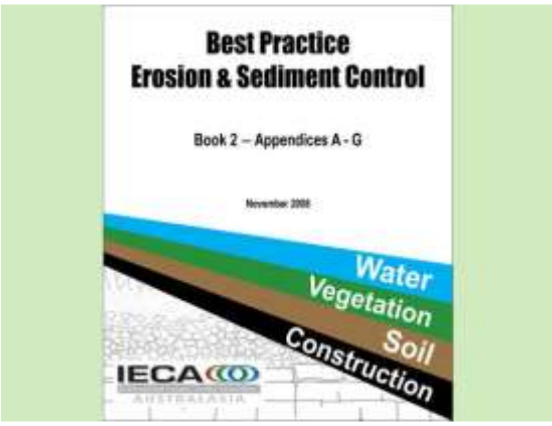
Type C sediment basin (NSW)

Type D sediment basin retained for permanent stormwater treatment (Qld)

1. Design Steps

Step 1: Assess the need for a sediment basin

Step 1: Assess the need for a sediment basin



IECA (2008) – Book 2

Assess the need for a sediment basin

- Refer to the sediment control standard specified for your area.
- Below is the Sediment Control Standard presented in IECA (Australasia), 2018, Appendix B.
- In Table 1, a sediment basin is considered a Type 1 sediment control system.

Table 1 – Sediment control standard (IECA, Australasia, 2018)

Catchment Area (m^2) [1]	Soil loss (t/ha/yr) [2]			Soil loss (t/ha/month) [3]		
	Type 1	Type 2	Type 3	Type 1	Type 2	Type 3
250	N/A	N/A	[4]	N/A	N/A	[4]
1000	N/A	N/A	All cases	N/A	N/A	All cases
2500	N/A	> 75	75	N/A	> 6.25	6.25
>2500	> 150	150	75	> 12.5	12.5	6.25
> 10,000	> 75	N/A	75	> 6.25	N/A	6.25

[1] Area is defined by the catchment area draining to a given site discharge. Sub-dividing a given drainage catchment shall not reduce its 'effective area' if runoff from these sub-areas ultimately discharges from the site at the same general location. The 'area' does not include any 'clean' water catchment that bypasses the sediment trap. The catchment area shall be defined by the 'worst case' scenario, i.e. the largest effective area that exists at any instance during the soil disturbance.

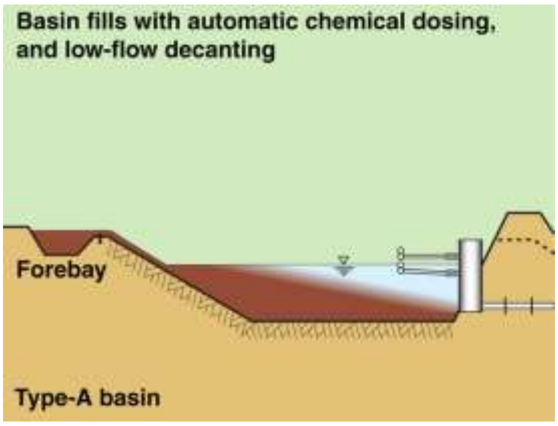
[2] Soil loss defines the maximum allowable soil loss rate (based on RUSLE analysis) from a given catchment area. A slope length of 80 m should be adopted within the RUSLE analysis unless permanent drainage or landscape features reduce this length.

[3] RUSLE analysis on a monthly basis shall only apply in circumstances where the timing of the soil disturbance is/shall be regulated by enforceable development approval conditions. When conducting monthly RUSLE calculations, use the worst-case monthly R-Factor during the nominated period of disturbance.

[4] Refer to the relevant regulatory authority for assessment procedures. The default standard is a Type 3 sediment trap.

[5] Exceptions to the use of sediment basins shall apply in circumstances where it can be demonstrated that the construction and/or operation of a sediment basin is not practical, such as in many forms of linear construction where the available work space or Right of Way does not provide sufficient land area. In these instances, the focus must be erosion control using techniques to achieve an equivalent outcome. The 'intent' shall always be to take all reasonable and practicable measures to prevent or minimise potential environmental harm.

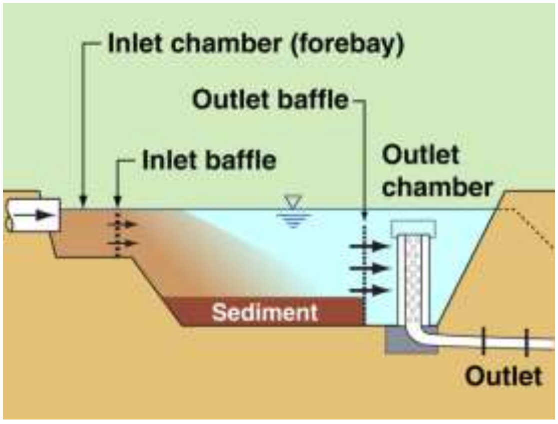
Step 2: Select the type of sediment basin



Type A sediment basin

Type A and B basins

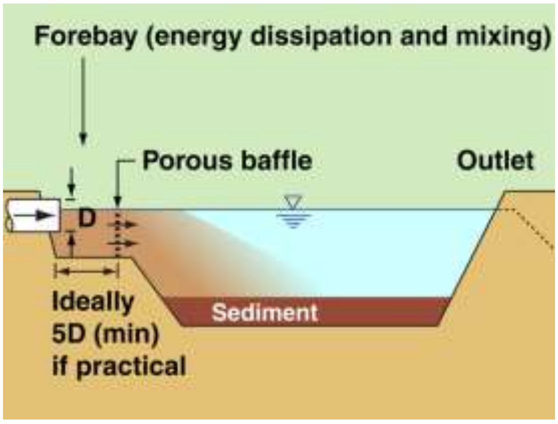
- Type A basins are recommended when the duration of the soil disturbance, within a given drainage catchment, exceeds 12 months.
- Type B basins are recommended when the duration of the soil disturbance, within a given drainage catchment, does not exceed 12 months.



Type C sediment basin

Type C basins

- Type C basins are recommended when:
  - less than 33% of the soil is finer than 0.02 mm (i.e. $d_{33} > 0.02$ mm), and
  - no more than 10% of the soil is dispersive (in other words, the soil is not dispersive).
- Slaking soils are likely to satisfy these requirements.
- Type C basins represent the older style basins that are largely discontinued by many authorities.



Type D sediment basin

Type D basins

- Type D basins are an alternative to Type A and B basins, and are used when it can be demonstrated that the installation of an automatic chemical flocculation system cannot be justified for the site conditions.
- These are a 'plug flow' system (i.e. not a continuous treatment system like Type A, B and C basins).
- Sediment-laden water enters the empty basin, is captured, treated, then decanted once a suitable water quality is achieved.

Select the type of sediment basin

Type A sediment basin (Qld)

Type A basins

- **Sizing:** based on minimum volume (V_S) and surface area (A_S) requirements
- **Operation:** continuous flow process
- **Chemical dosing;** automatic system
- **Decant:** floating decant system
- **Design storm:** typically Q1, but Q5 for long-term operations such as quarries and mine sites
- **Extras:** a forebay is required.

Type B sediment basin (Qld)

Type B basins

- **Sizing:** based on minimum volume (V_S) and surface area (A_S) requirements
- **Operation:** continuous flow process
- **Chemical dosing;** automatic system
- **Decant:** manual decent
- **Design storm:** typically 0.5Q1
- **Extras:** a forebay is required, but no floating decant system, and the basin can retain the captured water for dust control or plant watering on the construction site.

Type C sediment basin (NSW)

Type C basins

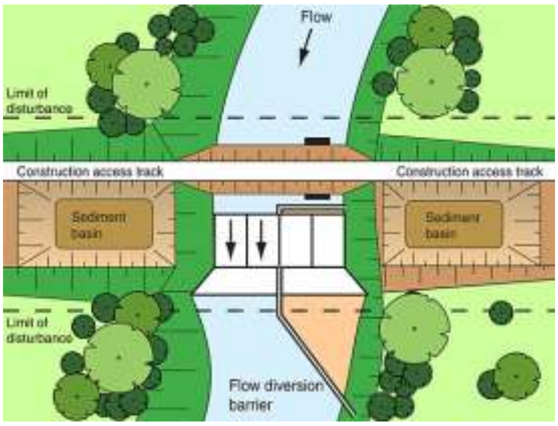
- **Sizing:** based on a minimum surface area (A_S) requirement
- **Operation:** continuous flow process
- **Chemical dosing;** no chemical dosing, but it can be added to the process
- **Decant:** automatic gravity system, but a floating decant system can be used
- **Design storm:** typically 0.5 Q1
- **Conditions of use:** used when working in sandy soils.

Type D sediment basin (Qld)

Type D basins

- **Sizing:** based on a minimum volume (V_S) requirement
- **Operation:** plug flow, i.e. a ‘start-stop’ batching process
- **Chemical dosing;** manual dosing
- **Decant:** manual decant
- **Design storm:** typically sized for an 80%ile, 5-day rainfall depth
- **Extras:** basins must be drained before the next storm.

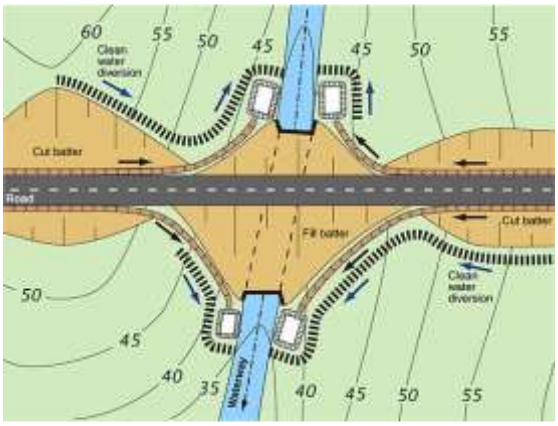
Step 3: Determine basin location



Basins located within a road reserve

Basin location

- Locate all basins within the relevant property boundary, unless permission from the adjacent land-holder has been provided (e.g. a farm adjacent to a road construction project).
- Locate all basins to maximise the collection of sediment-laden runoff generated within the site throughout the construction period, which extends up until the site is adequately stabilised against soil erosion, including raindrop impact.



Road construction over a waterway

Basins adjacent to waterways

- Do <u>not</u> locate a sediment basin within a waterway.
- For construction works that cross a waterway, it is typical for there to be four basins, located each side of the waterway, and each side of the crossing.
- Where practical, locate sediment basins above the 1 in 5 year ARI (18% AEP) flood level; **however**, common sense must apply—the basin must be in a position to perform its required task.

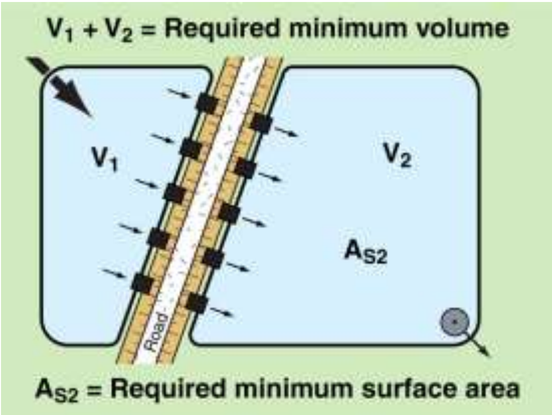
Limited access for basin de-silting

Locating a basin within an active work area

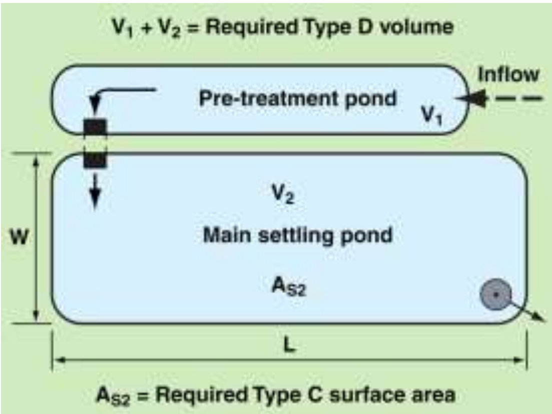
- Avoid locating a basin in an area where future construction works may limit the operational life of the basin.
- Minimise disturbance to the roots of retained or protected trees—refer to AS4970: *'Protection of trees on development sites'*.
- Ensure basins have suitable access for maintenance and de-silting.

Determine basin location

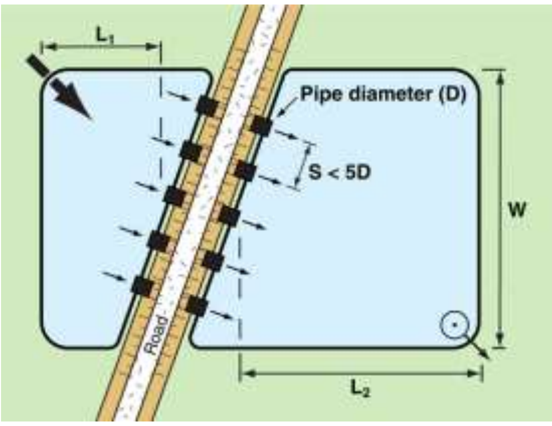
Several basins operating in 'series'



Divided Type A basin



Type C basin with pre-treatment pond



Divided Type C basin

Sediment basins operating in 'series'

- Operating basins in 'series' means the water from one basin flows into the next basin, and so on.
- Several basins operating in series can have significantly less sediment trapping efficiency than a single basin, even though the series of smaller basins may have the same total surface area and volume as a single large basin.
- Critical to the operation of these basins is maintaining an even flow distribution from one basin to another.

Circumstances where a series of divided basins can be used

- Basins that possibly could operate in series include:
  - Type A basins where the combined basin volume satisfies the minimum volume requirement, and at least one of the basins is able to satisfy the minimum surface area requirement.

Further to the above

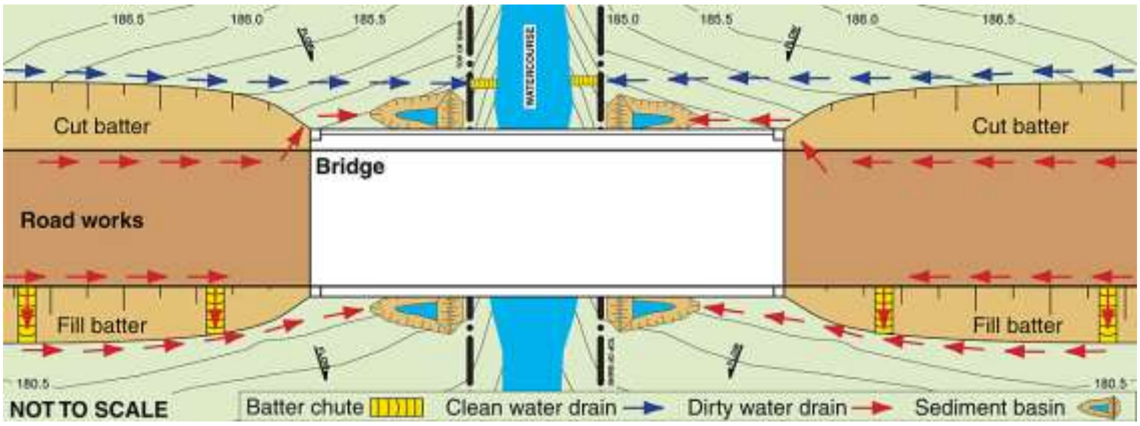
- Other basins that possibly could operate in series include:
  - Type D basins where at least one of the basins has sufficient surface area and length to width ratio to satisfy the requirements of a Type C basin.
    - The combined *settling volume* of the basins must not be less than that specified for a Type D basin.
  - A series of Type C or D basins where each settling pond is connected by several pipes evenly spaced across the basin.
    - Such a design must minimise the effects of inflow jetting from each pipe, and allow an even distribution of flow across the full basin width.
    - In such cases the minor sediment remixing that occurs as flow passes through the pipes is usually compensated by the improved hydraulic efficiency of the overall basin surface area.

Sediment controls for road construction over a waterway

Sediment basins (Type 1 sediment trap)

Typical basin layout

- Placement of Type 1 sediment traps each side of a small drainage pipe (culvert) is appropriate when:
  - the contributing catchment area is greater than 0.25 ha, or
  - soil loss rate > 150 t/ha/yr.
- Not all of the clean and dirty water drains shown below will be operational during each phase of the road construction.
- The contributing catchment area can include both the road and batter runoff.

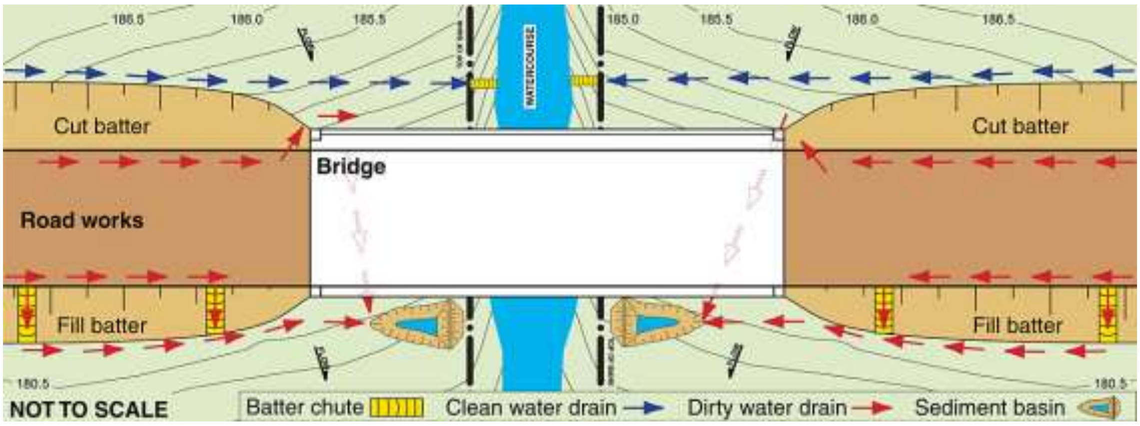


ESC measures for road works over a waterway with significant dirty water runoff

Sediment basin (Type 1 sediment trap)

Alternative drainage layouts

- The number of sediment traps can be reduced if sediment-laden runoff from both sides of the roadway can be diverted to a single sediment trap located each side of the waterway.
- The above examples apply equally to the construction of bridges and culverts; however, this alternative drainage layout (below) can only be employed on bridge construction.



Alternative layout with dirty water directed under the bridge towards the basins

Determine basin location (examples only)

Brisbane, Qld

Brisbane, Qld

Brisbane, Qld

Pacific Highway, NSW

Pacific Highway, NSW

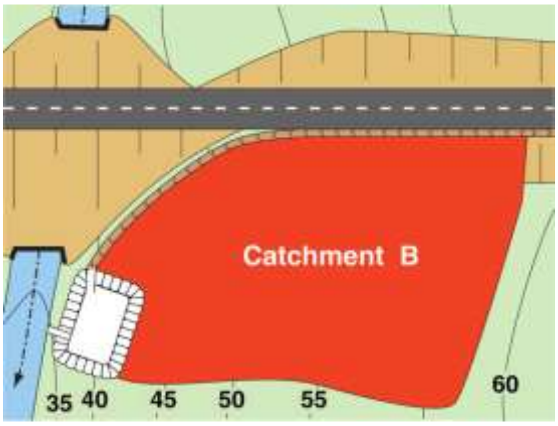
Pacific Highway, NSW

Pacific Highway, NSW

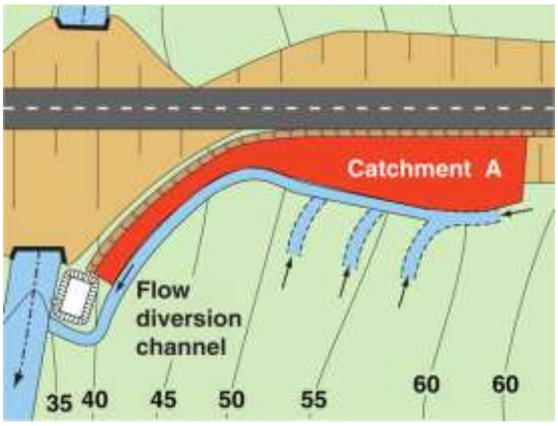
Road construction over stormwater pipe

Step 4: Divert up-slope ‘clean’ water

Step 4: Divert up-slope ‘clean’ water



Catchment area <u>without</u> flow diversion



Catchment area <u>with</u> flow diversion

Catch drain diverting ‘clean’ bush runoff

Introduction

- Wherever reasonable and practicable, up-slope ‘clean’ water should be diverted around the sediment basin to decrease the required size of the basin, and increase the basin’s sediment trapping efficiency.

Intent

- The <u>intent</u> is to minimise the volume of uncontaminated water flowing to a basin at any given time during the operation of the basin, even if the basin has been sized for the full catchment area.

What is considered ‘clean’ water?

- ‘Clean’ water is defined as
  - water that enters the property from an external source and has not been further contaminated by sediment within the property; or
  - water that has originated from the site and is of such quality that it either does not need to be treated in order to achieve the required water quality standard, or would not be further improved if it were to pass through the basin.

Step 5: Select internal and external bank gradients

Step 5: Select internal and external bank gradients

Very steep and slippery bank slope (Qld)

Partially fenced sediment basin (NSW)

Grassed banks (NSW)

Introduction

- The basin's internal bank gradient is important because it can alter the mathematical relationship between the pond's surface area (**A**$_S$) and volume (**V**$_S$).
- Recommended maximum gradients are:
  - 1:2 for good, erosion-resistant clay or clay-loam soils
  - 1:3 sandy-loam soil
  - 1:4 sandy soils
  - 1:5 unfenced sediment basins that are accessible to the public.

Fencing

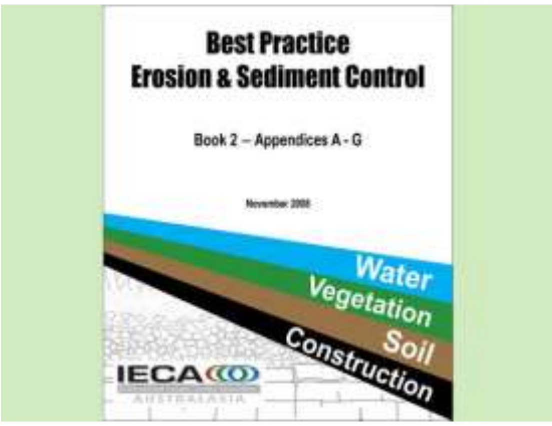
- If public safety is a concern, and the basin's internal banks are steeper than 1:5 (V:H), and the basin will not be fenced, then a suitable method of egress during wet weather needs to be installed.
- Examples include a side ladder, steps cut into the bank, or at least one bank turfed for a width of at least 2 m from the top of bank to the toe of bank.

Grassing banks

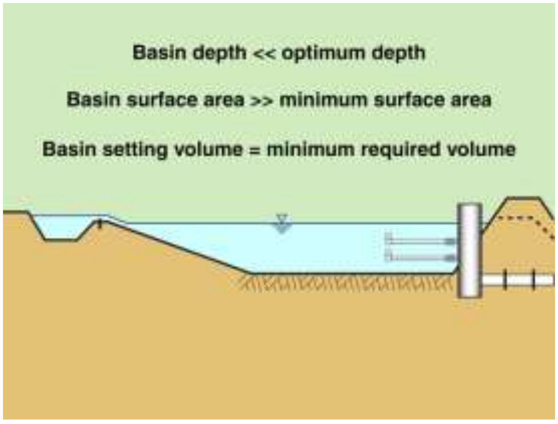
- Grassing the basin's embankment and internal banks can:
  - reduce soil erosion and sediment runoff from the basin's banks
  - improve safety by increasing a person's foot grip (climbing ability) when the soil is wet.
- A bank slope of 1:6 (V:H) is recommended for banks that will be mown on a regular basis.

Step 6a: Sizing Type A basins

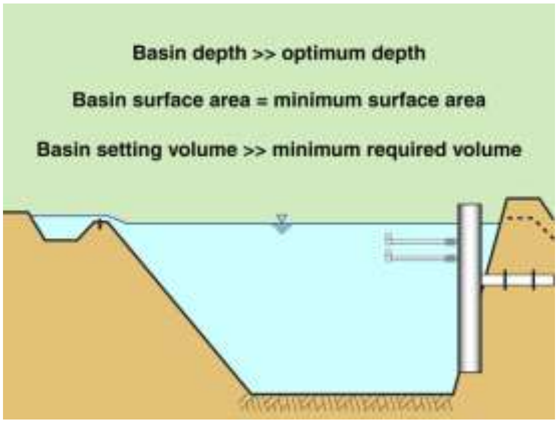
Step 6a: Sizing Type A basins



IECA (2008) – Book 2



Basin depth less than the 'optimum'



Basin depth greater than the 'optimum'

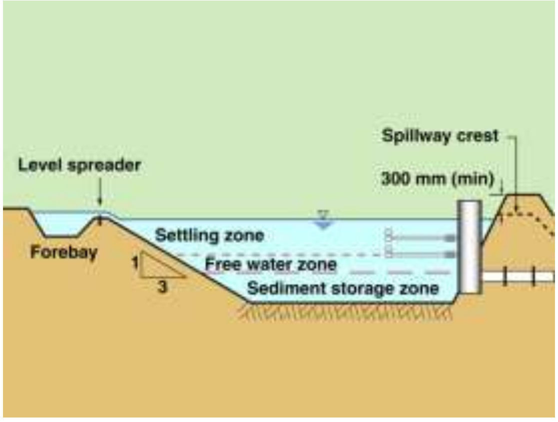
Introduction

- The requirements for the sizing of sediment basins varies from region to region.
- The following design steps are based on the recommendations outlined in IECA (Australasia), 2008, and the updated Appendix B (2018).
- Readers should check for further updates to Appendix B.
- Readers will need to refer to Appendix B to obtain the necessary tabulated data.

Sizing of Type A basins

- It has been said that the mathematical procedure presented in Appendix B for the sizing of Type A basins is unnecessarily complex, especially when compared to the sizing of an Auckland-style (NZ) basin.
- Auckland basins have the advantage of only having to work in one location, while Type A basins must work across Australia.
- What makes the sizing of Type A basins so complex is the fact that the basins need to satisfy both a minimum surface area requirement (A_S), and a minimum settling volume requirement (V_S).
- Consequently, a critical aspect of the design procedure is the selection of an 'optimum' settling pond depth.
- It is only at this optimum depth that the minimum surface area and settling volume are achieved simultaneously.
- If the basin is too shallow, then the basin's surface area will exceed the minimum requirements.
- If the basin is too deep, then the basin's volume will exceed the specified minimum while the basin's surface area requirement dictates the basin's sizing.

Sizing Type A basins



Type A basin

Components of a Type A basin

- Type A basins typically contain the following components:
  - mixing zone
  - forebay and level spreader
  - settling zone (upper zone)
  - free water zone (middle zone)
  - sediment storage zone (bottom zone)
  - floating decant system
  - emergency spillway
  - energy dissipater.

Table B6 – Components of the settling pond depth and volume (Type A basin)

Component			Term	Minimum depth	Term	Min. volume as a percentage of V_S
Total depth	Settling zone		D_S	0.6 m	V_S	100%
	Retained water zone	Free water	D_{FW}	0.2 m	V_F	—
		Sediment storage zone	D_{SS}	0.2 m	V_{SS}	30%

Minimum requirements of a Type A sediment basin (Table B6, IECA, 2018)

Table B7 – Recommended design storm for Type A basins

Design storm	Type of soil disturbance
1 yr	• Short-term soil disturbances, such as civil construction and urban development.
5 yr	• Long-term soil disturbances, such as landfill sites, quarries and mine sites.

IECA (2018) Table B7

Step 1A – Design storm

- Determine the design event from Table B7 (IECA, 2018).
  - Adopt a 1 year storm event for short-term soil disturbances, such as civil construction and urban development.
  - Adopt a 5 year storm event for long-term soil disturbances, such as landfill sites, quarries and mine sites.

Table B5 – Suggested 'trial value' of the optimum low-flow decant rate, Q_A

Likely optimum Q_A	Locations
4 L/s/ha	Mildura, Adelaide, Mt Gambier (D_S = 1.0 to 1.5 m)
5 L/s/ha	Wagga, Melbourne, Bendigo, Ballarat, Hobart (D_S = 1.0 m) Bourke, Dubbo, Bathurst, Goulburn (D_S = 1.5 m)
6 L/s/ha	Bourke, Bathurst, Canberra, Perth (D_S = 1.0 m) Toowoomba (based on D_S = 2.0 m)
7 L/s/ha	Dubbo, Tamworth, Goulburn (based on D_S = 1.0 m) Roma, Toowoomba (based on D_S = 1.5 m)
8 L/s/ha	Dalby, Roma, Armidale (based on D_S = 1.0 m)
9 L/s/ha	Darwin, Cairns, Townsville, Mackay, Rockhampton, Emerald, Caloundra, Brisbane, Toowoomba (D_S = 1.0 m), Lismore, Port Macquarie, Newcastle, Sydney, Nowra

IECA (2018) Table B5

Step 2A – Decant rate

- Select a trial low-flow decant rate (Q_A) from Table B5 (IECA, 2018).
- Alternatively, use equations B1 or B3 to determine an optimum decant rate.
- This is the low-flow decant rate at maximum water level, i.e. when all decant arms (if used) are operational.
- The decant rate may be based on locally available (commercial) decant systems.
- A maximum decant rate of 9 L/s/ha is currently (2018) recommended.

Sizing Type A basins

(ii) Optimum settling pond depth:

Alternatively, the designer may choose to nominate a low-flow decant rate (Q_A) based on the desired number of riser pipes and floating decant arms (refer to Figure B29), and then determine an optimum settling pond depth (D_S).

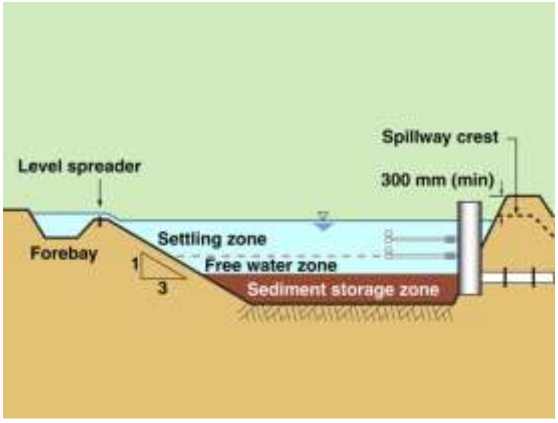
For all ARI events: $D_{S(optimum)} = (K \cdot I^{[illegible]})/(K_S \cdot Q_A)$ (B4)

For a 1 yr ARI design: $D_{S(optimum)} = 0.684\,(I^{[illegible]})/(K_S \cdot Q_A^{[illegible]})$ (B5)

For the Auckland-type decant system:

Q_A = 0.0045(number of decant arms) / (catchment area) [$m^3/s/ha$]

IECA (2018) equations B4 & B5



Type A basin

Table B8 – Type A basin sizing equation coefficient 'K'

Low-flow decant rate 'Q_A'		Coefficient 'K' for specific design events		
L/s/ha	$m^3/s/ha$	1 year	2 year	5 year
2	0.002	45.0	46.0	46.9
3	0.003	34.5	36.7	39.5
4	0.004	28.4	30.8	33.9
6	0.006	22.7	22.9	26.0
8	0.008	17.6	18.8	20.9
9	0.009	16.2	17.4	19.3

IECA (2018) Table B8

Table B9 – Assessment of a design coefficient (K_S) from Jar Test results

Jar test settlement after 15 min (mm)	50	75	100	150	200	300
Laboratory settlement rate (m/hr)	0.20	0.30	0.40	0.60	0.80	1.20
Factor of safety	1.33	1.33	1.33	1.33	1.33	1.33
Design settlement rate, v_S (m/hr)	0.15	0.23	0.30	0.45	0.60	0.90
Design settlement coefficient, K_S (s/m)	24000	16000	12000	8000	6000	4000
Minimum depth of the settling zone:						
Minimum settling zone depth, D_S (m)	0.6	0.6	0.6	0.68	0.90	1.35

Table B10 – Recommended water temperature for use in performing a Jar Test

City	Suggested water temperature (°C)
Darwin	30
Brisbane	20
Adelaide	15
Perth	15
Sydney	15
Canberra	10
Melbourne	10
Hobart	10

IECA (2018) tables B9 & B10

Step 3A – Optimum pond depth

- Determine the optimum settling pond depth using either equations B4 or B5.
- For a given low-flow decant rate (Q_A), there is an 'optimum' settling zone depth (D_S) that will allow the minimum settling volume and minimum settling zone surface area requirements to be achieved concurrently.

Where:

- Q_A = the low-flow decant rate per hectare of contributing catchment [$m^3/s/ha$]
- K = equation coefficient that varies with the design event (X) and the low-flow decant rate (Q_A) refer to Table B8
- $I = I_{X\ yr,\ 24\ hr}$ the average rainfall intensity for an X-year, 24-hour storm
- K_S = inverse of the settling velocity of the critical particle size (Table B9)
- D_S = depth of the settling zone measured from the spillway crest [m].

- If site conditions place restrictions on the total depth of the sediment basin (D_T), then this will impact upon the maximum allowable depth of the settling zone (D_S);

Determination of equation variable 'K'

- K = equation coefficient that varies with the design event (X) and the low-flow decant rate (Q_A) refer to Table B8.

Determination of equation variable 'K_S'

- 'K_S' is the inverse of the settling velocity of the critical particle size (Table B9).
- Sediment settling tests (Jar Tests) should be performed at the water temperature that is expected to exist in the basin.
- Typical water temperatures for Australian capital cities are provided in Table B10.
- The water temperature within the settling pond is likely to be equal to the temperature of rainwater (approximately the air temperature) at the time of year when rainfall intensity is the highest.

Sizing Type A basins

Table B13 – Typical Type A settling zone, free water & sediment storage depths

Type A basin geometry with sediment storage volume, V_{SS} = 30% (V_S):						
Inlet bank slope, 1 in 3	All other bank slopes, 1 in 2		Total depth, D_T = 1.5 m			
Typical basin dimensions based on a length:width ratio of 3:1 at top of the settling zone:						
Settling zone volume, V_S [m³]	**50**	**100**	**200**	**400**	**800**	**1600**
Total basin volume, V_T [m³]	75	147	292	585	1176	2364
Settling zone surface area [m²]	85	136	241	449	863	1682
Settling zone depth (D_S) [m]	0.59	0.73	0.83	0.89	0.93	0.95
Ratio D_S/D_T as a percentage	39%	49%	55%	60%	62%	64%
Free water depth (D_{FW}) [m]	0.20	0.20	0.20	0.20	0.20	0.20
Ratio D_{FW}/D_T as a percentage	13%	13%	13%	13%	13%	13%
Sediment storage (D_{SS}) [m]	0.71	0.57	0.47	0.41	0.37	0.35
Ratio D_{SS}/D_T as a percentage	48%	38%	32%	27%	25%	23%

[1] The settling zone surface area represents the 'average' surface area, $A_S = V_S/D_S$.

IECA (2018) Table B13

Step 4A – Pond depth

- If you choose a settling zone depth equal to the optimum depth determined in Step 3A, then a minimum basin volume and surface area will be achieved.
- However, a minimum depth of 0.6 m is recommended.
- Tables B13 to B15 can be used to estimate an appropriate settling zone depth (D_S) based on a desirable maximum basin depth (D_T), and a bank slope of 1 in 2 (excluding the inlet bank slope of 1 in 3).

Table B14 – Typical Type A settling zone, free water & sediment storage depths

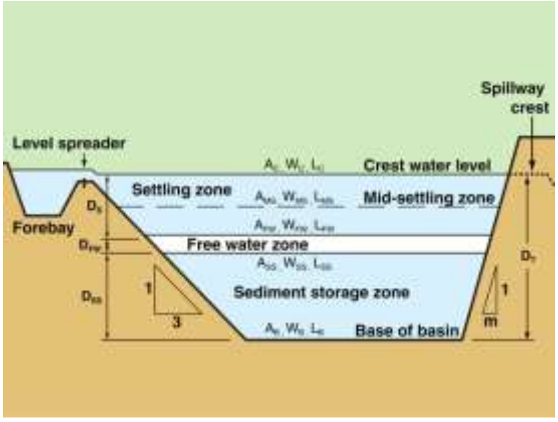
Type A basin geometry with sediment storage volume, V_{SS} = 30% (V_S):						
Inlet bank slope, 1 in 3	All other bank slopes, 1 in 2		Total depth, D_T = 2.0 m			
Typical basin dimensions based on a length:width ratio of 3:1 at top of the settling zone:						
Settling zone volume, V_S [m³]	**120**	**200**	**400**	**800**	**1600**	**3200**
Total basin volume, V_T [m³]	172	282	559	1119	2247	4514
Settling zone surface area [m²]	143	202	351	648	1240	2412
Settling zone depth (D_S) [m]	0.83	0.98	1.13	1.23	1.29	1.32
Ratio D_S/D_T as a percentage	42%	49%	57%	61%	64%	66%
Free water depth (D_{FW}) [m]	0.20	0.20	0.20	0.20	0.20	0.20
Ratio D_{FW}/D_T as a percentage	10%	10%	10%	10%	10%	10%
Sediment storage (D_{SS}) [m]	0.97	0.82	0.67	0.57	0.51	0.48
Ratio D_{SS}/D_T as a percentage	48%	41%	33%	29%	26%	24%

IECA (2018) Table B14

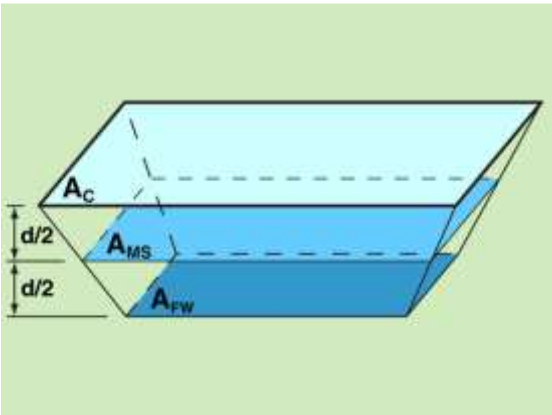
Table B15 – Typical Type A settling zone, free water & sediment storage depths

Type A basin geometry with sediment storage volume, V_{SS} = 30% (V_S):						
Inlet bank slope, 1 in 3	All other bank slopes, 1 in 2		Total depth, D_T = 3.0 m			
Typical basin dimensions based on a length:width ratio of 3:1 at top of the settling zone:						
Settling zone volume, V_S [m³]	**400**	**800**	**1600**	**3200**	**6400**	**12,800**
Total basin volume, V_T [m³]	558	1075	2146	4302	8632	17310
Settling zone surface area [m²]	305	488	867	1623	3124	6102
Settling zone depth (D_S) [m]	1.33	1.62	1.83	1.96	2.04	2.10
Ratio D_S/D_T as a percentage	44%	54%	61%	65%	68%	70%
Free water depth (D_{FW}) [m]	0.20	0.20	0.20	0.20	0.20	0.20
Ratio D_{FW}/D_T as a percentage	7%	7%	7%	7%	7%	7%
Sediment storage (D_{SS}) [m]	1.47	1.18	0.97	0.84	0.76	0.70
Ratio D_{SS}/D_T as a percentage	49%	39%	32%	28%	25%	23%

IECA (2018) Table B15



Terminology (Type A basins)



Simpson's Rule

Step 5A – Surface area

- Calculate the minimum, average, settling zone surface area (A_S) based on Equation B10 and the following design conditions:
  - (i) the expected settling rate of the treated sediment floc
  - (ii) the expected water temperature within the pond during its critical operational phase (i.e. the local wet/rainy season).
- In most cases it can be assumed that this average surface area is the same as the surface area at the mid-depth of the settling zone (A_{MS}).

Simpson's Rule

- If a more accurate determination of volume is required, then the Simpson's Rule can be used (Equation B11).

$$V_S = (D_S/6).(A_C + 4.A_{MS} + A_{FW}) \qquad \text{(B11)}$$

V_S = settling volume [m³]

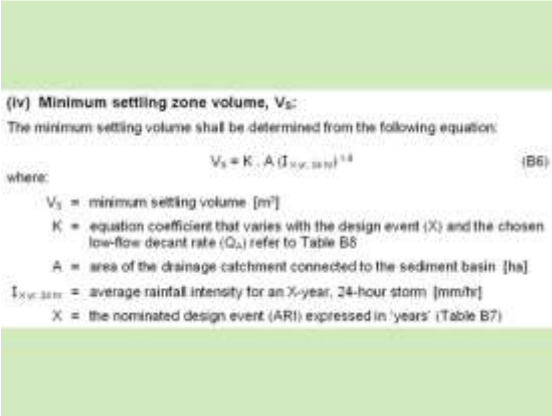
D_S = depth of settling zone [m]

A_C = surface area at spillway crest [m²]

A_{MS} = surface area at mid settling zone

A_{FW} = surface area at the top of the Free Water Zone.

Sizing Type A basins



(iv) Minimum settling zone volume, V_S:

The minimum settling volume shall be determined from the following equation:

$$V_S = K \,.\, A \,(I_{X\text{yr},\,24\text{hr}})^{1.6} \qquad \text{(B6)}$$

where:

V_S = minimum settling volume [m³]

K = equation coefficient that varies with the design event (X) and the chosen low-flow decant rate (Q_A) refer to Table B8

A = area of the drainage catchment connected to the sediment basin [ha]

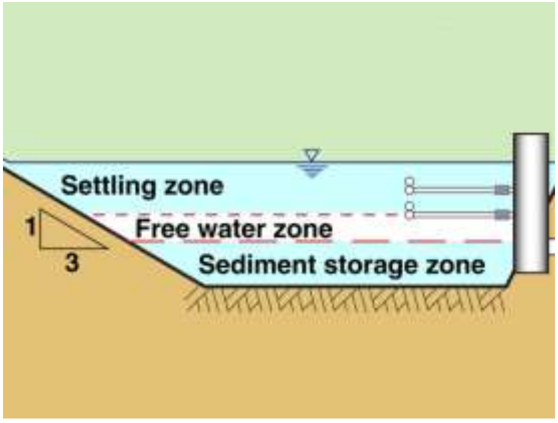
$I_{X\text{yr},\,24\text{hr}}$ = average rainfall intensity for an X-year, 24-hour storm [mm/hr]

X = the nominated design event (ARI) expressed in 'years' (Table B7)

IECA (2018) Equation B6

Step 6A – Pond volume

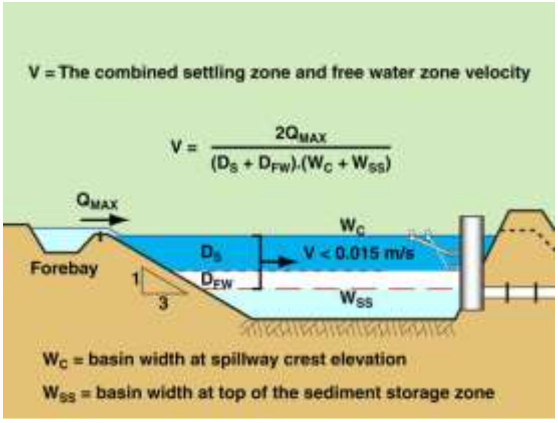
- Calculate the required settling zone volume (V_S), being the greater of:
  - (i) the minimum volume based on Equation B6
  - (ii) the settling zone volume determined from the minimum average surface area obtained from Step 5A.
- Where the equation coefficient 'K' was previously determined from Table B8 (previous page).



Free water zone

Step 7A – Free water zone

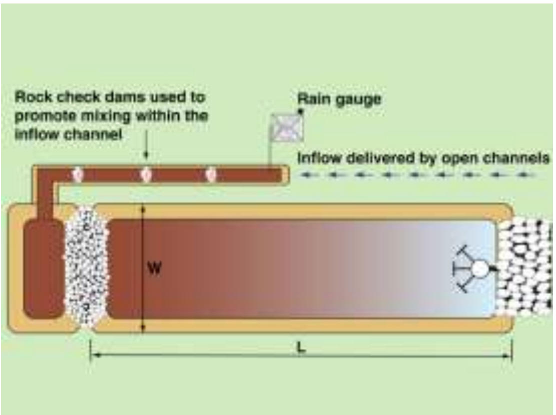
- Nominate the depth of the free water zone (D_F).
- The free water zone is used to separate the settled sediment from the low-flow decant system to prevent settled sediment from being drawn into the decant system at the start of the next storm.
- The minimum recommended depth of the free water zone is 0.2 m.



Maximum settling zone velocity

Step 8A – Sediment re-suspension

- Check for the potential re-suspension of settled sediment.
- The maximum allowable flow velocity upstream of the overflow spillway (based on the combined settling zone and free water zone cross-sectional area) has been set at 1.5 cm/s (0.015 m/s) based on decant testing of settled sludge blankets in wastewater treatment plants.
- There are outstanding questions about the validity of this re-suspension velocity, so check with your supervisor/regulator.



Type A basin (plan view)

Step 9A – Pond dimensions

- Determine the length (L_C) and width (W_C) of the settling zone.
- It is recommended that the settling zone length (L_C) > 3 times its width (W_C).
- These dimensions are measured at the height of the spillway crest.

Sizing Type A basins

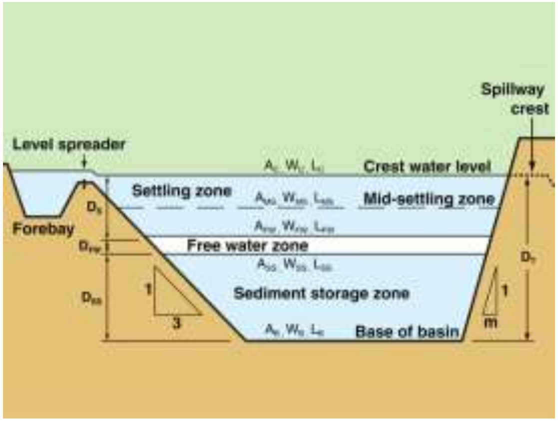
Table B6 – Components of the settling pond depth and volume (Type A basin)

Component			Term	Minimum depth	Term	Min. volume as a percentage of V_S
Total depth	Settling zone		D_S	0.6 m	V_S	100%
	Retained water zone	Free water	D_{FW}	0.2 m	V_F	—
		Sediment storage zone	D_{SS}	0.2 m	V_{SS}	30%

IECA (2018) Table B6

Step 10A – Overall basin dimensions

- Once the volume and dimensions of the settling zone are known, the remaining basin dimensions can be determined based on the sizing requirements outlined in Table B6.
- The internal bank slope adjacent the forebay should not be steeper than 1 in 3 (refer to Figure B6).



Type A basin

Determination of the sediment storage zone depth (D_{SS})

- Because of the basin's shape, it is <u>not</u> possible to assume that the depth of the sediment storage zone (D_{SS}) is simply 30% of the settling zone depth.
- One option is to utilise a spreadsheet to complete the analysis of a Type A basin, including the sizing of the sediment storage zone.
- Technical notes B2 to B4 and tables B13 to B15 provide a manual method to estimate D_{SS}.

Technical Note B2 – Determination of basin dimensions given Vs and Ds

The initial design steps for a Type A basin result in the determination of two key parameters:

- the settling zone volume, V_S (m^3)
- the settling zone depth, D_S (m)

The settling zone volume (V_S) is taken as the greater of:

- the minimum settling zone volume determined from Equation B6; or
- the settling zone volume based on the minimum average settling zone surface area (A_S). This condition would dictate the settling zone volume in cases where the basin's design is controlled by the minimum surface area requirement presented by Equation B10.

The next step is to determine the depth of the basin (D_T), the bank slope (m), and the basin's width and length. Once the bank slope and base dimensions are known, all other dimensions can be determined (the following analysis assumes the slope of the inlet bank is 1 in 3).

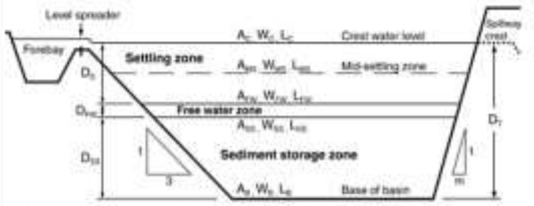


Figure B6 – Basin long-section with suggested dimensional terminology

If the parameters, V_S & D_S are known, then the basin's total depth (D_T) can be determined by one of the following methods:

- trial and error analysis of the basin's dimensions in order to achieve the various dimensional requirements of a Type A basin, including those outlined in Table B6
- utilisation of a spreadsheet program to determine suitable basin dimensions
- utilisation of the equations listed below to determine an 'approximation' of the sediment storage depth (D_{SS}) and total depth (D_T) based on the basin taking the shape of a trapezoidal prism.

Approximation of sediment storage depth (D_{SS}) and the total basin depth (D_T):

$$D_S/D_{SS} = K_1 \cdot \log_{10}(V_S - K_2) + K_3 \quad \text{(for values of } K_1, K_2, K_3 \text{ see Note B3)} \quad \text{(B12)}$$

$$D_T = D_S + 0.2 + D_{SS} \quad \text{(B13)}$$

Determination of the basin's length and width:

The basin's length and width is typically defined by its dimensions at the <u>crest</u> of the overflow weir (W_C & L_C); however, the basin's average surface area (A_S) is defined at the mid-elevation of the settling zone. It is recommended that basins are designed with a length:width = 3:1 at the elevation of the spillway crest; however, to simplify the design process, designers can choose to apply this recommended length:width ratio to the basin's dimensions at the mid-elevation of the settling zone, thus:

$$W_{MS} = (A_S/3)^{0.5} \quad \text{(B14)}$$

$$L_{MS} = 3.W_{MS} \quad \text{(B15)}$$

$$H_{MS} = 0.5D_S + 0.2 + D_{SS} \quad \text{(B16)}$$

IECA (2018) Technical Note B2

Technical Note B4 – Interpolation of basin dimensions for low values of 'Vs'

Low range values of V_S that can be used to interpolate an estimate of the sediment storage depth D_{SS} that achieves the minimum sediment storage volume, $V_{SS} = 0.3V_S$, are provided below.

Table B12 – Basin dimensions for low-range values of V_S

D_S	m	Minimum workable value				Low-range value			
		V_{S1}	D_{SS}	W_B	L_B	V_{S2}	D_{SS}	W_B	L_B
(m)	(slope)	(m^3)	(m)	(m)	(m)	(m^3)	(m)	(m)	(m)
0.6	1	15	0.73	0	4.0	22	0.42	1.5	7.0
0.6	1.5	31	0.70	0	7.7	46	0.41	2.0	11.6
0.6	2	53	0.73	0	11.1	76	0.40	2.6	16.1
0.6	3	117	0.69	0	18.8	168	0.39	3.7	25.6
0.6	4	205	0.73	0	28.0	295	0.38	4.9	34.7
0.8	1	31	1.01	0	4.7	45	0.56	1.9	8.7
0.8	1.5	64	1.00	0	9.2	92	0.54	2.5	14.4
0.8	2	111	0.96	0	14.0	160	0.52	3.2	20.2
0.8	3	244	0.93	0	23.4	351	0.51	4.6	32.0
0.8	4	430	0.91	0	32.8	619	0.50	6.1	43.8
1	1	56	1.25	0	5.7	81	0.69	2.2	10.5
1	1.5	116	1.20	0	11.2	167	0.68	3.0	17.4
1	2	201	1.16	0	16.9	289	0.66	3.8	24.4
1	3	442	1.14	0	28.3	636	0.63	5.6	38.6
1	4	779	1.12	0	39.6	1122	0.62	7.3	52.9
1.2	1	91	1.54	0	6.3	131	0.83	2.6	12.2
1.2	1.5	190	1.46	0	13.0	274	0.79	3.5	20.3
1.2	2	329	1.40	0	19.7	474	0.77	4.5	28.6
1.2	3	726	1.35	0	33.2	1046	0.75	6.5	45.3
1.2	4	1280	1.33	0	46.5	1848	0.74	8.5	62.0
1.5	1	168	1.95	0	7.5	242	1.03	3.1	14.9
1.5	1.5	352	1.81	0	15.8	507	0.97	4.3	24.8
1.5	2	611	1.72	0	24.1	880	0.96	5.5	35.0
1.5	3	1349	1.67	0	40.5	1943	0.93	8.0	55.4
1.5	4	2380	1.65	0	56.9	3427	0.91	10.5	75.9

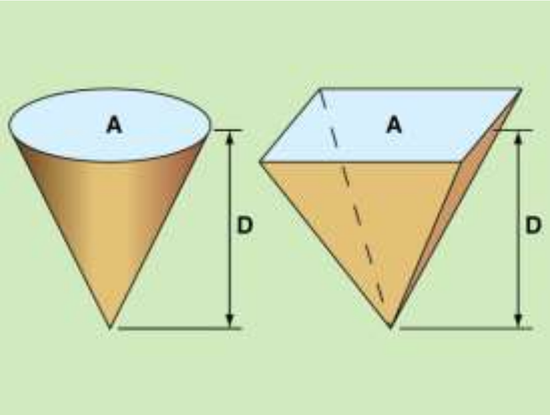
The term 'V_{S1}' defines the minimum possible settling zone volume that can exist for given values of D_S, m, and $V_{SS} = 0.3V_S$ at the point where the base width (W_B) approaches zero metres.

The term 'V_{S2}' defines a low-range value of the settling zone volume for which Equation B12 is considered to provide a suitable estimate of the term D_S/D_{SS}. Equation B12 can produce questionable values of D_S/D_{SS} for settling volumes between the values of V_{S1} and V_{S2}.

In some cases the basin's preferred dimensions will be governed by a desirable maximum total basin depth (D_T). In such cases, tables B13 to B15 can be used to interpolate typical values of D_S and D_{SS} for a basin with side slopes of 1 in 2.

IECA (2018) Technical Note B4

Determine the overall dimensions of the basin – Volume calculations



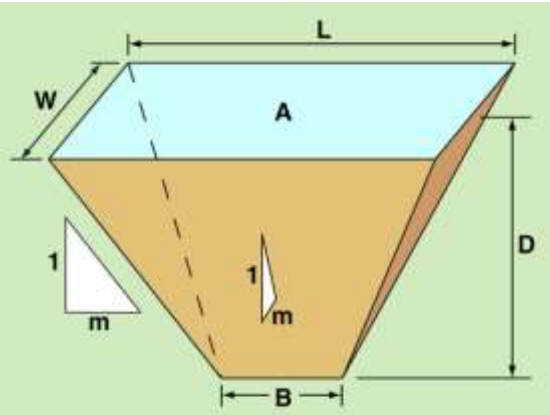
Cone and rectangular pyramid

Cone and pyramid shapes

$$V = (1/3).A.D$$

where:

- V = pond volume [m^3]
- A = top surface area [m^2]
- D = depth of pond [m]



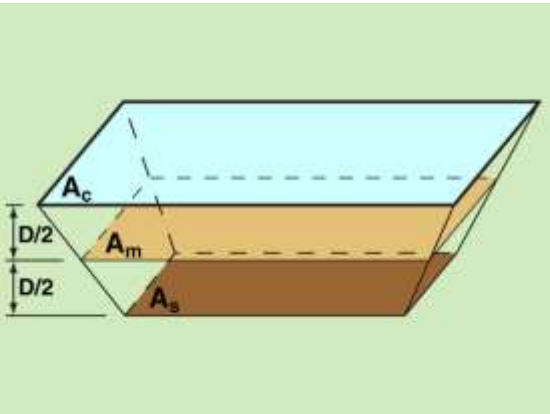
Rectangular prism

Rectangular prism

$$V = (1/3).W.(L - B).D + (1/2).W.B.D$$

where:

- V = pond volume [m^3]
- W = width of top surface [m]
- L = length of top surface [m]
- B = width of bottom edge [m]
- D = depth of volume [m]



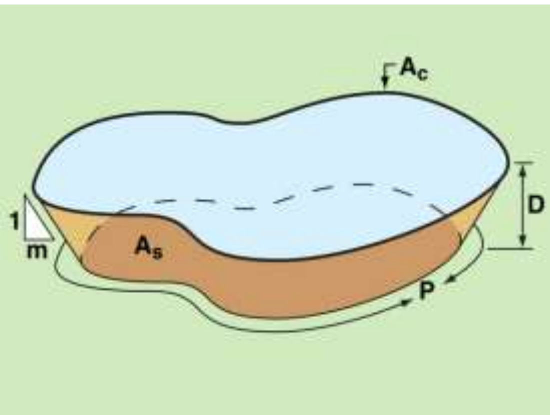
Truncated pyramid (Frustum)

Simpson's Rule

$$V = (D/6).(A_C + 4.A_M + A_S)$$

where:

- V = pond volume [m^3]
- D = depth of volume [m]
- A_C = surface area at top of volume [m^2]
- A_M = surface area at mid depth [m^2]
- A_S = surface area at base of volume [m^2]



Curved three-dimensional shape

Estimation of required basin depth given the pond surface area and bank slope

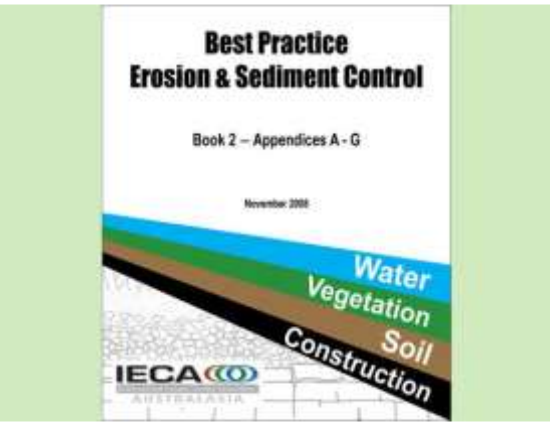
$$D \approx \frac{-A_s + \sqrt{(A_s^2 + 2.P.m.V)}}{P.m}$$

where:

- D = pond depth [m]
- A_S = pond surface area at base [m^2]
- P = circumference of the base of the volume [m]
- V = required basin volume [m^3]
- m = constant bank slope around the volume

Step 6b: Sizing Type B basins

Step 6b: Sizing Type B basins



IECA (2008) – Book 2

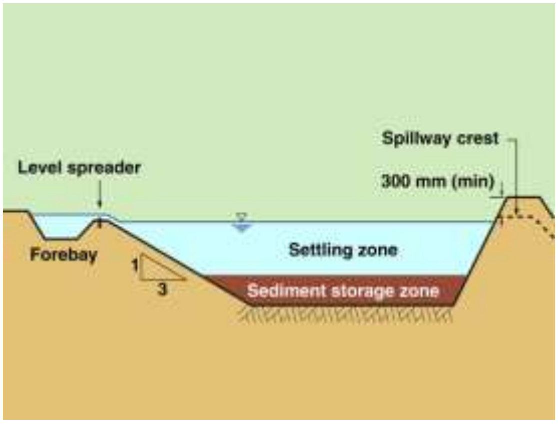
Introduction

- The requirements for the sizing of sediment basins varies from region to region; therefore, check your local rules.
- The following design steps are based on the recommendations outlined in IECA (Australasia), 2008, and the updated Appendix B (2018).
- Readers should check for further updates to Appendix B.
- Readers will need to refer to Appendix B to obtain the necessary tabulated data.

Type B basin (Qld)

Type B basins

- **Sizing:** based on minimum volume and surface area requirements
- **Operation:** continuous flow process
- **Chemical dosing;** automatic system
- **Decant:** manual decant
- **Design storm:** typically 0.5Q1
- **Extras:** a forebay is required, but no floating decant system, and the basin can retain the captured water for dust control or plant watering on the construction site.



Type B basin

Type B basin zones

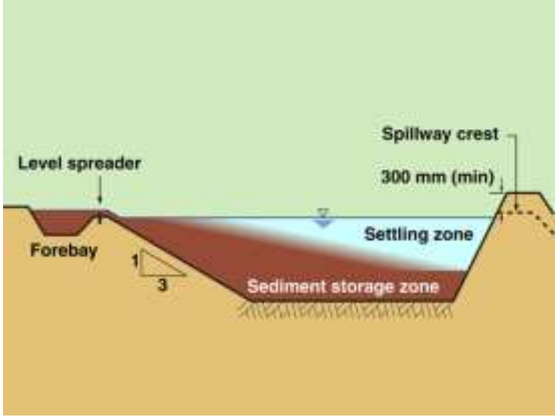
- The settling pond within a Type B sediment basin is divided horizontally into two zones:
  - the upper settling zone and
  - the lower sediment storage zone.
- Type B basins incorporate the same forebay, level spreader and automatic dosing system as a Type A basin.

Sizing Type B basins

Table B16 – Components of the settling pond depth and volume (Type B basin)

	Component	Term	Minimum depth	Term	Min. volume as a percentage of V_S
Total depth	Settling zone	D_S	0.5 m Option 1B 0.6 m Option 2B	V_S	100%
	Sediment storage zone	D_{SS}	0.2 m	V_{SS}	30%

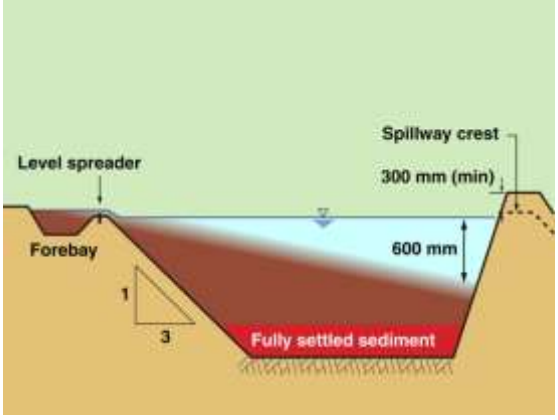
Minimum requirements of a Type B basin (Table B16, IECA, 2018)



Design option 1B

Design option 1B

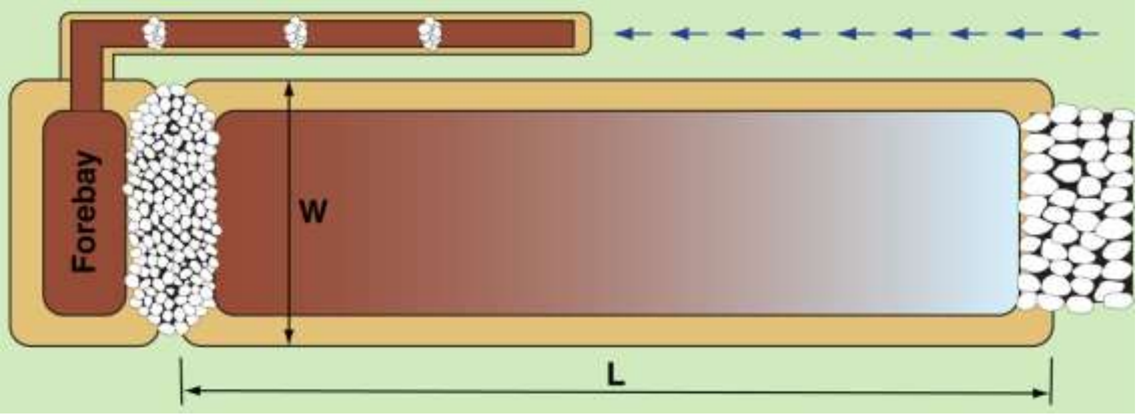
- Option 1B is based on setting a minimum settling pond surface area (A_S) and depth (D_S) such that the settled sediment has sufficient settling time to reach the existing settled sediment layer
- This means the sediment floc is able to form a 'compact' sediment blanket.
- It is anticipated that such a blanket would have a greater resistance to the effects of surface scour caused by the forward movement of the above supernatant layer.



Design option 2B

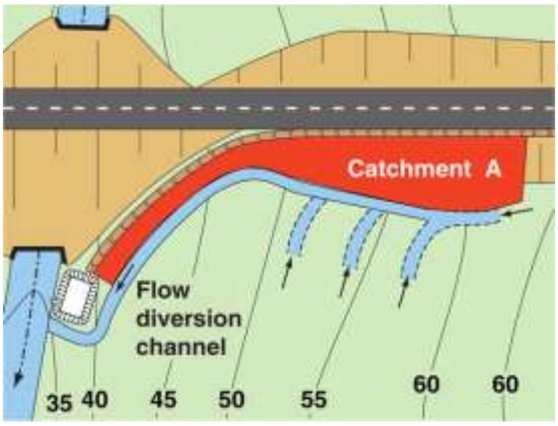
Design option 2B

- Option 2B is based on providing sufficient time to allow the sediment floc to settle at least 600 mm below the spillway crest, but not sufficient time to allow full settlement.
- This design option has an increased risk of sediment re-suspension.
- However, this design option does allow for a greater basin depth and smaller surface area when compared to an Option 1B design.



Type B basin (plan view)

Sizing Type B basins – Design option 1B

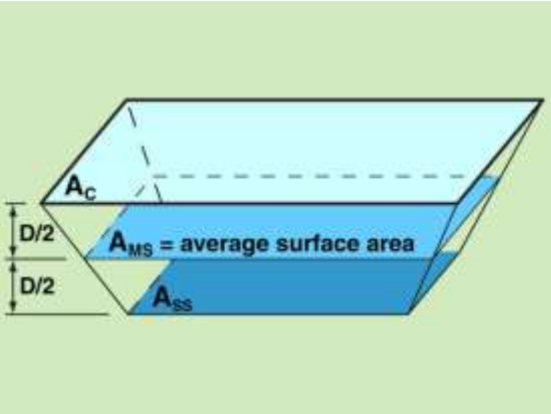


Catchment area

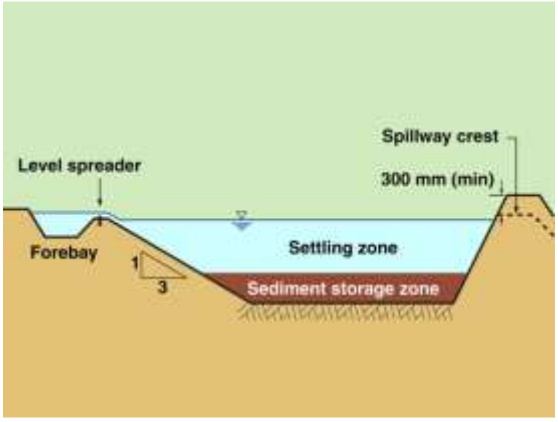
Table B17 – Sediment settlement characteristics for design option 1B

Jar test settlement after 15 min (mm)	50	75	100	150	200	300
Laboratory settlement rate (m/hr)	0.20	0.30	0.40	0.60	0.80	1.20
Factor of safety	1.33	1.33	1.33	1.33	1.33	1.33
Design settlement rate, v_s (m/hr)	0.15	0.23	0.30	0.45	0.60	0.90
Design settlement coefficient, K_S (s/m)	24000	16000	12000	8000	6000	4000
Minimum depth of the settling zone:						
Minimum settling zone depth, D_S (m)	0.5	0.5	0.5	0.68	0.90	1.35
Critical settling zone length before Step 5B begins to dictate the basin size:						
Critical settling zone length (L_C) before Step 5B and Equation B21 begin to dictate the basin size (m)	180	120	90	81	81	81

IECA (2018) Table B17



Average settling pond surface area



Type B basin

Step 1B – Determine the design discharge

- Note: the design discharge (Q) may be governed by state, regional, or local design standards
- If a local standard does not exist, then the recommended design storm is 0.5 times the peak 1 year ARI discharge.

$$Q = 0.5\ Q1 \qquad \text{(B18)}$$

- where: Q1 = peak discharge for the 1 in 1 year ARI design storm, which places the design standard of a Type B basin well below that of a Type A basin.

Step 2B – Determine a design value for the sediment settling coefficient

- The determination of the settling coefficient (K_S) should be based on the results of *Jar Testing* of the anticipated chemically treated sediment floc at the correct temperature.
- Select an appropriate value of 'K_S' from Table B17.
- If Jar settling test results are not available, then choose K_S = 12,000.

Step 3B – Calculate the minimum required average surface area

$$A_S = K_S\ Q \qquad \text{(B19)}$$

where:

A_S = minimum average settling zone surface area [m^2]

K_S = sediment settlement coefficient (refer to Table B17, Step 2B)

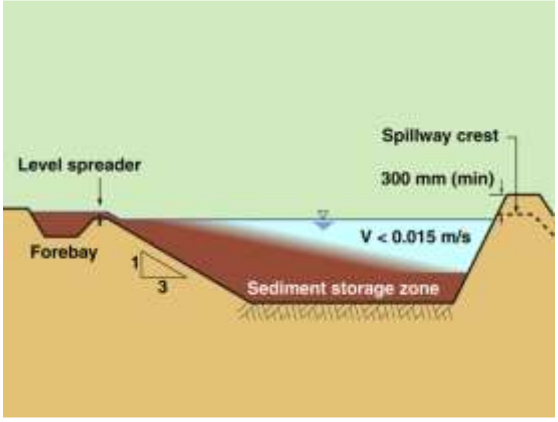
= inverse of the settling velocity of the treated sediment blanket.

Q = design discharge = 0.5 Q1 [m^3/s]

Step 4B – Determine the minimum depth of the settling zone

- If the sediment-flocculant partnership results in a poor settlement rate, such as less than 100 mm in 15 minutes, then the minimum depth of the settling zone (D_S) is governed by the minimum recommended depth of 0.5 m from Table B17.
- This increases the volume of the settling zone compared to those basins that utilise a more effective flocculant.

Sizing Type B basins – Design option 1B



Check for sediment re-suspension

Step 5B – Check for the potential re-suspension of the settled sediment

- To avoid the re-suspension of the settled sediment, the clear water (supernatant) flow velocity (v_C) should not exceed 0.015 m/s (1.5 cm/s).
- This only becomes critical when the length of the settling zone (L_S) exceeds the critical value given by Equation B21.

$$L_{S(critical)} = 0.015 \, . \, K_S \, . \, D_S \quad [m] \qquad (B21)$$

where: L_S = average length of the settling zone

Basin spillway (Qld)

Step 6B – Determine the width of the overflow spillway

- In order to reduce the risk of sediment re-suspension, the overflow spillway should have the maximum possible width.
- However, this may not always be practical.
- Ideally, designers should take all reasonable measures to achieve a spillway crest width just less than the top width of the settling zone.

Table B19 – Typical Type B settling zone and sediment storage depths

Type B basin geometry with sediment storage volume = 30% (V_S):						
Inlet bank slope, 1 in 3	All other bank slopes, 1 in 2		Total depth, D_T = 1.0 m			
Typical basin dimensions based on a length:width ratio of 3:1 at top of the settling zone:						
Settling zone surface area [m²]	**36**	**50**	**100**	**200**	**400**	**800**
Settling zone volume, V_S [m³]	18	29	65	139	288	589
Total basin volume, V_T [m³]	24	37	84	180	374	765
Settling zone depth (D_S) [m]	0.50	0.56	0.65	0.69	0.72	0.74
Ratio D_S/D_T as a percentage	50%	56%	65%	69%	72%	74%
Sediment storage (D_{SS}) [m]	0.50	0.44	0.35	0.31	0.28	0.26
Ratio D_{SS}/D_T as a percentage	50%	44%	35%	31%	28%	26%
Top length of settling zone [m]	12.6	14.7	20.1	27.5	37.7	52.1
Top width of settling zone [m]	4.2	4.9	6.7	9.2	12.6	17.4

* The settling zone surface area represents the 'average' surface area, $A_S = V_S/D_S$

IECA (2018) Table B19

Step 7B – Determine the remaining dimensions of the sediment basin

- Once the volume and dimensions of the settling zone are known, the remaining basin dimensions need to be determined based on the sizing requirements outlined in Table B16.
- Determining the depth of the sediment storage zone can be complex given the basin geometry; however, tables B19 to B21 can be used to estimate the storage depth.

Table B20 – Typical Type B settling zone and sediment storage depths

Type B basin geometry with sediment storage volume = 30% (V_S):						
Inlet bank slope, 1 in 3	All other bank slopes, 1 in 2		Total depth, D_T = 2.0 m			
Typical basin dimensions based on a length:width ratio of 3:1 at top of the settling zone:						
Settling zone surface area [m²]	**150**	**300**	**600**	**1200**	**2400**	**4800**
Settling zone volume, V_S [m³]	154	373	815	1706	3506	7131
Total basin volume, V_T [m³]	200	484	1058	2215	4553	9262
Settling zone depth (D_S) [m]	1.02	1.23	1.35	1.42	1.46	1.48
Ratio D_S/D_T as a percentage	51%	62%	68%	71%	73%	74%
Sediment storage (D_{SS}) [m]	0.98	0.77	0.65	0.58	0.54	0.52
Ratio D_{SS}/D_T as a percentage	49%	38%	32%	29%	27%	26%
Top length of settling zone [m]	25.6	35.3	48.2	66.1	91.1	126
Top width of settling zone [m]	8.5	11.8	16.1	22.0	30.4	42.1

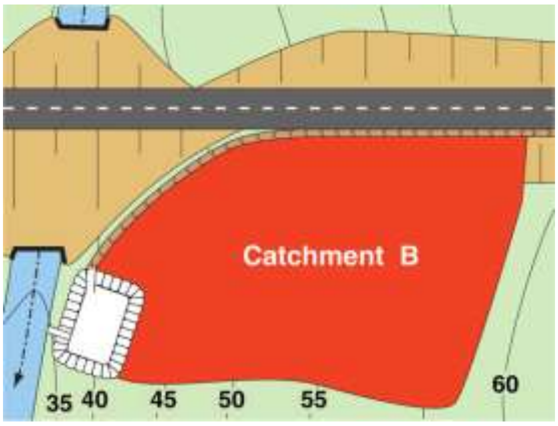
IECA (2018) Table B20

Table B21 – Typical Type B settling zone and sediment storage depths

Type B basin geometry with sediment storage volume = 30% (V_S):						
Inlet bank slope, 1 in 3	All other bank slopes, 1 in 2		Total depth, D_T = 3.0 m			
Typical basin dimensions based on a length:width ratio of 3:1 at top of the settling zone:						
Settling zone surface area [m²]	**300**	**600**	**1200**	**2400**	**4800**	**9600**
Settling zone volume, V_S [m³]	438	1094	2416	5086	10475	21343
Total basin volume, V_T [m³]	569	1421	3138	6605	13605	27720
Settling zone depth (D_S) [m]	1.44	1.81	2.00	2.11	2.18	2.22
Ratio D_S/D_T as a percentage	48%	60%	67%	70%	73%	74%
Sediment storage (D_{SS}) [m]	1.56	1.19	1.00	0.89	0.82	0.78
Ratio D_{SS}/D_T as a percentage	52%	40%	33%	30%	27%	26%
Top length of settling zone [m]	38.2	50.2	68.8	93.9	129	179
Top width of settling zone [m]	12.1	16.7	22.9	31.3	43.1	59.7

IECA (2018) Table B21

Sizing Type B basins – Design Option 2B



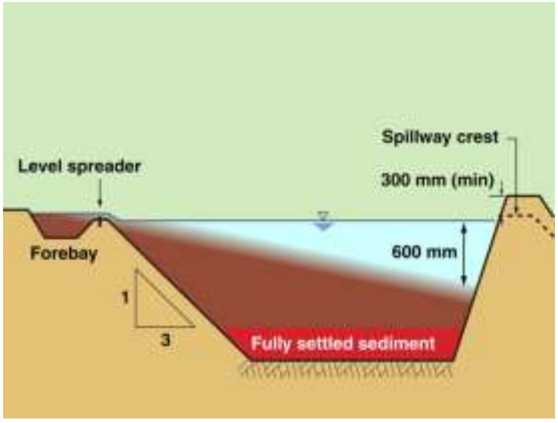
Catchment area

Step 1B – Determine the design discharge

- Note: the design discharge (Q) may be governed by state, regional, or local design standards
- If a local standard does not exist, then the recommended design storm is 0.5 times the peak 1 year ARI discharge.

$$Q = 0.5\ Q1 \qquad \text{(B23)}$$

- where: Q1 = peak discharge for the 1 in 1 year ARI design storm [m^3/s]

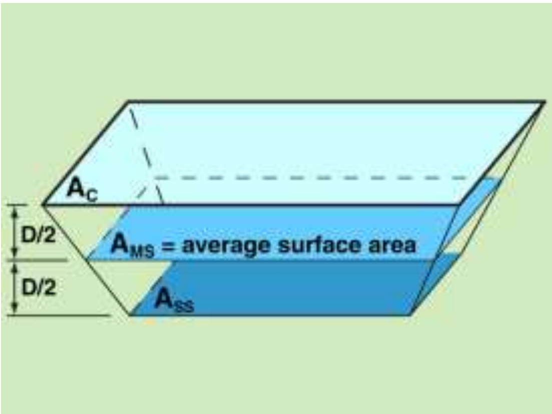


Design option 2B

Step 2B – Nominate the desired settling zone depth, and the floc settling depth

- The settling zone depth (D_S) can be within the range of 0.6 to 2.0 m.
- The greater the depth, the smaller the required surface area (A_S) of the basin, but the volume of the settling zone (V_S) will essentially remain unchanged.
- Increasing this depth will reduce the forward velocity, which increases the time available for the sediment floc to settle the required depth, D_F.

$$D_S \geq D_F \qquad \text{(B25)}$$



Average settling pond surface area

Step 3B – Calculate the average surface area (A_S) of the settling zone

$$A_S = (D_F/D_S)\ K_S\ Q \qquad \text{(B26)}$$

where:

A_S = minimum average settling zone surface area [m^2]

K_S = sediment settlement coefficient (refer to Table B18) = inverse of the settling velocity of the treated sediment blanket

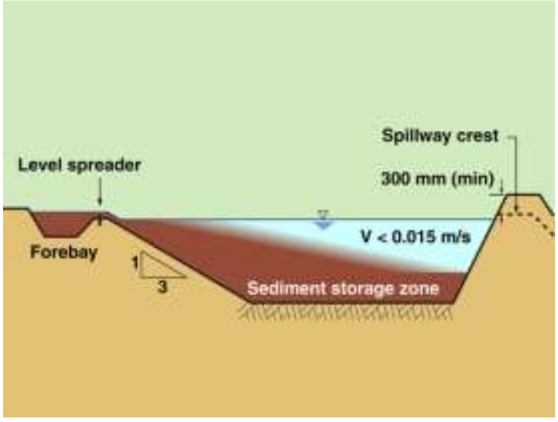
Q = design discharge = 0.5 Q1 [m^3/s]

Table B18 – Sediment settlement characteristics for design option 2B

Jar test settlement after 15 min (mm)	50	75	100	150	200	300
Laboratory settlement rate (m/hr)	0.20	0.30	0.40	0.60	0.80	1.20
Factor of safety	1.33	1.33	1.33	1.33	1.33	1.33
Design settlement rate, v_F (m/hr)	0.15	0.23	0.30	0.45	0.60	0.90
Design settlement coefficient, K_S (s/m)	24000	16000	12000	8000	6000	4000

Sediment settlement characteristics (Table B18, IECA, 2018)

Sizing Type B basins – Design option 2B



Check for sediment re-suspension

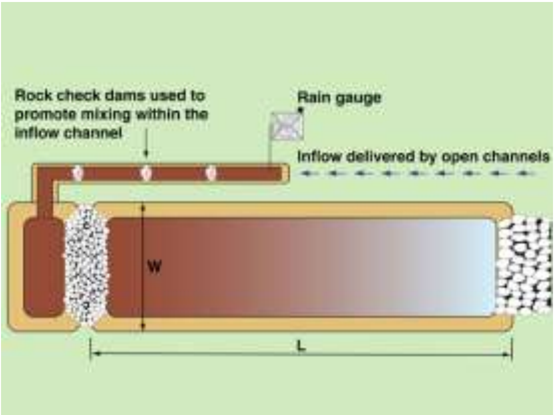
Step 4B – Check for the potential re-suspension of the settled sediment

- To avoid the re-suspension of the settled sediment, the clear water (supernatant) flow velocity (v_C) should not exceed 0.015 m/s (1.5 cm/s).

$$v_C = Q/(D_F \cdot W_{SF}) \ \text{[m/s]} \qquad \text{(B27)}$$

- In order to satisfy Equation B27, the minimum average basin width (W_{SF}) can be determined from Equation B28.

$$W_{SF} = 66.7(Q/D_F) \ \text{[m]} \qquad \text{(B28)}$$



Type B basin (plan view)

Step 5B – Determine the width of the overflow spillway

- In order to reduce the risk of sediment re-suspension, the overflow spillway should have the maximum possible width.
- However, this may not always be practical.
- Ideally, designers should take all reasonable measures to achieve a spillway crest width just less than the top width of the settling zone.

Table B19 – Typical Type B settling zone and sediment storage depths

Type B basin geometry with sediment storage volume = 30% (V_S):
Inlet bank slope, 1 in 3 | All other bank slopes, 1 in 2 | Total depth, D_T = 1.0 m
Typical basin dimensions based on a length:width ratio of 3:1 at top of the settling zone:

Settling zone surface area [m²]	36	50	100	200	400	800
Settling zone volume, V_S [m³]	18	29	65	139	288	589
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Ratio D_S/D_T as a percentage	50%	56%	65%	69%	72%	74%
Sediment storage (D_{SS}) [m]	0.50	0.44	0.35	0.31	0.28	0.26
Ratio D_{SS}/D_T as a percentage	50%	44%	35%	31%	28%	26%
Top length of settling zone [m]	12.6	14.7	20.1	27.5	37.7	52.1
Top width of settling zone [m]	4.2	4.9	6.7	9.2	12.6	17.4

* The settling zone surface area represents the 'average' surface area, $A_S = V_S/D_S$

IECA (2018) Table B19

Step 6B – Determine the remaining dimensions of the sediment basin

- Once the volume and dimensions of the settling zone are known, the remaining basin dimensions need to be determined based on the sizing requirements outlined in Table B16.
- Determining the depth of the sediment storage zone can be complex given the basin geometry; however, tables B19 to B21 can be used to estimate the storage depth.

Table B20 – Typical Type B settling zone and sediment storage depths

Type B basin geometry with sediment storage volume = 30% (V_S):
Inlet bank slope, 1 in 3 | All other bank slopes, 1 in 2 | Total depth, D_T = 2.0 m
Typical basin dimensions based on a length:width ratio of 3:1 at top of the settling zone:

Settling zone surface area [m²]	150	300	600	1200	2400	4800
Settling zone volume, V_S [m³]	154	373	815	1706	3506	7131
Total basin volume, V_T [m³]	200	484	1058	2215	4553	9262
Settling zone depth (D_S) [m]	1.02	1.23	1.35	1.42	1.46	1.48
Ratio D_S/D_T as a percentage	51%	62%	68%	71%	73%	74%
Sediment storage (D_{SS}) [m]	0.98	0.77	0.65	0.58	0.54	0.52
Ratio D_{SS}/D_T as a percentage	49%	38%	32%	29%	27%	26%
Top length of settling zone [m]	25.6	35.3	48.2	66.1	91.1	126
Top width of settling zone [m]	8.5	11.8	16.1	22.0	30.4	42.1

IECA (2018) Table B20

Table B21 – Typical Type B settling zone and sediment storage depths

Type B basin geometry with sediment storage volume = 30% (V_S):
Inlet bank slope, 1 in 3 | All other bank slopes, 1 in 2 | Total depth, D_T = 3.0 m
Typical basin dimensions based on a length:width ratio of 3:1 at top of the settling zone:

Settling zone surface area [m²]	300	600	1200	2400	4800	9600
Settling zone volume, V_S [m³]	438	1094	2418	5066	10475	21343
Total basin volume, V_T [m³]	569	1421	3138	6605	13605	27720
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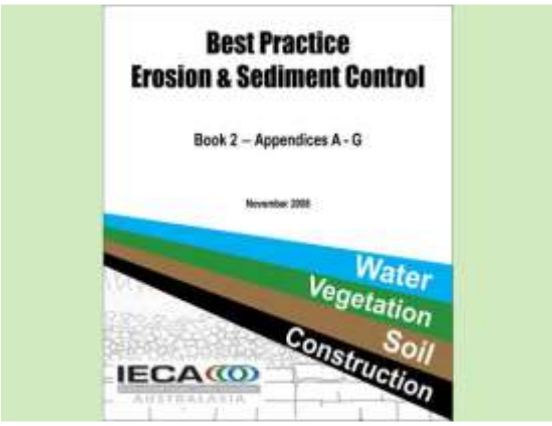
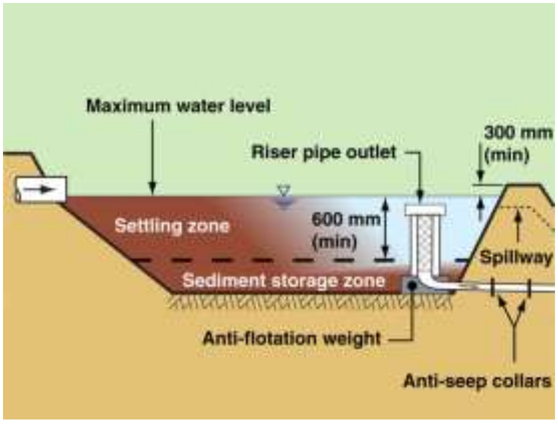
Ratio D_S/D_T as a percentage	48%	60%	67%	70%	73%	74%
Sediment storage (D_{SS}) [m]	1.56	1.19	1.00	0.89	0.82	0.78
Ratio D_{SS}/D_T as a percentage	52%	40%	33%	30%	27%	26%
Top length of settling zone [m]	38.2	50.2	68.8	93.9	129	179
Top width of settling zone [m]	12.1	16.7	22.9	31.3	43.1	59.7

IECA (2018) Table B21

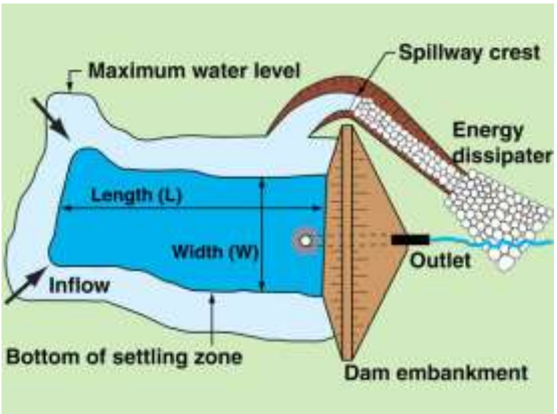
Step 6c: Sizing Type C basins



IECA (2008) – Book 2



Type C basin



Type C basin (plan view)

Introduction

- The requirements for the sizing of sediment basins varies from region to region; therefore, check your local rules.
- The following design steps are based on the recommendations outlined in IECA (Australasia), 2008, and the updated Appendix B (2018).
- Readers should check for further updates to Appendix B.
- Readers will need to refer to Appendix B to obtain the necessary tabulated data.

Average settling zone surface area

- The minimum 'average' surface area of the settling zone (A_S) is given by Equation B30.

$$A_S = K_S H_e Q \qquad \text{(B30)}$$

where:

A_S = average surface area of settling zone = V_S/D_S [m^2]

K_S = sediment settlement coefficient = the inverse of the settling velocity of the 'critical' particle size (Table B22)

H_e = hydraulic efficiency correction factor (Table B23)

Q = design discharge = 0.5 Q1 [m^3/s]

Q1 = peak discharge for the critical storm duration 1 in 1 year ARI event

V_S = volume of the settling zone [m^3]

D_S = depth of the settling zone [m]

- The hydraulic efficiency correction factor depends on flow conditions entering the basin and the basin shape (Table B23).
- The minimum recommended depth of the settling zone (D_S) is 0.6 m.
- The desirable minimum length to width ratio at the mid-elevation of the settling zone is 3:1 (L:W).

Sizing Type C basins: Tables B22 and B23

Table B22 – Sediment settlement coefficient (K_S)

Water temperature (degrees C)	5	10	15	20	25	30
Kinematic viscosity ($m^2/s \times 10^6$)	1.519	1.306	1.139	1.003	0.893	0.800
Critical particle characteristics	**Sediment settlement coefficient (K_S)**					
d = 0.02 mm and s = 2.2	5810	4990	4350	3830	3410	3060
d = 0.02 mm and s = 2.4	4980	4280	3730	3290	2930	2620
d = 0.02 mm and s = 2.6 (default)	4360	3740	3270	2880	2560	2290
d = 0.02 mm and s = 2.8	3870	3330	2900	2560	2280	2040
d = 0.02 mm and s = 3.0	3480	3000	2610	2300	2050	1840
d = 0.02 mm and s = 3.2	3170	2720	2380	2090	1860	1670

Sediment settlement coefficient (Table B22, IECA, 2018)

Relative density (specific gravity) of rock

Rock type	Relative density (s_r)
Sandstone	2.1 to 2.4
Granite	2.5 to 3.1 (commonly 2.6)
Limestone	2.6
Basalt	2.7 to 3.2

Table 2 – Relative density of sediment particles 's_r' as used in Table B22

Table B23 – Hydraulic efficiency correction factor (H_e)

Flow condition within basin	Effective [1] length:width	H_e
Uniform or near-uniform flow conditions across the full width of basin. [2]	1:1	1.2
For basins with concentrated inflow, uniform flow conditions may be achieved through the use of an appropriate inlet chamber arrangement (refer to Step 9).	3:1	1.0
Concentrated inflow (piped or overland flow), primarily at one inflow point, and no inlet chamber to evenly distribute flow across the full width of the basin.	1:1	1.5
	3:1	1.2
	6:1	1.1
	10:1	1.0
Concentrated inflow with two or more separate inflow points, and no inlet chamber to evenly distribute flow across the full width of the basin.	1:1	1.2
	3:1	1.1

Hydraulic efficiency correction factor (Table B23, IECA, 2018)

Sizing Type C basins: Typical dimensions based on a bank slope of 1 in 2

Table B25 – Typical Type C & D settling zone and sediment storage depths

Type C & Type D basin geometry:						
Sediment storage = 50% (V_S)	All bank slopes, 1 in 2			Total depth, D_T = 1.5 m		
Typical basin dimensions based on a length:width ratio of 3:1 at mid-elevation of settling zone:						
Settling zone surface area [m^2]	**80**	**100**	**200**	**400**	**800**	**1600**
Settling zone volume, V_S [m^3]	48	65	158	346	730	1507
Total basin volume, V_T [m^3]	72	97	235	516	1090	2250
Settling zone depth (D_S) [m]	0.60	0.65	0.78	0.86	0.91	0.94
Ratio D_S/D_T as a percentage	39%	43%	52%	58%	61%	63%
Sediment storage (D_{SS}) [m]	0.91	0.85	0.72	0.64	0.59	0.56
Ratio D_{SS}/D_T as a percentage	61%	57%	48%	42%	39%	37%
Mid length of settling zone [m]	15.5	17.3	24.5	34.6	49.0	69.3
Mid width of settling zone [m]	5.2	5.8	8.2	11.5	16.3	23.1

* The settling zone surface area represents the 'average' surface area, $A_S = V_S/D_S$.

Table B26 – Typical Type C & D settling zone and sediment storage depths

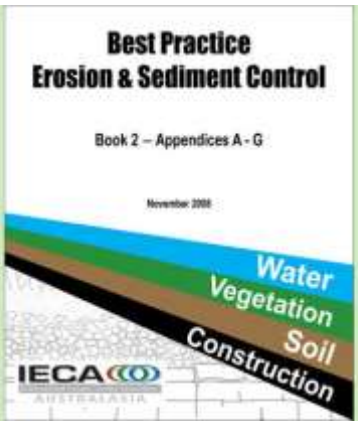
Type C & Type D basin geometry:						
Sediment storage = 50% (V_S)	All bank slopes, 1 in 2			Total depth, D_T = 2.0 m		
Typical basin dimensions based on a length:width ratio of 3:1 at mid-elevation of settling zone:						
Settling zone surface area [m^2]	**150**	**300**	**600**	**1200**	**2400**	**4800**
Settling zone volume, V_S [m^3]	121	304	680	1444	2995	6128
Total basin volume, V_T [m^3]	181	454	1015	2155	4470	9146
Settling zone depth (D_S) [m]	0.81	1.01	1.13	1.20	1.25	1.28
Ratio D_S/D_T as a percentage	40%	51%	56%	60%	62%	64%
Sediment storage (D_{SS}) [m]	1.19	0.99	0.87	0.80	0.75	0.72
Ratio D_{SS}/D_T as a percentage	60%	49%	44%	40%	38%	36%
Mid length of settling zone [m]	21.2	30.0	42.4	60.0	84.9	120
Mid width of settling zone [m]	7.1	10.0	14.1	20.0	28.3	40.0

Table B27 – Typical Type C & D settling zone and sediment storage depths

Type C & Type D basin geometry:						
Sediment storage = 50% (V_S)	All bank slopes, 1 in 2			Total depth, D_T = 3.0 m		
Typical basin dimensions based on a length:width ratio of 3:1 at mid-elevation of settling zone:						
Settling zone surface area [m^2]	**350**	**500**	**1000**	**1500**	**3000**	**6000**
Settling zone volume, V_S [m^3]	433	706	1634	2577	5450	11276
Total basin volume, V_T [m^3]	646	1054	2438	3847	8135	16830
Settling zone depth (D_S) [m]	1.23	1.40	1.63	1.71	1.81	1.88
Ratio D_S/D_T as a percentage	41%	47%	54%	57%	60%	63%
Sediment storage (D_{SS}) [m]	1.77	1.60	1.37	1.29	1.19	1.12
Ratio D_{SS}/D_T as a percentage	59%	53%	46%	43%	40%	37%
Mid length of settling zone [m]	32.4	38.7	54.8	67.1	94.9	134.2
Mid width of settling zone [m]	10.8	12.9	18.3	22.4	31.6	44.7

Step 6d: Sizing Type D basins

Step 6d: Sizing Type D basins



IECA (2008) – Book 2

Table B31 – Typical single storm event volumetric runoff coefficients [1]

Rainfall (mm) [2]	Soil Hydrologic Group (refer to Section A3.1, Appendix A)			
	Group A Sand	Group B Sandy loam	Group C Loamy clay	Group D Clay
10	0.02	0.10	0.09	0.20
20	0.02	0.14	0.27	0.43
30	0.08	0.24	0.42	0.56
40	0.16	0.34	0.52	0.63
50	0.22	0.42	0.58	0.69
60	0.28	0.48	0.63	0.74
70	0.33	0.53	0.67	0.77
80	0.36	0.57	0.70	0.79
90	0.41	0.60	0.73	0.81
100	0.45	0.63	0.75	0.83

Notes: [1] Sourced from Fifield (2001) and Landcom (2004).
[2] Rainfall depth based on the nominated 5-day rainfall depth, $R_{(Y\%,5\text{-day})}$

IECA (2018) Table B31

Table B28 – Recommended equation constants

Recommended application	Y%	K_1	K_2
Basins with design life less than 6 months	75%	12.9	9.9
Basins with a design life greater than 6 months	80%	17.0	11.2
Basins discharging to sensitive receiving waters	85%	23.2	12.6
At the discretion of the regulatory authority	90%	33.5	14.2
At the discretion of the regulatory authority	95%	56.7	14.6

IECA (2018) Table B28

Introduction

- The requirements for the sizing of sediment basins varies from region to region; therefore, check your local rules.
- The following design steps are based on the recommendations outlined in IECA (Australasia), 2008, and the updated Appendix B (2018).
- Readers should check for further updates to Appendix B.
- Readers will need to refer to Appendix B to obtain the necessary tabulated data.

Settling zone volume (V_S)

- The minimum volume of the upper settling zone is defined by Equation B35.

$$V_S = 10 . R_{(Y\%,5\text{-day})} . C_V . A \qquad \text{(B35)}$$

where:

V_S = volume of the settling zone [m^3]

$R_{(Y\%,5\text{-day})}$ = Y%, 5-day rainfall depth [mm]

C_V = volumetric runoff coefficient (refer to Table B31)

A = effective catchment surface area connected to the basin [ha]

Design rainfall depth (R)

- It is highly recommended that actual $R_{(Y\%,5\text{-day})}$ values be determined for each region based on analysis of local rainfall records wherever practicable.

$$R_{(Y\%,5\text{-day})} = K_1 . I_{(1yr,\ 120hr)} + K_2 \qquad \text{(B36)}$$

where:

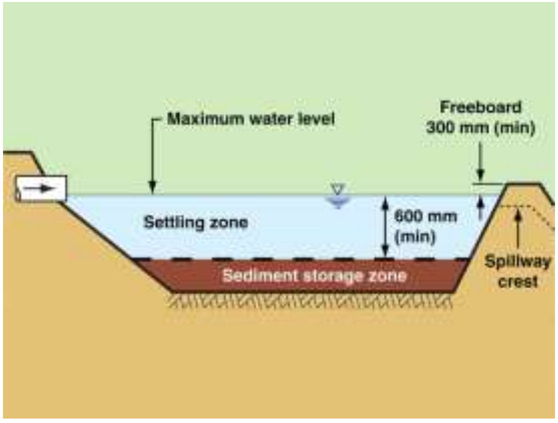
K_1 = constant (Table B28)

K_2 = constant (Table B28)

$I_{(1yr,\ 120hr)}$ = average rainfall intensity for a 1 in 1 year ARI, 120 hr storm [mm/hr]

Step 7: Determine the sediment storage volume

Step 7: Determine the sediment storage volume



Sediment storage in a Type D basin

Introduction

- The minimum recommended volume of the sediment storage zone is defined below in Table B32.
- If less sediment storage volume is provided, then the basin will need to be de-silted more frequently.
- If a greater sediment storage volume is provided, then the cost of ongoing basin de-silting should be reduced.

Table B32 – Sediment storage volume

Basin type	Minimum sediment storage volume
Type A and Type B	30% of settling volume (V_S)
Type C	50% of settling volume
Type D	50% of settling volume

Recommended sediment storage volume (Table B32, IECA, 2018)

Sediment storage depth marker (Qld)

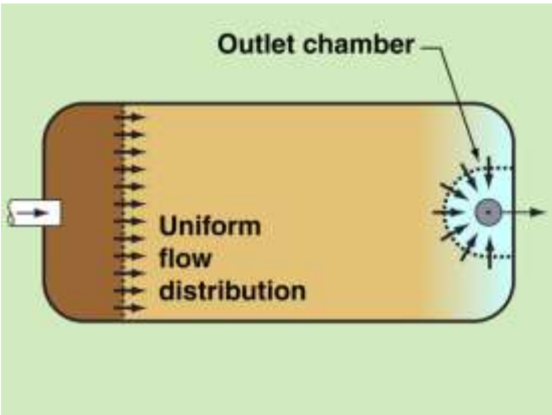
Sediment storage depth marker

- Some type of indicator or marker board is required that can be used to identify when the settled sediment reaches the top of the nominated sediment storage zone.
- Along with flood marker posts (left), a simple timber cross can be installed with the horizontal member set at the top of the sediment storage zone (over time, numbers on the marker post can become difficult to read).

Step 8: Design of flow control baffles

Internal baffle (USA)

Permeable internal baffles (not ideal)



Outlet chamber (plan view)

Internal baffles: Flow redirection

- Internal baffles are used to increase the effective length-to-width ratio of the basin.
- Figure B12 (over page) demonstrates the possible arrangement of flow control baffles for various settling pond layouts.
- If internal baffles are used, then the flow velocity within the settling pond must not exceed the sediment scour velocity as defined in Table B33.
- The crest of these baffles should be set level with, or just below, the crest of the emergency spillway.

Internal baffles: In-line permeable baffles

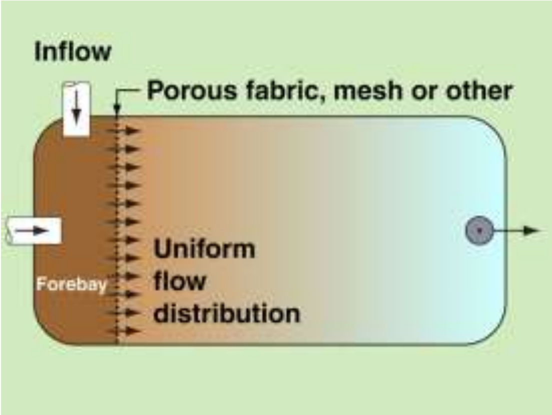
- Internal baffles can also be used to ensure uniform flow through a basin.
- These permeable internal baffles can assist performance of all basin types even in standard basin shapes (Figure B13).
- Use of 75% weave shade cloth, or equivalent open weave fabric, but **not** sediment fence fabric.

In the photo (left) the internal baffles would **not** encourage the uniform flow conditions required to optimise the basin's performance.

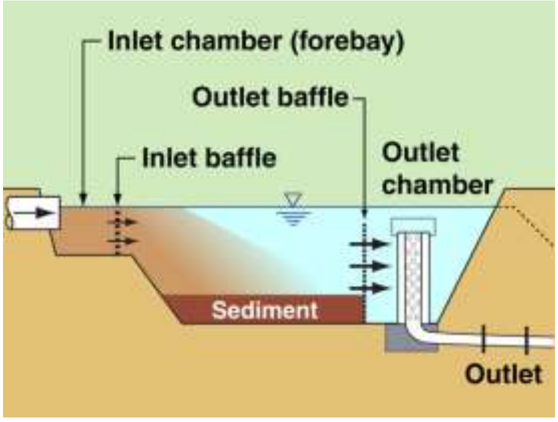
Outlet chambers

- Outlet chambers are used to keep the bulk of the settled sediment away from the low-flow outlet system, particularly riser pipe outlets and floating outlet pipe systems.
- Maintenance of a sediment basin can be expensive if the basin's low-flow outlet system becomes blocked with sediment, or if the outlet is damaged during the de-silting operation.

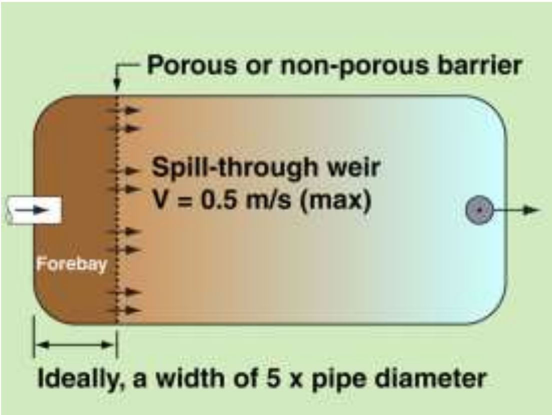
Design of flow control baffles



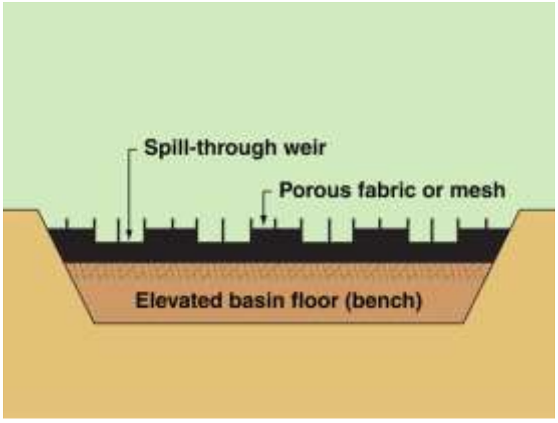
Flow through a shade cloth baffle



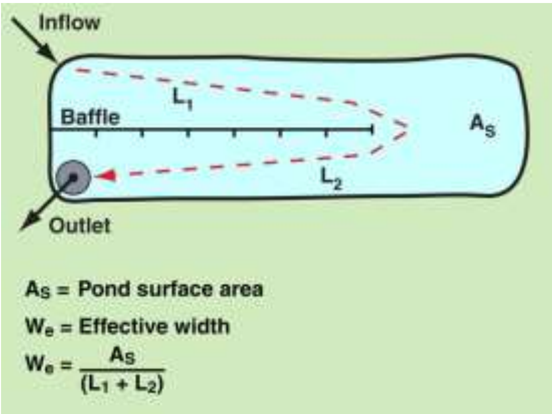
Inlet baffle and outlet chamber



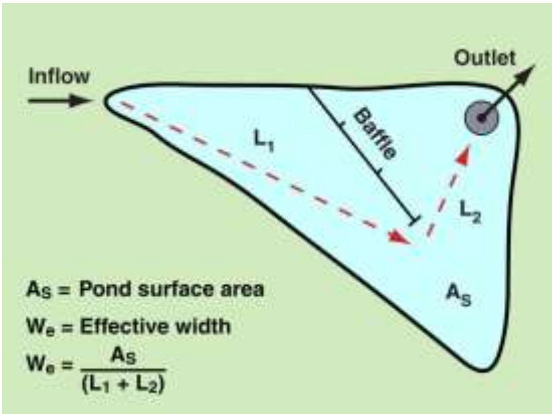
Flow via a sediment fence baffle



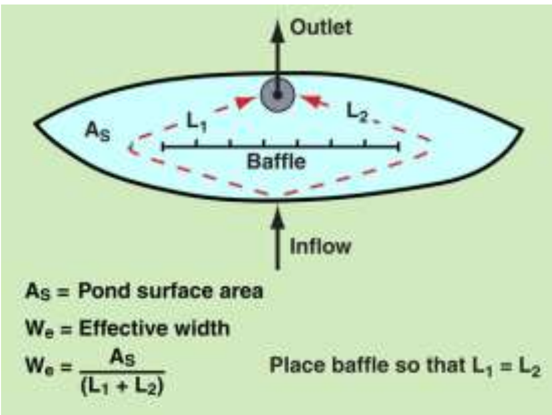
Sediment fence baffle profile



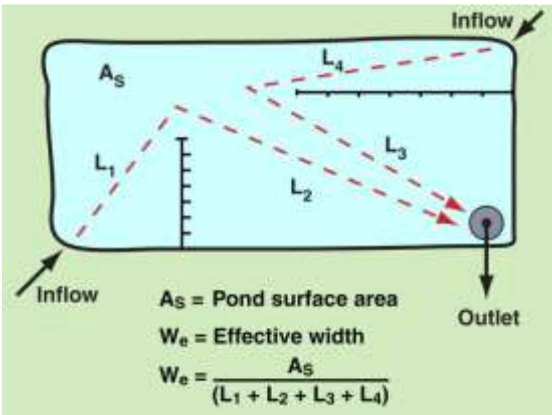
Internal flow control baffle



Internal flow control baffle



Internal flow control baffle



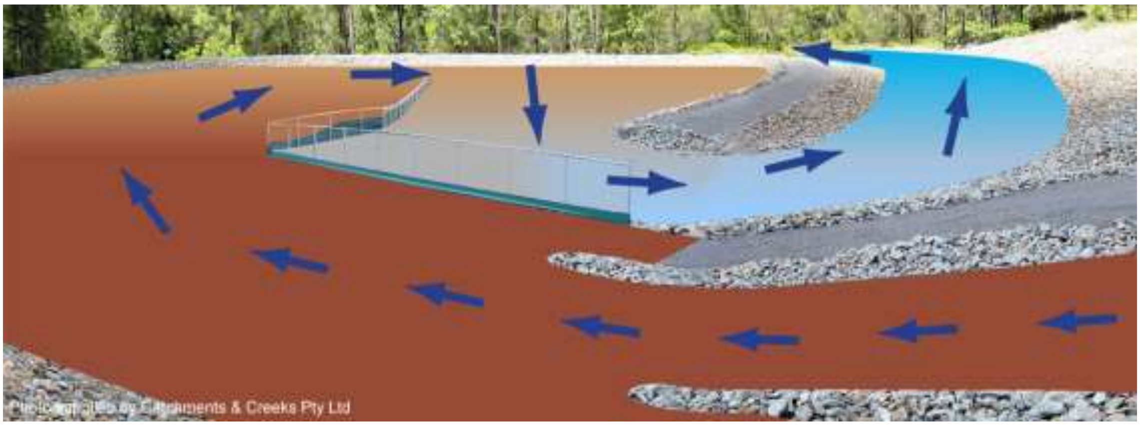
Internal flow control baffle

Design of flow control baffles

Internal flow control baffle on a quarry site (Qld)

Internal flow control baffle on a quarry site (Qld)

Internal flow control baffle on a quarry site (Qld)



Internal flow control baffle on a quarry site (above basin)

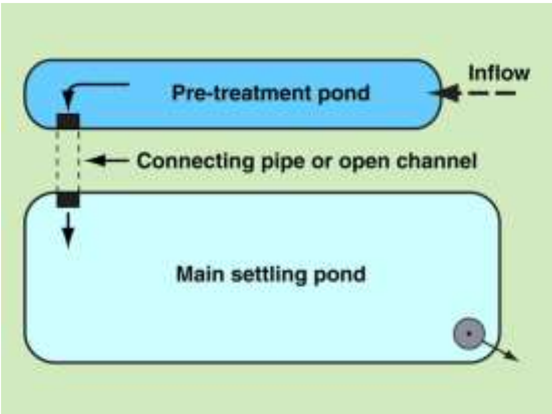
Step 9: Design the basin's inflow system

Step 9: Design the basin's inflow system

Multi-stage sediment basin (Qld)

Mixing zones and energy dissipation

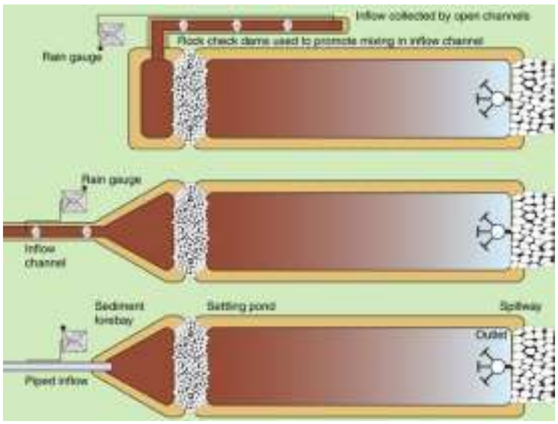
- Energy dissipation is required at the inflow points of all sediment basins in order to reduce the potential for flow short-circuiting to occur within the basin.
- For Type A and B basins it is necessary to establish both energy dissipation and the mixing of any chemicals added to the water.
- Mixing can occur within:
  - the inflow channel or pipe
  - a pre-treatment pond or forebay.



Pre-treatment pond

Pre-treatment ponds

- Where space is available, the construction of an inlet (pre-treatment) pond, or inlet chamber, can significantly reduce the cost of regular de-silting activities for large and/or long-term basins.
- Their size and location should allow de-silting by readily available on-site equipment.



Examples of forebays (Type A basins)

Forebays (flow distribution)

- Forebays can be designed to achieve all or some of the following outcomes:
  - energy dissipation
  - the mixing of coagulants and flocculants
  - the uniform distribution of inflows across the full width of the basin via a level spreader.

Flow entry into a sediment basin

Litter screen at basin inlet (Qld)

Introduction

- Surface flow entering the basin should not cause erosion down the banks of the basin.
- If concentrated surface flow enters the basin (e.g. via a batter chute), then an appropriate channel lining will be required.
- On permanent basins, all inflows may need to pass through a litter screen to trap gross pollutants that would otherwise pass over the spillway.

Geotextile-lined drain (NSW)

Batter chutes

- Temporary batter chutes can be lined with filter cloth.
- If the soil is dispersive, then an impervious channel lining is recommended, such as plastic sheeting.
- Permanent batter chutes (i.e. batter chutes attached to sediment basins that remain as part of the permanent stormwater treatment system) can be lined with concrete.

Constructed drain flowing into a basin

Open channels

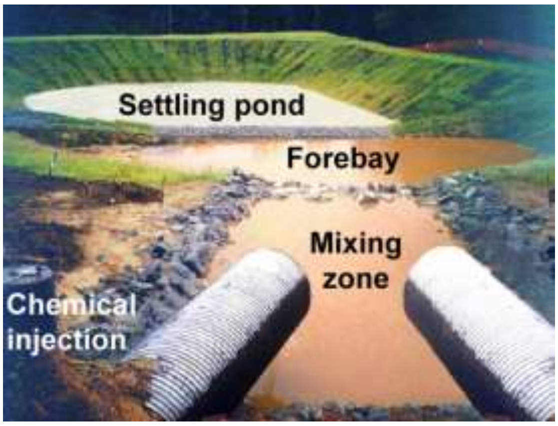
- Open channels leading into a sediment basin should not be allowed to erode and therefore become just another source of sediment flowing into the basin.

Piped inflow (Qld)

Piped inflow

- Piped inflows can cause some additional problems, including:
  - sediment deposition within the stormwater pipe
  - high-velocity inflows
  - increased potential for flow short circuiting to occur within the basin.
- Typical controls for high-velocity piped inflows are discussed at the end of this design step (Design Step 9).

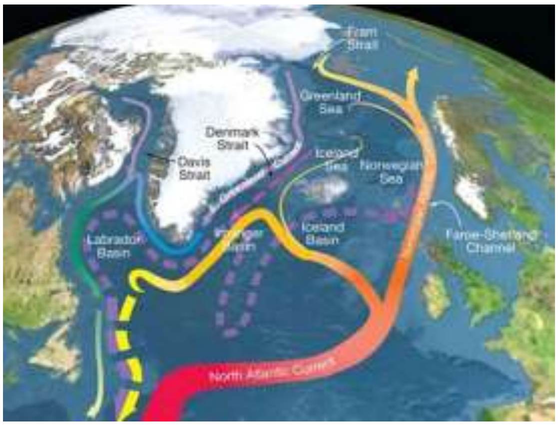
Mixing zones (Type A & B basins)



Three-stage Type C sediment basin

Introduction

- There are several parts of a sediment basin's inlet structure that incorporate specific hydraulic requirements, including:
  - mixing zones
  - forebays
  - pre-treatment ponds
  - flow distribution system aimed at reducing the 'jetting' caused by concentrated inflows.
- Not all of these features will be required on all types of sediment basins.



Poorly mixed ocean currents

Mixing zones

- When chemicals, such as coagulants and flocculants, are injected into sediment-laden inflows, it is essential to achieve the correct amount of **mixing**.
- Good mixing is required to achieve the necessary chemical reactions.
- It is not good enough to simply spread the chemicals over the surface of the pond.
- It can also be detrimental to over-mix the water, which can result in damage to the molecular bonds of some chemicals.

Good milk circulation, but poor mixing

Good mixing vs good circulation

- A study of ocean currents (above, left) shows us that ocean waters of different 'densities' do not like to mix.
- Our oceans are said to be well-circulated, but poorly-mixed—this is because good 'mixing' requires significantly more energy input than does water 'circulation'.
- When milk is poured into coffee or tea, initially it experiences good circulation, but poor mixing (photo left)—good mixing requires the input of energy; i.e. stirring to achieve an even colour.

Pre-treatment mixing zone

Good mixing requires energy and turbulence

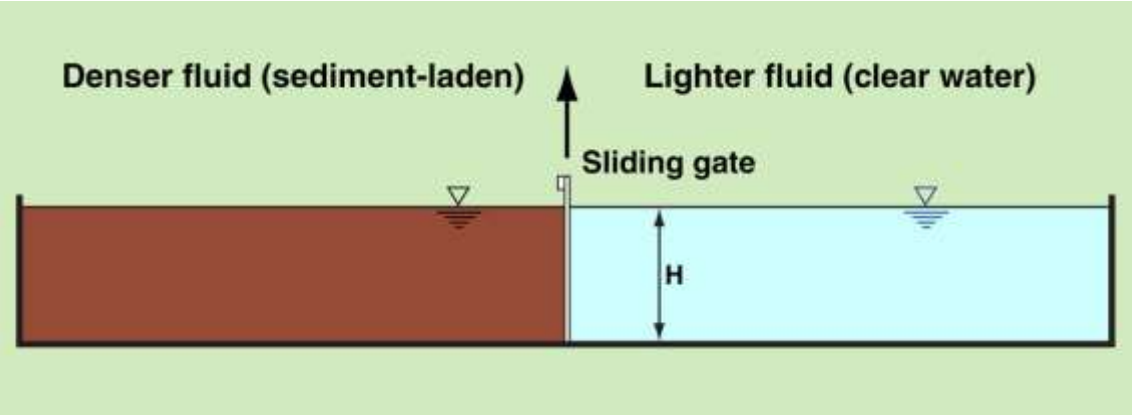
- In basins, chemical mixing can occur:
  - by injecting the chemicals into a pipe 10 times the pipe diameter upstream of the forebay; or for open channels, 10–20 times the hydraulic radius
  - or in a high-turbulence chamber upstream of the forebay (as shown the top image).
- **Warning:** Some chemicals require gentle mixing in order to prevent damage to the chemical's molecular bonds.

Loch exchange test – A demonstration of poor mixing

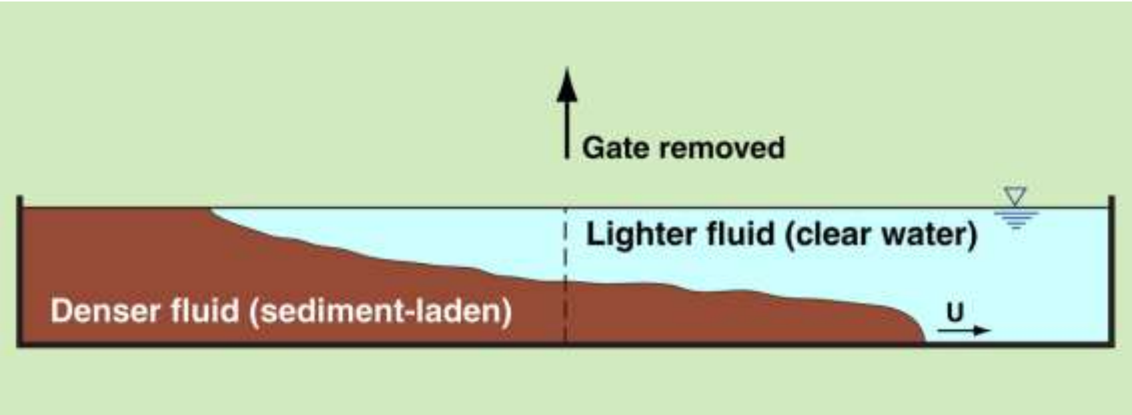
Loch exchange test prior to gate removal

Examples of poor 'mixing' in waters

- A lock exchange test can be used to measure the density difference of two fluids that have only a slight variation in density.
- The test relies on the fact that waters of different densities (caused by salt content, temperature, or sediment) will flow as a 'wedge' over and under each other with minimal mixing.
- A 'salt water wedge' can form when fresh floodwater discharges into an ocean; or when shipping lochs cause the interaction of fresh and saline water when they open.



Lock exchange tank prior to removal of the dividing gate

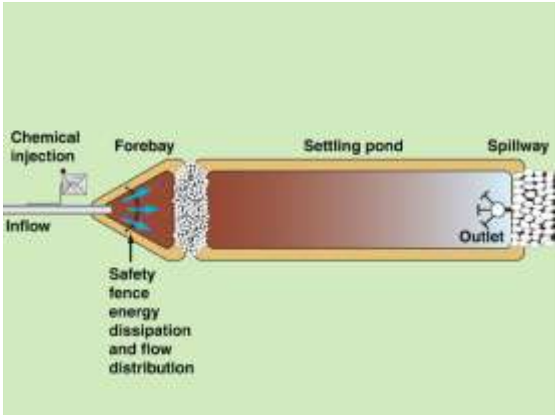


Movement of the two fluids after removal of the dividing gate

Lock exchange tank used to measure density difference of wastewater floc (NSW)

Energy dissipation zone (used as required)

Energy dissipation zone (Qld)



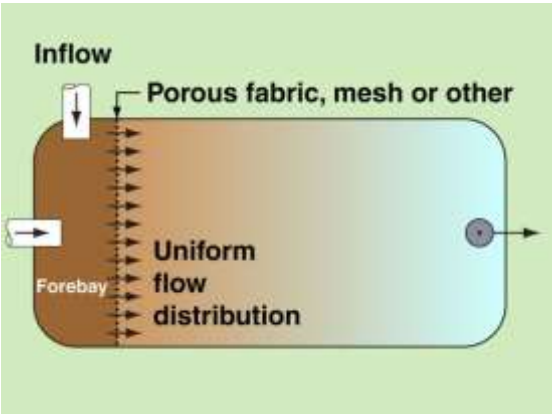
High-velocity piped inflow

Introduction

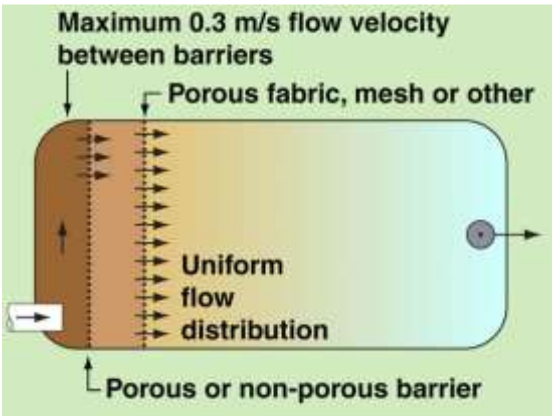
- The need for an energy dissipation zone depends on the approach velocity of the basin's inflow.
- Energy dissipation may be required if:
  - the inflow channel or pipe points directly towards the settling pond, or the level spreader; and
  - flow expansion begins to occur less than 10 times the flow depth from the level spreader; and
  - the approach velocity exceeds 1.5 m/s.

Controlling high inflow velocities

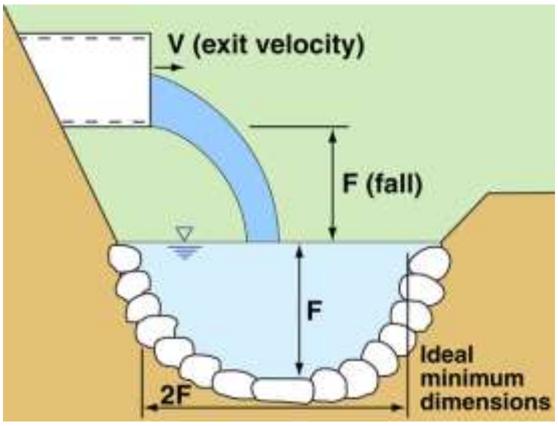
- If flows enter the sediment basin via a stormwater pipe, then special care must be taken to control any 'jetting'.
- One simple solution is to install a well-secured length of the orange PVC safety fencing in a semi-circle around the end of the pipe (left) with a typical radius of 3 times the pipe diameter.
- If there are several points of inflow, then a coarse-weave shade cloth can be used to form an inlet chamber (see below).



Flow distribution inlet chamber



Flow distribution inlet chamber



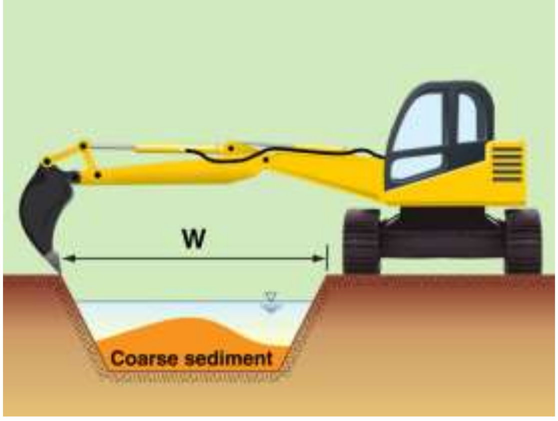
Plunge pool arrangement

Controlling spilling inflows

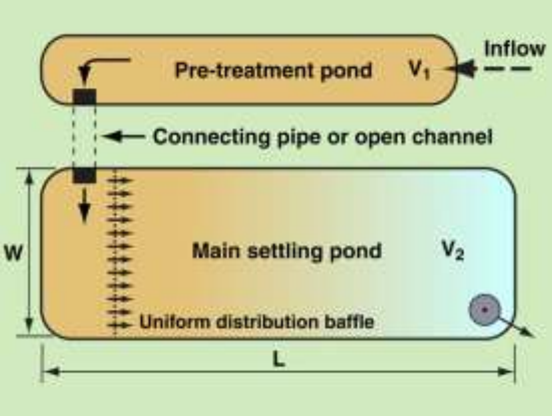
- If the piped inflow spills into the forebay or the inlet chamber, then:
  - the water must spill into a plunge pool (i.e. a deeper zone within the forebay) where the energy can dissipate
  - the plunge pool should have a depth at least equal to the 'fall' height.
- If soil scour within the plunge pool could potentially damage or weaken an earth embankment, then the pool should be lined with 300 mm rock.

Pre-treatment ponds (Type C and D basins, optional)

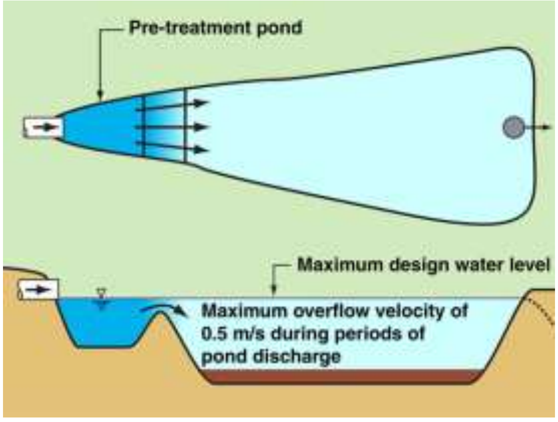
Deposition of coarse sediment (USA)



Width of pre-treatment pond



Pre-treatment pond



Pre-treatment pond and flow distribution

Introduction

- Pre-treatment ponds should be designed:
  - to be easily de-silted using readily available equipment
  - to primarily capture coarse sediments, such as 'sand'
  - to aid in energy dissipation and/or mixing, as required.
- These ponds are most commonly used on Type C and Type D basins, but can be used on any sediment basin to reduce the frequency and cost of basin de-silting.

Typical dimensions

- If the inflow arrives via a pipe, the length of the pre-treatment pond should ideally be at least 13 times the pipe diameter, otherwise try a length of: L > 3W.
- The width of the pond (W) should be based on the reach capabilities of the available equipment; for example:
  - 4–5 m for 5 tonne excavator
  - 5–6 m for 10 tonne excavator
  - 6–7 m for 15 tonne excavator
  - 8–9 m for 20 tonne excavator.

Side ponds

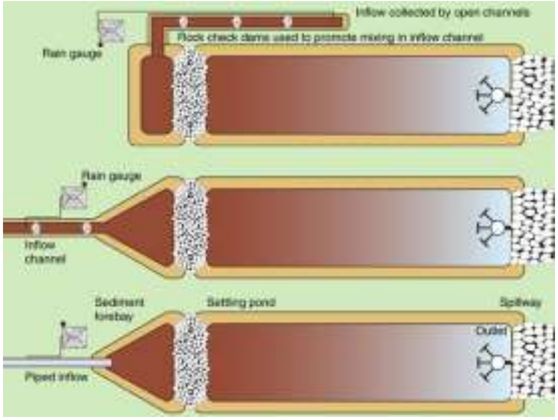
- Side ponds can either be constructed:
  - adjacent to the main settling pond to reduce space; or
  - separated from the main basin to allow better access for de-silting operations.
- Side ponds can be connected to the main basin via a pipe or open channel.

Flow distribution ponds

- Pre-treatment ponds can also be used to help spread the inflow.
- These ponds usually incorporate a level spreader weir, as used in a Type A forebay.

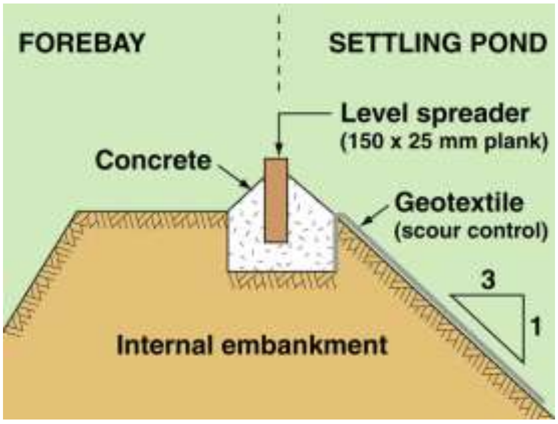
Forebays (critical component of Type A and B basins)

Inflow chute lined with filter cloth (Qld)



Type A basin forebays

Bank slope downstream of level spreader



Standard level spreader crest profile

Introduction

- The forebay can be designed to achieve several different roles, including:
  - energy dissipation (in some cases parts of the forebay may need to be lined with rock)
  - mixing of coagulants or flocculants with the sediment-laden water
  - to remove turbulence from the inflow
  - to distribute the inflow evenly into the main settling pond.

Forebay geometry

- The size varies with the site conditions, but the following rules are recommended:
  - Type A: a volume equal to 10% of the settling pond volume, but typically a width (perpendicular to the level spreader) not exceeding 5 m.
  - Type B: a width of 2–5 m depending on expected sediment inflow.
  - Depth of 2 m (Type A); 1–2 m (Type B).
  - A level spreader length (width) of at least 80% of the settling pond width.

Level spreader bank slope

- The recommended gradient of the internal bank slope immediately downstream of the level spreader is:

  Bank slope: 1:3 (V:H)

- This bank slope is important for the following reasons:
  - reduces flow turbulence
  - removes the ‘dead water’ volume that would otherwise exist below the weir.

Level spreader

- Care should be taken in:
  - providing sufficient design details and drawing of the level spreader
  - supervising the construction of a ‘level’ Level Spreader.
- Using a timber plank (shown left) can allow ‘fine-tuning’ of the weir in order to achieve uniform flow conditions through the basin.

Forebays – What NOT to do!

Inadequate forebay width

Problem 1

- The width of the forebay (parallel to the level spreader) should be at least 80% of the width of the settling pond in order to achieve near-uniform flow conditions within the settling pond.
- The narrow forebay and level spreader width in this example is highly likely to compromise the settling rate and overall performance of the sediment basin.[

Forebay not aligned with basin

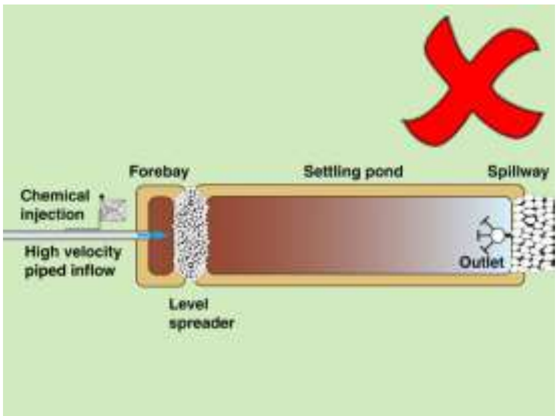
Problem 2

- The forebay and level spreader should align with the direction of the settling pond in order to achieve uniform flow conditions within the settling pond.
- The misalignment of the forebay and level spreader in this example is expected to send a jet of water flow towards the side of the basin, compromising the settling rate and reducing basin efficiency.

Steep downstream face of forebay

Problem 3

- The internal bank slope immediately downstream of the level spreader should have a maximum gradient of 1:3 (V:H).
- Otherwise, consider the viability of a porous (shade cloth) internal barrier.



Piped inflow directed at the level spreader

Problem 4

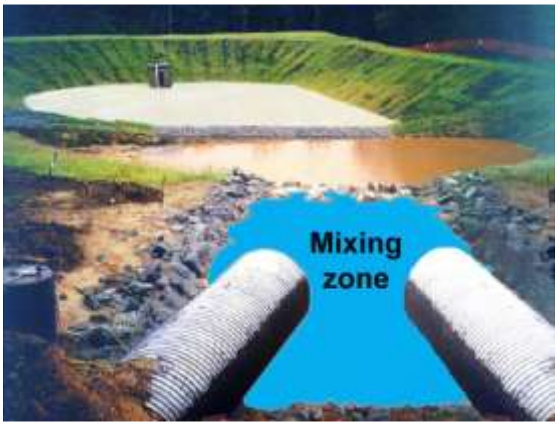
- Piped inflows must not be allowed to cause concentrated inflows (water jetting) to pass over the level spreader.

Summary of sediment basin inlet components (Type C basin shown)

Chemical injection and mixing

Chemical injection and mixing

- Chemical injection is usually required if:
  - the disturbed soil is dispersive
  - the disturbed soil has a moderate (> 10%) to high clay content
  - the disturbed soil contains a very fine clay that would otherwise take an excessive time to settle
  - the local regulator mandates its use.



Energy dissipation and further mixing

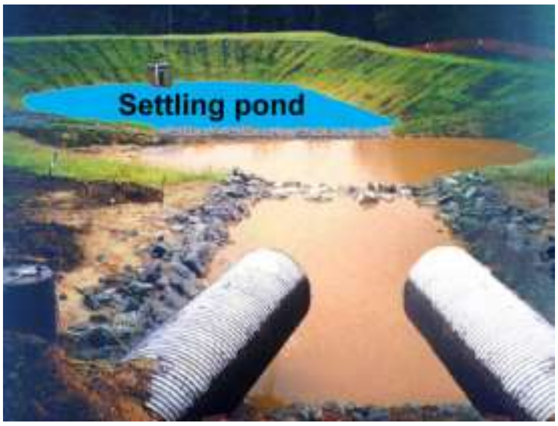
Energy dissipation and mixing

- Energy dissipation may be required if flows enter the basin at high velocity.
- Appropriate mixing may also be required if chemicals are added to the water to improve particle settlement.
- **Warning:** Some chemicals require gentle mixing in order to prevent damage to the chemical's molecular bonds.

Forebay

Forebay

- A forebay, inlet chamber, or pre-treatment pond may be required for the following reasons:
  - energy dissipation
  - chemical mixing
  - capture of coarse sediment
  - aid in achieving uniform flow entry into the main settling pond
  - a mandatory requirement of Type A & B sediment basins.



Settling pond

Settling pond

- Critical dimensions of a settling pond are:
  - All basins length:width ratio > 3:1
  - Type A: volume and surface area
  - Type B: volume and surface area
  - Type C: surface area
  - Type D: volume.
- The pond discharge conditions:
  - Type A: floating decant (skimmer pipe)
  - Type B & D: manual pumping
  - Type C: a filtered, free-draining outlet.

Step 10: Design the primary outlet system

Floating decant arms (NZ)

Type A basins

- Various floating decant (skimmer pipe) designs exist, each with their own flow characteristics.
- Type A basins typically utilise a T-bar decant system developed in Auckland, New Zealand.

Pumped decant system (Qld)

Type B and D basins

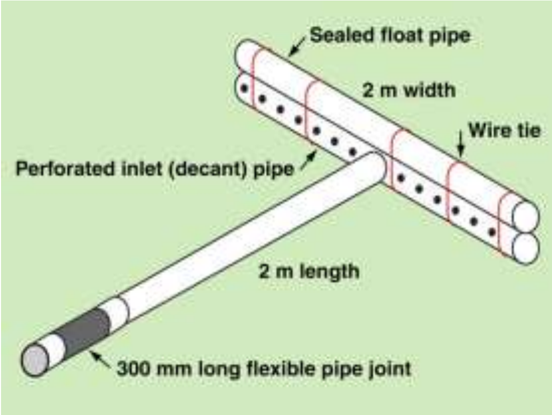
- Pumps can be used to decant Type B and Type D sediment basins.
- The intake hose must be kept away from the settled sediment, such as being suspended via a high-buoyancy float.

Riser pipe outlet (NSW)

Type C basins

- Type C basins utilise a free draining system that is designed to fully decant the basin over a period of 24 to 48 hours.
- The discharge system normally utilises a riser pipe arrangement that uses either aggregate or filter cloth as the final filtration process.
- These outlet systems are <u>not</u> suitable for clayey soils (e.g. soils with a clay content exceeding 10%).

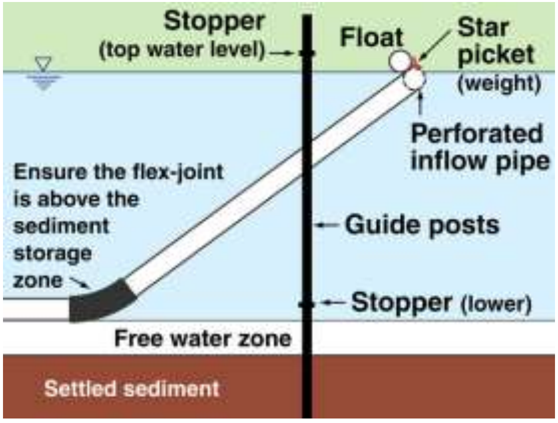
Design the primary outlet system – The Auckland decant system (NZ)



Floating T-bar decant arm

Use

- Used on Type A basins.
- The preferred option for Type C basins.
- Note: the floating decant arms must be allowed to move freely, which means:
  - the lower decant arm will usually require two rubber couplings to provide sufficient flexibility
  - sediment must <u>not</u> be allowed to collect around the decant arm.



Floating decant arm

Design

- The specified decant system can operate at a flow rate of 4.5 L/s per decant arm.
- This decant rate is achieved with six (6) rows of 10 mm diameter holes placed at 60 mm spacings.
- This represents a total of 200 holes along a 2 m long decant arm.
- The decant arm must be appropriately weighted (approx. 4 kg steel star pickets—see bottom photo) to allow it to rest in the water at the correct elevation.

Multiple outlet system (Qld)

Operating range

- A single decant arm must be able to operate through the full depth of the settling zone.
- If two decant arms are required, then the lower T-bar decant operates through the full settling depth; the upper arm operates through the upper 50%.
- If more than two T-bar decant arms are used, then each subsequent arm should be set at least 100 mm above the previous arm (unless otherwise directed).

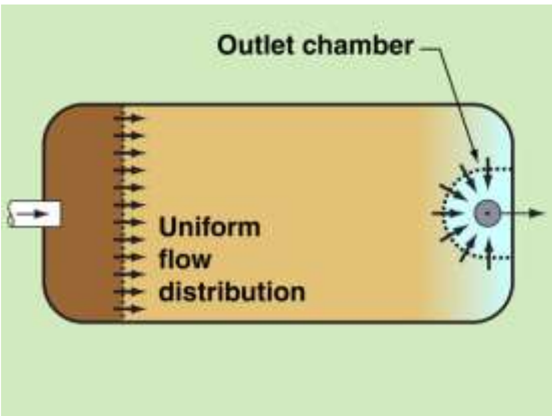
Auckland style floating (T-bar) de-watering system with two steel star-picket weights

Design the primary outlet system – The Faircloth Skimmer (USA)

Faircloth skimmer

Faircloth skimmer (USA)

Skimmer pipe inside an outlet chamber



Outlet chamber (plan view)

Use

- Typically used on Type C basins.
- A floating decant system is the preferred option for Type B basins if automatic free draining of the basin is required.
- Available sizes include: 1.5” (38 mm), 2” (50 mm), 2.5” (64 mm), 3” (75 mm), 4” (100 mm), 5” (127 mm), 6” (150 mm), and 8” (200 mm).
- Manufactured in North Carolina, USA.

Design flow rate

- Decant rates for the current 2022 (Faircloth) design are:
  - 0.57 L/s for the 1.5” (38 mm)
  - 1.08 L/s for the 2” (50 mm)
  - 2.04 L/s for the 2.5” (64 mm)
  - 3.20 L/s for the 3” (75 mm)
  - 6.59 L/s for the 4” (100 mm)
  - 10.8 L/s for the 5” (127 mm)
  - 17.0 L/s for the 6” (150 mm)
  - 32.1 L/s for the 8” (200 mm).

Operation

- The skimmer pipe normally starts to decant the basin as soon as the basin water reaches a depth that can flood the skimmer pipe.
- Type C basins are required to be empty between storm events.
- Consequently, skimmer pipes are normally connected near the base of the outlet pit so the basin can fully drain.
- This means settled sediment must be prevented from collecting around the swinging skimmer arm.

Outlet chambers

- Outlet chambers can be used to keep the bulk of the settled sediment away from the base of the swinging skimmer arms.
- Outlet chambers can also reduce the cost of de-silting operations by preventing sediment blockage of the outlet structure.

Design the primary outlet system – Riser pipe outlets

Riser pipe with aggregate filter (NSW)

Use

- Used on Type C basins
- Used when working in very sandy soils with less than 10% clay content (i.e. sandy to sandy loam soils).
- Can be used for most slaking soils.
- <u>Not</u> used when working in clayey soils or dispersive soils.

Riser pipe with aggregate filter (NSW)

Riser pipe decant system

- The riser pipe normally incorporates either a geotextile or aggregate filter.

Riser pipe with aggregate filter (Qld)

Top of riser pipe

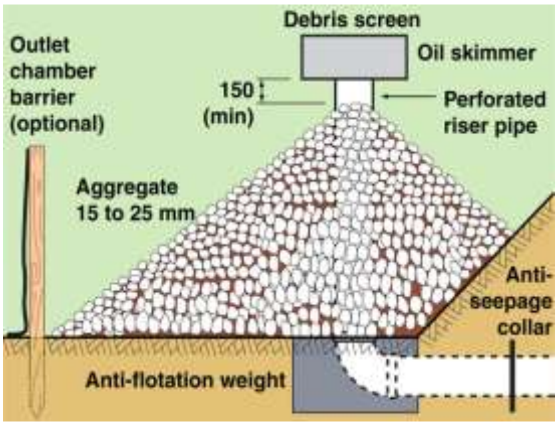
- Different design specifications for the top of a riser pipe may exist in states (and contries), including:
  - weir crest
  - debris screen
  - anti-vortex plate
  - oil skimmer.
- The open top of the riser pipe acts as a medium flow weir, which activates before water reaches the crest of the emergency spillway.

Riser pipe with aggregate filter (USA)

Use of oil skimmers

- Some regions require oil skimmers to be attached to the top of the riser pipe to prevent oils from the construction site entering downstream waterways.
- However, in general, construction sites are not a significant source of oil pollutants.
- If the sediment basin is to be retained as part of the site's permanent stormwater treatment system, then oil skimmers are often considered mandatory items.

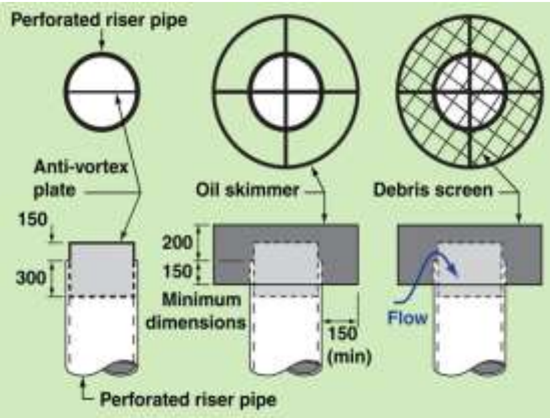
Design the primary outlet system – Riser pipe outlets



Riser pipe with aggregate filter

Aggregate filter

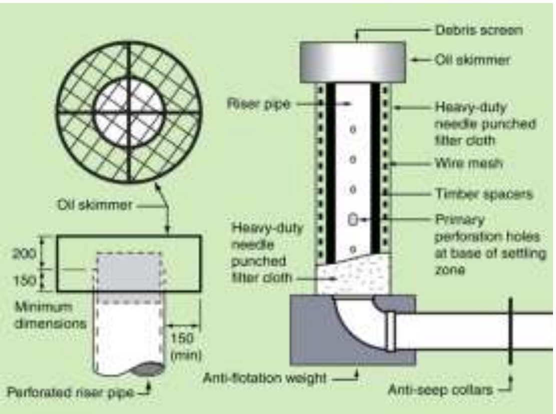
- Aggregate filters are best used on long-term structures because the 'filtration' process relies on the partial sand blockage of the aggregate, which takes several storm events to work properly.
- The recommended maximum surface area (A_O, mm^2) of all decant holes based on the decant holes being spaced evenly up the riser pipe, and an initial blockage factor of 1.0, is given in Table 4 (over page).



Top-of-pipe assembly

Top-of-pipe hydraulics

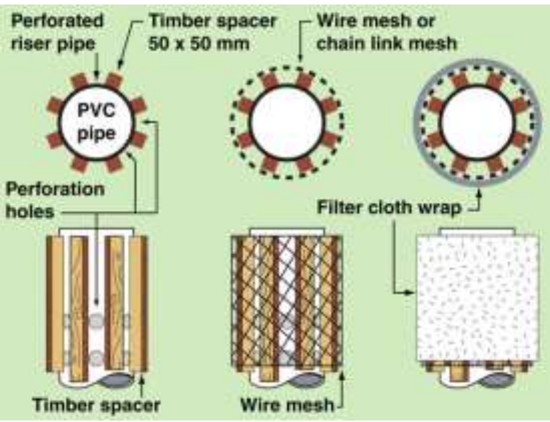
- Anti-vortex plates are used to reduce the risk of a 'bathtub type' vortex forming when flows spill into the open top of the riser pipe.
- Vortex control is necessary (along with debris screens) to prevent floating debris (e.g. organic storm debris) from being drawn (sucked) into the riser pipe.



Riser pipe with fabric filter

Fabric filter

- Geotextile fabric (filter cloth) filters are best used on short-term construction sites because the 'filtration' process starts working from the first storm event.
- Filter cloth equivalent to bidim A44 or A64 is recommended.
- The adoption of a fabric filter does not mean that such a decant system can be used in clayey soil construction sites.



Riser pipe with fabric filter

Assembly of fabric filter

- It is essential for the filter cloth to be:
  - separated from the riser pipe (i.e. an air gap is needed) so that the full surface area of the filter cloth is operational, and not just the sections of filter cloth covering the perforation holes in the riser pipe
  - placed around the outside surface of the riser pipe to allow easy replacement during basin de-silting and maintenance.

Design the primary outlet system – Understanding material usage

Filter cloth (a non woven fabric)

Filter cloth

- The things to know about filter cloth are:
  - it will <u>not</u> slow down the flow of water—unless it becomes blocked with sediment
  - it will <u>not</u> filter clay particles from the water, it will only capture silts and sands—if the water approaches with a brown colour, then it will pass through the cloth with the same brown colour.

If you want to <u>slow</u> the flow (in order to pond water and aid sediment capture), then use a woven cloth (e.g. sediment fence fabric).

Aggregate

Aggregate (not gravel!)

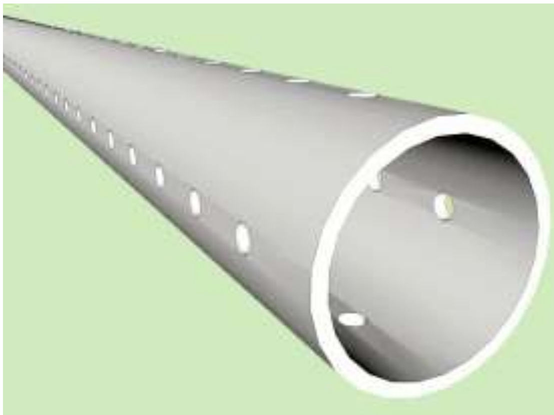
- In 90% of cases, aggregate will <u>not</u> act as a 'filter'.
- Aggregate is normally used:
  - as a means of slowing the flow rate so that a sediment basin will drain at the required decant rate
  - as a means of separating filter cloth from a slotted or perforated PVC pipe.

Aggregate is different from 'gravel' because it is more uniform in size, and contains very few 'fines'.

Ag-pipe (Agricultural pipe)

Ag-pipe (Agi-pipe or corrugated pipe)

- The things to know about Ag-pipe are:
  - the pipe is flexible
  - the drainage holes are located within the 'inner' ring, which means the outer ring can be used to prevent aggregate or filter cloth from coming into direct contact with the drainage holes
  - filter cloth can be wrapped directly around the pipe (an aggregate layer is not required), but only for short-term usage, such as in sediment basins.

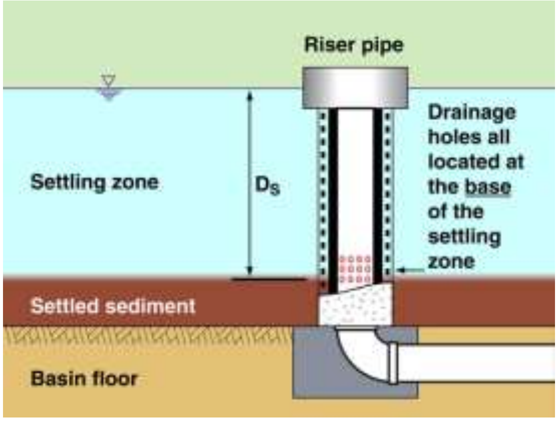


Perforated pipe

Slotted or perforated PVC pipe

- The things to know about slotted or perforated PVC pipe are:
  - the pipe is solid and has minimal flex
  - the smooth inner surface allows better drainage than Ag-pipe when placed at a low gradient (i.e. bed slope)
  - filter cloth should <u>**not**</u> be wrapped directly around a slotted or perforated pipe, because only the cloth directly covering the holes will allow flow, which means the filter cloth will quickly block with sediment and stop flowing.

Design the emergency spillway – Riser pipe decant hydraulics



Holes drilled at one elevation

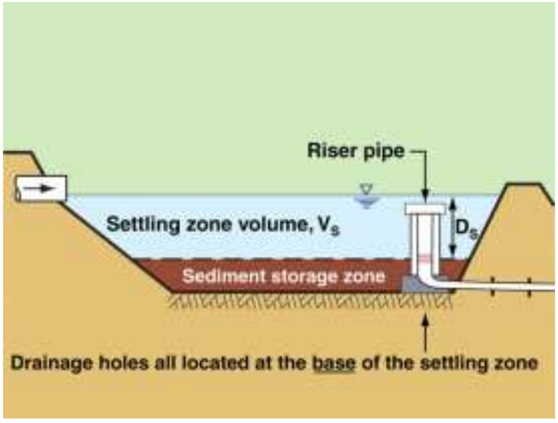
Design flow rate

- Discharge rate for a single orifice, or localised group of drainage holes is:

$$Q = BF \, . \, C_d \, . \, A_O \, (2g \, . \, H)^{1/2}$$

where:

- Q = discharge [m^3/s]
- BF = blockage factor (see below)
- C_d = orifice coefficient = 0.6
- A_O = total area of drainage holes [m^2]
- g = acceleration due to gravity = 9.81
- H = hydraulic head ($H_{MAX} = D_S$) [m]



Holes drilled at one elevation

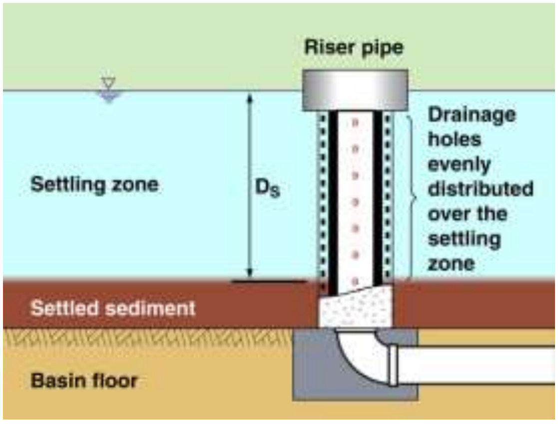
Time to decant a specific volume

- The time it would take for a riser pipe with drainage holes at the base of the settling zone to decant is given by:

$$T = [\, V_S \,] \,/\, [1800 \, BF \, C_d \, A_O \, (2.g.D_S)^{1/2} \,]$$

where:

- T = time to decant full volume [hours]
- V_S = volume of the settling zone [m^3]
- BF = blockage factor (see below)
- C_d = orifice coefficient = 0.6
- A_O = total area of drainage holes [m^2]
- D_S = depth of the settling zone [m]



Holes drilled at multiple elevations

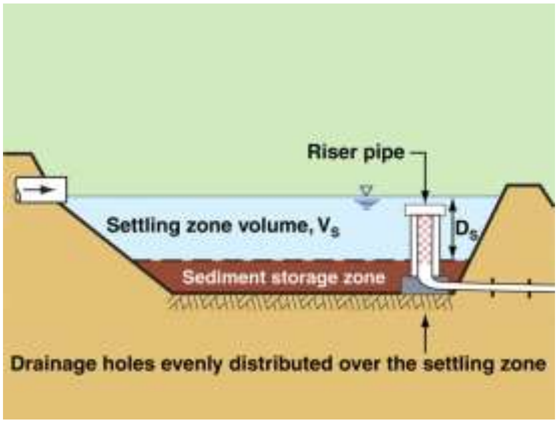
Design flow rate

- If the drainage holes are evenly spaced up the riser pipe, then the discharge rate for the riser pipe is given by:

$$Q = 0.67 \, BF \, . \, C_d \, . \, A_O \, (2g \, . \, H)^{1/2}$$

where:

- Q = discharge [m^3/s]
- BF = blockage factor (see below)
- C_d = orifice coefficient = 0.6
- A_O = total area of drainage holes [m^2]
- g = acceleration due to gravity = 9.81
- H = hydraulic head ($H_{MAX} = D_S$) [m]



Holes drilled at multiple elevations

Time to decant a specific volume

- The time it would take for a riser pipe with evenly distributed drainage holes to drain a specific volume is given by:

$$T = [\, V_S \,] \,/\, [1200 \, BF \, C_d \, A_O \, (2.g.D_S)^{1/2} \,]$$

Recommended blockage factors:

- BF = 1.0 (for filter cloth wrap, new)
- BF = 1.0 (for aggregate filter, new)
- BF = 0.5 (for sand filter, new)
- BF = 0.1 (for filter cloth wrap, old)
- BF = 0.5 (for aggregate filter, old)
- BF = 0.2 (for sand filter, old).

Table 3 – Maximum surface area (mm²) of all decant holes based on the decant holes being located near the base of the decant volume, and an initial blockage factor of 1.0 [1]

Depth D_S (m)	Required decant volume (m³)							
	100	**200**	**500**	**1000**	**2000**	**5000**	**10000**	**20000**
0.6	1124	2249	5622	11,244	22,489	56,222	112,445	224,890
0.8	974	1948	4869	9738	19,476	48,690	97,380	194,760
1.0	871	1742	4355	8710	17,420	43,550	87,099	174,199
1.2	795	1590	3976	7951	15,902	39,755	79,511	159,021
1.4	736	1472	3681	7361	14,722	36,806	73,612	147,225
1.6	689	1377	3443	6886	13,772	34,429	68,858	137,716
1.8	649	1298	3246	6492	12,984	32,460	64,920	129,840
2.0	616	1232	3079	6159	12,318	30,794	61,589	123,177
2.2	587	1174	2936	5872	11,744	29,361	58,722	117,445
2.4	562	1124	2811	5622	11,244	28,111	56,222	112,445
2.6	540	1080	2701	5402	10,803	27,008	54,017	108,034
2.8	521	1041	2603	5205	10,410	26,026	52,052	104,104
3.0	503	1006	2514	5029	10,057	25,143	50,287	100,574
4.0	435	871	2177	4355	8710	21,775	43,550	87,099
5.0	390	779	1948	3895	7790	19,476	38,952	77,904

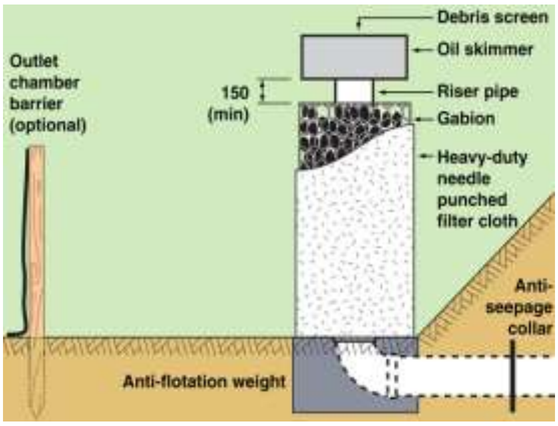
[1] This analysis adopts a minimum decant time of 24 hours when the sediment basin is new and the blockage factor is 1.0. If the initial blockage factor is less than 1.0 at the start of operation of the sediment basin, then the required surface area of the drainage holes may be determined by dividing the tabulated values by the revised blockage factor.

Table 4 – Maximum surface area (mm²) of all decant holes based on the decant holes being spaced evenly up the riser pipe, and an initial blockage factor of 1.0 [1]

Depth D_S (m)	Required decant volume (m³)							
	100	**200**	**500**	**1000**	**2000**	**5000**	**10000**	**20000**
0.6	1687	3373	8433	16,867	33,733	84,334	168,667	337,335
0.8	1461	2921	7304	14,607	29,214	73,035	146,070	292,140
1.0	1306	2613	6532	13,065	26,130	65,325	130,649	261,298
1.2	1193	2385	5963	11,927	23,853	59,633	119,266	238,532
1.4	1104	2208	5521	11,042	22,084	55,209	110,419	220,837
1.6	1033	2066	5164	10,329	20,657	51,644	103,287	206,575
1.8	974	1948	4869	9738	19,476	48,690	97,380	194,760
2.0	924	1848	4619	9238	18,477	46,191	92,383	184,766
2.2	881	1762	4404	8808	17,617	44,042	88,084	176,167
2.4	843	1687	4217	8433	16,867	42,167	84,334	168,667
2.6	810	1621	4051	8103	16,205	40,513	81,025	162,050
2.8	781	1562	3904	7808	15,616	39,039	78,078	156,156
3.0	754	1509	3772	7543	15,086	37,715	75,430	150,861
4.0	653	1306	3266	6532	13,065	32,662	65,325	130,649
5.0	584	1169	2921	5843	11,686	29,214	58,428	116,856

[1] This analysis adopts a minimum decant time of 24 hours when the sediment basin is new and the blockage factor is 1.0. If the initial blockage factor is less than 1.0 at the start of operation of the sediment basin, then the required surface area of the drainage holes may be determined by dividing the tabulated values by the revised blockage factor.

Design the primary outlet system – Riser pipe with gabion surround



Riser pipe with fabric filter

Gabion riser wrapped in filter cloth

Gabion riser without fabric wrap

Type C basin with riser pipe outlet

Use

- Used on Type C basins.
- Used when working in very sandy soils with less than 10% clay content (sandy loam).
- Can be used for most slaking soils.
- Not used when working in clayey soils (> 10% clay content) or dispersive soils.
- If suitably arranged, the gabion can replace the need for an anti-flotation weight.

Filtration system

- It is a mistake to assume that the rocks will provide any form of sediment filtration.
- Final filtration is provided by filter fabric (filter cloth) that is wrapped around the outside of the gabion.
- Wrapping filter cloth around the outside of the gabion allows the filter cloth to be easily replaced during maintenance.
- Type C basins utilise a free draining system that is designed to fully decant the basin over a period of 24 hours (new) to 48 hours (in need of maintenance).

Filtration system

- Wrapping filter cloth around the riser pipe means the filter cloth cannot be easily replaced once the cloth becomes blocked with sediment.
- If a smooth-wall riser pipe is used (e.g. a PVC pipe) then the filter cloth must NOT be wrapped directly around the pipe—a 'spaced' must be used, such as aggregate, or timber spacers covered with wire mesh.

Required orifice decant flow area

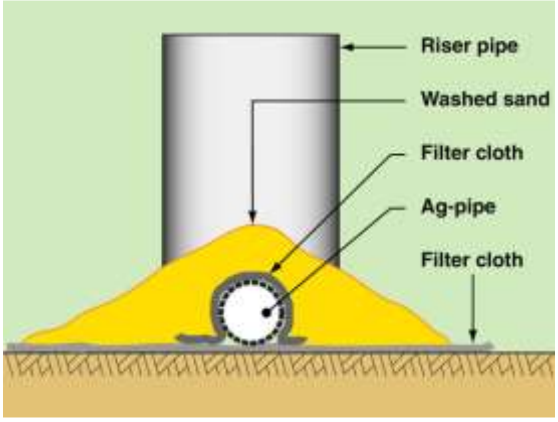
- The maximum total flow area of the drainage holes in the riser pipe is:

$$A_O = [\, V_S \,] \,/\, [1200\ BF\ C_d\ T\ (2.g.D_S)^{1/2}\ ]$$

where:

A_O = total area of drainage holes [m^2]
V_S = volume of the settling zone [m^3]
BF = blockage factor = 1.0 (as new)
C_d = orifice coefficient = 0.6
T = time to decant = 24 hours
g = acceleration due to gravity = 9.81
D_S = depth of the settling zone [m]

Design the primary outlet system – Riser pipe with Ag-pipe drainage



Riser pipe with Ag-pipe drainage

Riser pipe with Ag-pipe drainage (Qld)

Sand filter surrounded by sed-fence (USA)

Perforated HDPE Ag-pipe

Use

- This type of drainage system is suitable for:
  - the drainage of wide, shallow, Type C basins
  - good settling, sandy soils with less than 10% clay content
  - non-dispersive soils.
- In general, this type of decant system is not favoured because it decants from the floor of the basin where, in theory, the sediment storage zone exists.

Aggregate and geotextile filter outlets

- The drainage 'fingers' can be constructed from Ag-pipe (shown left), or perforated PVC pipe (shown over the page).
- The critical design feature is the design flow rate, which must be slow enough to allow the full drainage of the basin over a minimum period of 24 hours.
- The recommended maximum surface area (A_O, mm^2) of all decant holes based near the base of the settling zone, and an initial blockage factor of 1.0, is given in Table 3 (previous pages).

Design flow rate

- It is difficult to be accurate with these calculations—on-site calibration is recommended, which may require reducing the total length of the pipe.
- Orifice equation for perforated pipe:

$$Q\ (m^3/s) = BF \cdot C_d\ (L \cdot A_L) \cdot (2g \cdot H)^{1/2}$$

BF = blockage factor (BF = 0.5 for sand)
C_d = coefficient = 0.6
L = length of pipe [m]
A_L = area of drainage holes [m^2/m]
H = hydraulic head [m]

Typical surface area of drainage holes per metre length of pipe

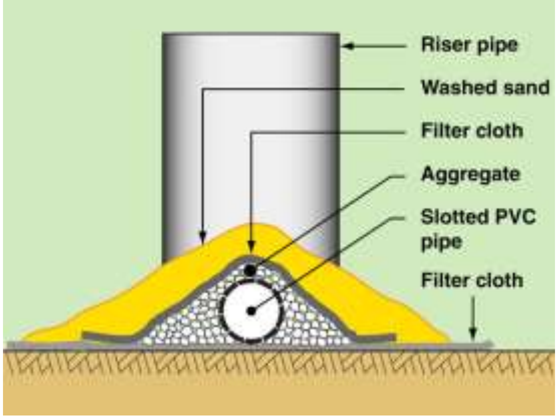
AS2439.1 Class 400, HDPE flex pipe

- Vinidex, Draincoil, Class 400, diameter = 50–100 mm, A_O = 0.00150 m^2/m
- Poly Pipe, DrainFlex, Class 400, diameter = 65–160 mm, A_O = 0.00150 m^2/m

Also:

- Drainflow (65 mm) = 0.00556 m^2/m
- Drainflow (110 mm) = 0.00767 m^2/m
- Drainflow (160 mm) = 0.00918 m^2/m

Design the primary outlet system – AS2439.1 Class 1000 slotted PVC pipe



Slotted PVC pipe drainage system

Sand filter under construction (USA)

Sediment basin converted to OSD (USA)

Perforated PVC pipe

Use

- This type of drainage system is suitable for:
  - the drainage of wide, shallow, Type C basins
  - good settling, sandy soils with less than 10% clay content
  - non-dispersive soils.
- This type of decant system is sometimes favoured (but still rare) when the sediment basin is intended to be converted to a permanent stormwater treatment pond.

Design flow rate

- Discharge flow rate can be based on the orifice equation:

$$Q = BF \cdot C_d \cdot A_O \, (2g \cdot H)^{1/2}$$

where:

Q = discharge [m^3/s]
BF = blockage factor (BF = 0.5 as new)
C_d = orifice coefficient = 0.6
A_O = total area of drainage holes [m^2]
g = acceleration due to gravity = 9.81
H = hydraulic head ($H_{MAX} = D_S$) [m]

Conversion of a sediment basin to part of the permanent stormwater system

- In most cases the low-flow decant system will need to be removed and totally reconstructed at the end of the construction phase in order to comply with the site's stormwater requirements.
- The benefit gained by having such a sand filter outlet system is the retention of the primary drainage network.
- 'OSD' means 'on-site detention', in the example (left), the low-flow outlet system is a sand filter over a perforated pipe.

Time to decant a specific volume

- The time it would take for such a discharge system to drain a specific volume is given by:

$$T = [\,V\,] \,/\, [1800 \; BF \; C_d \; A_O \; (2.g.D)^{1/2}\,]$$

where:

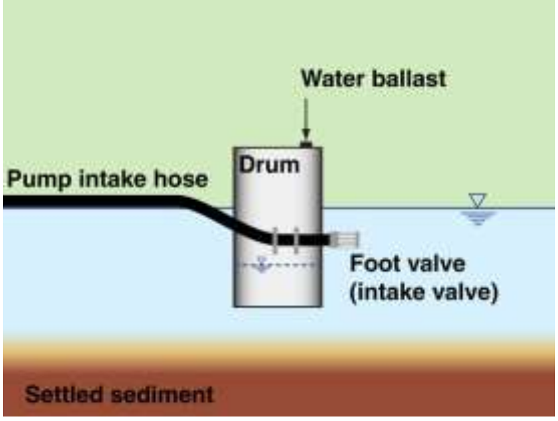
T = time to decant the volume [hours]
V = specified decant volume [m^3]
BF = blockage factor
C_d = orifice coefficient = 0.6
A_O = total area of drainage holes [m^2]
D = depth at maximum water level [m]

Design the primary outlet system – Pumped decant system

Pumped decant system (Qld)

Use

- Manual pumping is often used to de-water Type B and Type D sediment basins.



Floating intake system

Preventing the extraction or re-suspension of settled sediment

- The pump's intake pipe (foot valve) must remain suspended above the settled sediment.
- Besides attaching the foot valve to a floating drum (left), the foot valve can also be placed inside a larger diameter PVC pipe that rests on the basin's bank, and:
  - is sealed at the base; and
  - has intake holes drilled only along the top of the pipe.

Intake pipe resting on the basin floor

Poor practice

- The intake pipe must <u>not</u> rest on the muddy floor of the sediment basin, otherwise settled sediment will be drawn into the intake pipe.

Intake pipe resting on the basin floor

Poor practice

- The de-watering pump should not be used to de-water and de-silt the basin in one process!

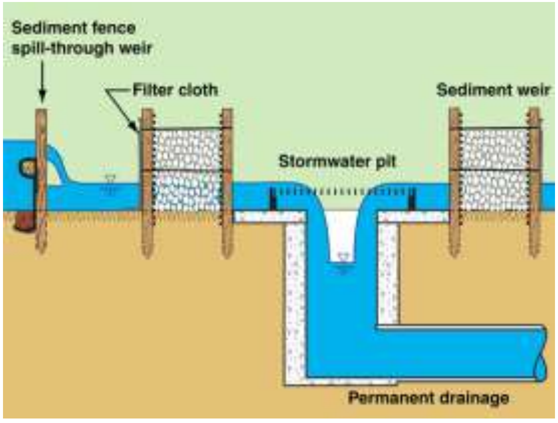
(Two decant pumps can be seen in this photo. One in the foreground, which has its intake pipe resting on the floor of the sediment basin: and one placed on the central 'level spreader', which is currently not in operation.)

Design the primary outlet system – Type-2 sediment trap drainage

Sediment weir outlet system (USA)

Use of sediment weir outlet systems

- Type 2 (IECA, 2008) outlet systems can consist of:
  - sediment weirs
  - rock filter dams.
- Type 2 sediment traps are used on small site disturbances where a Type 1 sediment trap is not required.
- This type of decant system can be used when an existing, or recently constructed park, is temporarily used as a sediment basin.



Sediment weir outlet system

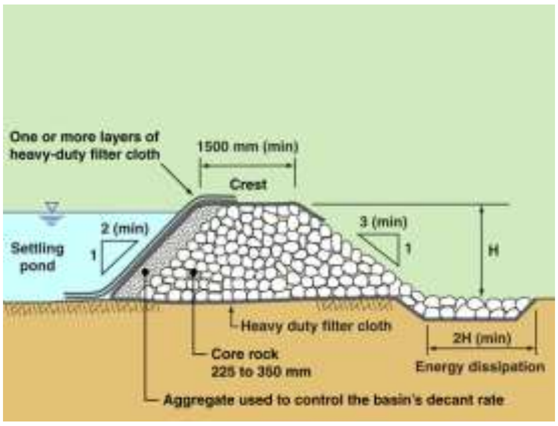
Design of sediment weir outlet systems

- This type of outlet structure is normally used on Type 2 sediment traps, but the outlet can be adapted to Type 1 status if the pond surface area and decant rate satisfy the Type 1 specifications discussed earlier in this chapter.
- Filter cloth must be wrapped around the outside of the sediment weir.
- Unfortunately the photo (above) does not show filter cloth wrapped around the outside of the sediment weir.

Rock filter dam outlet system (USA)

Use of rock filter dam outlet systems

- Rock filter dam outlet structures are commonly utilised as Type 2 (IECA, 2008) sediment traps on road construction projects.
- IECA (Australasia) 2008 specification of Type 2 sediment trap requirements are:
  - catchment area < 2500 m^2
  - estimated soil loss rate > 75 t/ha/yr.



Rock filter dam outlet system

Design of rock filter dam outlet systems

- The critical design parameter is the surface area of the settling pond, which needs to be maximised.
- The use of filter cloth as the primary 'filter' is the preferred construction technique.
- Aggregate filters are normally used on long-term (extractive industry) projects.
- If the disturbed soil has a clay content greater than 10%, then consider covering the aggregate with sand instead of filter cloth, because filter cloth will not capture the clay-sized particles.

Step 11: Design the emergency spillway

Overtopped spillway (NSW)

Rock handling

Sediment basin spillway (USA)

Introduction

- The 'emergency spillway' is an essential safety feature of all sediment basins.
- Flows don't just pass over the spillway only during severe storms, in fact, depending on the type of sediment basin, flows could pass over the spillway on average four times a year.
- Just because flows are passing over the spillway does not mean that the basin no longer needs to perform its role as a major sediment trap—the basin should continue to capture silt and sand-sized particles.

Sizing rock for placement on spillways

- This design step includes providing information on:
  - the Manning's 'n' roughness of rock-lined surfaces
  - typical properties of rock
  - thickness of rock placement
  - minimum dimensions, freeboard and safety factor
  - sizing rock for sediment basin spillways
  - rock sizing table.

Hydraulic analysis

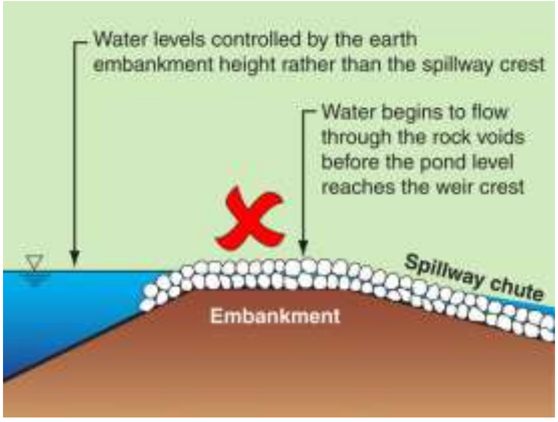
- A detailed description of the hydraulic analysis of complex spillway conditions can be found in Chapter 2 of this field guide.
- The information provided in Chapter 2 is aimed at designers who have previous training in hydraulic analysis.

Design the emergency spillway – Problems to be avoided

Spillway aimed at neighbour's fence

Spillway location problems

- Before discussing the various issues associated with the design of spillways, it would be appropriate to first highlight those design issues that in the **past** have resulted in on-site problems.
- The first issue is 'spillway location'.
- The location of the spillway can influence the hydraulic efficiency of the settling pond, and, the potential impact the spillway's operation could have on neighbouring properties.



Flow leakage through rock voids

Water flow through spillway rocks

- In a Type A and B sediment basins, both the maximum pond volume and surface area are important design parameters.
- If pond water is allowed to escape by passing through the open voids of the rock lining, then the settling pond's maximum water level (and storage capacity) will be determined by the elevation of the earth embankment and not the theoretical top of the spillway weir.

Spillway with insufficient freeboard

Embankment freeboard

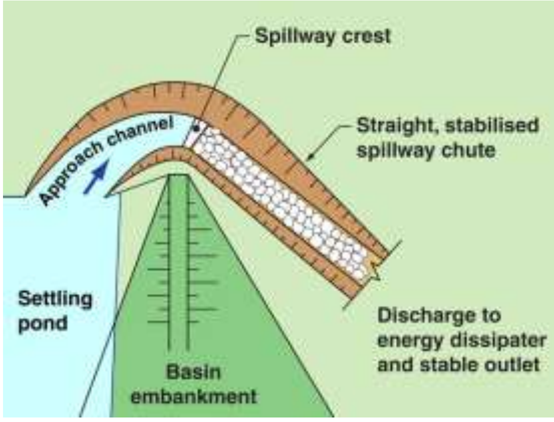
- Far too often sediment basin spillways are found to have insufficient freeboard between the crest of the spillway and the top of the adjacent earth embankment.
- Such occurrences greatly increase the risk of embankment failure during major storms.

Spillway chute with not side walls!

Spillway chute with insufficient cross-sectional profile (depth)

- The spillway chute must have sufficient 'depth' to fully contain the flow, and any splash resulting from the turbulent flow.
- Other problems can include:
  - insufficient rock size
  - using round rock instead of angular rock
  - rock placement stops at the base of the embankment, i.e. there is no energy dissipater formed at the base of the spillway.

Design the emergency spillway – Spillway location



Spillway formed within virgin soil

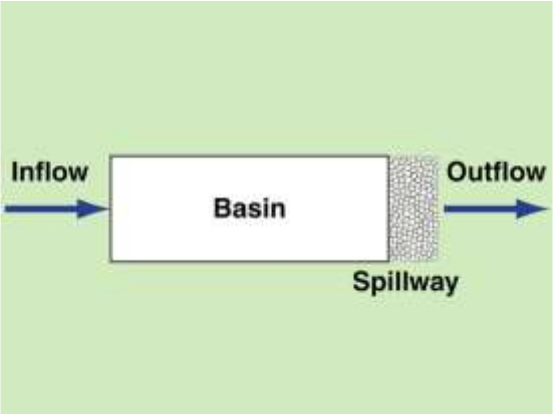
Preferred location of spillways

- Ideally, the emergency spillway should be constructed in virgin soil (i.e. adjacent to any constructed fill embankments).
- This hopefully means the spillway will be cut into a stable soil.
- However, if the in-situ subsoils are dispersive, then it may be more appropriate to construct the spillway over the fill embankment (assuming this fill embankment has been formed from a well-compacted, treated soil).

Embankment spillway (USA)

Combined spillway and low-flow outlet

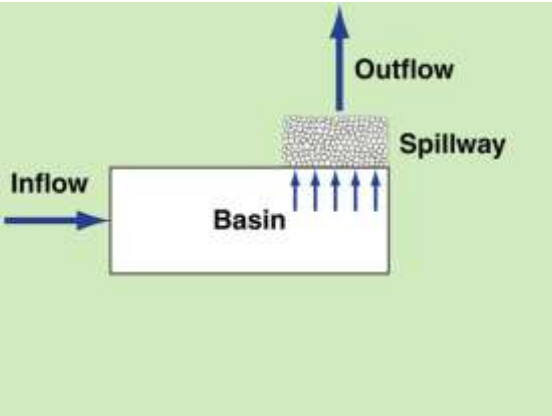
- The spillway and riser pipe outlet can discharge into the same energy dissipation area (rock pad).



Spillway aligned with the settling pond

Spillway located at the end of the settling pond

- Ideally, the spillway should be located as close to the downstream end of the settling pond as is practical.
- The design standards and discharge water quality performance of several sediment basin types (e.g. Type A and B basins) rely on this type of spillway alignment.

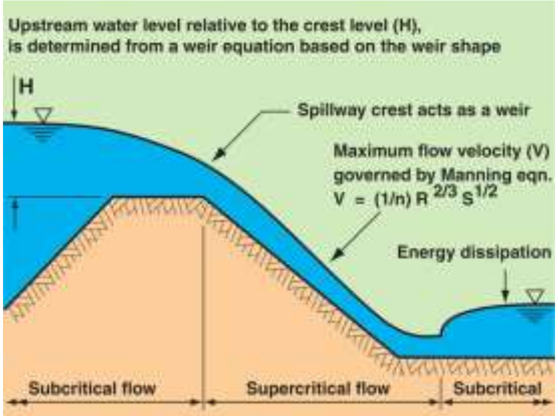


Side-flow spillway

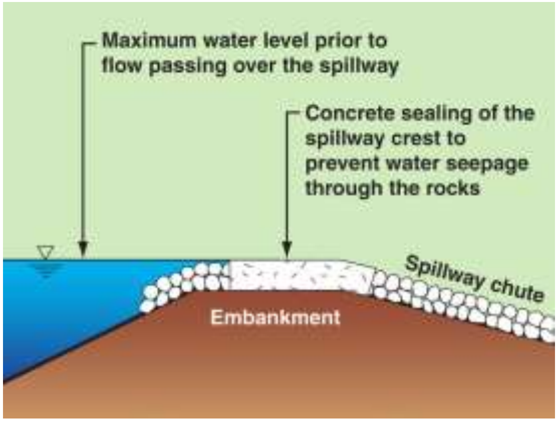
Side-flow spillways

- If the spillway is cut into in-situ (virgin) soil, then it will usually be necessary for the spillway to be located along the side of the settling pond.
- Such a layout may be optimum for stability of the spillway (if the subsoils are stable), but this layout can reduce the hydraulic efficiency of the settling pond during major storms.
- Designers are required to determine the best spillway layout on a case-by-case basis.

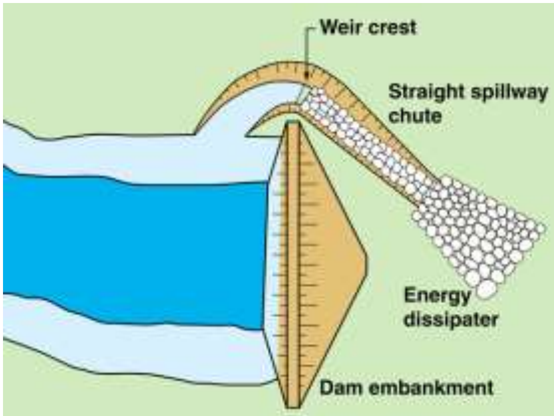
Design the emergency spillway – Hydraulics



Spillway hydraulics



Sealing the spillway crest



Basin spillway cut into virgin soil

Energy dissipation pond (Qld)

Hydraulic design

- Basin spillways are hydraulic structures that need to be designed for a specified design storm using standard hydraulic equations.
- The hydraulic design can be broken down into three components:
  - design of the spillway crest using an appropriate weir equation
  - scour protection down the face of the spillway based on Manning's equation
  - design of the energy dissipater.

Design of spillway crest

- Flow conditions at the spillway crest may be determined using an appropriate weir equation.
- It is important to ensure that the maximum potential water level within the basin at peak discharge will be fully contained by the basin's embankments.
- The concrete sealing of the spillway crest is necessary to maximise basin volume and surface area during the design storm.

Design of the spillway chute

- Determination of rock size on the spillway is based on either the maximum unit flow rate (q) or the maximum flow velocity (V) down the spillway.
- The upstream segment of the spillway's inflow channel can be curved (i.e. that section upstream of the weir crest).
- Once the spillway moves past the weir crest (i.e. where the flow is supercritical) the spillway **must** be straight.

Design of the energy dissipater

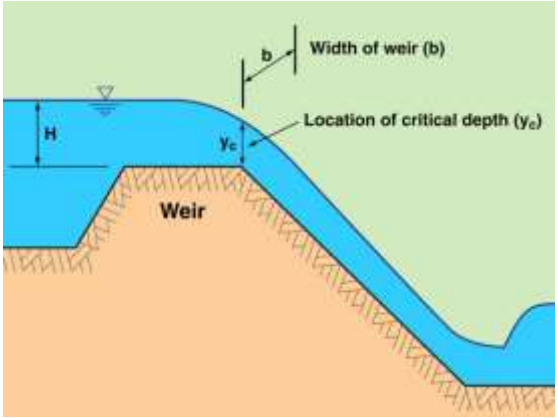
- An appropriate energy dissipater is required at the base of the spillway.
- The design of the energy dissipater **must** be assessed on a case-by-case basis.
- There are very few design procedures available for sizing rock placed within an energy dissipater.
- The best advice is to separate out the largest rocks from those delivered to the site, and use these larger rocks to line the energy dissipation basin.

Design the emergency spillway – Weir flow equations

Crest of a rock-lined chute (USA)

Introduction

- The face of the chute should have a constant cross-section from the crest of the chute to the energy dissipater.
- Flow conditions at the crest of a chute will be governed by a specific weir equation.
- The weir equation defines the relationship between the flow rate (Q), the width of the weir crest (b), and the **upstream** water depth relative to the weir crest (H).
- Manning's equation can then be used to calculate the max velocity down the chute.



Weir flow head (H) and width (b)

Weir equation for a wide flat weir crest

- For wide chutes (say, b > 5H), it can be acceptable to adopt a simple rectangular weir equation, such as:

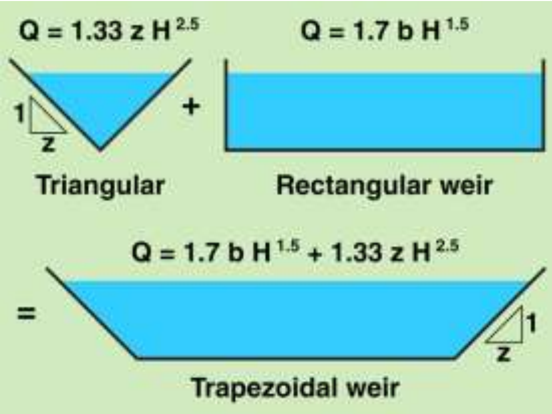
$$Q = 1.7\ b\ H^{1.5} \qquad [1]$$

where:

Q = total flow rate [m^3/s]

b = width of the weir crest [m]

H = upstream hydraulic head (water level) relative to the weir crest [m]



Components of a trapezoidal weir

Weir equation for a trapezoidal weir crest

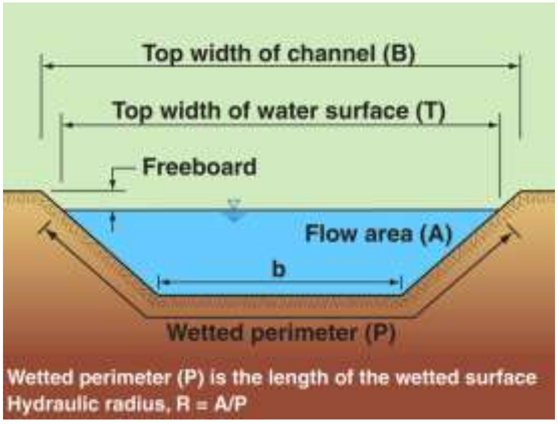
- For trapezoidal weirs, either use a numerical model, or analyse by combining rectangular weir equation (above) plus a triangular weir equation (below).
- The equation for a triangular weir is:

$$Q = 1.33\ z\ H^{2.5} \qquad [2]$$

where: z = the side slope (1 in z) of the trapezoidal weir.

- Thus the trapezoidal weir equation is:

$$Q = 1.7\ b\ H^{1.5} + 1.33\ z\ H^{2.5} \qquad [3]$$



Chute cross-section

Manning's equation

The flow velocity down a chute may be estimated using Manning's equation:

$$V = (1/n)\ R^{2/3}\ .\ S^{1/2} \qquad [4]$$

where:

V = average flow velocity [m/s] = Q/A

n = Manning's roughness coefficient

A = cross-sectional area of flow [m^2]

R = hydraulic radius [m] = A/P

P = wetted perimeter of flow [m]

S = channel slope [m/m]

Design the emergency spillway – Scour protection options

Rock-lined spillway (Qld)

Rock

- A common scour protection material.
- Rock-sizing equations and tables are provided over the page.
- Spillway failures are all too common, but most likely due to:
  - inadequate rock size
  - spillway too steep, compacted smooth, then lined with filter cloth, which turns the spillway into a giant 'slippery dip' (if the spillway is steep, then stair-step the surface).

Rock mattress spillway (NSW)

Rock mattress

- Rock mattresses can be expensive and time-consuming to assemble, but reliable.
- Can be vegetated if it remains as a permanent stormwater feature.
- Usually 'thinner' than two layers of loose rock.

Plastic sheeting (NZ)

Geotextile fabric with plastic sheeting

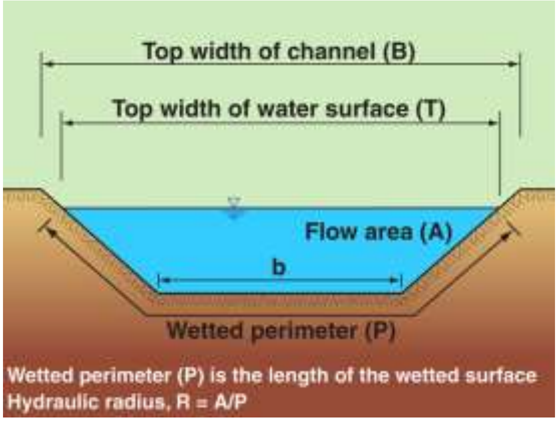
- In many cases, thick plastic sheeting can be all that is required to control scour over the crest of the spillway.
- Plastic or rubber sheeting may not be appropriate down the face of the spillway chute due to the turbulence and higher flow velocities. **Use with caution!**
- The upstream end of the sheeting must be well pinned and buried in a 200 mm (min) deep trench. **Avoid any processes that may initiate tearing of the fabric.**

Concrete spillway crest (Qld)

Concrete or grouted rock

- Reinforced concrete can be used if the basin remains as part of the permanent stormwater infrastructure.
- If the spillway is a temporary feature, then concrete can be poured over a single layer of rocks.
- A cut-off trench is recommended if the embankment soil is susceptible to tunnel erosion.

Design the emergency spillway – Rock protection



Channel geometry and flow conditions

Rock chute (Qld)

Deep water flow over rocks (NSW)

Shallow water flow over rocks (Qld)

Manning's equation

- The **average** channel flow velocity may be calculated using Manning's equation:

$$V = (1/n)\, R^{2/3}\, S^{1/2} \qquad [5]$$

where:

V = average flow velocity (m/s)
n = Manning's roughness coefficient
R = hydraulic radius (m) = A/P
A = effective flow area of channel (m^2)
P = wetted perimeter of flow (m)
S = channel slope (m/m)

Factors affecting the hydraulic roughness of rock-lined surfaces

- The effective Manning's roughness (n) of rock-lined surfaces depends on:
  - average rock size (d_{50})
  - the distribution of rock sizes, defined in this case by a ratio: d_{50}/d_{90}
  - the depth of water flow, defined by the hydraulic radius of flow (R)
  - the existence of vegetation
  - the occurrence of aerated 'whitewater' (not directly considered here).

Manning's roughness in deep water

- The Strickler equation for deep water may be presented in the modified form:

$$n = ((d_{50})^{1/6})/21.1 \qquad [6]$$

- An alternative equation was developed by Meyer-Peter & Muller:

$$n = ((d_{90})^{1/6})/26.0 \qquad [7]$$

  - d_{50} = rock size for which 50% of rocks (by weight) are smaller [m]
  - d_{90} = rock size for which 90% of rocks (by weight) are smaller [m]

Manning's roughness in shallow water

- The Manning's roughness (n) of rock-lined surfaces in both shallow-water and deep-water flow conditions is provided below.

$$n = \frac{d_{90}^{\,1/6}}{26(1 - 0.3593^{m})} \qquad [8]$$

  - $m = [(R/d_{90})(d_{50}/d_{90})]^{0.7}$
  - R = hydraulic radius of flow [m]
- The relative roughness (d_{50}/d_{90}) of rock extracted from streambeds is typically in the range 0.2 to 0.5; while quarried rock is commonly in the range 0.5 to 0.8.

Manning’s roughness of rock-lined surfaces

The Manning’s (n) roughness for rock-lined surfaces can be determined from Table 5 or Equation 8.

Table 5 – Manning’s (n) roughness of rock-lined surfaces

	d_{50}/d_{90} = 0.5				d_{50}/d_{90} = 0.8			
d_{50} =	**200mm**	**300mm**	**400mm**	**500mm**	**200mm**	**300mm**	**400mm**	**500mm**
R (m)	Manning’s roughness (n)				Manning’s roughness (n)			
0.2	0.10	0.14	0.17	0.21	0.06	0.08	0.09	0.11
0.3	0.08	0.11	0.14	0.16	0.05	0.06	0.08	0.09
0.4	0.07	0.09	0.12	0.14	0.04	0.05	0.07	0.08
0.5	0.06	0.08	0.10	0.12	0.04	0.05	0.06	0.07
0.6	0.06	0.08	0.09	0.11	0.04	0.05	0.05	0.06
0.8	0.05	0.07	0.08	0.09	0.04	0.04	0.05	0.06
1.0	0.04	0.06	0.07	0.08	0.03	0.04	0.05	0.05

Equation 8 is considered to produce significantly better estimates of the Manning’s roughness of rock-lined surfaces in shallow water flow compared to the use of traditional deep water equations such as the Strickler, Meyer-Peter & Muller or Limerinos equations.

Given the high variability of Manning’s n and the wide range of variables that are believed to influence the hydraulic roughness of a rock-lined channel, Equation 8 is considered well within the limits of accuracy expected for Manning’s n selection.

Data analysis during the development of Equation 8 indicated that the Meyer-Peter & Muller equation (Eqn 7) produced more reliable estimates of the deep water Manning's roughness values than the Strickler equation (Eqn 6). Possibly the choice between the two equations would come down to how reliable the determination of the d_{50} and d_{90} values were. If the estimate of d_{90} is not reliable, then it would be more appropriate to rely on the Strickler equation for the determination of the deep water Manning's n value, and vice versa.

Typical properties of rock

Crushed rock is generally more stable than natural rounded rock; however, rounded rock has a more ‘natural’ appearance. A 36% increase in rock size is recommended if rounded rock is used (i.e. K_1 = 1.36).

The rock should be durable and resistant to weathering, and should be proportioned so that neither the breadth nor the thickness of a single rock is less than one-third of its length.

Maximum rock size generally should not exceed twice the nominal (d_{50}) rock size, but in some cases a maximum rock size of 1.5 times the average rock size may be specified.

Typical rock densities (s_r) are presented in Table 6.

Table 6 – Relative density (specific gravity) of rock

Rock type	Relative density (s_r)
Sandstone	2.1 to 2.4
Granite	2.5 to 3.1 (commonly 2.6)
Limestone	2.6
Basalt	2.7 to 3.2

Thickness of rock placement

On basin spillways, the minimum height of the rock protection placed on the spillway banks should be equal to the critical flow depth (at the crest) plus 0.3 m.

The thickness of the armour layer should be sufficient to allow at least two overlapping layers of the nominal rock size. The thickness of rock protection must also be sufficient to accommodate the largest rock size. It is noted that increasing the thickness of the rock placement will **not** compensate for the use of undersized rock.

In order to allow at least two layers of rock, the minimum thickness of rock protection (T) can be approximated by the values presented in Table 7.

Table 7 – Minimum thickness (T) of rock lining

Min. thickness (T)	Size distribution (d_{50}/d_{90})	Description
1.4 d_{50}	1.0	Highly uniform rock size
1.6 d_{50}	0.8	Typical upper limit of quarry rock
1.8 d_{50}	0.67	Recommended lower limit of distribution
2.1 d_{50}	0.5	Typical lower limit of quarry rock

Note: d_X = nominal rock size (diameter) of which X% (by weight) of the rocks are smaller.

Minimum dimensions, freeboard and safety factor

Recommended absolute minimum spillway depth is 300 mm plus freeboard.

A freeboard of 150 mm should exist. A greater freeboard may be required if it is necessary for the spillway to fully contain any splash.

The descending spillway chute must be straight from its crest to outlet (i.e. no bends or curves).

Vegetating rock-lined spillways can significantly increase the spillway's long-term stability. Flexible, mat-forming grasses (non-woody plants) must be used. Stiff grasses, such as *Lomandra* or *Vetiveria zizanioides,* are **not** recommended for basin spillways.

Table 8 – Recommended safety factor for use in determining rock size

Safety factor (SF)	Recommended usage	Example site conditions
1.2	• Low risk structures. • Failure of structure is most unlikely to cause loss of life or irreversible property damage. • Permanent rock chutes with all voids filled with soil and pocket planted.	• Basin spillways where failure of the structure is likely to result in easily repairable soil erosion. • Basin spillways that are likely to experience significant sedimentation and vegetation growth before experiencing high flows. • Temporary (< 2 yrs) spillways with a design storm of 1 in 10 years or greater.
1.5	• High risk structures. • Failure of structure may cause loss of life or irreversible property damage. • Temporary structures that have a high risk of experiencing the design discharge while the voids remain open (i.e. prior to sediment settling within and stabilising the voids between individual rocks).	• Basin spillways where failure of the structure may cause severe gully erosion. • Basin spillways located up-slope of a residential area or busy roadway where an embankment failure could cause property flooding or loss of life. • Spillways designed for a storm frequency less than 1 in 10 years.

Sizing rock for sediment basin spillways

Application of equation	Primary rock sizing equation:
• This is the preferred design equation • Applicable for uniform flow conditions only, $S_e = S_o$ • Batter slopes (S_o) less than 50% (1 in 2)	$d_{50} = \dfrac{127.SF.K_1.K_2.S_o^{0.5}.q^{0.5}.y^{0.25}}{(s_r - 1)}$ [9]

Tables 9 and 10 provide mean rock size (rounded up to the next 0.1 m unit) for angular rock, and a safety factor of both 1.2 and 1.5. These tables are based on Equation 9, and are best used in the design of long drainage chutes. Use of the 'unit flow rate' (q) as the primary design variable is preferred to the use of 'flow velocity' (V) because it avoids errors associated with the selection of Manning's roughness.

Alternatively, tables 11 and 12 provide mean rock size for angular rock and a safety factor of 1.2 and 1.5, based on Equation 9 with flow velocity presented as the primary variable. These tables are best used in the design of short drainage chutes where uniform flow conditions are unlikely to be achieved down the face of the chute.

Definition of equation symbols

- d_X = nominal rock size (diameter) of which X% (by weight) of the rocks are smaller [m]
- d_{15} = rock size of 'coarse' layer of which 15% of the rocks are smaller [m]
- d_{85} = rock size of 'fine' underlay of which 85% of the rocks are smaller [m]
- A & B = equation constants
- K = equation constant based on flow conditions
  = 1.1 for low-turbulent deep water flow, 1.0 for low-turbulent shallow water flow, and 0.86 for highly turbulent and/or supercritical flow
- K_1 = correction factor for rock shape
  = 1.0 for angular (fractured) rock, 1.36 for rounded rock (i.e. smooth, spherical rock)
- K_2 = correction factor for rock grading
  = 0.95 for poorly graded rock ($C_u = d_{60}/d_{10} < 1.5$), 1.05 for well graded rock ($C_u > 2.5$), otherwise $K_2 = 1.0$ ($1.5 < C_u < 2.5$)
- n_o = Manning's roughness value for deep water conditions [dimensionless]
- q = flow per unit width down the embankment [$m^3/s/m$]
- s_r = specific gravity of rock (e.g. sandstone 2.1–2.4; granite 2.5–3.1, typically 2.6; limestone 2.6; basalt 2.7–3.2)
- S_e = slope of energy line [m/m]
- S_o = bed slope = tan(θ) [m/m]
- SF = safety factor
- V = actual depth-average flow velocity at location of rock [m/s]
- V_o = depth-average flow velocity based on **uniform** flow down a slope, S_o [m/s]
- y = depth of flow at a given location [m]
- θ = slope of channel bed [degrees]

Table 9 – Uniform flow depth [1], y (m) and mean rock size, d_{50} (m) for SF = 1.2

Safety factor, SF = 1.2			Specific gravity, s_r = 2.4			Size distribution, d_{50}/d_{90} = 0.5		
Unit flow rate (m^3/s/m)	Bed slope = 1:2		Bed slope = 1:3		Bed slope = 1:4		Bed slope = 1:6	
	y (m)	d_{50}	y (m)	d_{50}	y (m)	d_{50}	y (m)	d_{50}
0.1	0.09	**0.20**	0.09	**0.20**	0.09	**0.10**	0.09	**0.10**
0.2	0.14	**0.30**	0.14	**0.20**	0.14	**0.20**	0.15	**0.20**
0.3	0.18	**0.30**	0.19	**0.30**	0.19	**0.20**	0.20	**0.20**
0.4	0.22	**0.40**	0.23	**0.30**	0.23	**0.30**	0.24	**0.20**
0.5	0.26	**0.40**	0.26	**0.40**	0.27	**0.30**	0.27	**0.30**
0.6	0.29	**0.50**	0.30	**0.40**	0.30	**0.40**	0.31	**0.30**
0.8	0.35	**0.60**	0.36	**0.50**	0.37	**0.40**	0.37	**0.40**
1.0	0.41	**0.70**	0.42	**0.60**	0.42	**0.50**	0.44	**0.40**
1.2	0.46	**0.70**	0.47	**0.60**	0.48	**0.50**	0.49	**0.50**
1.4	0.51	**0.80**	0.52	**0.70**	0.53	**0.60**	0.54	**0.50**
1.6	0.56	**0.90**	0.57	**0.70**	0.58	**0.70**	0.60	**0.50**
1.8	0.60	**1.00**	0.62	**0.80**	0.63	**0.70**	0.64	**0.60**
2.0	0.65	**1.00**	0.66	**0.90**	0.67	**0.70**	0.69	**0.60**
3.0	0.85	**1.30**	0.87	**1.10**	0.88	**1.00**	0.90	**0.80**
4.0	1.02	**1.60**	1.05	**1.30**	1.07	**1.20**	1.10	**1.00**
5.0	1.19	**1.80**	1.22	**1.50**	1.24	**1.30**	1.27	**1.10**

[1] Flow depth is expected to be highly variable due to whitewater (turbulent) flow conditions.

Table 10 – Uniform flow depth [1], y (m) and mean rock size, d_{50} (m) for SF = 1.5

Safety factor, SF = 1.5			Specific gravity, s_r = 2.4			Size distribution, d_{50}/d_{90} = 0.5		
Unit flow rate (m^3/s/m)	Bed slope = 1:1		Bed slope = 1:3		Bed slope = 1:4		Bed slope = 1:6	
	y (m)	d_{50}	y (m)	d_{50}	y (m)	d_{50}	y (m)	d_{50}
0.1	0.10	**0.20**	0.10	**0.20**	0.10	**0.20**	0.10	**0.10**
0.2	0.15	**0.30**	0.15	**0.30**	0.16	**0.20**	0.16	**0.20**
0.3	0.20	**0.40**	0.20	**0.30**	0.21	**0.30**	0.21	**0.30**
0.4	0.24	**0.50**	0.25	**0.40**	0.25	**0.40**	0.26	**0.30**
0.5	0.28	**0.50**	0.28	**0.50**	0.29	**0.40**	0.30	**0.30**
0.6	0.31	**0.60**	0.32	**0.50**	0.33	**0.40**	0.34	**0.40**
0.8	0.38	**0.70**	0.39	**0.60**	0.40	**0.50**	0.41	**0.40**
1.0	0.44	**0.80**	0.45	**0.70**	0.46	**0.60**	0.47	**0.50**
1.2	0.50	**0.90**	0.51	**0.80**	0.52	**0.70**	0.53	**0.60**
1.4	0.55	**1.00**	0.57	**0.90**	0.58	**0.80**	0.59	**0.60**
1.6	0.60	**1.10**	0.62	**0.90**	0.63	**0.80**	0.64	**0.70**
1.8	0.65	**1.20**	0.67	**1.00**	0.68	**0.90**	0.70	**0.70**
2.0	0.70	**1.30**	0.72	**1.10**	0.73	**0.90**	0.75	**0.80**
3.0	0.92	**1.70**	0.94	**1.40**	0.96	**1.20**	0.98	**1.00**
4.0	1.11	**2.00**	1.14	**1.70**	1.16	**1.50**	1.19	**1.20**
5.0	1.29	**2.30**	1.32	**1.90**	1.34	**1.70**	1.38	**1.40**

[1] Flow depth is expected to be highly variable due to whitewater (turbulent) flow conditions.

Table 11 – Velocity-based design table for mean rock size, d_{50} (m) for SF = 1.2 [1]

Safety factor, SF = 1.2			Specific gravity, s_r = 2.4			Size distribution, d_{50}/d_{90} = 0.5		
Local velocity (m/s)	**Bed slope (V:H)**							
	1:2	**1:3**	**1:4**	**1:6**	**1:10**	**1:15**	**1:20**	**1:30**
0.5	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
0.8	0.10	0.10	0.05	0.05	0.05	0.05	0.05	0.05
1.0	0.20	0.10	0.10	0.10	0.10	0.10	0.05	0.05
1.3	0.20	0.20	0.20	0.20	0.10	0.10	0.10	0.10
1.5	0.30	0.30	0.20	0.20	0.20	0.20	0.20	0.10
1.8	0.40	0.30	0.30	0.30	0.20	0.20	0.20	0.20
2.0	0.50	0.40	0.40	0.30	0.30	0.30	0.20	0.20
2.3	0.60	0.50	0.50	0.40	0.30	0.30	0.30	0.30
2.5	0.70	0.60	0.60	0.50	0.40	0.40	0.30	0.30
2.8	0.80	0.70	0.70	0.60	0.50	0.40	0.40	0.40
3.0	1.00	0.90	0.80	0.70	0.60	0.50	0.50	0.40
3.5	1.30	1.10	1.00	0.90	0.80	0.70	0.60	0.60
4.0	1.70	1.50	1.30	1.20	1.00	0.90	0.80	0.70
4.5	2.10	1.90	1.70	1.50	1.20	1.10	1.00	0.90
5.0				1.80	1.50	1.30	1.20	1.10
6.0						1.90	1.70	1.60

[1] Based on uniform flow conditions, **safety factor = 1.2**, rock specific gravity of 2.4, and a rock size distribution such that the largest rock is approximately twice the size of the mean rock size.

Table 12 – Velocity-based design table for mean rock size, d_{50} (m) for SF = 1.5 [1]

Safety factor, SF = 1.5			Specific gravity, s_r = 2.4			Size distribution, d_{50}/d_{90} = 0.5		
Local velocity (m/s)	**Bed slope (V:H)**							
	1:2	**1:3**	**1:4**	**1:6**	**1:10**	**1:15**	**1:20**	**1:30**
0.5	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
0.8	0.10	0.10	0.10	0.10	0.05	0.05	0.05	0.05
1.0	0.20	0.20	0.20	0.20	0.10	0.10	0.10	0.10
1.3	0.30	0.30	0.20	0.20	0.20	0.20	0.20	0.10
1.5	0.40	0.30	0.30	0.30	0.20	0.20	0.20	0.20
1.8	0.50	0.50	0.40	0.40	0.30	0.30	0.30	0.20
2.0	0.70	0.60	0.50	0.50	0.40	0.40	0.30	0.30
2.3	0.80	0.70	0.60	0.60	0.50	0.40	0.40	0.40
2.5	1.00	0.90	0.80	0.70	0.60	0.50	0.50	0.40
2.8	1.20	1.00	0.90	0.80	0.70	0.60	0.60	0.50
3.0	1.40	1.20	1.10	1.00	0.80	0.70	0.70	0.60
3.5	1.90	1.70	1.50	1.30	1.10	1.00	0.90	0.80
4.0			1.90	1.70	1.40	1.30	1.10	1.00
4.5					1.80	1.60	1.40	1.30
5.0						1.90	1.80	1.60
6.0								2.20

[1] Based on uniform flow conditions, **safety factor = 1.5**, rock specific gravity of 2.4, and a rock size distribution such that the largest rock is approximately twice the size of the mean rock size.

Common spillway design and construction problems

Insufficient freeboard

Inadequate spillway crest profile

- It is important to ensure that the spillway crest has sufficient depth and width to fully contain the design storm discharge.
- A poorly defined spillway crest profile (e.g. insufficient cross-sectional width or depth) can result in flows bypassing the spillway.
- In such cases (left, and below) damage to the earth embankment is likely to occur.

Insufficient freeboard

Insufficient freeboard

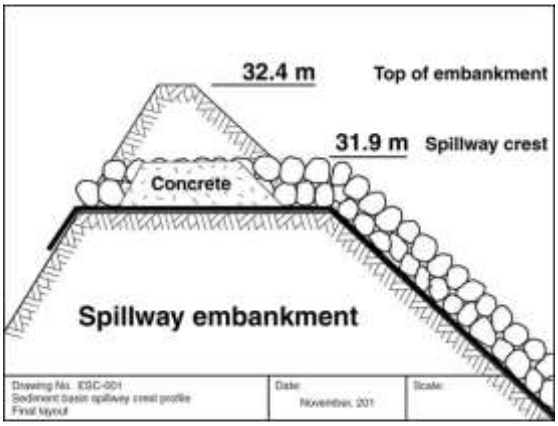
Insufficient freeboard

Insufficient freeboard

Insufficient freeboard

Insufficient freeboard

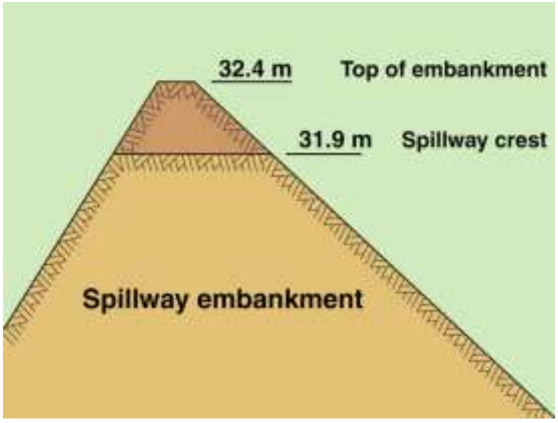
Possible spillway design and construction problem



Design details of the spillway crest

What the designer drafted

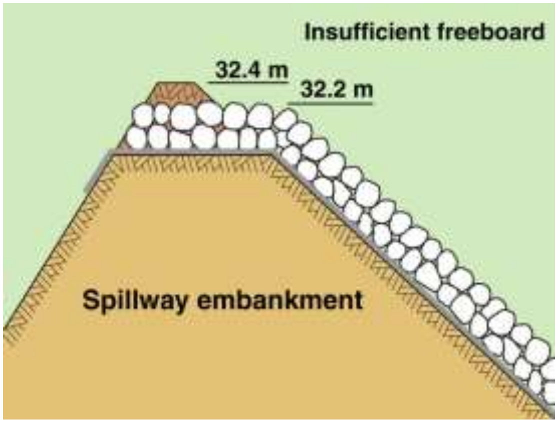
- Sediment basin plans should specify the 'top of the earth embankment' as well as the 'spillway crest elevation'.
- This will require designers to know the likely thickness of the scour protection placed over the spillway crest.
- It is noted (over page) that the scour protection of the spillway crest may be different from that used down the face of the spillway.



Finished earthworks

What the earthworks contractor constructed

- It would appear that too often the earth spillway embankment is formed to the stated crest elevation (i.e. the expected top of the scour protection).



Final spillway profile

What the spillway looked like after placement of the rocks

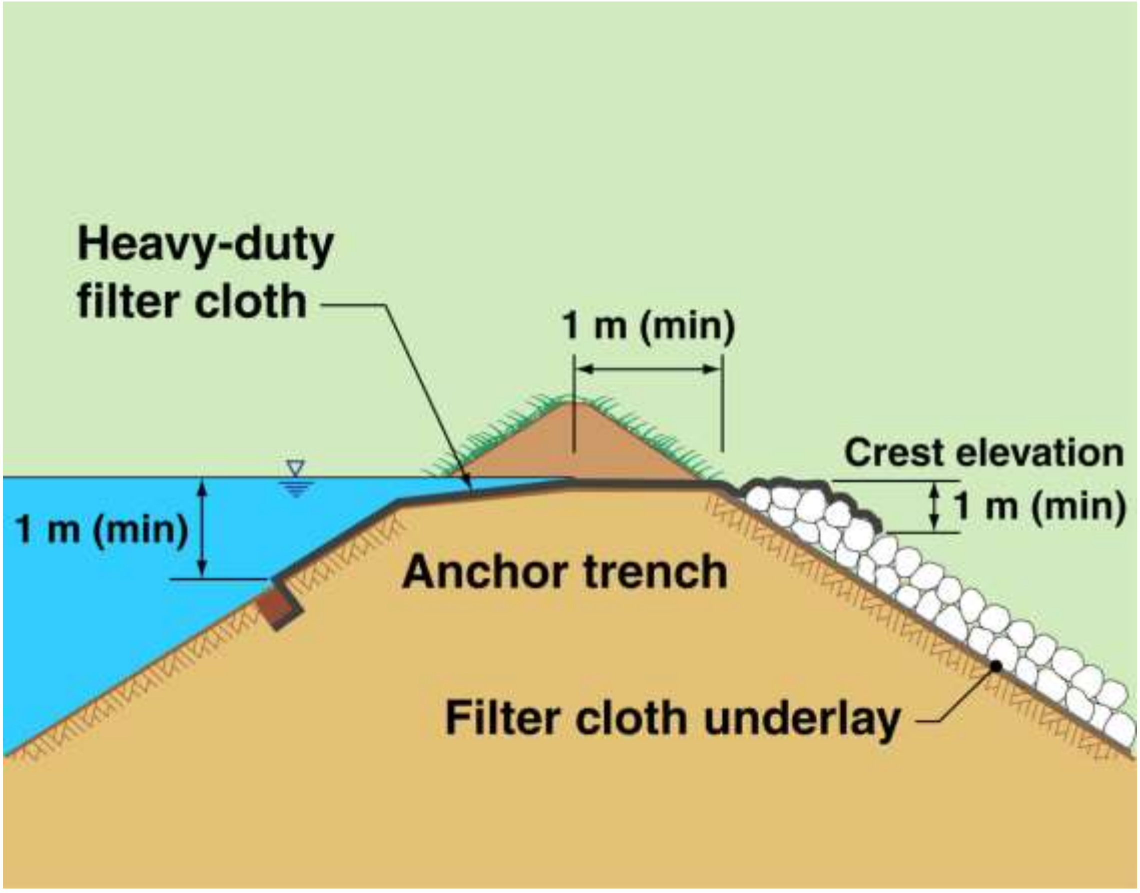
- Once rocks are placed on the spillway crest, the 'design' freeboard almost totally disappears.
- If sufficient freeboard is difficult to achieve, then consider replacing the rocks on the 'crest' of the spillway with:
  - plastic or rubber sheet
  - heavy-duty filter cloth if the hydraulic head (H) is less than 300 mm.

Embankment failure

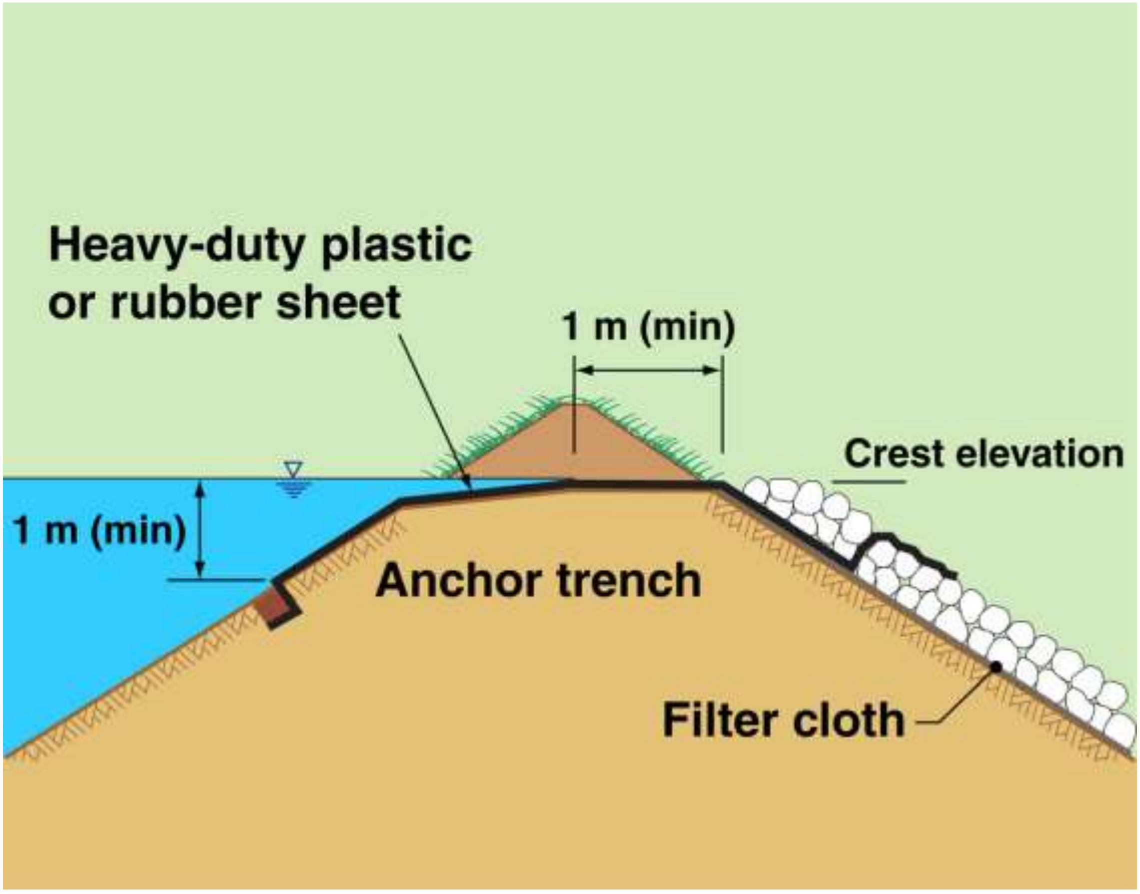
What happened to the basin embankment during a storm event

- These spillway embankment failures should not be considered a normal or acceptable consequence of civil construction activities!

Design the emergency spillway – Optional spillway crest details



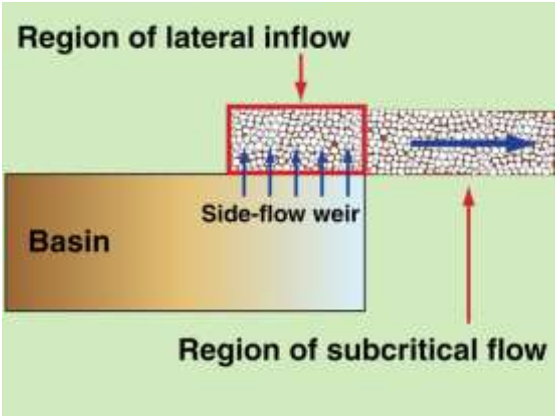
Recommended placement of filter cloth when used for spillway crest scour protection



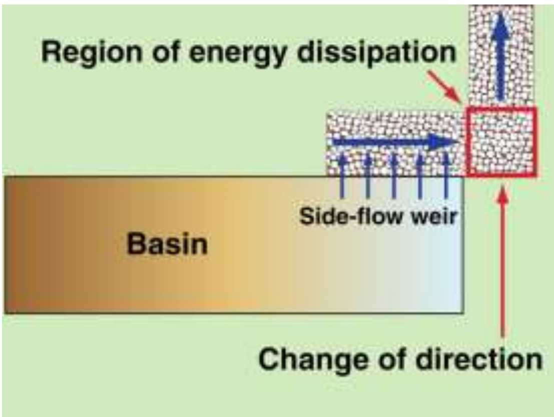
Recommended placement of plastic sheet when used for spillway crest scour protection

Design the emergency spillway – Spillway hydraulics

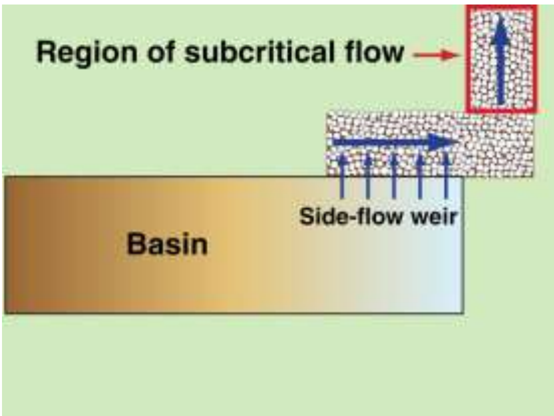
Sediment basin spillway (USA)



Lateral inflow and energy dissipation



Sudden change of direction



Subcritical flow conditions

Supercritical flow conditions

- The hydraulic analysis of the typical sediment basin spillway requires several different types of flow analysis.
- Flow conditions at the crest are usually defined by a **weir equation**.
- Flow conditions immediately downstream of the weir are usually **supercritical**, which can be analysed by a one-dimensional numerical model, such as HecRas.

Subcritical flow with lateral inflow

- If the spillway incorporates a side-flow weir from where the water then enters a low gradient (subcritical) channel that flows in a direction 90-degrees to the weir, then:
  - the **gradient** of the water surface in this section of the channel will be steeper than that predicted by a standard backwater model
  - this means the flow conditions **cannot** be analysed using a one-dimensional numerical model, such as HecRas.

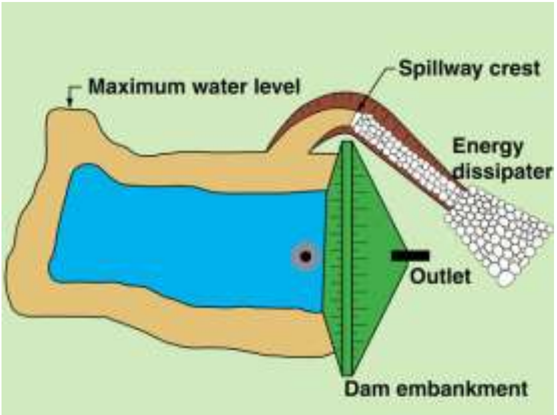
Energy dissipation at channel bends

- If the spillway chute incorporates a sudden change of direction, then
  - this sudden change of direction should only occur in a region of subcritical flow (do not try to change the direction of supercritical flow)
  - this change of direction will incorporate an added energy loss that **cannot** be assessed by a one-dimensional numerical model, such as HecRas.

Subcritical flow conditions

- Eventually the flow passing down a spillway will enter a region of gradually varied, subcritical flow.
- In some cases a hydraulic jump and region of energy dissipation will occur as the water moves from supercritical to subcritical flow conditions.
- A detailed description of the hydraulic analysis of complex spillway conditions is provided in Chapter 2 of this field guide.

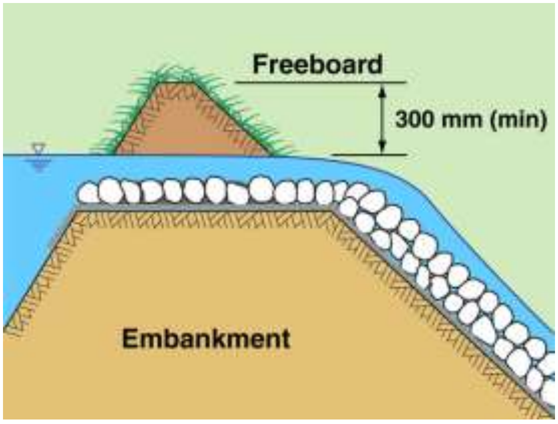
Step 12: Determine the overall dimensions of the basin



Sediment basin (plan view)

Introduction

- The overall dimensions of the sediment basin can be significantly larger than the dimensions of the settling pond.
- It is important to ensure the overall dimensions of the basin can fit within the construction site (i.e. lawful property boundary), including the spillway and energy dissipater.



Recommended minimum freeboard

Embankment freeboard requirements

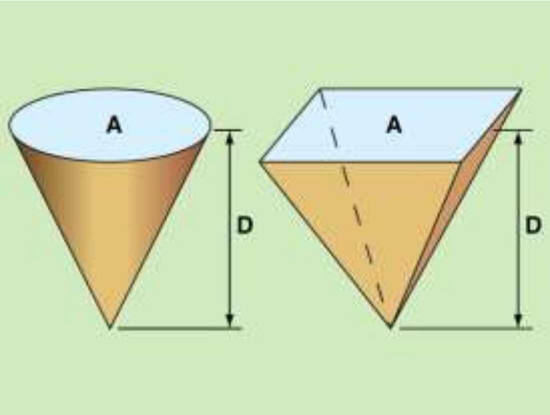
- The minimum recommended freeboard between the 'top water level' and any 'earth embankments' is 300 mm.

Sediment basin spillway

Located within the property boundary

- All aspects of the basin must be located within the construction site boundaries, including the spillway and energy dissipater.
- Running out of room to construct a proper spillway and/or energy dissipater is **not** a valid reason for not constructing a proper spillway and/or energy dissipater!

Determine the overall dimensions of the basin – Volume calculations



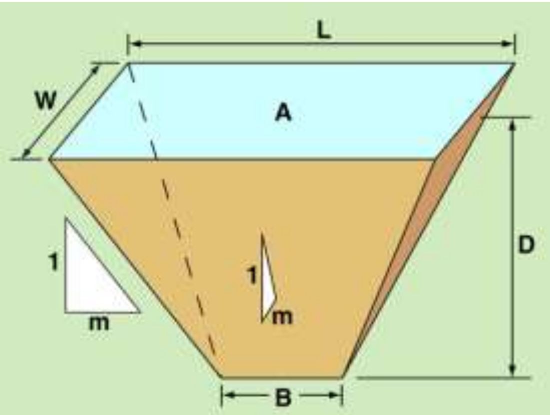
Cone and rectangular pyramid

Cone and pyramid shapes

$$V = (1/3).A.D$$

where:

- V = pond volume [m^3]
- A = top surface area [m^2]
- D = depth of volume [m]



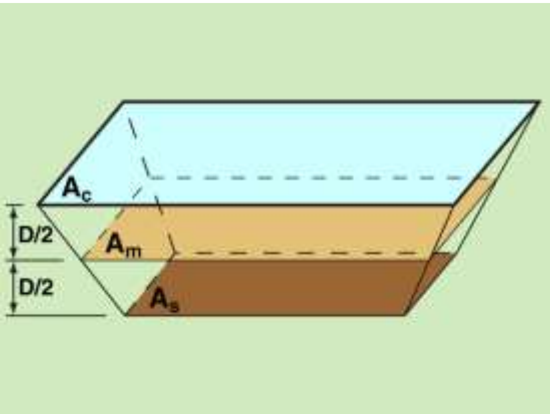
Rectangular prism

Rectangular prism

$$V = (1/3).W.(L - B).D + (1/2).W.B.D$$

where:

- V = pond volume [m^3]
- W = width of top surface [m]
- L = length of top surface [m]
- B = width of bottom edge [m]
- D = depth of volume [m]



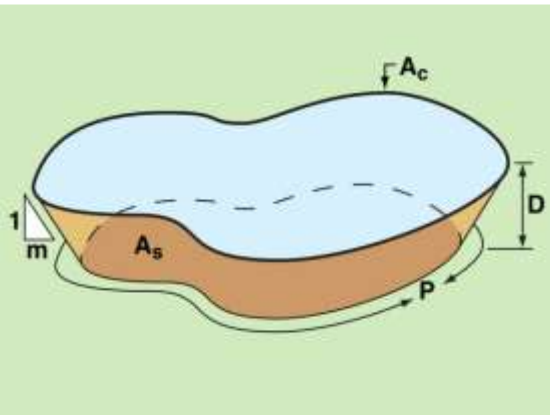
Truncated pyramid (Frustum)

Simpson's Rule

$$V = (D/6).(A_C + 4.A_M + A_S)$$

where:

- V = pond volume [m^3]
- D = depth of volume [m]
- A_C = surface area at top of volume [m^2]
- A_M = surface area at mid depth [m^2]
- A_S = surface area at base of volume [m$_2$]



Curved three-dimensional shape

Estimation of required basin depth given the pond surface area and bank slope

$$D \approx \frac{-A_s + \sqrt{(A_s{}^2 + 2.P.m.V)}}{P.m}$$

where:

- D = pond depth [m]
- A_S = pond surface area at base [m^2]
- P = circumference of the base of the volume [m]
- V = required basin volume [m^3]
- m = constant bank slope around the volume

Step 13: Locate maintenance access (de-silting)

Basin de-silting (Qld)

Introduction

- Sediment basins are usually de-silted either:
  - using excavators operating from the sides of the basin; or
  - by allowing excavators direct access into the basin.

Trafficked sediment (Qld)

Access into the basin

- If excavation equipment needs to enter into the basin, then it is better to:
  - design the access ramp so that trucks can be brought to the edge of the sediment
  - rather than trying to transport the sediment up a ramp to trucks located at the top of the embankment—such access ramps can quickly become covered with slippery mud.
- Thus a maximum 1:6 (ideally 1:10, V:H) access ramp will need to be constructed.

De-watering sediment (Qld)

Transportation of 'dry' sediment

- If the sediment is to be removed from the site, then a suitable sediment drying area should be made available adjacent to the basin, or at least somewhere within the basin's catchment area.

Step 14: Define the sediment disposal method

Step 14: Define the sediment disposal method

Sediment disposal area (Qld)

Introduction

- Trapped sediment can be mixed with on-site soils and buried, or removed from the site.
- If sediment is removed from the site, then it should be de-watered prior to transportation.
- De-watering must occur within the catchment area of the sediment basin.

Soil testing

Regulations

- If coagulants or flocculants have been used in the treatment of runoff within the basin, guidance should be sought from the chemical supplier on the requirements for sludge disposal in accordance with state government requirements.

Park rehabilitation (Qld)

Land replenishment

- Opportunities may exist for the use of the sediment, in association with a compost blanket, to improve the surface soils on council parks and sporting fields.
- Always seek expert advice, and of course, council approval /coordination.

Step 15: Assess need for safety fencing

Fenced sediment basin adjacent to a creek

Fencing (Qld)

Preparation of ESCP

Introduction

- Construction sites are often located in publicly accessible areas.
- In general it is not reasonable to expect a parent or guardian of a child to be aware of the safety risks (drowning) associated with a near-by construction site.
- Thus fencing of a sediment basin is usually warranted even if the basins are located adjacent to other permanent water bodies such as a stream, lake, or wetland.

Workplace health and safety

- Responsibility for safety issues on a construction site ultimately rests with the site manager; however, each person working on a site has a duty of care in accordance with the state's work place safety legislation.

Designer's responsibility

- Designers of sediment basins have a duty of care to investigate the safety requirements of the site on which the basins are to be constructed.

Step 16: Define the rehabilitation process

De-commissioned roadside basin (NSW)

Introduction

- The *Erosion and Sediment Control Plan* needs to include details on the required decommissioning and rehabilitation of the sediment basin area.
- On subdivisions and major road works, construction site sediment basins often represent a significant opportunity for conversion into either: a detention basin, retention basin, bio-retention basin, wetland, or pollution containment system.

Pollution containment pond (USA)

Pollution containment basins

- Pollution containment basins function by 'containing' any pollutants released from traffic accidents, including fire-fighting chemicals.
- Any pollutants captured by the basins must be later removed for off-site treatment and disposal.

'Stop-board' outlet control system (SA)

Outlet structures

- Pollution containment traps need to be fitted with outlet structures that will allow emergency services to isolate the basin to prevent the release of captured pollutants.
- Some agencies require such outlet structures to be fitted to a wide range of permanent stormwater treatment systems.

Define the rehabilitation process for the basin area

Basin converted to a detention basin, garden and area for factory works to lunch (USA)

Sediment basin converted to a detention basin and park (Qld)

Sediment basin converted to a bio-retention basin (NSW)

Define the rehabilitation process for the basin area

Sediment basins in operation during road construction works (NSW)

Sediment basins in the above site converted into wetlands (NSW)

Sediment basin (associated with adjacent road works) retained as a farm dam (NSW)

Step 17: Define the basin's operational procedures

Type A sediment basin (Qld)

Type A and B basin

- Type A basins were developed from the Auckland style basins used in Auckland, New Zealand.
- The basins use an automatic dosing system that introduces coagulants or flocculants into the sediment-laden inflow.
- These basins are also designed to be hydraulically efficient (i.e. flow short circuiting is minimised), and as such are sometimes referred to as *High Efficiency Basins*.

Type C sediment basin (NSW)

Type C basin

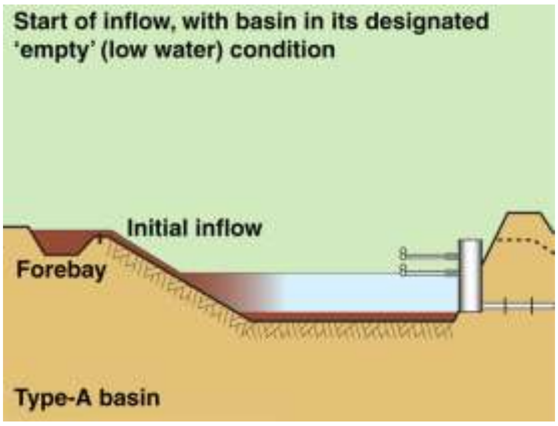
- Type C basins are the traditional continuous flow settling ponds that have proved effective for coarse-grained soils that do not experience high turbidity issues.
- These basins incorporate a low-flow decant system that allows the basin to fully drain under gravity.
- Automatic chemical dosing can be introduced to these basins, but in such cases a Type A basin would be preferred.

Type D sediment basin (Qld)

Type D basin

- Type D basins utilise a 'plug flow' system where under normal flow conditions, the basin is empty at the start of a storm.
- During a storm, the basin accepts inflows without decanting any water.
- After the storm, coagulants or flocculants are mixed with the water to aid settlement, and the basin is only decanted:
  - when a specified water quality is reached; or
  - a new storm approaches.

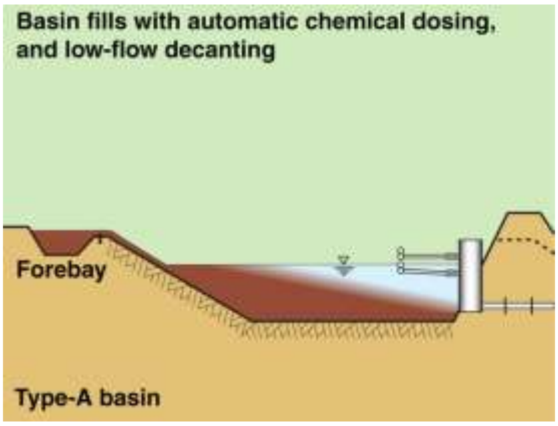
Operation of Type A basins



Prior to storm event

Prior to storm event

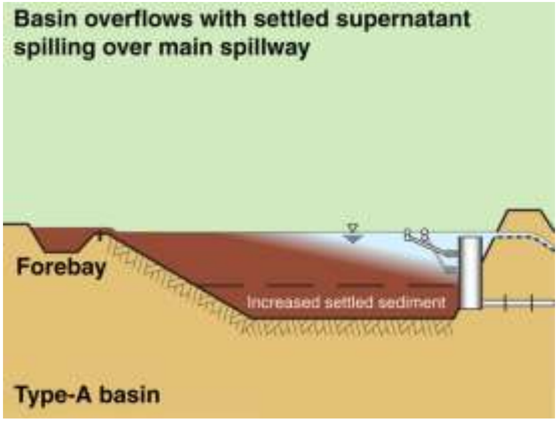
- Basin must be de-watered to the bottom of the settling zone between storm events.



Operation during a design storm

Operation during storms <u>less</u> than the design event

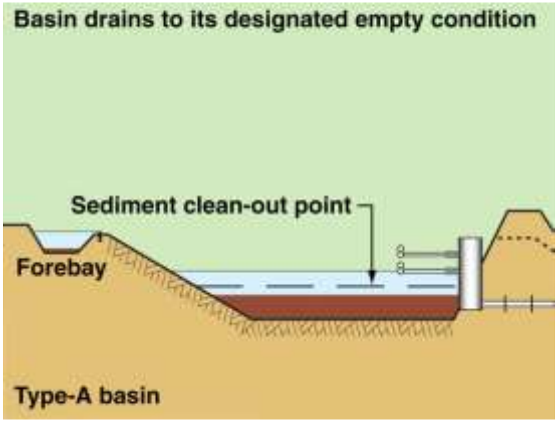
- For storm events less than the nominated design storm, sediment particle settlement must have occurred before the water reaches the floating decant pipes.



Operation during major storms

Operation during storms <u>greater</u> than the design event

- During major storms, the length-to-width ratio of the settling basin is designed such that the basin still captures coarse sediment, even though turbid water may pass over the emergency spillway.

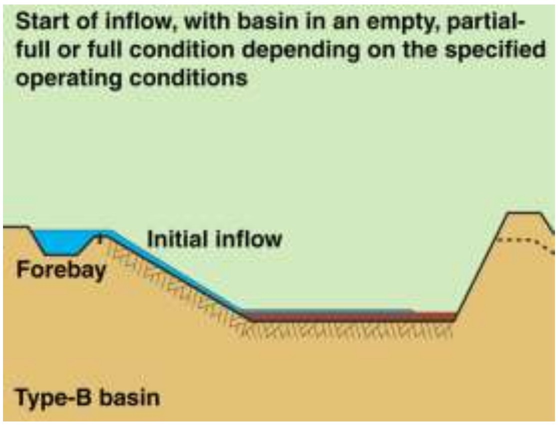


Free-draining de-watering

Post storm de-watering

- The free-draining de-water system should allow the basin to drain to the base of the settling zone.
- Once sediment reaches the top of the sediment storage zone, the basin must be de-silted.

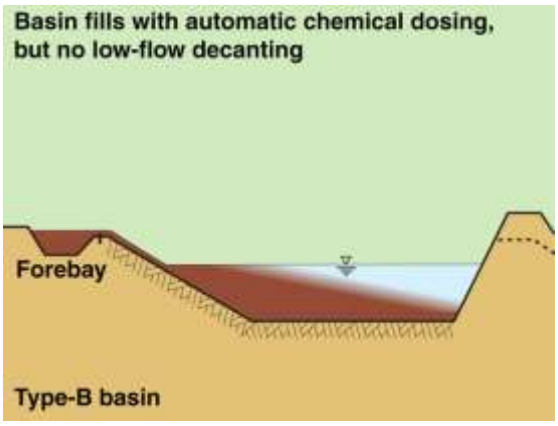
Operation of Type B basins



Prior to storm event

Prior to storm event

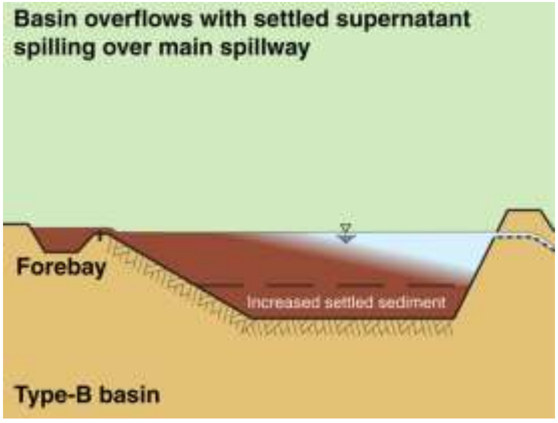
- Type B basins do not necessarily need to be de-watered immediately after storm events, but wherever practical, the basin should be de-watered prior to an imminent, runoff-producing storm.



Operation during a design storm

Operation during storms <u>less</u> than the design event

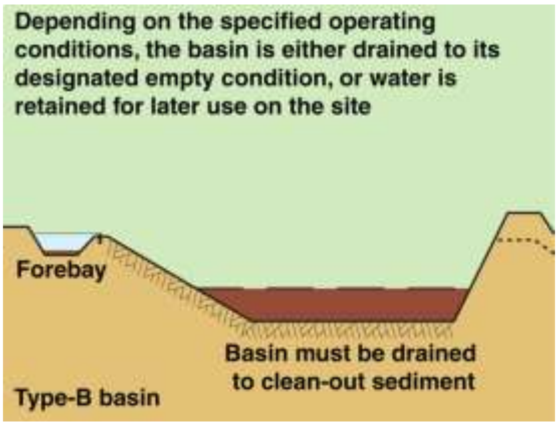
- For storm events less than the nominated design storm, sediment particle settlement must have occurred before the water reaches the spillway.



Operation during major storms

Operation during storms <u>greater</u> than the design event

- During major storms, the length-to-width ratio of the settling basin is designed such that the basin still captures coarse sediment, even though turbid water may pass over the emergency spillway.

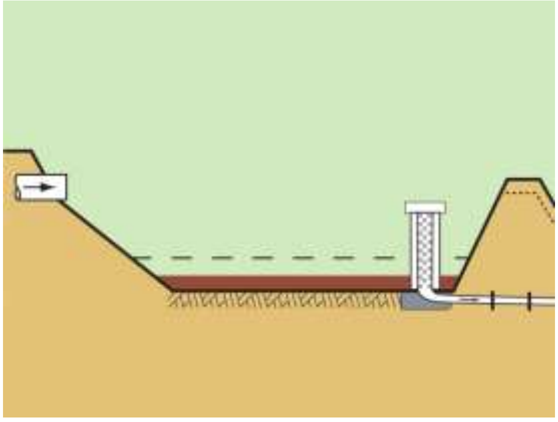


Free-draining de-watering

Post storm de-watering

- Type B basins do not necessarily need to be de-watered immediately after storm events.
- Captured water can be used for on-site purposes, such as dust control and plant watering.
- Once sediment reaches the top of the sediment storage zone, the basin must be de-silted.

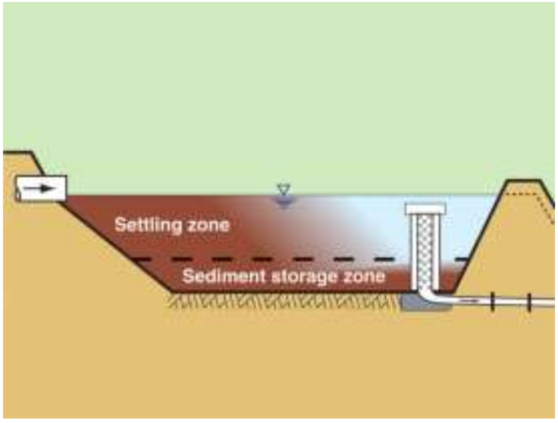
Operation of Type C basins



Prior to storm event

Prior to storm event

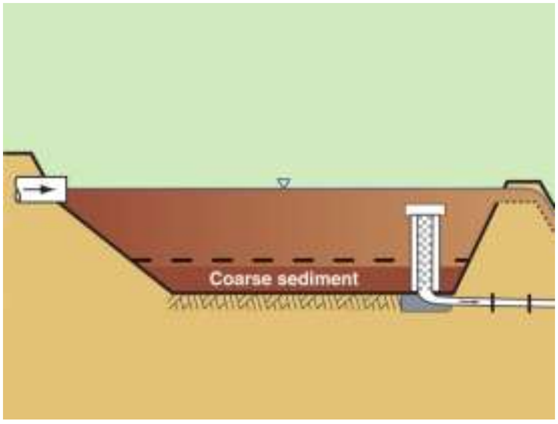
- Basin must be de-watered between storm events.



Operation during a design storm

Operation during storms less than the design event

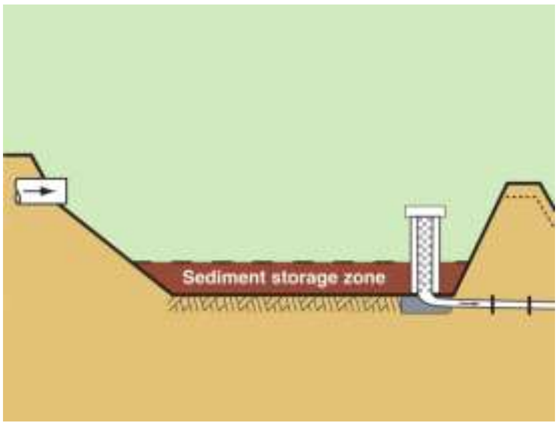
- For storm events less than the nominated design storm, sediment particle settlement must have occurred before the water reaches the decant riser pipe.



Operation during major storms

Operation during storms greater than the design event

- During major storms, the length-to-width ratio of the settling basin is designed such that the basin still captures coarse sediment, even though turbid water passes over the emergency spillway.

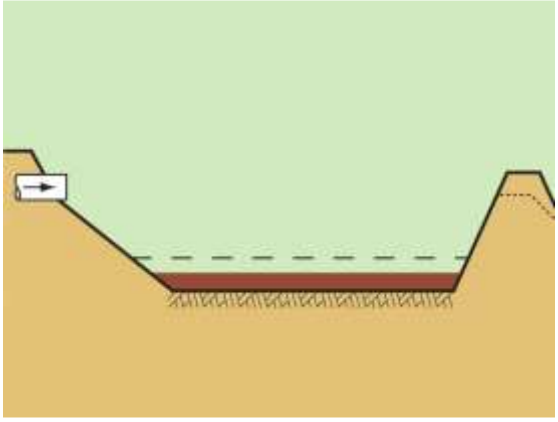


Free-draining de-watering

Post storm de-watering

- The free-draining de-water system should allow the basin to fully drain, even if this means 'clean' water is decanted from the sediment storage zone.
- Once sediment reaches the top of the sediment storage zone, the basin must be de-silted.

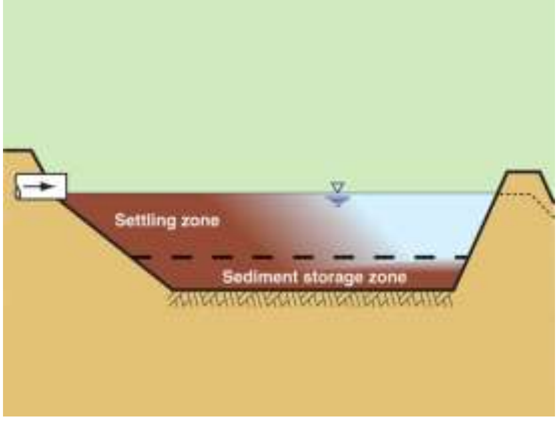
Operation of Type D basins



Prior to storm event

Prior to storm event

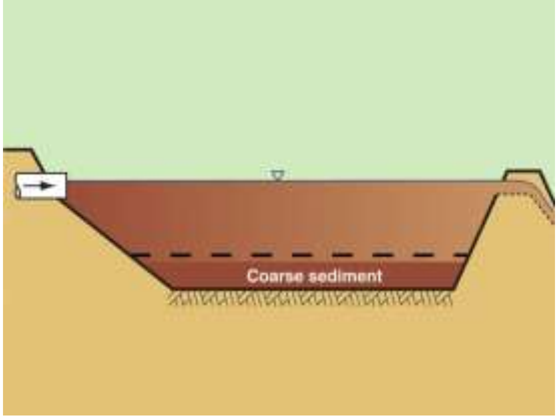
- Basin must be de-watered between storm events.



Operation during a design storm

Operation during storms less than the design event

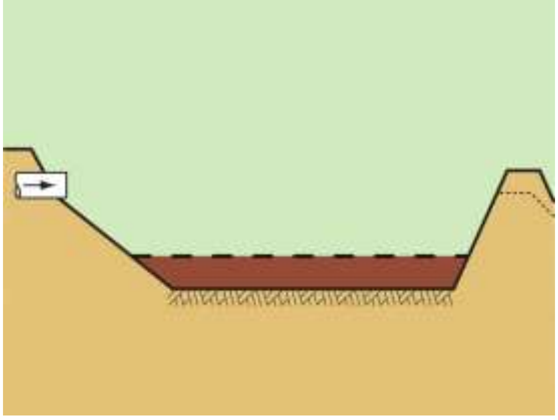
- For storm events less than the nominated design storm, sediment particle settlement must have occurred before the water reaches the spillway.



Operation during major storms

Operation during storms greater than the design event

- During major storms, the length-to-width ratio of the settling basin is designed such that the basin still captures coarse sediment, even though turbid water passes over the emergency spillway.



Free-draining de-watering

Post storm de-watering

- After the storm, the basin must be de-watered, and if possible, any 'clean' water should also be decanted from the sediment storage zone.
- Once sediment reaches the top of the sediment storage zone, the basin must be de-silted.

Step 18: Complete the Standard Basin Data forms

Step 18: Complete the Standard Basin Data forms


BASIN PERFORMANCE REPORT

Basin Performance table

Introduction

- Some authorities may require specific data forms to be completed for each basin design.


Part J: Sediment Basins

Sediment basin checklist

Design check lists

- Design check lists exist in various ESC publications, including IECA (Australasia) 2008.


Floc Performance Report

Floc Performance Report

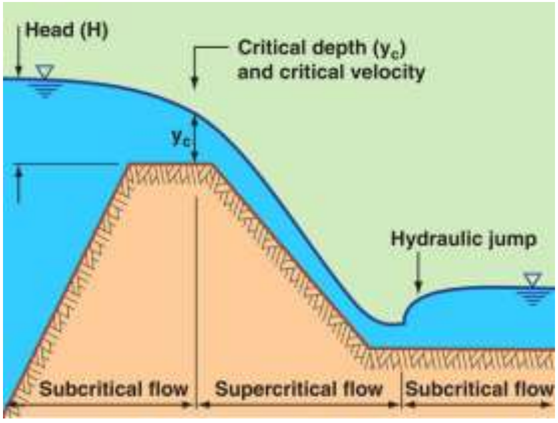
Flocculant testing

- Forms also exist for the recording of sediment settling rates for use in the design of Type A and B basins.

2. Hydraulic Analysis of Sediment Basin Spillways

Overview of ‘subcritical’ and ‘supercritical’ flow conditions

Flow conditions on a basin spillway (USA)



Flow conditions on a basin spillway

Supercritical flow down a spillway (NSW)

Subcritical flow downstream of a spillway

Introduction

- Most people are aware that there are two types of air flow, subsonic and supersonic, which relates to the speed of sound.
- The speed of sound is important because it is the speed of a pressure wave in air.
- Well, there are also two types of water flow, subcritical and supercritical, which relates to the speed of a surface wave.
- The speed of a surface wave is important because it is the speed that water pressure is allowed to change in open channel flow.

Critical flow conditions

- Critical flow is the flow condition that exists at the point where water flow converts from subcritical to supercritical.
- Critical flow is important to hydraulic engineers because:
  - it helps define the various weir equations
  - it controls the point where weirs can become ‘drowned’ by downstream flow conditions; and
  - it is used in the analysis of some lateral inflow conditions.

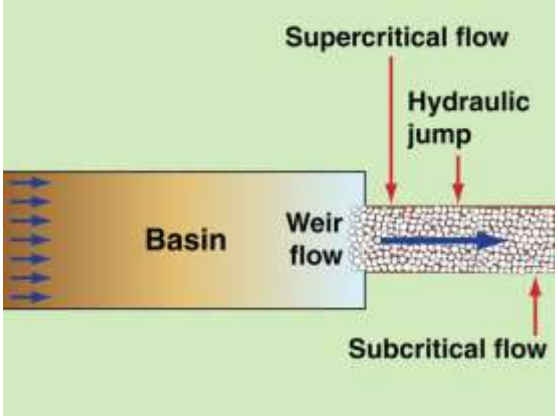
Supercritical flow conditions

- Most people would have observed supercritical flow by simply watching stormwater flowing down a roadside gutter.
- Only on very flat roads will stormwater be moving at subcritical velocities.
- Supercritical flow conditions normally exist on the steep sections of spillways.
- During supercritical flow, the elevation of the water, and its flow velocity, will be governed by the flow conditions that exist upstream of the channel or chute.

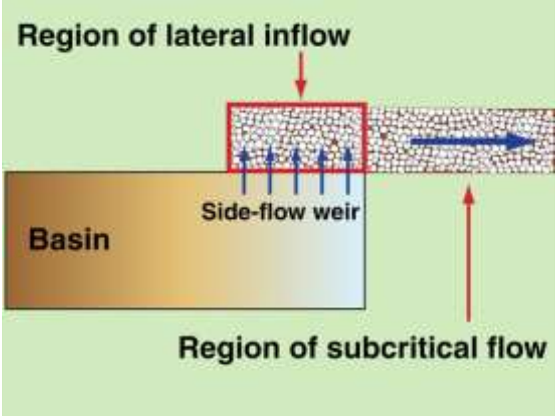
Subcritical flow conditions

- Well downstream of a sediment basin spillway the flow conditions are likely to be subcritical.
- During subcritical flow, the elevation of the water at any given location is governed by the downstream channel conditions.
- This means water levels may be governed by the flow conditions outside the construction site (property boundary).

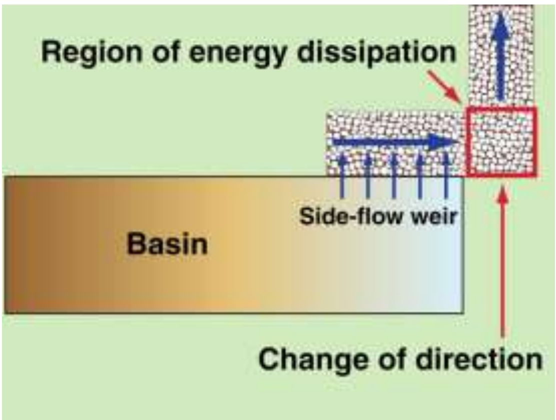
Complex flow conditions experienced on sediment basin spillways



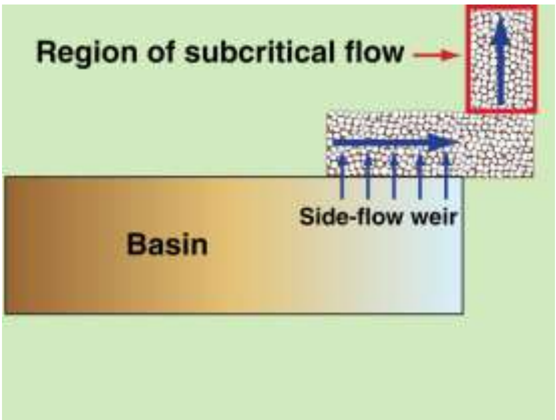
Spillway located at the end of the basin



Lateral inflow and energy dissipation



Sudden change of direction



Subcritical discharge channel

Supercritical flow conditions

- The hydraulic analysis of the typical sediment basin spillway requires several different types of flow analysis.
- Flow conditions at the crest are usually defined by a **weir equation**.
- Flow conditions immediately downstream of the weir is usually **supercritical**, which can be analysed by a one-dimensional numerical model, such as HecRas.
- A hydraulic jump is usually formed when supercritical flow converts into subcritical flow.

Subcritical channel with lateral inflow

- If the spillway incorporates a side-flow weir from where the water spills into a low gradient (subcritical) channel that flows in a direction 90-degrees to the weir, then:
  - the gradient of the water surface in this section of the channel will be steeper than that predicted by a standard backwater model
  - this means the flow conditions **cannot** be analysed by a one-dimensional numerical model, such as HecRas.

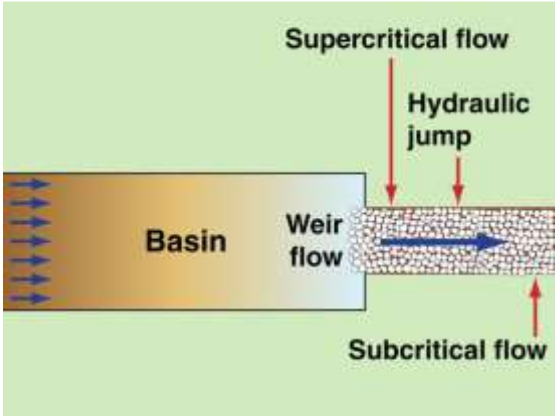
Energy dissipation at channel bends

- If the spillway incorporates a sudden change of direction, then:
  - this sudden change of direction should only occur in a region of subcritical flow (do not try to change the direction of supercritical flow)
  - this change of direction will generate an added energy loss that **cannot** be analysed by a one-dimensional numerical model, such as HecRas.

Subcritical discharge channel

- Eventually the flow will pass along a subcritical discharge channel.
- The flow conditions in these discharge channels can be easily analysed using a one-dimensional numerical model, such as HecRas.
- If the spillway is straight (not the case shown left), then the flow may pass through a hydraulic jump as the water moves from supercritical to subcritical flow conditions (as per top image).

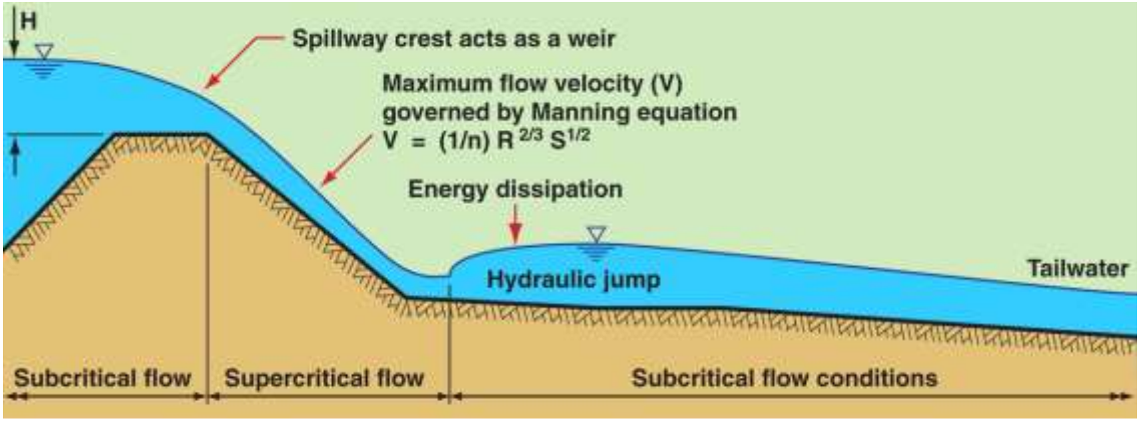
Alternative spillway layouts



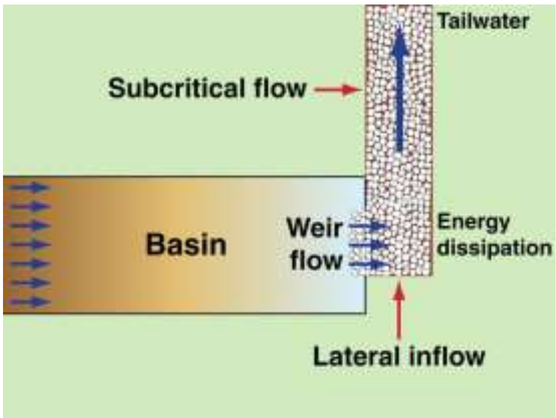
Spillway located at the end of the basin

Spillway layout 1 (preferred option)

- Optimum settling conditions are achieved within the sediment basin.
- Spillway typically consists of a steep chute (supercritical flow), followed by a low gradient channel (subcritical flow).
- Hydraulic analysis can be conducted using a one-dimensional hydraulic model, such as HecRas.



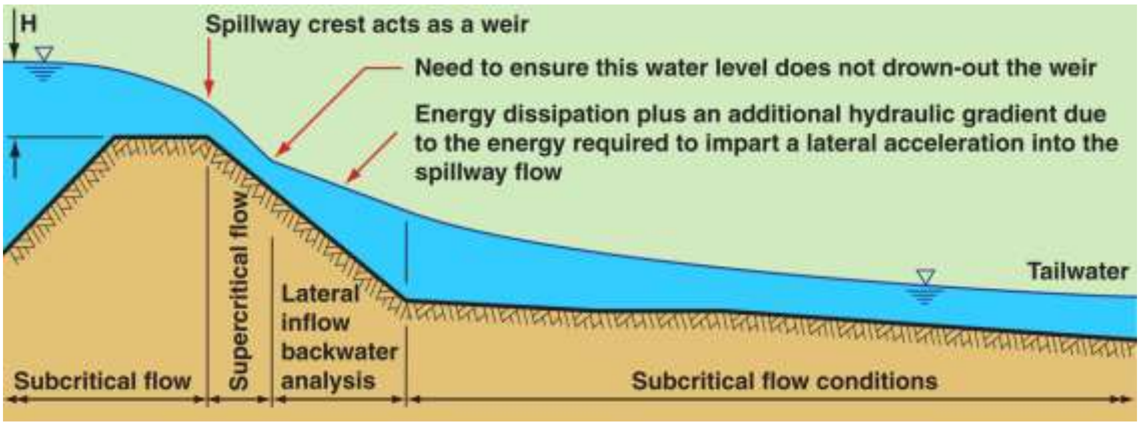
Components of the hydraulic gradeline for spillway layout 1



Spillway at end of basin

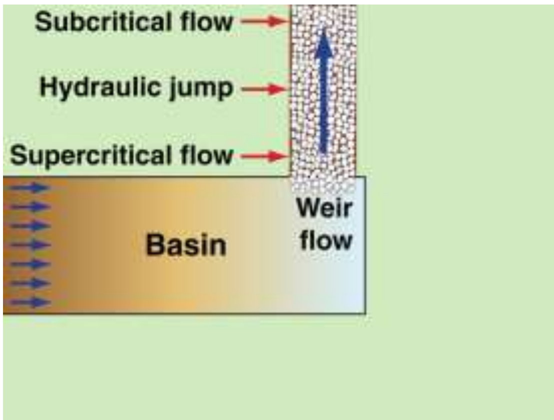
Spillway layout 2

- Optimum settling conditions achieved within the sediment basin.
- The lateral inflow of water into the energy dissipation area means that an additional hydraulic gradient will be required in this channel in order to impart a lateral acceleration into the water (i.e. get the water to turn 90-degrees and pick up velocity in a new direction).
- The hydraulic analysis **cannot** be completed using a one-dimensional numerical model, such as HecRas.



Components of the hydraulic gradeline for spillway layout 2

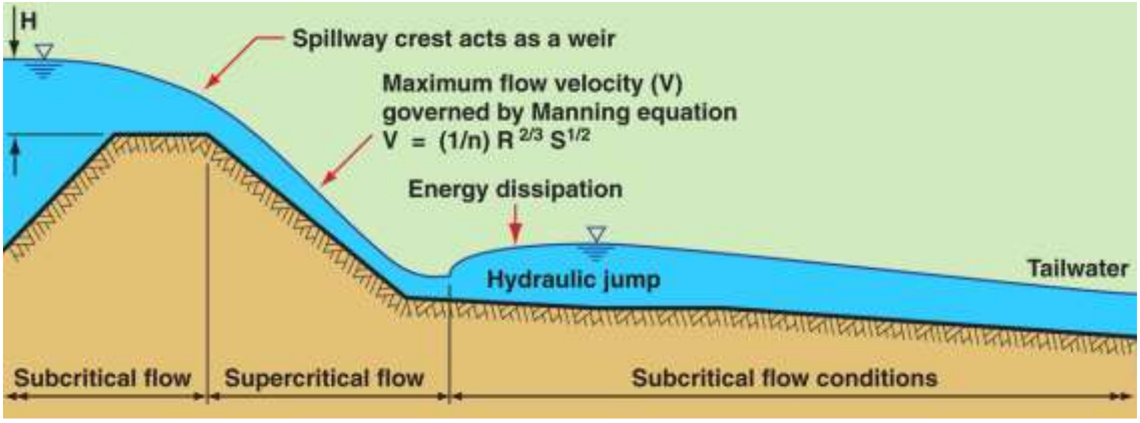
Alternative spillway layouts (continued)



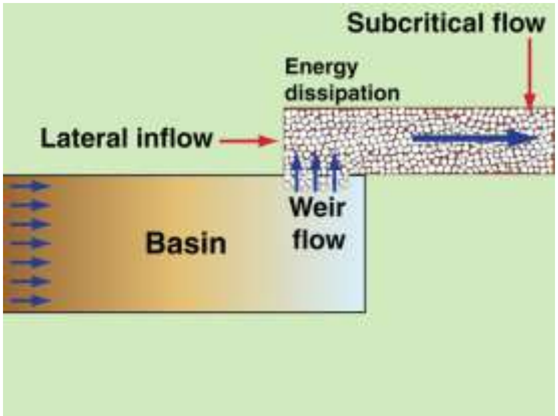
Spillway located on the side of the basin

Spillway layout 3

- Flow conditions within the basin will be compromised during major storms.
- Spillway should be located as far from the inflow points as possible.
- Spillway typically consists of a steep chute (supercritical flow), followed by a low gradient channel (subcritical flow).
- Hydraulic analysis can be conducted using a one-dimensional hydraulic model, such as HecRas.



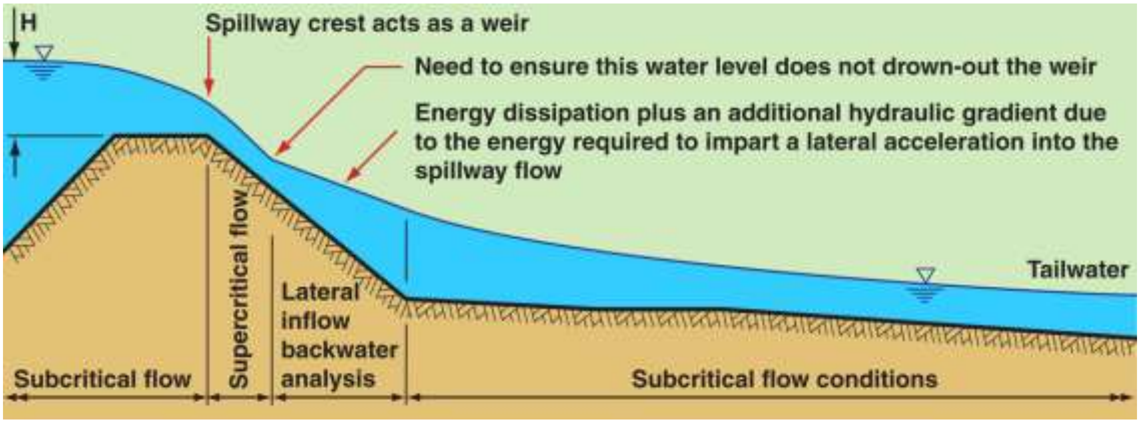
Components of the hydraulic gradeline for spillway layout 3 (same as layout 1)



Spillway located on the side of the basin

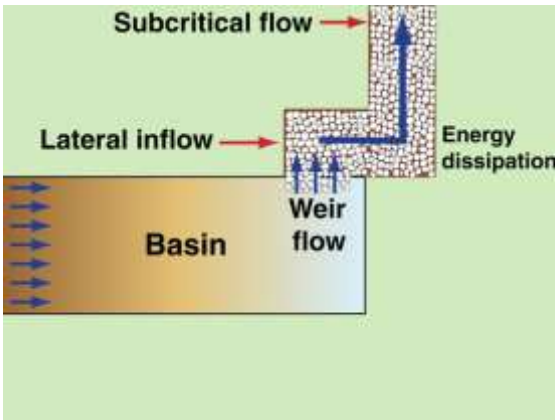
Spillway layout 4

- Flow conditions within the basin will be compromised during major storms.
- The lateral inflow of water into the energy dissipation area means that an additional hydraulic gradient will be required in this region in order to impart a lateral acceleration into the water.
- The analysis of this region **cannot** be completed using a 1-dimensional numerical model, such as HecRas.



Components of the hydraulic gradeline for spillway layout 4 (same as layout 2)

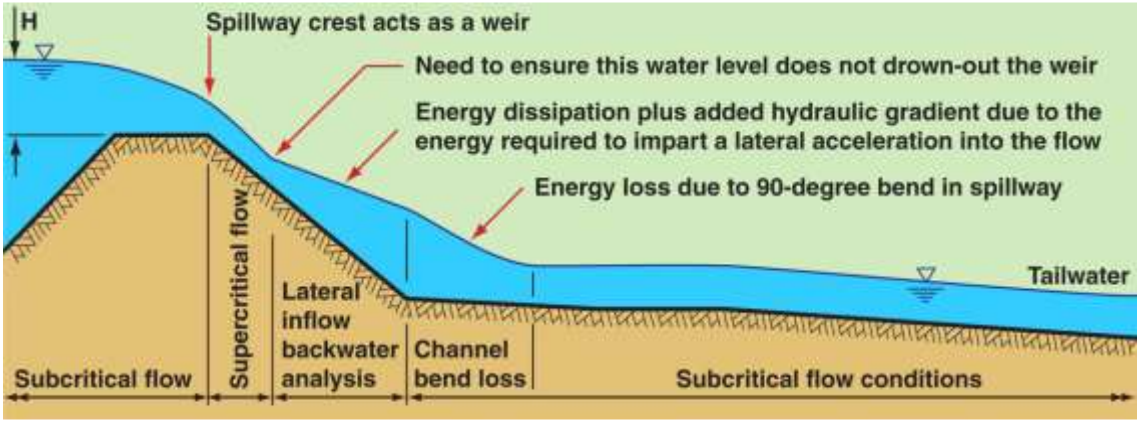
Alternative spillway layouts (continued)



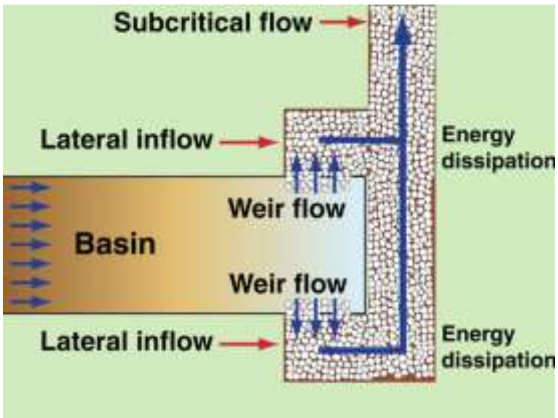
Spillway located on the side of the basin

Spillway layout 5

- Flow conditions within the basin will be compromised during major storms.
- The hydraulic gradient within the section of spillway subjected to lateral inflows cannot be analysed using a 1-dimensional numerical model, such as HecRas.
- The energy loss at the channel bend cannot be analysed using a 1-dimensional numerical model, such as HecRas—such models must be 'tricked' into giving a representative energy loss.



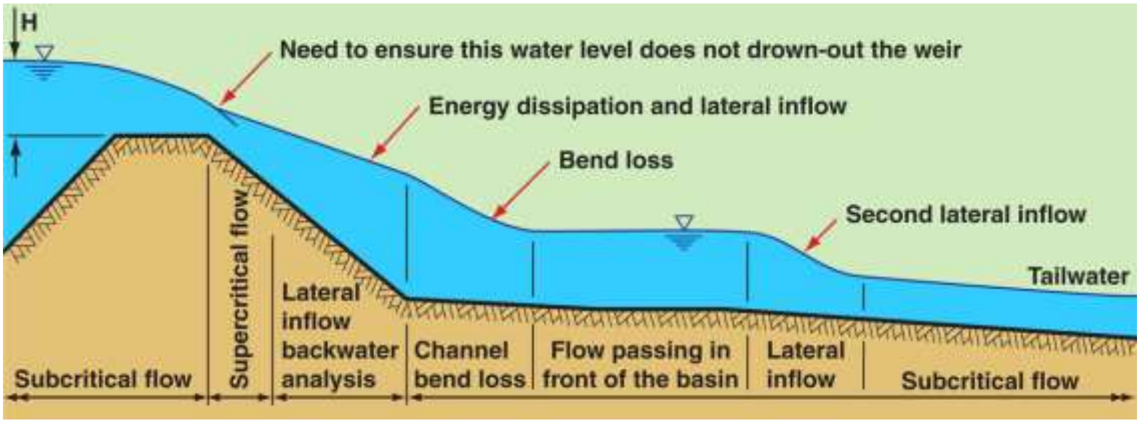
Components of the hydraulic gradeline for spillway layout 5



Dual side spillways

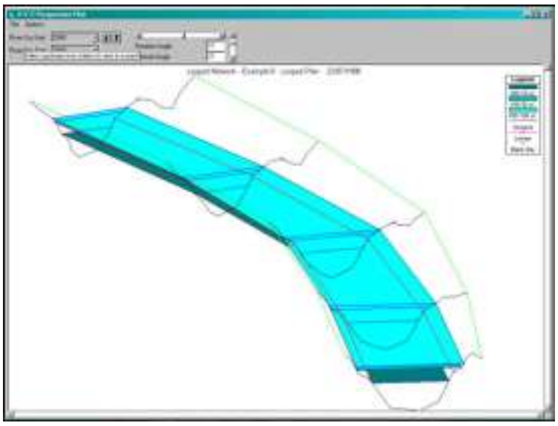
Spillway layout 6

- Flow conditions within the basin will be compromised during major storms.
- The hydraulic gradient within the two sections of lateral inflow cannot be analysed using a one-dimensional numerical model, such as HecRas.
- The energy loss at the channel bend cannot be analysed using a one-dimensional numerical model, such as HecRas—such models must be 'tricked' into giving a representative energy loss.



Components of the hydraulic gradeline for spillway layout 6

Understanding the limitations of one-dimensional numerical models



HecRas hydraulic analysis of a channel

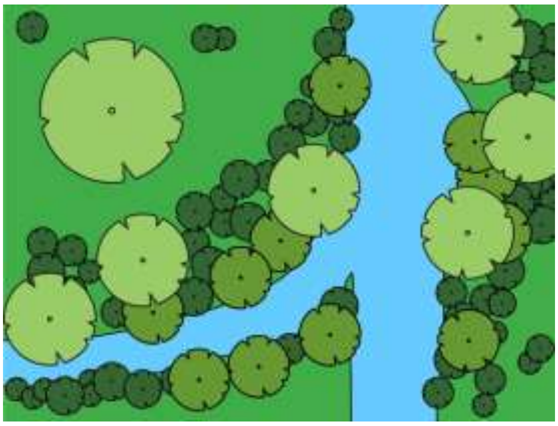
Introduction

- The hydraulic analysis of a sediment basin spillway is often performed using hand calculations or a simple one-dimensional (1D) numerical model, such as HecRas.
- What makes HecRas a 1D model is the fact that all flow is assumed to be moving in the same direction.
- The fact that the computer graphics can show a 3D channel, or that the model simulates both channel flow and floodplain flow, does <u>not</u> make the analysis a 2D simulation.

Failed sediment basin spillway

One-dimensional numerical models

- The first lesson about numerical models is: *They are <u>not</u> the sole measure of what is true in hydraulics, they are just a <u>simulation</u> of the hydraulic conditions.*
- The second lesson is: *If the model predicts the existence of non-scouring flow velocities, but in real life the soil continues to erode, then:*
  - *the erosion may be the result of excess sodium in the soil (i.e. dispersive); or*
  - *the model could be wrong!*



How a 1D model simulates lateral inflows

How 1D models manage energy loss induced by lateral inflows

- Because a 1D model assumes all flow is moving in the same direction, it also assumes that any lateral inflows blend smoothly with the main channel.
- Any energy loss resulting from two streams joining would be accounted for by the energy loss coefficient linked to changes in the velocity head.
- But this is <u>not</u> appropriate for lateral inflows which approach the main channel at 90-degrees.

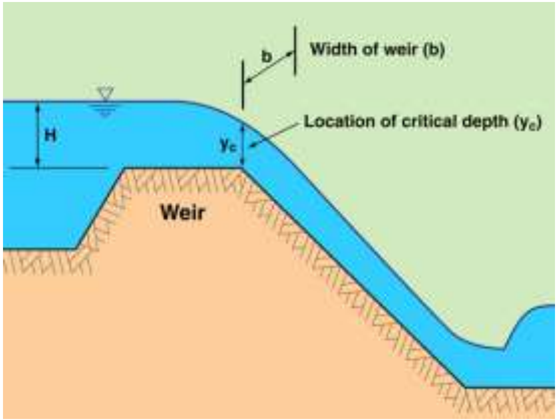
90-degree bend in a constructed channel

How one-dimensional models manage energy loss at channel bends

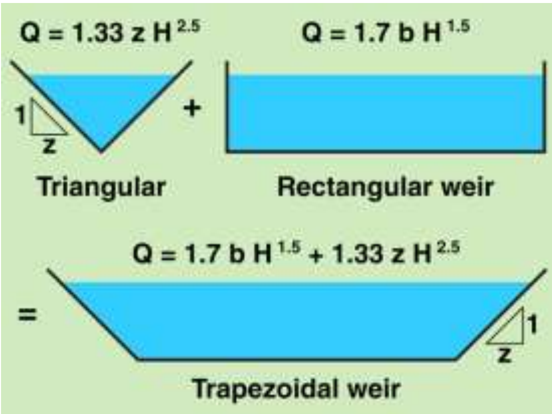
- One-dimensional models account for energy losses at the types of channel bends found in meandering river channels by increasing the Manning's roughness of the main channel.
- If an open channel contains a sharp bend, such as a 90-degree bend, then the model must be forced, or tricked, into introducing an appropriate energy loss.
- The model operator would be required to determine what this energy loss would be.

Weir flow conditions on sediment basin spillways

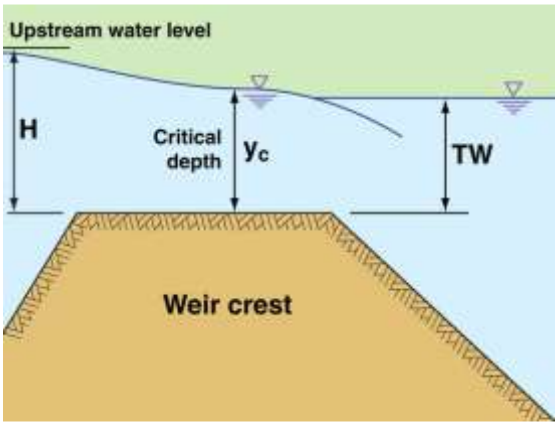
Crest of a rock-lined chute (USA)



Weir flow head (H) and width (b)



Components of a trapezoidal weir



A spillway weir close to being drowned

Introduction

- The face of the chute should have a constant cross-section from the weir crest to the energy dissipater.
- Flow conditions at the crest of a chute will be governed by a specific weir equation.
- The weir equation defines the relationship between the flow rate (Q), the width of the weir crest (b), and the upstream water depth relative to the weir crest (H).
- Manning's equation can then be used to calculate the maximum velocity down the chute.

Weir equation for a wide, flat, weir crest

- For wide chutes (say, b > 5H), it can be acceptable to adopt a simple rectangular weir equation, such as:

$$Q = 1.7\ b\ H^{1.5} \qquad (2.1)$$

where:

Q = total flow rate [m^3/s]

b = width of the weir crest [m]

H = upstream hydraulic head (water level) relative to the weir crest [m]

Weir equation for a trapezoidal weir crest

- For trapezoidal weirs, either use a numerical model, or analyse by combining the rectangular weir equation (above) with a triangular weir equation (below).
- The equation for a triangular weir is:

$$Q = 1.33\ z\ H^{2.5} \qquad (2.2)$$

where: z = the side slope (1 in z) of the trapezoidal weir.

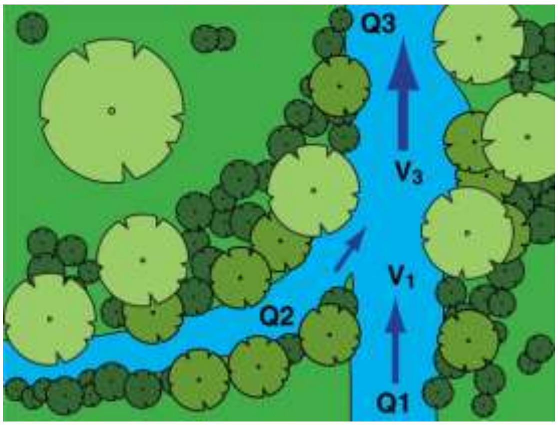
- Thus the trapezoidal weir equation is:

$$Q = 1.7\ b\ H^{1.5} + 1.33\ z\ H^{2.5} \qquad (2.3)$$

Drowned weir conditions

- If the elevation of the tailwater (relative to the weir crest, 'TW') is more than 0.8 times the upstream water level (relative to the weir crest, 'H'), then the weir is said to be partly or fully drowned (i.e. TW > 0.8H).
- This effectively means that in order to avoid drowning the weir, the water elevation within the downstream spillway chute needs to be below the water elevation associated with 'critical depth' (y_c) on the weir crest.

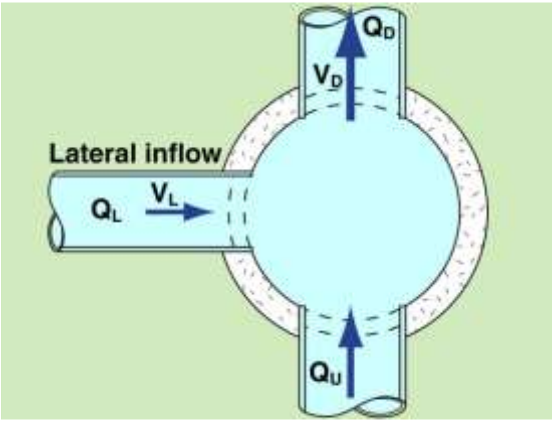
Analysis of open channels which have significant lateral inflow



How a 1D model simulates lateral inflows

Introduction

- In 1-dimensional numerical modelling it is assumed that lateral inflows:
  - blend smoothly with the main channel flow; and
  - the flows enter the main channel moving in the same flow direction as the main channel flow.
- This means the main channel does not need to impart significant energy into changing the direction of flow of the new inflow.



Stormwater pit with lateral inflow

Pipe flow analysis

- In the analysis of drainage pipes, a significant proportion of the energy loss that occurs within a junction pit is due to the energy required to accelerate the lateral inflow in the direction of the outlet pipe.
- It takes energy to accelerate flows.
- It takes even more energy to decelerate flows.
- It also takes energy to rapidly change the direction of flows.

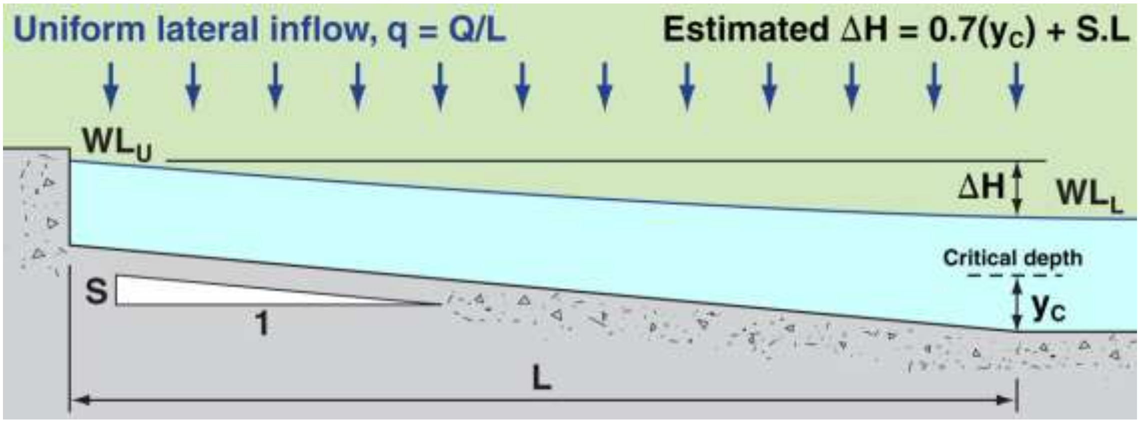
Lateral inflow spilling into a channel (Qld)

Hydraulic analysis of energy loss in channels with significant lateral inflow

- Hydraulic analysis:
  - determine the water level (WL_L) at the downstream (lower) limit of the inflow
  - determine the critical depth (y_C) at the downstream limit of the lateral inflow
  - an estimate of the upstream water level (WL_U) is given by:

$$WL_U = WL_L + 0.7(y_C) + S.L \qquad (2.4)$$

S = Bed slope (m/m), and **L** = bed length (m).



Hydraulic gradient within a subcritical channel subjected to lateral inflows

Table 2.1 – Geometric properties of channels

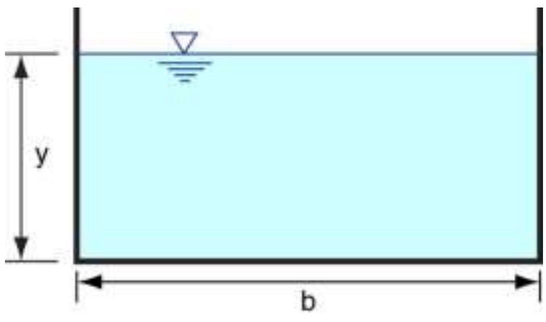
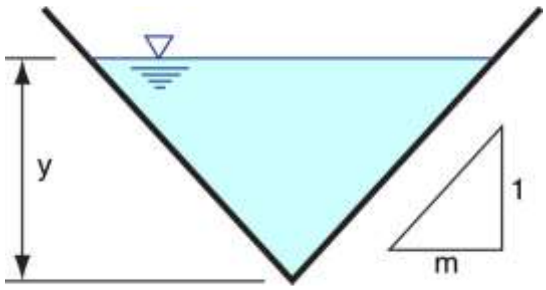
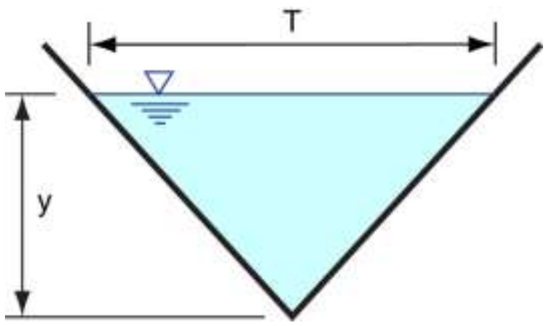
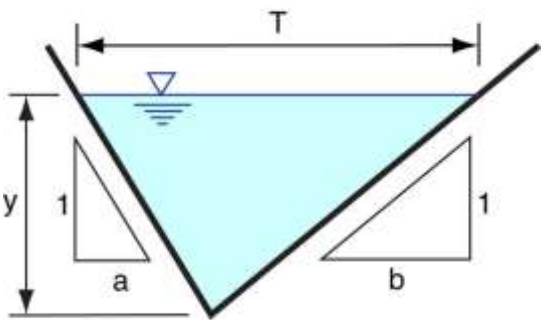
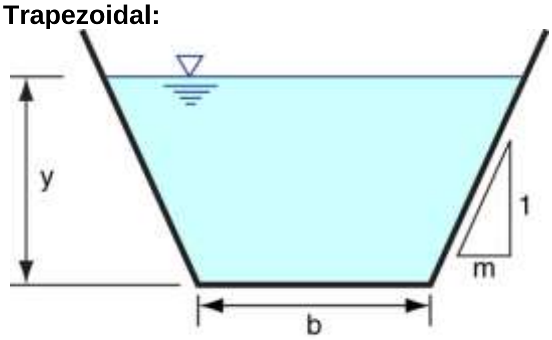
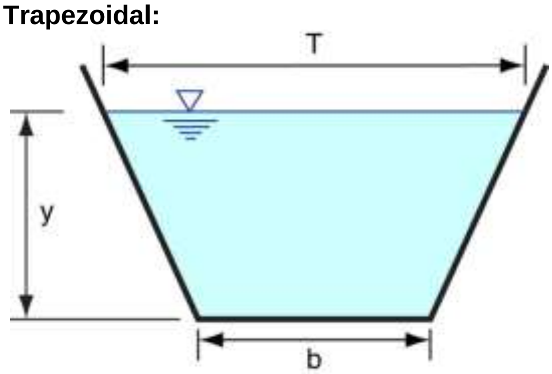
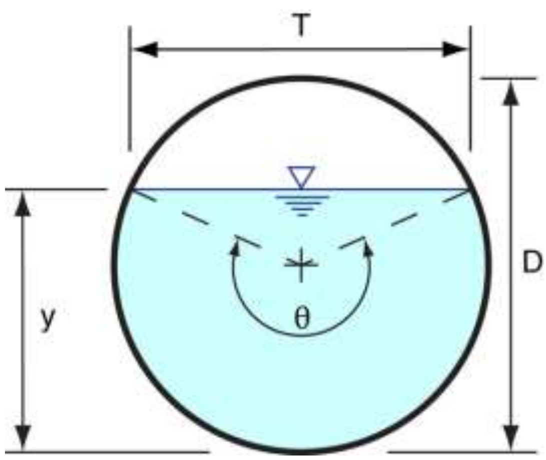
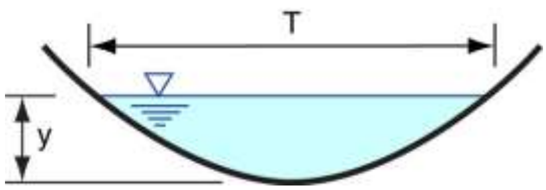
Rectangular: 	**Area (A):** $A = by$
	Wetted perimeter (P): $P = b + 2y$
	Critical depth (y_c): $y_c = (q^2/g)^{1/3} = 0.67(EL)$
Triangular (V-drain): 	**Area (A):** $A = my^2$
	Wetted perimeter (P): $P = 2y\sqrt{(1+m^2)}$
	Hydraulic radius (R): $R = \dfrac{my}{2\sqrt{(1+m^2)}}$
Triangular (V-drain): 	**Area (A):** $A = 0.5\,T\,y$
	Wetted perimeter (P): $P = \sqrt{T^2 + 4y^2}$
	Hydraulic radius (R): $R = \dfrac{Ty}{2\sqrt{T^2 + 4y^2}}$
Asymmetric Triangular (V-drain): 	**Area (A):** $A = \left(\dfrac{a+b}{2}\right)y^2$
	Wetted perimeter (P): $P = y\left[\sqrt{(1+a^2)} + \sqrt{(1+b^2)}\right]$
	Hydraulic radius (R): $R = \dfrac{0.5(a+b)y}{\sqrt{(1+a^2)} + \sqrt{(1+b^2)}}$

Table 2.2 – Geometric properties of channels

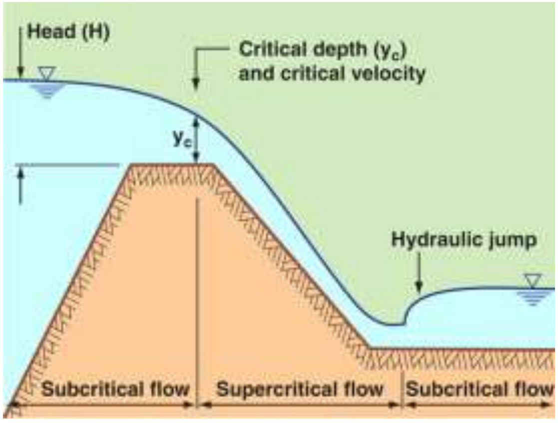
Trapezoidal: 	**Area (A):** $A = y(b+my)$
	Wetted perimeter (P): $P = b+2y\sqrt{(1+m^2)}$
	Hydraulic radius (R): $R = \dfrac{y(b+my)}{b+2y\sqrt{(1+m^2)}}$
Trapezoidal: 	**Area (A):** $A = 0.5y(T+b)$
	Wetted perimeter (P): $P = b+\sqrt{4y^2+(T-b)^2}$
	Critical depth (y_c): **$y = y_c$ when: $Q^2.T = g.A^3$**
Circular: 	**Area (A):** $A = \dfrac{D^2}{8}(\theta-\sin\theta)$
	Wetted perimeter (P): $P = 0.5(D.\theta)$
	Hydraulic radius (R): $R = \dfrac{D}{4}\left(1-\dfrac{\sin\theta}{\theta}\right)$
	Top width (T): $T = 2\sqrt{y(D-y)} = D.\sin\left(\dfrac{\theta}{2}\right)$ where: $\theta = 2\cos^{-1}\left(1-\dfrac{2y}{D}\right)$
Parabolic:  **Parabolic profile: y = constant(T^2)**	**Area (A):** $A = 0.67(T.y)$
	Wetted perimeter (P): $P = T+\dfrac{8y^2}{3T}$
	Hydraulic radius (R): $R = \dfrac{2T^2.y}{3T^2+8y^2}$

Hydraulic analysis of a hydraulic jump

Hydraulic jump flow conditions (USA)

Introduction

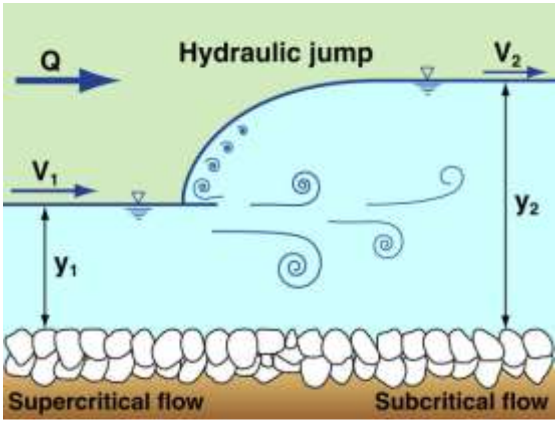
- A hydraulic jump (HJ) occurs when supercritical flow (typically at the base of a steep spillway chute) enters a low gradient channel and converts the flow back to subcritical flow conditions.
- Significant energy loss can occur within a hydraulic jump, and consequently their existence is often encouraged at the base of spillways to help dissipate energy.



Sediment basin spillway hydraulics

Location of a hydraulic jump (HJ)

- Diagrams often show the hydraulic jump located at the base of the spillway chute, but:
  - if the tailwater conditions are high, then the hydraulic jump can move up the spillway chute; alternatively
  - if the tailwater conditions are low, then the hydraulic jump can move well downstream of the chute.
- Designers often use 'impact blocks' or 'end sills' to force the HJ to stay in a location where there is good scour control.



Hydraulic jump terminology

The hydraulics of hydraulic jumps

- The hydraulic conditions upstream (y_1) and downstream (y_2) of a hydraulic jump can be determined through the use of a 1D numerical model; BUT, only if the flow is 1D, which means the HJ is not occurring at a channel bend.
- However, in many cases the hydraulics of a temporary basin spillway is not critically important, so it may be OK to approximate the flow conditions.
- If hand calculations are being used, then it should be OK to assume the spillway has a rectangular cross-section, which means in a straight, flat-bed, channel:

$$y_1 = 0.5\,(y_2)[(1 + 8(F_{R2})^2)^{1/2} - 1] \qquad (2.5)$$

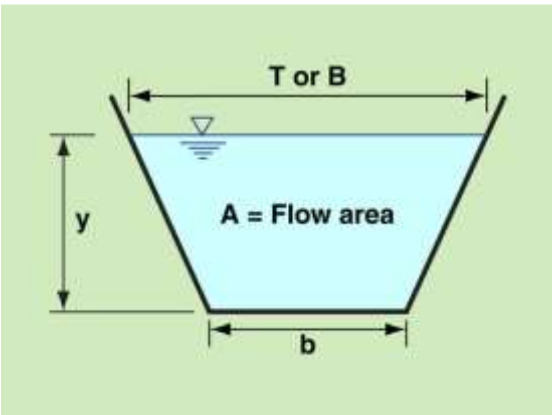
- If only upstream conditions are known:

$$y_2 = 0.5\,(y_1)[(1 + 8(F_{R1})^2)^{1/2} - 1] \qquad (2.6)$$

- Energy loss in a straight channel is:

$$\Delta H = (y_1 + V_1^2/2g) - (y_2 + V_2^2/2g) \qquad (2.7)$$

Froude No: $F_R = [(T.Q^2)/(g.A^3)]^{1/2}$ **(2.8)**

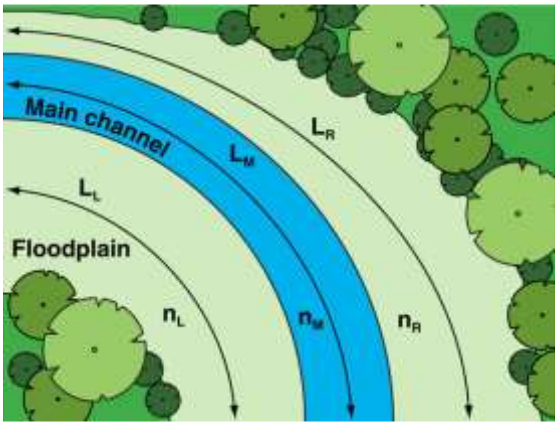


Spillway chute cross-section

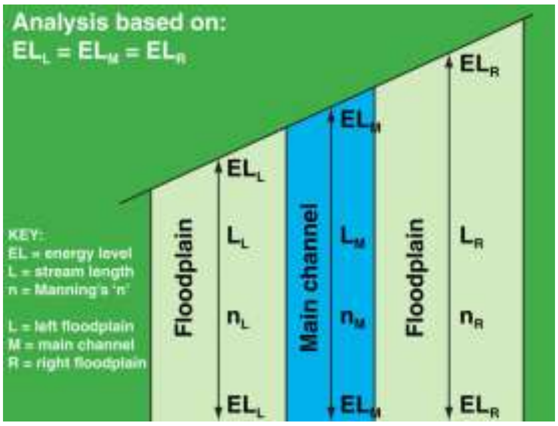
- If the HJ occurs at a sharp bend, then assume a total energy loss is given by:

$$\text{Total energy loss} = \Delta H + (V_2)^2/2g \qquad (2.9)$$

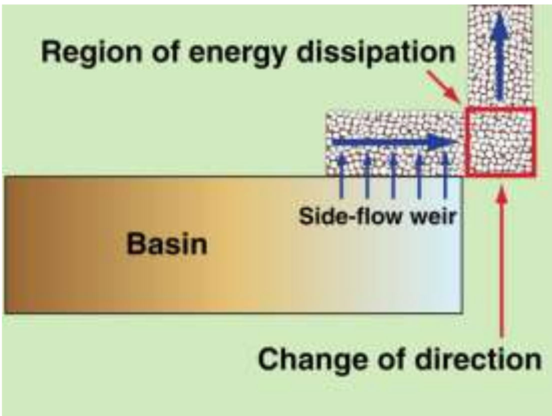
Hydraulic analysis at a sudden change of direction



How a 1D model 'displays' a channel bend



How a 1D model analyses a channel bend



Sudden change of direction

Basin spillway with two 90-degree bends

Introduction

- In a 1D numerical model, the energy loss at channel bends is often simulated by entering a different reach length and roughness values for the left overbank (L_L), main channel (L_M), and right overbank (L_R).
- In HecRas, the graphics package allows the user to draw a curved flow path on the computer screen, but this is not what happens inside the 1D numerical analysis.

The hydraulic analysis that actually occurs within a 1D model

- In the above situation of a channel bend, the mathematics that actually occurs within a 1D model is equivalent to that shown in this diagram (left).
- Mathematically, the 1D model simply models the left overbank, main channel, and right overbank as straight (1D) flow paths.
- The aim of the model is to ensure that the energy levels are equal at the beginning and end of all three flow paths.

Don't try to change the direction of supercritical flow

- When designing a sediment basin spillway there are a few essential rules that should be followed:
  - don't aim high-velocity water into a neighbouring property
  - don't forget to build an energy dissipater at the base of the chute, and
  - don't try to change the direction of supercritical flow—you need to first convert it to subcritical flow.

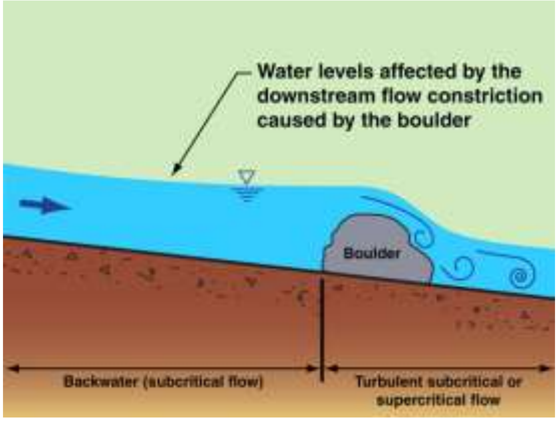
Assumed energy loss at a sharp bend

- The energy loss associated with a sharp bend in an open channel is based on:
  - partially-full flow in stormwater drainage systems; and
  - closed conduit flow systems (*Internal Flow Systems*, D.S.Miller, 1990).
- Energy loss for an isolated sharp bend:

$$\Delta H = (V_1)^2/2g \qquad (2.10)$$

V_1 = the approaching flow velocity, not the exit velocity (V_2) which is used if the bend is associated with a hydraulic jump.

Hydraulic analysis of the subcritical discharge channel



Example of backwater effects

Subcritical flow in an urban creek (Qld)

Debris blockage of a property fence (Qld)

High tide backing-up into a coastal drain

Introduction

- The term 'backwater' has two meanings.
- It can mean a region of a channel or floodplain where there is no measurable flow velocity, which means water levels will be totally controlled by downstream conditions.
- The term is also used in hydraulic engineering to refer to the subcritical flow conditions that exist when water levels and flow velocities are strongly influenced by the channel conditions downstream of the point of interest.

The importance of subcritical flow in the design of sediment basin spillways

- Most sediment basin spillways will eventually discharge into:
  - a street
  - a discharge channel; or
  - a creek or gully.
- In each case the flow conditions within these receiving water bodies will likely be subcritical.

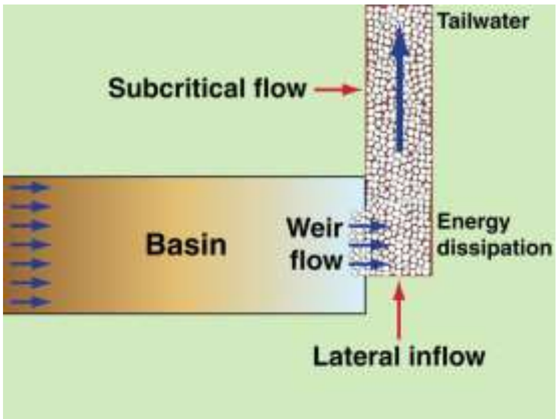
The backwater effects of property fencing

- If the discharge channel passes through a property fence, such as a security fence surrounding a construction site, then it can be very difficult to determine the backwater effects due to the potential impacts of debris blockages.
- Professional judgement is required on a case-by-case basis.

Flood and tide conditions

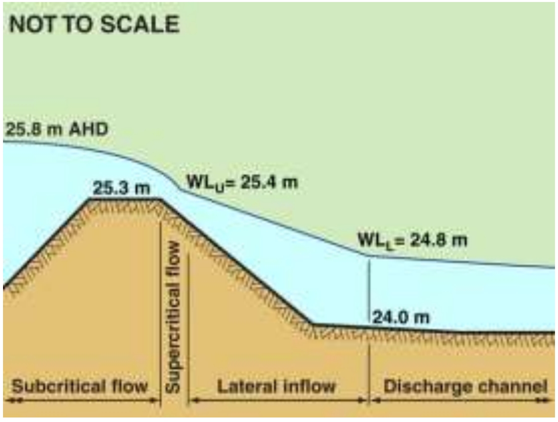
- Accounting for the effects of local flooding and tides is done through the consideration of 'coincident flooding'.
- The severity of this coincident flooding will depend on the ratio of the time of concentration (t_c) of the spillway discharge to that of the receiving waterway.
- Suggested analytical procedures are provided in Chapter 8 of the Queensland Urban Drainage Manual (QUDM).

Worked example 1: Hydraulic analysis of Spillway Layout No. 2

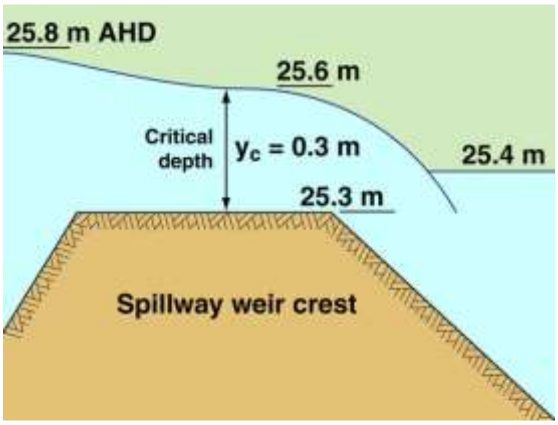


Spillway at end of basin

Flow must turn 90-degrees at base of spillway



Centreline of spillway and discharge channel



Flow conditions at the spillway weir

1. Tailwater level (TWL)

- In this case, supercritical flow only occurs as water initially spills out of the basin.
- The discharge channel is subcritical.
- The hydraulic analysis will be based on a backwater analysis starting at the TWL.
- At this site it has been determined that the tailwater level at the downstream end of the discharge channel during maximum design discharge (Q) is:

TWL = 24.0 m (AHD)

AHD means: Australian Height Datum

2. Hydraulic analysis along the channel

- The most common way to perform a backwater analysis along the discharge channel is to establish a HecRas model.
- In some cases it may be acceptable to use Manning's equation; in this case let the fall in the channel bed and the water level be 0.8 metres.

$WL_L = 24.0 + 0.8 = 24.8$ m (AHD)

- However, a HecRas model cannot 'accurately' simulate the complex 3D conditions at the base of the spillway.

3. Flow conditions at the base of the main spillway

- Lateral inflow conditions exist at the base of the main spillway.
- Downstream of the lateral inflow: bed level = 24.0 m; flow depth = 0.8 m; water level (WL_L) = 24.8 m (as above).
- Critical depth (y_C) for the water flowing along the discharge channel must be calculated based on the geometry of the channel, in this case assume:
- Critical depth: $y_C = 0.5$ m (calculated).
- Equation 2.4 gives an estimate of the water level (WL_U) at the upstream end of the lateral inflow.

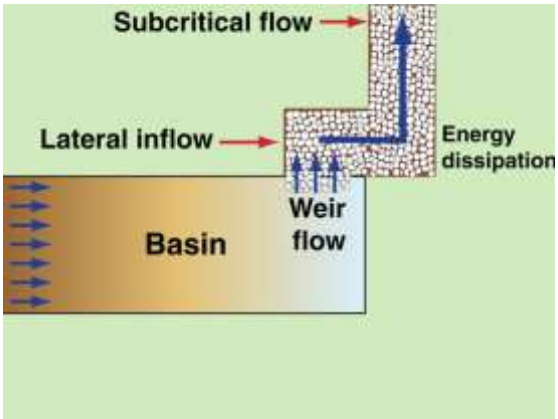
$$WL_U = WL_L + 0.7(y_C) + S.L$$

$$WL_U = 24.8 + 0.7(0.5) + 0.2 = 25.4 \text{ m}$$

where: S.L = fall in bed level over the section of channel affected by the lateral inflow.

- Therefore, the embankment that contains the flow in the discharge channel must have an elevation > 25.4 m (plus freeboard), <u>and</u> critical depth on the spillway crest must be higher than 25.4 m (AHD) otherwise the weir will be partially drowned.

Worked example 2: Hydraulic analysis of Spillway Layout No. 5



Spillway located on the side of the basin

1. Tailwater level (TWL)

- The hydraulic analysis will be based on a backwater analysis starting at the TWL.
- At this site it has been determined that the tailwater level at the downstream end of the spillway's channel, during the maximum design discharge (Q) is:

TWL = **WL_1** = 10.4 m (AHD)

AHD means: Australian Height Datum

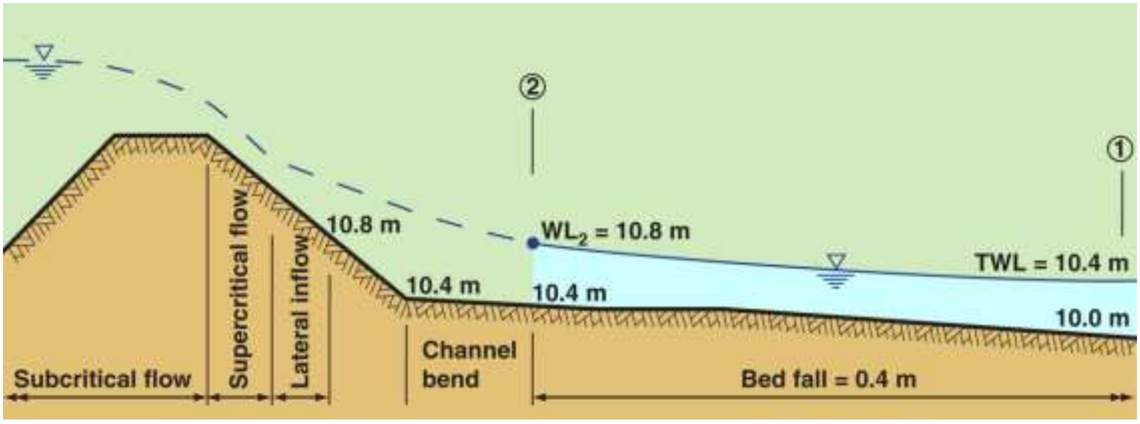
Basin spillway with two 90-degree bends

2. Backwater flow analysis along the subcritical discharge channel

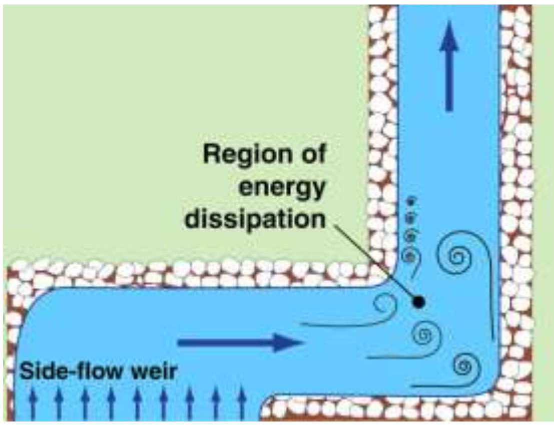
- The most common way to perform a backwater analysis along the discharge channel is to establish a HecRas model.
- In some cases it may be acceptable to use Manning's equation; in this case let the fall in the channel bed slope and the water level be 0.4 metres.

WL_2 = 10.4 + 0.4 = 10.8 m (AHD)

- However, a 1D model cannot accurately simulate the complex 3D conditions.



Adopted bed levels along the spillway chute and discharge channel for Layout 5



90-degree bend in a spillway channel

3. Energy loss associated with sharp bend

- Downstream of the bend: bed level = 10.4 m (AHD); y_2 = 0.4 m; V_2 = 1.5 m/s.

$EL_2 = 10.4 + 0.4 + (1.5)^2/2g = 10.91$ m

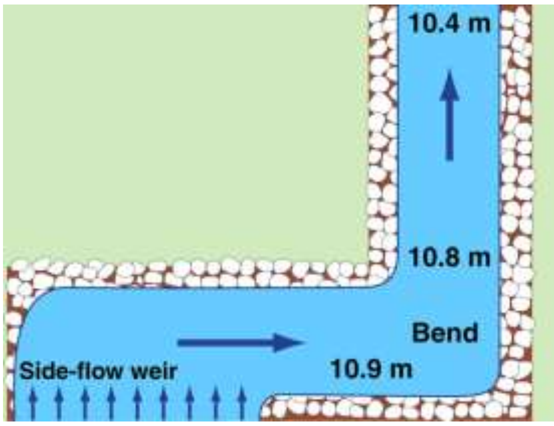
- As a first guess: assume the flow velocity upstream of the bend: V_3 = 1.2 m/s.
- Based on Equation 2.10, assume the energy loss caused by the bend is:

$\Delta H = (V_U)^2/2g = (1.2)^2/2g = 0.07$ m

- Therefore the assumed energy level upstream of the bend is:

First guess: $EL_3 = 10.91 + 0.07 = 10.98$ m

Worked example 2: Hydraulic analysis of Spillway Layout 5 (continued)



Water level upstream of channel bend

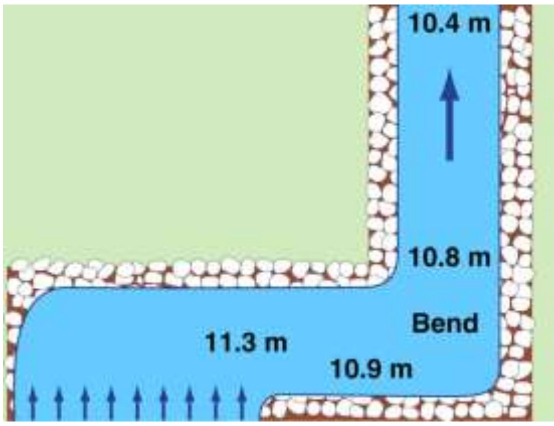
4. Check on the first guess of the flow conditions upstream of the bend

- Our first guess of the flow conditions upstream of the 90-degree channel bend gave: bed level = 10.4 m (AHD); y_3 = 0.5 m; V_3 = 1.2 m/s, thus:

$$EL_3 = 10.4 + 0.5 + (1.2)^2/2g = 10.99 \text{ m.}$$

- This energy level is close enough to our first guess, so we can assume the water level immediately upstream of the channel bend is:

$$WL_3 = 10.4 + 0.5 = 10.9 \text{ m (AHD)}$$



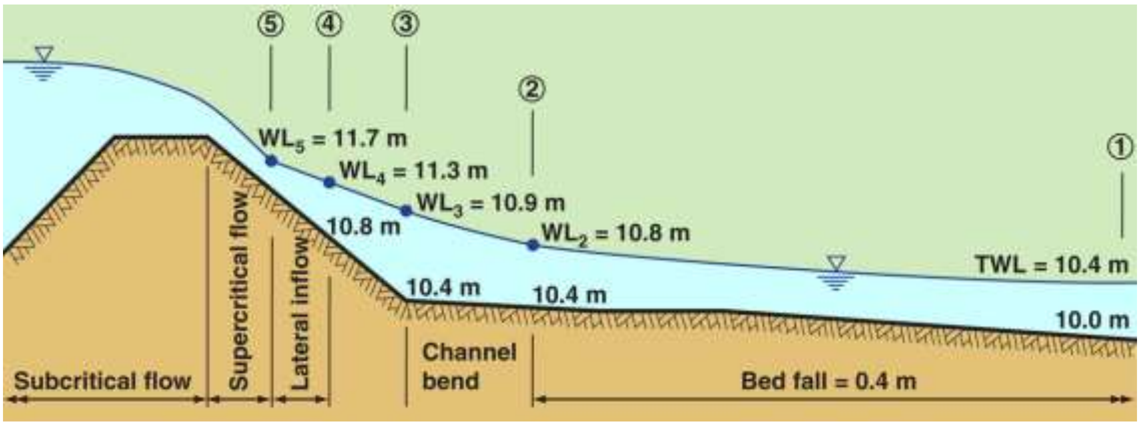
Water at downstream end of lateral inflow

5. Water level at the downstream end of the lateral inflow (WL_4)

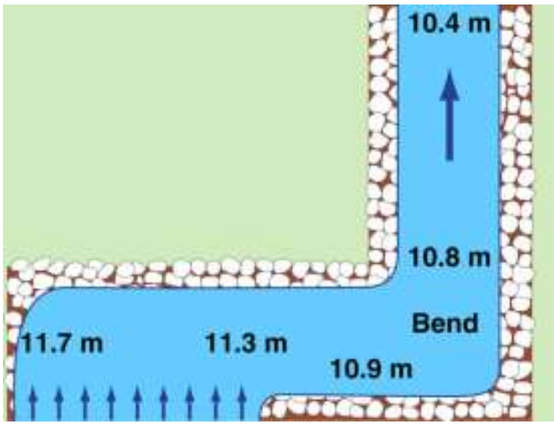
- Now the fall in bed level (and the assumed fall in water level) between the downstream end of the lateral inflow and the channel bed is 0.4 m, thus:

$$WL_4 = 10.9 + 0.4 = 11.3 \text{ m}$$

- This fall in water level can be checked using a 1-dimensional numerical model.



Water levels along the spillway chute and discharge channel for Layout 5



Water at upstream end of lateral inflow

6. Water level at the upstream end of the lateral inflow (WL_5)

- Critical depth (y_C) in the spillway channel is based on the channel geometry at the downstream end of the lateral inflow.
- Critical depth: y_C = 0.3 m (calculated).
- Equation 2.4 gives an estimate of the water level (WL_5) at the upstream end of the lateral inflow.

$$WL_5 = WL_4 + 0.7(y_C) + S.L$$

$$WL_5 = 11.3 + 0.7(0.3) + 0.2 = 11.7 \text{ m}$$

- Where S.L = fall in bed level = 0.2 m.