Section 4 Detailed device description and design procedures

In this section:

Detailed descriptions and designs for:

4.1 Filter

4.2 Infiltration trench

4.3 Rain garden

4.4 Stormwater planter

4.5 Rain tank

4.6 Swale / filter strip

4.7 Wetland

References

For each device:

- description
- capability
- applicability
- design approach
- design steps
- design detailing
- implementation provisions
- operation and maintenance
- worked example

The format of the step-by-step design procedures is:

- **device description**: explains the form of the device (with variants where applicable) and how it works, with the aid of diagrams and/or photos
- **capability**: treatment performance and other capability
- **applicability**: sets out the situations where the subject device can/cannot be used, with guidance notes on some of the key technical and/or operation and maintenance issues that should be accounted for before proceeding to the site-specific design of the device
- **summary of design approach**: summarises the steps involved
- **preparatory steps**: lists the site-specific information that will be needed to prepare the design and any related information
- **step-by-step design procedure**: explains the steps in the design procedure (note that this is kept quite succinct, with cross-referencing used to direct users to explanatory material where they need further guidance)
- **design detailing**: sets out the standard details to be shown in design drawings, with attendant guidance notes on alternatives where applicable
- **implementation provisions**: summarises key issues in implementing the specific device such as consenting, construction, operation and maintenance
- **references**: lists reference material providing general guidance, precedents, worked examples
- **worked example(s)**: numerical example(s) to illustrate use of the design procedure in typical applications

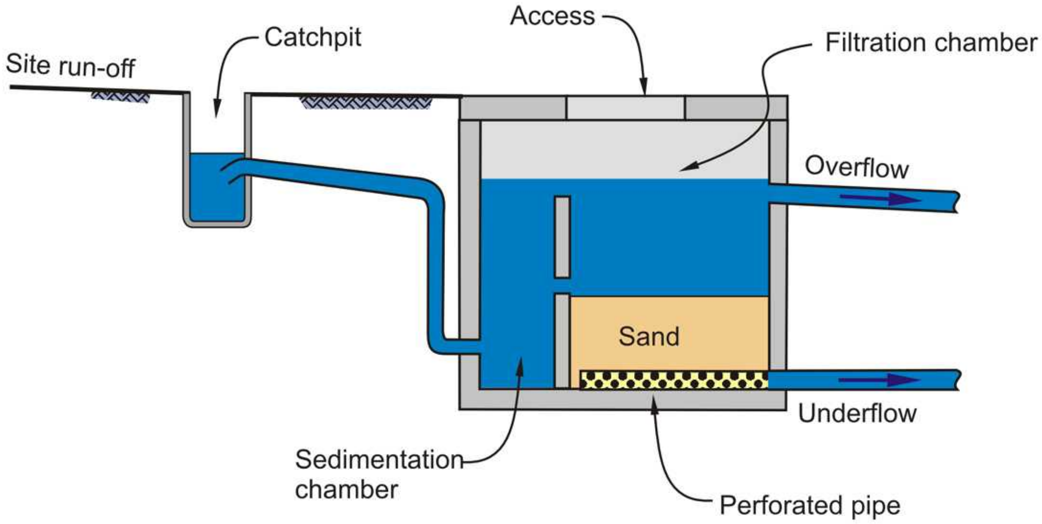
In using the design procedures, users should also note that the device selection and design should follow the steps in the flow chart in Figure 3.1.

4.1 Filter

4.1.1 Description

Filters are structures in which a bed of material such as sand traps and accumulates contaminants. Filters can include those with inert media in which only particulate pollutants are removed and those with absorptive media, which remove dissolved contaminants. Filters usually include a sedimentation unit to reduce sediment loads to the filter.

Figure 4.1.1 Filter operating principles



4.1.2 Capability

Filters are able to:

- treat runoff from impermeable hardstand ground surfaces in commercial, residential and industrial areas
- treat road or parking lot runoff

Filters are not able to:

- treat sediment-laden water from construction sites. Install after site works are complete and contributing areas have been fully stabilised in order to prevent excess sediment loading
- provide significant peak flow or volume control

The most common filter is the sand filter.

Expected contaminant removal rates for sand filters are (ARC TP10, EPA 1999a):

- suspended solids > 75%
- metals (copper, zinc, lead) (total) > 75 %
- total phosphorus 33 %
- total nitrogen 21%
- biochemical oxygen demand 70%
- hydrocarbons >75%

Filters other than sand filters include filters that use standard sand filter type hydraulic design but modify or replace the sand with other media such as:

- iron oxide coated sand
- iron wool
- polypropylene fabric
- leaf compost
- peat
- sphagnum moss
- limestone
- waste wood fibre
- bottom ash
- perlite
- zeolite
- iron oxide coated sand
- granular polymer
- iron amended resin
- proprietary filters with a variety of media, which can treat a variety of contaminants both particulate and dissolved

For other media, references are in work by Landcare Research (Reducing road runoff contaminants through low-cost treatment wall (filter) systems: Landcare Research studies (Surya Pandey pers. comm.), summarised below.

Territorial and regional authorities in New Zealand have identified stormwater management as a priority environmental issue in urban areas, with increasing attention being paid to the use of various filter systems to reduce the contaminant load in road runoff. In many cases, the effective application of such systems requires the development of improved filtration media, design and operational parameters (e.g. frequency of sediment or medium removal) to align construction, performance and maintenance to specific guidelines, such as those for stormwater interception devices as suggested by Auckland Regional Council in TP10.

Under laboratory conditions, Landcare Research examined five media that may be suitable as a medium in treatment walls through their ability to remove the heavy metals copper (Cu), lead (Pb), zinc (Zn), and also selected polyaromatic hydrocarbons (PAH) (fluoranthene and pyrene) from artificial road run-off. The media tested were commercially available *sphagnum moss*, crushed limestone, waste wood pulp, wood ash, and waste wool felt. Two media, sphagnum/lime and sphagnum/wood ash in layered (1 layer of each) and mixed configurations, containing 10% by weight of *sphagnum*, were also tested.

The individual, mixed and layered media were ranked according to their contaminant removal efficiency, 1 being the best performance (Table 1). The best-performed medium over the 5 contaminants studied; presence of PAH degrader; and hydraulic conductivity is given by the lowest total score. The best-performed media overall were lime, wood ash and the mixed *sphagnum*/wood ash combination.

Table 4.1 Ranked treatment matrix

Note: 1 is the best performing: lowest total is best performing overall

Medium	Copper	Lead	Zinc	Fluoranthene	Pyrene	PAH degraders	Hydraulic conductivity	Total
Sphagnum	1	3	1	2	2	1	5*	15
Lime	1	1	1	1	1	3	3	11
Wood Fibre	2	4	4	3	2	3	6	24
Wood Ash	1	1	1	1	1	2	2	9
Sphagnum/Ash mixed	1	2	1	1	1	1	1	8
Sphagnum/lime mixed	3	2	1	1	1	1	3	12
Sphagnum/Ash layered	1	4	3	1	1	1	4	15
Sphagnum/lime layered	1	3	2	1	1	1	4	13

Although sphagnum had the highest hydraulic conductivity, the use as filter media on its own will be limited due to very small contact time between dissolved pollutants and sphagnum, hence the higher ranking. Based on the above results, the *sphagnum*/wood ash media (1:1 by volume, 1:10 by weight) was chosen for field-testing. A treatment wall/filter was constructed at the corner of River Road and Wairere Drive in Hamilton in December 2000, to intercept the run-off from a portion of a roundabout. Subsequently, an additional wall was constructed in Cambridge on the side of State Highway 1. In contrast to the Hamilton trial, we are testing an increased ratio of *sphagnum* (20% by weight).

Comparison of input and output pollutants through the treatment walls show that both treatment walls greatly reduce the quantities of pollutants being discharged into the aquatic environment.

Landcare Research also determined the types and amounts of contaminants (Cu, Pb, Zn, fluoranthene, pyrene, and suspended solids) removed from stormwater during typical storm events from the Henderson aquatic centre car park in Waitakere City. We tested wood-ash, sand, and green-waste compost as filter media in a filtration system designed to standard TP10 filtration criteria. The results indicated that wood-ash was the most effective medium, removing more Cu and Zn than the compost, or the sand filter medium. All three media removed fluoranthene and pyrene. A ranked treatment matrix for the media tested is presented in Table 2 below. A ranking of 1 indicates the best overall performance for the removal of the contaminant indicated.

Table 4.2 Overall ranked treatment matrix for filter media

Note: 1 is the highest overall performance

Media	Cu	Pb	New Zealand	Fluoranthene	Pyrene
Wood-ash	1	1=	1	1	2=
Sand	2	1=	2	2=	1
Compost	3	2	3	2=	2=

In May 2003 a fourth treatment wall was constructed at the Hewletts Rd/Tasman Quay roundabout at the entrance to the port in Tauranga, to intercept the runoff from a portion of a roundabout. The treatment wall consists of a *sphagnum* “basket” on top of 300 mm wood ash housed in a shallow rectangular tank (0.5 m deep by 1 m wide by 4 m long). This study is continuing and initial results are similar to those found at other study sites in New Zealand.

Additional references for filter media are:

- discussion of sorptive media filtration in Minton, 2002
- discussion of sorbent materials for removal of hydrocarbons in stormwater applications (EPA 2002)

4.1.3 Applicability

- specific applications include:
  - commercial and industrial parking areas or yards
  - service stations
  - high density residential housing
- on line or off line location
- suitable for retrofits
- can be constructed completely underground with surface access lids or can be constructed using a pond or other structure that is open at the surface
- device catchment area no more than 4 ha (ARC TP10)
- New Zealand suppliers of proprietary filters include:
  - Hynds Environmental Systems
  - Ingal Environmental

Care is needed if using media other than sand for which design methodologies have been well established. In such situations assessment of long term permeabilities or allowance for reduction in permeability with time should be addressed.

The use of compost or similar materials should consider the possibility of viral or bacterial contamination from the compost.

4.1.4 Summary of design approach

1. Determine the nature of contaminants to be removed, including whether particulate or dissolved, and determine the type of filter required, i.e. sand or modified type of sand filter or proprietary filter
2. Calculate water quality volume or other parameters if required for sizing a proprietary filter
3. Size the filter per appropriate method. The design method for a sand filter is set out below. Design of proprietary filters as to the supplier’s recommendation

4.1.5 Preparatory steps

1. Confirm quality objective: refer section 3.6
2. Define key site parameters and device needs that determine design details:
    - device catchment land use (this is required to be used in design calculations)
    - device catchment impervious area (roof and on-ground areas)
    - device catchment pervious area and cover type (e.g. grass, shrubs, forest). This should be minimal or zero
    - adequate hydraulic head between entry and discharge from the filter
    - location of filter:
        - clearance to services and boundaries
        - subsoil materials and costs for excavation (beware of rock)
        - water table to be below base of filter
        - access for maintenance
    - define maximum flow capacity requirements for the area to be drained and locate overland flow paths for flows in excess of the capacity of the swale/filter strip
    - check any regional, city or district council requirements for resource consent, building consent or drainage permit or compliance with other standards

4.1.6 Design steps

4.1.6.1 Sizing for water quality design

Sand filter (and similar types) design parameters:

- determine the water quality volume (refer to section 3.6)
- choose media type and sizing

For sand, ARC TP10 specifies sand size as:

Sieve size (mm)	Percentage passing
9.5	100
6.3	95-100
3.17	80-100
1.5	50-85
0.8	25-60
0.5	10-30
0.25	2-10

For sand that complies with the above or is close to compliance, a permeability (k) value of 1 m per day is used in design. If other media are used, or mixtures of sand with other media are used, the permeability should be carefully assessed and a conservative value used for filter design.

Design/sizing methodology (refer to Figure 4.1.2):

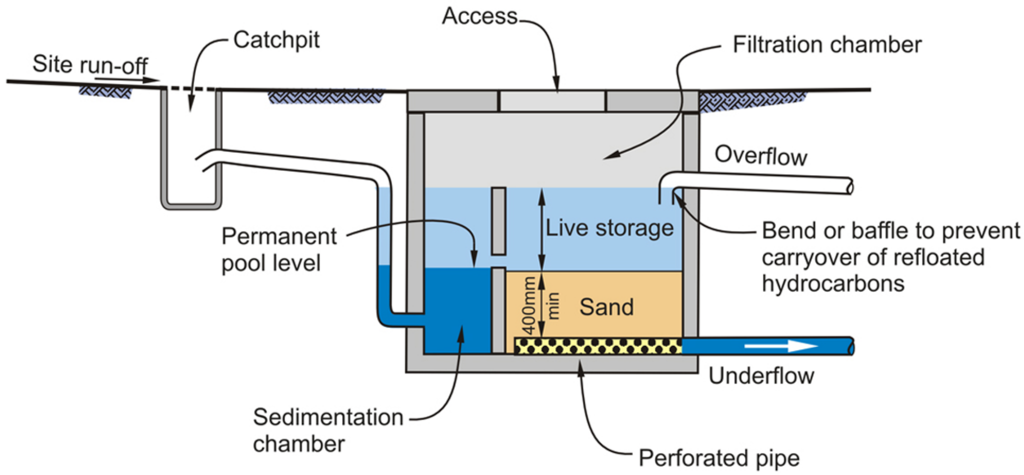
Sand filtration chamber to be sized using the equation:

$$A = WQV \times d / [k \times (h+d) \times t]$$

Where:

- A = surface area of the sand bed
- WQV = water quality volume
- d = depth of sand = 0.4 m minimum
- k = permeability of the sand or other media in m per day
- h = average depth of water during the WQV storm above the surface of the sand in metres, assume to be half the maximum depth
- t = time required for runoff to pass through the filter, in days. This relates to the inter-event period. ARC TP10 requires this to be a maximum of 2 days for the Auckland area. It is suggested that this is used as a default value throughout New Zealand, unless more specific local guidance is available

Figure 4.1.2 Filter detailed design



Provide adequate live storage. The live storage includes the water above the top of the sand in the filtration chamber together with the volume of water in the sedimentation chamber and any associated chambers or pipes that is above the permanent pool level but below the overflow level. Live storage determines the overall performance of the filter, i.e. the total amount of runoff it will treat, so should be maximised (within economical limits). Live storage can be maximised by installing additional separate chambers upstream of the filter. Pipes discharging to the filter can also be utilised to provide additional live storage, subject to suitable geometry and levels. Where peak flow control or extended detention is required, detention tanks can be incorporated before the filtration chamber to provide further live storage and possibly act as sedimentation chambers. The minimum live volume required in ARC TP10 is 37% of the WQV, based on modelling of Auckland conditions. This guideline recommends a minimum live volume of 37% of the WQV unless analysis of local rainfall records and other conditions indicate a larger live volume should be used.

Check that the area of the sedimentation chamber is at least 25% of the filtration area.

Flow velocities in the sedimentation chamber must be less than 0.25 m/s to avoid re-suspension of sediment.

The sedimentation chamber must have a permanent pool with a minimum depth of 0.4 m to reduce re-suspension of trapped sediments.

4.1.7 Design detailing and drawings

Inlet and overflow bypass

- provide flow bypass when live storage is completely utilised; it is better to pass excess flow through the filter chamber than bypassing it before the filter, providing re-suspension of sediment can be avoided
- If inlet flows drop some distance into the sediment chamber, provide energy dissipation before the sediment chamber to avoid re-suspension of sediment

Sedimentation chamber

- configure to avoid short-circuiting of the flow, by using a long narrow pool or tank, the use of baffles to lengthen the flow path and/or provide flow resistance at the inlet

Flow from sedimentation chamber to filter chamber

- design the transfer structure to avoid velocities that will scour the filter bed, using baffles and erosion protection, if necessary, where the flow enters the filter compartment

Access

- provide surface access to sedimentation chamber to allow removal of sediment;
- provide access to filter chamber to allow removal of accumulated material on filter surface

Underdrainage

The filter chamber must have an underdrainage system which can be:

- horizontal perforated pipes in a clean gravel layer or pocket covered with filter cloth
- horizontal perforated pipes covered with filter fabric
- proprietary rectangular drainage product incorporating filter fabric cover

Filter fabric to be chosen and underdrainage system sized and designed to:

- allow maximum filtered flow to pass through with negligible head loss
- pore size suitable to retain sand
- robust fixing of the edges of the filter fabric to prevent short circuiting of sand or water around the edges

Collector pipe system

- sized to pass the design filter flow at the pipe gradient
- provide for flushing of collector pipes
- slope of pipes exceeding 10 m length to be preferably 3% or more

Council requirements

Check any regional, city or district council requirements for resource consent, building consent or drainage permit or compliance with other standards.

4.1.8 Implementation provisions

Following the issuing of the consent, the steps implementing the on-site device are:

- construction: requires close attention to ensuring that the following are met:
  - design details
  - materials specifications in particular topsoil and grass
  - specifications

- commissioning:
  - once constructed, the device will need to be commissioned and tested
  - in the event that the device is commissioned during a dry spell, in some cases it may be appropriate to test the device using a high-capacity hose (e.g. from hydrant or tanker, feeding water to the roof or site impervious area)
  - checks need to be made for “flaws” such as leaks, blockages, evidence of scour, etc

- certification: once commissioned and operating satisfactorily, the device will need to be certified under the provisions of the Building and/or Resource Consent – ARC TP10 provides examples of the checklists used by certification authorities

- O&M (ongoing): the routine maintenance provisions set out below will need to be undertaken, in accordance with either (as applicable):
  - the provisions of the consent (where nominated), or
  - as per an appropriate O&M model (refer to Appendix D2.0)

Filter operation and maintenance

Item	**Frequency**
Check depth of and removal of accumulated sediment in the sedimentation chamber, remove if depth of accumulated sediment exceeds 25% of the permanent pool depth.	As required, at least annually
Remove excess vegetation, litter, debris from surface of filter bed	As required, at least quarterly
Maintain surface of filter bed by removing accumulated sediment from the surface of the sand.	As required, at least annually, areas with significant contaminant loading may require six monthly
Rejuvenation of the filter bed if emptying times exceed the design time by 50%. This may involve tilling the surface or removal and replacement of the upper part of the bed.	As required

4.1.9 Filter: worked example

Job name	Example			
Location	Gisborne			

design objective	water quality		5 year ARI	
catchment land use	industrial			
impervious area type	seal			
pervious area type	not applicable			
catchment impervious area	800	m^2	800	m^2
catchment pervious area	0	m^2	0	
catchment time of concentration	10	min	10	min
rain intensity source			HIRDS	
rain intensity			54	mm/hr
C impervious	0.9		0.9	
C pervious	0.18		0.18	
catchment CA			0.072	ha
Design Flow			**0.011**	**m^3/s**

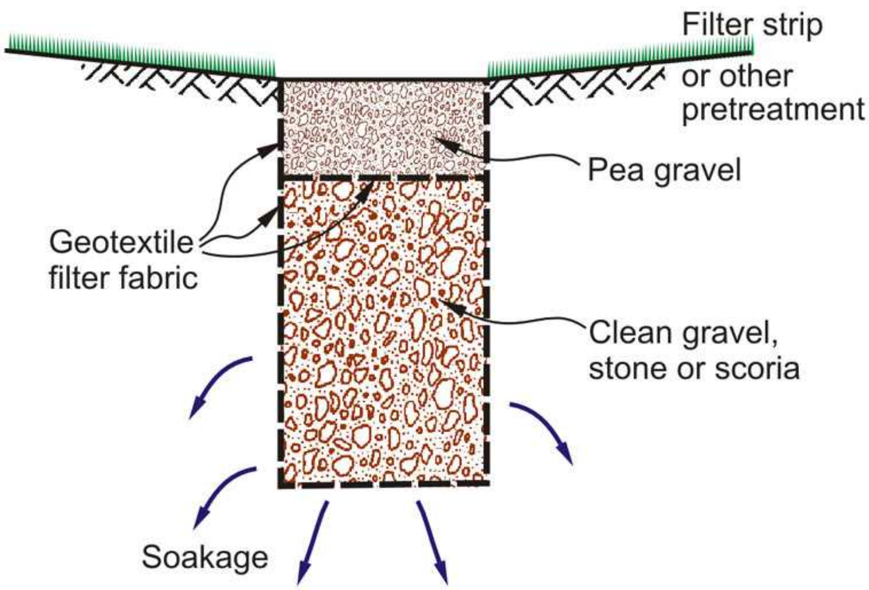
water quality design storm depth	32.6	mm	1/3 of 2 year 24 hour rainfall from HIRDS
runoff from impervious area = rainfall - 2 mm	30.6	mm	
pervious area depression storage and infiltration	na	mm	
pervious area runoff	0.0	mm	
total runoff = runoff from imp & perm area = WQV	24.5	m^3	
depth of sand , d	0.4	m	
coeff perm k	1	m/day	sand 0.25 mm to 9.5 mm
maximum height of ponded water hmax	1	m	from geometry of filter chamber
average height water h = half max height	0.5	m	half maximum height
time to pass WQV t_f	2	day	
area of filter, Af = WQV x d / k(h+d) x tf			
thus required filter area, Af =	5.4	m^2	
minimum live storage required = 37% of WQV =	9.1	m^3	
total required area of filter chamber and sed chamber = min live storage/ max height ponded water			
A_{f+S} =	9.1	m^2	
for filter chamber, nominate inside width, w of	1	m	
required filter length = Af / w	5.4	m	
for width of sed chamber same as for filter chamber, ie = w =	1	m	
total filter chamber & sed chamber length for 37% WQV =	Af+s / w		
=	9.1	m	
min. sed chamber length based on 37% WQV =	total filter & sed chamber length - filter length		
	3.6	m	
Min sed chamber area based on 37% WQV = length x width	3.6	m^2	
check that minimum sedimentation chamber area = 0.25 x A =	1.4	m^2	OK
check velocities in sed chamber for 5 year ARI event			
A rea of flow = w x h =	1	m	
vel = Q_5 / area of flow =	0.01	m/s	< 0.25 OK

4.2 Infiltration trench

4.2.1 Description

An excavated trench, backfilled with stone or scoria media. Stormwater from paved areas enters the trench and trickles through the trench media. Infiltration trenches are used where final disposal is via infiltration of stormwater into surrounding insitu soils. In these cases most of the treatment is provided by adjacent soils provided they are of suitable texture.

Figure 4.2.1 Infiltration trench operating principles



4.2.2 Capability

Infiltration trenches are able to:

- o treat runoff from impermeable hardstand ground surfaces in commercial, residential and industrial areas
- o treat road or parking lot runoff
- o be located so as to take up a small amount of space
- o may in some situations, provide peak flow detention up to the two year ARI event and thus can be used for stream channel protection

Infiltration trenches are not able to:

- treat sediment-laden water from construction sites

Expected contaminant removal rates for trenches where disposal is by infiltration to adjacent soil are listed below, from ARC TP10 and EPA, 1999b. Note that treatment is provided primarily by the insitu soil and will be dependant on its texture:

sediment	90%
metals (copper, zinc, lead) (totals)	85 to 90 %
total phosphorus	60 to 70%
total nitrogen	55 to 60%
organics	90%
bacteria	90%

4.2.3 Applicability

- care is needed to avoid groundwater contamination: refer section 3.5, 3.6 and 3.8
- for car parks and other areas with high or hydrocarbon loads, inflow should be pre-treated to reduce sediment loads, for example by using shallow flow over grass (6 to 8 m wide)
- check that adequate soakage is available and other requirements for infiltration are complied with; refer sections 3.8 and 3.10. Trench preferably horizontal along its length, maximum slope along trench less than 5% to avoid wastage of trench volume. Works best if upgradient drainage slope is less than 5%
- ensure minimum separation distance of 600 mm between bottom of the device and the seasonably high water table (Georgia Stormwater 2001)
- adequate clearance to existing utilities and to site boundaries
- provide downstream overland flow path to avoid scour damage or flood damage to assets
- can incorporate large pipes within trench to provide additional pore space to provide additional storage to help treat large volumes of stormwater
- can add organic matter to the subsoil to enhance removal of metals and nutrients
- device catchment area: no more than 4 hectares, preferably not more than 2 ha (ARC TP10)
- care is needed to prevent large amounts of sediment entering the trench

Infiltration trenches are not suitable for sites with risk of significant sediment runoff that could block up the trench.

Ensure trenches are not installed until after site works are complete and contributing areas are fully stabilised

4.2.4 Summary of design approach

Determine the size required to meet water quality objectives.

4.2.5 Preparatory steps

1. Confirm design imperatives
   - quality objective: refer section 3.6, confirm that an assessment has been made to ensure that discharge to ground will not have an adverse effect on groundwater
   - refer to ground disposal assessment requirements in Section 3.8 and 3.10 sensitivity of groundwater
2. Define key site parameters and device needs that determine design details
   - device catchment land use (this is required to be used in design calculations)
   - device catchment impervious area (roof and on-ground areas)
   - device catchment pervious area and cover type (e.g. grass, shrubs, forest)
   - check any regional, city or district council requirements for resource consent, building consent or drainage permit or compliance with other standards.
   - provision of adequate access for maintenance

4.2.6 Design steps

4.2.6.1 Sizing for water quality design

The recommended method for sizing for infiltration trenches is similar to that in ARC TP10 and other stormwater guidelines.

Design parameters:

- determine Water Quality Volume (WQV) from the appropriate method in section 3.6
- determine design percolation or soakage rate, based on the results of soakage tests or based on soil properties
- assess void ratio of trench media - for clean stone this is typically 0.35, for scoria 0.5 (ARC TP10)
- select the trench drain time in days this should be a minimum of 6 hours (EPA 1999b) and a maximum of 48 hours. ARC recommends a drain time of 48 hours be used for the Auckland region. It is recommended that a maximum drain time of 48 hours be used unless local conditions suggest a different value

4.2.6.1 Design / sizing methodology

The recommended method is a simplified version of that in ARC TP10, which allows complete infiltration within a nominated drain time:

$$A = WQV / (f \times i \times t)$$

where:

A = base area of trench
WQV = water quality volume, m^3 as per section 3.6 (include trench surface area in the calculation for WQV)
f = design infiltration rate (measured rate multiplied by a factor of safety of 0.5)
i = hydraulic gradient, assumed to =1
t = drain time, maximum 48 hours

Size the trench depth to provide storage in the trench voids equal to 37% of the water quality volume unless hydrologic analysis or local experience provide another more appropriate proportion of the WQV to be used to calculate trench storage.

Trench gross area = V = 0.37 x WQV / n
where
n = the stone void ratio, typically 0.35 for stone

Check that the trench sized to meet the storage requirements also meets the area requirements, using the formula above for trench area, resize trench as necessary.

4.2.7 Design detailing and drawings

Inlet

- provide appropriate pre-treatment to reduce sediment input, such as grassed swale, grass filter strip, permeable pavement

Trench dimensions

- typically 0.9 m wide and 0.9 to 2 m deep

Addition of organic material

provide details of amount and method of adding organic material, if required. Take care not to compromise disposal capacity

Stone or scoria media

- 25 to 75 mm , clean

Filter fabric

- use filter fabric on the side walls to prevent migration of in situ soils into the trench
- filter fabric to overlap across the top of the trench or at a depth of 300 mm to minimise entry of sediment form the surface

Observation well

use 100 mm perforated PVC pipe with a footplate and cap

Council requirements

check any regional, city or district council requirements for resource consent, building consent or drainage permit or compliance with other standards

4.2.8 Implementation provisions

Following issuing of the consent, the steps involved in implementing the on-site device are:

- construction: requires close attention to ensuring that the following are met:
  - design details
  - materials specifications in particular stone or scoria medium
  - use light equipment for trench excavation to minimise compaction of surrounding soils
  - trench base and side clear of roots etc that could damage filter fabric or impermeable liner
  - follow construction specifications

- commissioning:
  - once constructed, the device will need to be commissioned and tested
  - in the event that the device is commissioned during a dry spell, in some cases it may be appropriate to test the device using a high-capacity hose (e.g. from hydrant or tanker, feeding water to the roof or site impervious area)
  - checks need to be made for “flaws” such as leaks, blockages, evidence of scour, etc

- certification: once commissioned and operating satisfactorily, the device will need to be certified under the provisions of the Building and/or Resource Consent – ARC TP10 provides examples of the checklists used by certification authorities

- as-builts – preparation of as-built drawings for the TA and the property owner

- O&M (ongoing): the routine maintenance provisions set out below will need to be undertaken, in accordance with either (as applicable):
  - the provisions of the consent (where nominated), or
  - as per an appropriate O&M model (refer to Appendix D2.0)

Operation and maintenance

Item	Frequency
Clear debris, litter from entry and contributing areas	As required, at least quarterly
Monitor observation well to assess whether trench ids draining within the specified times	Annually
Remove small section of upper trench and inspect upper layer of filter fabric for sediment deposits. If clogged, restore to original condition	Every 2 years

Infiltration trench worked example

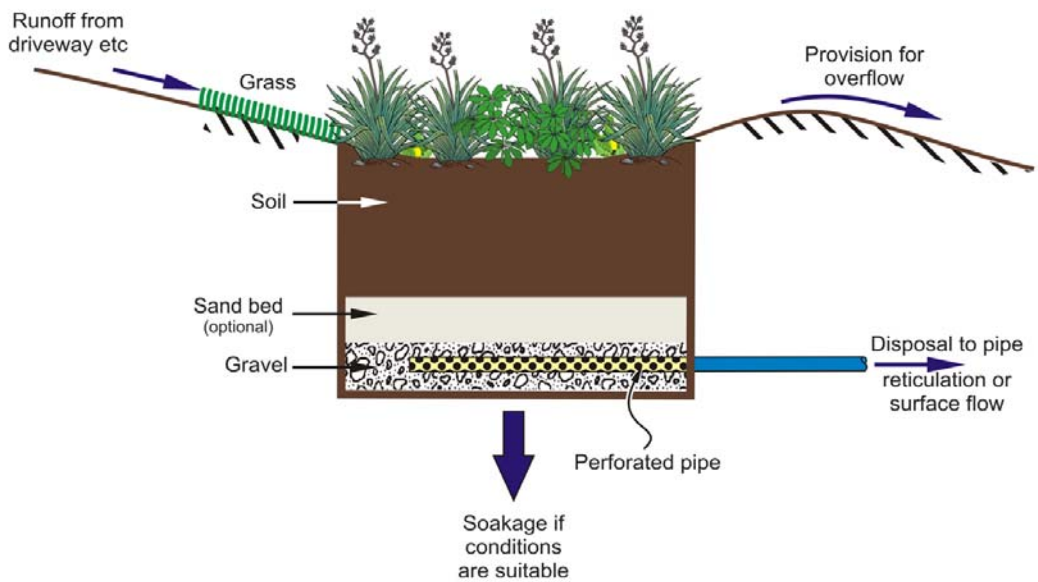
Job name	Example				
Location	Gisborne				
design objective	Water quality				
catchment land use	industrial				
impervious area type	seal				
pervious area type	not applicable				
catchment impervious area	800	m^2			
catchment pervious area	0	m^2			
catchment time of concentration	10	min			
rain intensity source					
rain intensity					
C impervious	0.9				
C pervious	0.18				
Catchment CA					
water quality design storm depth	32.6	mm	1/3 of 2 year 24 hour rainfall from HIRDS		
runoff from impervious area = rainfall - 2 mm	30.6	mm			
impervious area soil drainage	na				
pervious area depression storage and infiltration	na	mm			
pervious area runoff	0.0	mm			
total runoff = runoff from imp & perm area = **WQV**	24.5	m^3			
soil infiltration rate based on soakage test/soil type	26	mm/hr	sandy loam		
design percolation or soakage rate = half infiltration rate = f =	0.31	m/day			
media porosity, n =	0.35	gravel			
drain time, t =	2	days			
hydraulic gradient, i assumed =	1	m/m			
Required area, A = WQV / (f x i x t) =	39.5	m^2			
Required trench gross volume, V = 0.37 x WQV / n =	25.9	m^3			
if trench depth, D =	1.30	m			
choose trench width, W =	0.9	m			
Thus required trench length, based on gross volume requirement = L = V / (W x D)	22.1	m			
Check trench area, based on gross volume = L x W=	19	m^2			
Need to increase trench length and /or width to meet area requirement of 39.5 m^2					
Required trench length based on width of 0.9 m = A/L=	44	m			

4.3 Rain garden

4.3.1 Description

Also known as bioretention areas or stormwater planters, rain gardens are an in-ground filter, with the upper surface of the filter medium exposed to allow infiltration of collected stormwater ponded on it. The filter medium is a specially selected soil/sand mix with a surface mulch or organic layer. Small, shallow-rooting plants protect this medium (the 'soil medium') and provide some evapotranspiration.

Figure 4.3.1 Rain garden operating principles



Stormwater is conveyed by surface flow to the rain garden, ponds on the surface and slowly infiltrates through the planting medium. Treatment is provided by filtration in the soil medium together with bioretention provided by the plants and organic/mulch layer. After infiltrating through the soil medium, water is discharged either by infiltration to underlying soil, or is collected in a pipe and discharged to a reticulated service or surface disposal.

4.3.2 Capability

Rain gardens are able to:

- treat runoff from impermeable hardstand ground surfaces in commercial, residential and industrial areas, including parking lot runoff

Expected contaminant removal rates are (ARC TP10, EPA 1999c):

- sediment 90%
- metals (copper, zinc, lead) 93 to 98 %
- total phosphorus 70-83%
- total Kjeldahl nitrogen 68-80%
- organics 90%
- bacteria 90%
- hydrocarbons > 75%

Rain gardens may be able to:

- be used for flow attenuation and extended detention thus may be used for stream channel protection
- provide aesthetic benefit

Rain gardens are not able to:

- treat sediment-laden water from construction sites. Install after site works are complete and contributing areas have been fully stabilised in order to prevent excess sediment loading

4.3.3 Applicability

- can be located in median strips and islands
- on line or off line location (refer to glossary for definition)
- maximum ground slope: 20% (11°) from considerations of construction practicality: need to check for slope stability
- avoid unstable ground
- ensure minimum separation distance of 600 mm between bottom of the device and the seasonably high water table(Georgia Stormwater 2001)
- adequate clearance to existing utilities and to site boundaries
- inflow should be via shallow flow over grass, to prevent scour of the rain garden surface
- provide overland flow downhill path to avoid scour damage or flood damage to assets
- minimum head required between inlet and outlet is 1.5 m (Georgia stormwater 2001)
- location of piped outlet to discharge to pipe reticulation or surface dispersal
- device catchment area no more than 1000 m^2 (ARC TP10)

4.3.4 Summary of design approach

Determine the size required to meet:

- water quality objectives
- peak flow control and stream channel protection objectives

Check that a device of the required size can be built on the site for all relevant objectives. A device sized to meet the most onerous objective will meet the others.

If a device of the size required to meet a water quality/peak flow/quantity objective cannot be built on the site but a smaller device will be able to meet the most onerous objective, then adopt the sizing for that less onerous objective and select a separate device to meet the more onerous objective.

4.3.5 Preparatory steps

1. Confirm design imperatives

- quality objective: refer section 3.6
- peak flow quantity and stream channel protection: refer section 3.7

2. Define key site parameters and device needs that determine design details

- device catchment land use (this is required to be used in design calculations)
- device catchment impervious area (roof and on-ground areas)
- device catchment pervious area and cover type (e.g. grass, shrubs, forest)
- for final discharge by infiltration to ground, refer to ground disposal assessment requirements in Section 3.8 and 3.10
- for final discharge to pipe reticulation or to the surface, care is needed to avoid potential slope instability from infiltration from the rain garden to adjacent in situ soil. For slopes over 5%, an impermeable liner is required, or approval from geotechnical advisor obtained if a liner is not used
- for water quality treatment only, the maximum ponding depth recommended to avoid over wetting of plants is 220 mm (ARC,2004). Where the maximum water depth will be 220 mm, select suitable plants from Chapter 7, Table 7-3 of ARC TP10
- for flow control and extended detention for stream channel protection, maximum ponding depth may need to be over 220 mm. Obtain specialist plant selection advice for depth of ponding more than 220 mm, or use mulch instead of plants
- check any regional, city or district council requirements for resource consent, building consent or drainage permit or compliance with other standards
- provision of adequate access for maintenance

4.3.6 Design steps

4.3.6.1 Sizing for water quality design

These steps follow the ARC TP10 method, unless noted otherwise.

Design parameters

- determine water quality volume (WQV) from the appropriate method in section 3.6
- minimum live storage: 40% of WQV; This is recommended to be used unless local studies suggest a different value should be used
- detention time for WQV (time to pass through soil):
  - ARC TP10 recommends 1 day for residential sites, which is based on the amenity considerations, ie homeowners may not want ponding longer than this
  - ARC TP10 recommends a detention time up to 1.5 days for commercial/industrial sites
  - the above are recommended to be used unless local studies suggest different values should be used
- planting soil depth: minimum 1 metre for good root growth
- soil permeability: adopt 0.3 m/day default for soil per description in section 4.3.7
- ponding depth:
  - initial assumption: maximum 220 mm
  - average depth during device operation: half maximum
- once area has been calculated, check that depth based on 40% WQV is satisfied by the assumed ponding depth

Design/sizing methodology

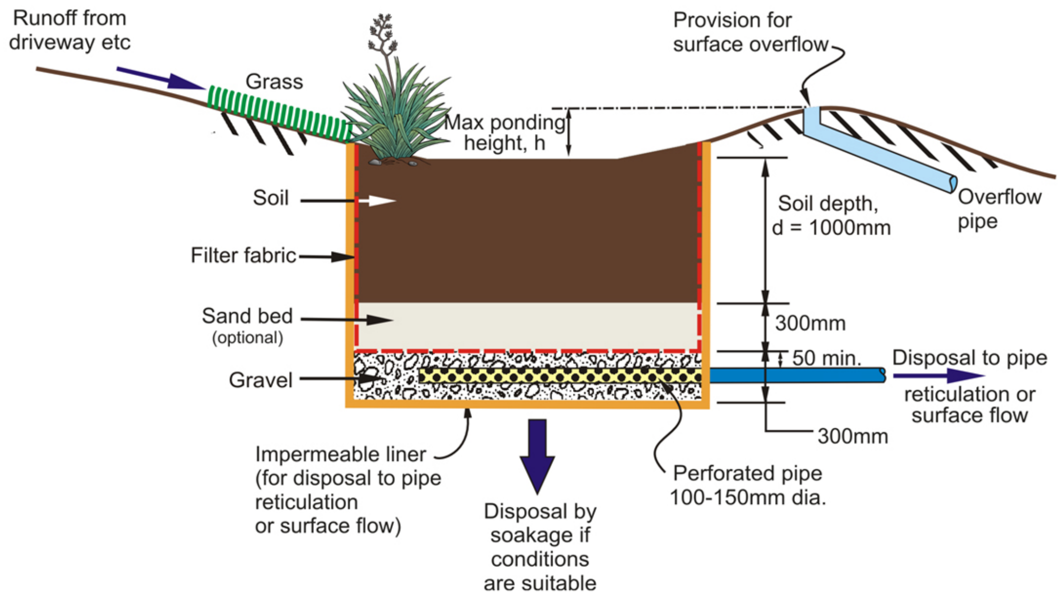
Refer to Figure 4.3.2.

$$A = WQV \times d / (k \times (h/2+d) \times t)$$

Where:

A = surface area, m^2
WQV = water quality volume, m^3
d = planting soil depth, m
k = coefficient of permeability, m/day
h = maximum depth of ponded water above surface, mm
t = time to pass WQV through soil

Figure 4.3.2 Rain garden detailed design



4.3.6.2 Sizing for peak flow / volume control

Design objectives

confirm design objectives, refer section 3.7, i.e. required peak flow control (ARI events to be considered) and extended detention requirements

Design parameters

- determine catchment rainfall losses, or land use runoff factors, refer to Appendix C
- determine rainstorm ARI and duration to be considered and associated rainfall depths
- assess a maximum ponding depth based on site topography. A maximum ponding depth of 0.6 metres is recommended, to avoid excessive inundation of plants. Obtain specialist plant selection advice for this depth of inundation
- assume average ponded depth = half maximum depth
- planting soil depth: minimum 1 metre
- soil permeability: adopt 0.3m/day for planting soil

Design/sizing methodology

- generate hydrographs for existing situation –for peak flow control ARI events under consideration
- generate inflow hydrographs for developed situation –for peak flow control ARI events and for rainfall depth for extended detention requirements
- adopt trial rain garden area

- calculate outflow characteristics (one or more of the following):
  - seepage through planting soil: Q = A x k x (h/2+d) / d
  - outflow through overflow pipe using appropriate standard equations
  - overflow via surface overflow using standard equations Mannings for gentle slope downstream of rain garden, broad crested weir for embankment
- route inflow hydrographs (developed) through the rain garden
- check whether peak flow and extended detention objectives are achieved. If they are not achieved, decide whether a larger device is practical for the site. If so, increase the surface area and maximum water height to the practical maximum and recalculate the routing calculations
- if the required peak flow and extended detention control objectives can be achieved by the revised design, confirm the device feasibility in relation to the site characteristics, especially slope and available area

Determine device size

- check that the required size can be achieved on the site for all relevant objectives. If so, the device is sized to meet the most onerous objective will meet other objectives
- if a device of the size required to meet a water quality/peak flow/extended detention objective cannot be built on the site but a smaller device will be able to meet a less onerous objective, then adopt the sizing for that less onerous objective and select a separate device to meet the more onerous objective
- if the required depth of ponding results in drainage time in excess of 1 to 1.5 days, select plants that can tolerate longer wetting times

4.3.7 Design detailing and drawings

Inlet

Provide a grass buffer between the downstream edge of paved areas and the edge of the rain garden of at least 1 m length in the direction of flow. Design inflow to be spread over as much of the full width of one side of the rain garden as possible to minimise scour of the surface. Need to address on line and off line and design implications.

Plants

Use the plant types and spacings in Section 7.5 and Tables 7-2 and 7-3 of ARC TP10.

Soil medium requirements

- loamy soil: 35 to 60% sand
- clay content: less than 25% (some clay is beneficial for treatment)
- permeability: at least 0.3 m per day
- free of stones, stumps, roots, seeds

Soil placement requirements

- place soil in lifts of 300-400 mm and loosely compact
- cover soil surface with a mulch layer
- use filter fabric on the side walls to prevent migration of in situ soils into the rain garden

Interface between soil and underdrain

There are two options for managing the potential migration of planting soil in the underlying gravel:

- option 1: do not place filter cloth between the planting soil and the gravel underdrain, to avoid potential clogging as recommended in ARC TP10.
- option 2: place a permeable filter cloth to stop planting soil migrating into the underdrainage system, recommended in Georgia Stormwater, 2001

Maintenance implications

Using filter cloth means accepting that the planting soil may need to be removed and the filter cloth cleaned or replaced at certain intervals. Not using filter cloth means potential clogging of underdrainage gravel material may occur which would be difficult to remove and clean but may need to be done infrequently.

Surface mulch

- standard landscape type shredded wood mulch or chips
- well aged, free of other materials such as weed seeds, soil, roots etc
- apply in a uniform thickness of between 50 and 75 mm deep

Impermeable liner

May be required on sites where ground soakage is not used in order to avoid raising local groundwater levels which may lead to instability or other problems. Options are an impermeable liner or a suitable impermeable container such as concrete or timber with an internal impermeable liner. For slopes over 5%, or where stability of adjacent land may be vulnerable to infiltration of water from the raingarden, an impermeable liner is required unless site-specific geotechnical advice is obtained that it is not necessary.

Underdrainage

The underdrainage system comprises gravel layer and a perforated pipe:

- gravel to be clean (no fines) with minimum thickness of 300 mm
- outlet pipe to be perforated 100 mm or 150 mm diameter
- minimum cover of gravel over the pipe to be 50 mm.

Outlet from surface of garden and overflow

A surface entry piped outlet can be used if the hydrologic design requires additional outflow. Whether or not a piped outlet for the garden surface is used, the minimum requirements for provision of overflow are:

- grassed or protected length of in situ or fully compacted soil for the full length of the downstream side of the rain garden
- use of a 50 x 150 mm horizontal timber level spreader to ensure even flow and minimise scour
- overflow directed clear of buildings or other assets or features that may cause obstructions to flow

Council requirements

Check any regional, city or district council requirements for resource consent, building consent or drainage permit or compliance with other standards.

4.3.8 Implementation provisions

Following issuing of the consent, the steps in implementing the on-site device are:

- construction: requires close attention to ensuring that the following are met:
  - design details
  - materials specifications in particular planting medium grading
  - specifications

- commissioning:
  - once constructed, the device will need to be commissioned and tested
  - in the event that the device is commissioned during a dry spell, in some cases it may be appropriate to test the device using a high-capacity hose (e.g. from hydrant or tanker, feeding water to the roof or site impervious area)
  - checks need to be made for "flaws" such as leaks, blockages, evidence of scour, etc

- certification: once commissioned and operating satisfactorily, the device will need to be certified under the provisions of the Building and/or Resource Consent – ARC TP10 provides examples of the checklists used by certification authorities

- O&M (ongoing): the routine maintenance provisions set out below will need to be undertaken, in accordance with either (as applicable):
  - the provisions of the consent (where nominated), or
  - as per an appropriate O&M model (refer to Appendix D2.0)

Operation and maintenance

Item	Frequency
Clear debris, litter from rain garden and contributing areas	As required
Remove noxious or invasive weeds and plants	As required but inspect at least quarterly
Check plant height and density, prune excessive vegetation, replace plants if necessary	As required, but at least 6 monthly
Check that the surface dewaters between storms: 220 mm of ponded water depth should empty within 1 or 1.5 days, depending on design (residential, commercial/industrial). If longer, check for surface clogging, remove sediment. Replace planting soil medium if required	6 monthly
Outlet /overflow spillway: check condition, scour, erosion, blockage	6 monthly
Sediment accumulation: remove if more than 30 mm depth, re-establish plants after sediment removal	Annually
Rain garden integrity: check device has not been blocked or filled in	Annually
Replace mulch	Every 2 to 3 years

4.3.9 Rain garden design worked example

Job name	Example		
Location	Gisborne		

Design objective	Water quality	
catchment land use	residential	
impervious area type	seal	
pervious area type	grass, shrub	
catchment impervious area	500	m^2
catchment pervious area	300	m^2
catchment time of concentration	10	min
rain intensity source		
rain intensity		
C impervious	0.83	
C pervious	0.18	
Catchment CA		

water quality design storm depth	32.6	mm	1/3 of 2 year 24 hour rainfall from HIRDS
runoff from impervious area = rainfall less 2 mm	30.6	mm	
pervious area soil drainage	slow		
pervious area depr storage and infiltration	15	mm	
pervious area runoff = rain – depr stor & infiltr =	17.6	mm	
total runoff = WQV	20.6	m^3	

planting soil depth d	1	m	
coeff perm k	0.3	m/day	
maximum height of ponded water h	0.22	m	
time to pass WQV t	1	day	residential

Area, A = WQV x d / (k x (h/2+d) x t)

Thus area	=	61.8	m^2	
check min live storage	=	area x max height ponded water		
	=	13.6	m^3	
	=	66.1	% of WQV	(> 40% OK)

4.4 Stormwater planter

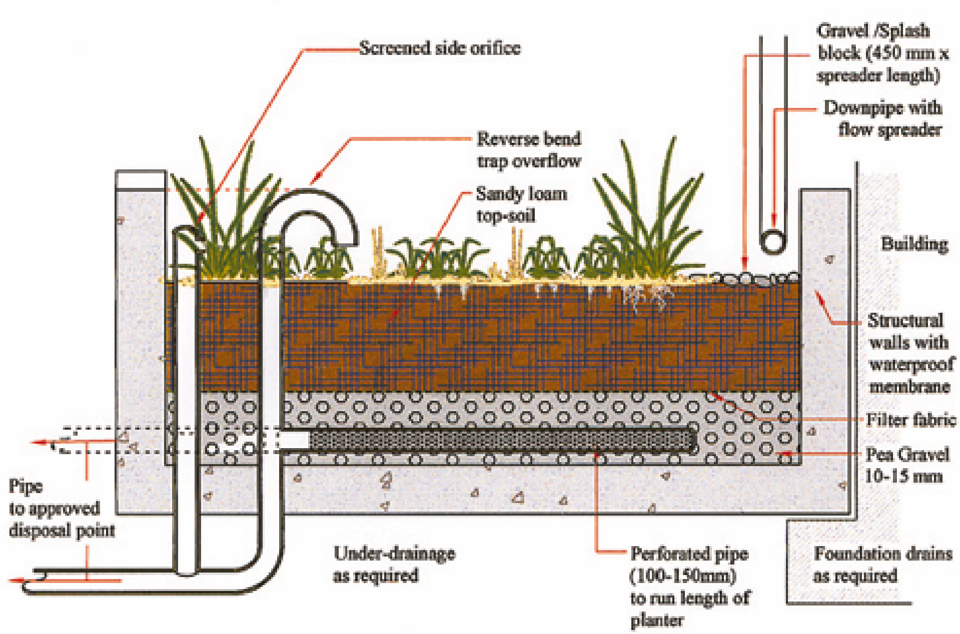
4.4.1 Description

The stormwater planter is essentially a variant of the rain garden (refer Section 4.3). The main differences are:

- it is fed from roof water only
- it is typically located above ground, or partially buried, designed to serve both stormwater and landscaping functions
- its outlet is normally connected to the public stormwater system, although it can be revamped to operate in a disposal-by-soakage mode

As documented in this guideline, the device is based on an arrangement in widespread use in Portland, Oregon, USA (CoP 2002) and adapted by Auckland City Council (ACC 2002). The stormwater planter, as illustrated in Fig. 4.4.1, functions as both a water quantity and quality control device.

Figure 4.4.1 Stormwater planter



The key components and function of the stormwater planter are:

- roof water is fed onto the surface of the stormwater planter, via a spreader device
- this water infiltrates through the top soil layer and then collects in the underlying drainage layer, from where it is piped to the public stormwater system
- when the inflow rate exceeds the soil infiltration rate, ponding occurs on top of the soil; this is contained by the wall of the stormwater planter

- two outlets[1] from the pondage, located at the end opposite the spreader inlet, feed to the public stormwater system via a standpipe, namely:
    - o an orifice [1] comes into operation when the ponding is nominally about half-full - ponding to this level is required to meet water quality requirements
    - o a half siphon which comes into operation when the ponding is nominally full[2]
- part of the wall rim is cut down to act as an emergency overflow

The stormwater planter is normally sited above-ground, rather like a planter box (Figure 4.4.2). However, the base can be below ground level, subject to suitable gradients being available to connect the outlet to the public stormwater system, and provided flooding by groundwater can be avoided. It is normally constructed in concrete (e.g. plaster-faced concrete blocks, cast-in-situ concrete or precast concrete), but can be constructed from timber, much like a retaining wall.

Figure 4.4.2 Stormwater planter example

4.4.2 Capability

The stormwater planter has the same broad capabilities as for the rain garden, but with a greater flow attenuation capacity. In summary, a stormwater planter is able to:

- provide detention to achieve peak flow attenuation of roof runoff (a stormwater planter alone can often meet the greenfield site runoff standard, by over-throttling the flow to compensate for the extra runoff from the site impervious area)
- filter-out the roof-derived sediment and allied contaminants (refer Section 4.3.2)

The stormwater planter is not able to:

- treat site runoff (refer Section 4.3 for rain gardens, which can serve this function)

4.4.3 Applicability

- receives roof runoff

[1] The plan for a second orifice-type outlet was introduced by ACC, 2002, designed to reduce the planter size over that required for a single siphon outlet arrangement (as used by City of Portland)
[2] With the orifice, the siphon is essentially superfluous, but is retained as a safety against blockage of the orifice

- performs a water quality and quantity control function (note that the former may not be as important an objective as for a rain garden which treats site runoff)
- is normally installed on the ground or partially buried, provided flooding by groundwater can be avoided
- must be sited at an elevation to allow adequate fall from outlet at the base of the planter box to the connection point with the stormwater receiving system, noting that provision may be needed for heading-up of the latter
- doubles as an attractive landscaping feature, thereby avoiding the need for a dedicated a space such as needed for say a rain tank
- allow access for maintenance

4.4.4 Summary of design approach

1. Confirm the suitability of the stormwater planter to the particular site and application
2. Establish device parameters and the applicable water quantity and quality performance standards
3. Establish the site parameters
4. Assemble the requisite hydrological data applicable to the general area in which the device to be sited
5. Size the capacity needed to meet the water quantity and quality control targets (note that the former is normally calculated first, and then the need for any incremental storage to meet the water quality target is computed)
6. Complete the attendant device sizing and hydraulic design

Note: *For details of the model-based approaches, refer Appendix C – Section C2.4; note that the procedure set out below used manual methods, assisted with spreadsheets*

4.4.5 Preparatory steps

4.4.5.1 General

1. Confirm the applicability of using a stormwater planter, noting that it accepts flow from roofs only
2. Confirm the water quality control performance standard (note that if water quality is a secondary objective – recognising that roof runoff is relatively clean compared to say site runoff (aside from zinc off metal roofs) - the planter can be designed as a flow control device, noting that the basic planter design achieves a degree of treatment)
3. Confirm the peak flow control performance standards, ie:
    - design storm frequency (e.g. 2% AEP, 10% AEP and/or 50% AEP, with the latter only applicable where there is a stream channel erosion protection imperative – refer Section 3.7)
    - the target peak site outflow; this is typically as existing, or greenfield – refer Section 3.7 (note that a stormwater planter alone can often meet the greenfield site runoff standard, by over-throttling the flow to compensate for the extra runoff from the site impervious area)

4. Establish applicable design time of concentration (Tc – refer Appendix C, Section C2.2 for further details), e.g.:
   - in the receiving stormwater system, at the dwelling connection point (= Tc_1 say)
   - at key points on down to the outfall (e.g. major watercourse, sea); = Tc_2 , Tc_3, etc
5. Establish key site parameters, e.g.:
   - site area
   - impervious area (roof and on-ground)
   - pervious area (and cover type)
6. Identify site/device layout constraints, e.g.:
   - device location
   - device above ground or partially buried
   - stormwater system connection points (and corresponding elevations)
   - overland flow paths (from the emergency overflow)

4.4.5.2 Hydrological data

1. Obtain rainfall depth-duration-frequency data applicable to the general area in which the planter is to be sited, for the following cases, as applicable (refer Section 3.12 for explanations/details):
   - 50%, 10% and 2% AEP
   - applicable Tc values (from 3 above)

Using the data from (1) above, establish design storm runoff peaks and hydrographs, according to the Rational Formula, or other method (refer Section 3.12 and Appendix C for details), for:

- target site outflow (only the peak flow is required)
- roof runoff
- rest-of-site runoff (ie surface impervious and pervious)

4.4.6 Design steps

(a) Summary:

1. Collate the design data/parameters from Section 4.4.5
2. Size the storage capacity required for water quality control
3. Select tentative planter dimensions and size the storage capacity required for flow control
4. Reconcile the storage volumes from (2) and (3) above
5. Size the allied hydraulic components

(b) Sizing water quality storage:

Note: *As explained in Section 4.4.5.1(2), where water quality control is a secondary objective, this computation step can be bypassed*

This should apply the water quality volume (WQV) based approach, as used for the rain garden (refer Section 4.3.6.1 – note that, because the method is the same, it is not repeated here).

From this calculation, the following will be derived:

- planter surface area, W (m^2)
- planter ponding height[3], h (mm)
- planter storage capacity, P (m^3) = W x h / 1000

(c) Sizing storage to meet flow control:

Typically, a spreadsheet is used to size the temporary storage capacity. This involves performing routing calculations to quantify the way the storage provided in the planter modifies the inflow hydrograph (refer Appendix C – Section C3.4 for details).

Because the method closely parallels that applied in sizing the temporary storage component of a rain tank - refer Section 4.5.6(b) and Section 4.5.10 for a sample of the spreadsheet – it is not repeated here. Note also the probable need for "trial and error" iterations, also accounting for the full range of applicable storm durations from the catchment Tc value up to the duration giving the maximum volume requirement [ie as set out in Section 4.4.5.1(4)], to arrive at the design sizings.

The following adjustments should be applied to the generalised spreadsheet in Appendix C (Table C3) to model the planter features:

- planter dimensions (refer Figs 4.4.3 and 4.4.4 for typical dimensions[2]):
    - o in place of the tank area, the planter area is used (<u>note</u>: *where the water quality-based design step (b) above applies, this area should match that used in (b), ie A m^2)*
    - o in place of the tank height, the heights of the orifice and siphon are used (ie both relative to the top of the planter soil surface)
- orifice:
    - o apply the orifice discharge formula to match its location in the stormwater planter (<u>note</u>: *where the water quality-based design step (b) above applies, this orifice should be located at height "h" – as derived in (b) - above the planter soil surface)*
    - o size the orifice so that when the water level in the planter reaches the siphon level, the sum of the orifice and the infiltration flows matches the required maximum device outflow rate
- o add the following outlets/outflows:
    - o an infiltration component (ie based on the infiltration rate, in m/day)
    - o the half siphon outlet[4]
    - o the emergency overflow[3]

An example of the planter spreadsheet is given in Section 4.4.10. Note that:

- the spreadsheet also incorporates a trial orifice diameter calculation (ie based on the simplifying assumption that the peak flows from both the planter and "rest of site" coincide in time)
- infiltration = infiltration rate (K) x planter area
- orifice discharge – refer formula in Table C4 (Appendix C)
- overflow; not accounted for in the spreadsheet example (refer spreadsheet footnote for an explanation)

[3] For aesthetic reasons, the height of the planter wall above the soil surface should not exceed about 300 mm

[4] In the spreadsheet, the hydraulics of these two high level outlets can be adequately approximated by applying the assumption that the outflow matches the inflow

From the spreadsheet, the following will be derived:

- planter surface area, W (m^2)
- maximum water depth above the planter soil surface, M (m)
- planter storage capacity, R (m^3) = W x M
- orifice: E (mm) diameter, set N (mm) above the planter soil surface

(d) Net storage:

Use the following procedure to reconcile the storage capacities derived in (b) and (c) above:

- water quantity control only: adopt the planter sizings derived by the spreadsheet in (c) above
- water quality and quantity control (the symbols used below are as defined in (b) and (c) above):
    - select the greater of the planter storage capacities, P and R (ie as derived in (b) and (c) respectively)
    - check that the orifice height, N [ie as computed in (c)] is set above the depth, h, needed for water quality control [ie as computed in (b)]

(e) Sizing of hydraulic components:

1. orifice standpipe diameter, F (mm) = 1.5 x orifice diameter E (mm)
2. siphon diameter, G (mm): select the larger of:
    - the orifice diameter, E (mm), or
    - 50 mm
3. emergency overflow:
    - takes the form of a “discharge slot”, namely a cut-down section of the planter wall, located to ensure flow is directed away from buildings and avoids damage to adjacent properties
    - design the overflow to pass the full 2% AEP flood, with the pipework assumed blocked
    - size the discharge slot by applying the sharp-crested weir formula (refer Appendix C – Section C4.0), allowing a modest freeboard
    - an illustrative sizing (from ACC, 2002) as shown in Fig 4.4.4,. takes the form of a slot, S = 30 mm deep x T(m) in length, where T = 0.1xwall perimeter length, or 0.2 x (Y+Z)
4. planter wall height, K = M (max water depth)+G (siphon diameter)+S (emergency overflow slot depth)

Figure 4.4.3 Stormwater planter – definition sketch

Note: Dimensions are illustrative only

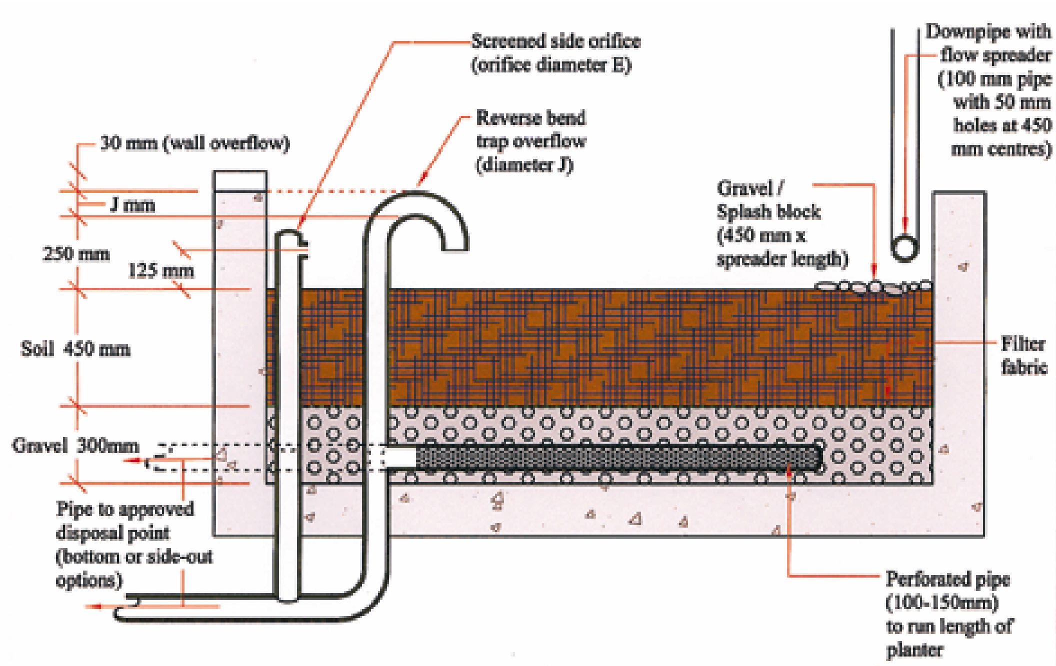
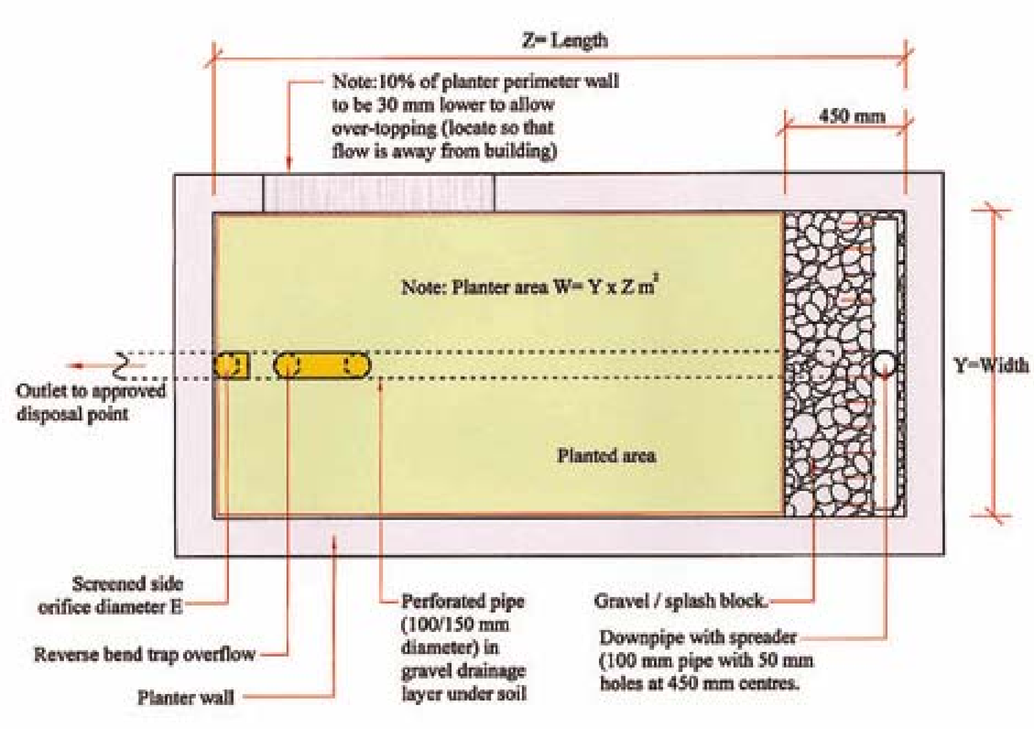


Figure 4.4.4 Stormwater planter – plan

Note: Dimensions are illustrative only



4.4.7 Design detailing and drawings

The standard design details applicable to stormwater planters, to be shown on drawings to be submitted with the consent application(s), are as listed below (adapted from ACC, 2002). These should be read in conjunction with Figs 4.4.3 and 4.4.4, noting that the dimensions on these drawings are illustrative only.

In parallel, it will be appropriate to check any regional, city or district council requirements for resource consent, building consent or drainage permit or compliance with other standards.

Stormwater planter worked example

Item:	**Requirement:**
Planter base elevation	Nominally at ground level, but can be sunk into the ground to a depth of not more than 500 mm, subject to suitable gradients being available to connect the outlet to the main/public stormwater system, and provided flooding by groundwater can be avoided by installing separate sub-surface drainage externally, at base level. In such cases, the applicant / developer is to provide full details of the proposed arrangement for: - the connection (including the reduced levels of the planter base and stormwater system connection) - the sub-surface drainage system
Key dimensions:	
- Minimum planter width	500 mm (no minimum length or prescribed planter shape)
- Gravel depth	300 mm
- Soil depth	450 mm
- Planter wall height	Approx. 300 mm maximum
Inlet: (from roof drainage)	
- Spreader	"Spreader-type" pipe inlet across the width of the shortest side (typically comprises 100 mm diameter pipe with 30 – 50 mm diameter holes at 300 - 450 mm centres)
- Erosion protection	Spreader flow to discharge onto a gravel bed (typical dimensions: spreader length x 450 mm x 75 mm depth)
Top outlets:	
- Location	At the end opposite the inlet
- Orifice	Machine drilled, to the calculated diameter; to be covered with wire mesh to protect against the ingress of debris
Emergency overflow	To discharge the full 2% AEP flood peak; overtopping to be directed away from buildings and avoid damage to adjacent properties
Bottom outlet	Perforated pipe, embedded in gravel, with the pipe length covering the full length of the planter (pipe diameter typically 100 mm)
Construction materials:	
- Concrete	Reinforced concrete, reinforced concrete blocks, or pre-cast concrete, painted on the inside face with two coats of a bitumen-based sealer
- Timber	Constructed as a retaining wall using H4 radiata; boards to be tongue and groove; the inside of the planter to be lined with 200µ grade black PVC sheeting; all joints to be sealed with approved tape
Planter media:	
- Soil	As for rain garden (refer Section 4.3.7)
- Filter cloth	As for rain garden (refer Section 4.3.7)
- Gravel	Gravel or scoria 10 – 15 mm sizes; minimum infiltration rate 4 m/day
- Plants	As for rain garden (refer Section 4.3.7); ie as specified in Section 7 of ARC TP10

4.4.8 Implementation provisions

Following the completion of the design and detailing, the steps to implementation are:

- consents:
  - apply for the appropriate consent (refer Section 3.13 for details of the type of information that will need to be included)
  - receive the consent and account for any design changes required under the consent
- construction: requires close attention to ensuring that the following are met:
  - design details
  - materials specifications in particular planting medium grading
  - specifications
- commissioning:
  - once constructed, the device will need to be commissioned and tested
  - checks need to be made for flaws such as leaks, blockages, etc.
- certification: once commissioned and operating satisfactorily, the device will need to be certified under the provisions of the Building and/or Resource Consent – ARC TP10, Chapter 7 provides examples of the checklists used by certification authorities
- O&M (ongoing): the routine maintenance provisions set out in Section 4.4.9 will need to be undertaken, in accordance with either (as applicable):
  - the provisions of the consent (where nominated), or
  - as per an appropriate O&M model (refer to Appendix D2.0)

4.4.9 Operation and maintenance

The routine maintenance activities that should be undertaken in respect to a stormwater planter are as tabulated below (note that Section 4.3.8 provides a checklist for the rain garden, which is closely comparable). It is recommended that the owner be issued with a copy of the checklist, along with a description of the rain tank, covering how it works and explaining the maintenance imperative (refer ACC, 2002 – Appendix C, pages C8 & C9 for an example of such a handout).

Stormwater planter operation and maintenance

Frequency			Action
After storm	**Quarterly**	**Annually**	
✓	✓	✓	**Spouting & downpipes:** check for problems such as debris/blockages and leaks & rectify
✓	✓	✓	**Spreader & splash pad:** check for blockage/erosion and rectify
	✓	✓	**Planter surface:** remove litter & sediment accumulation
		✓	**Vegetation:** maintain healthy plants & replace dying plants (to ensure at least 90% of the surface is covered); trim/prune
		✓	**Soil:** cultivate to a depth of 100 mm (insofar as possible without disturbing the plant root zone
		✓	**Planter box:** check for structural deficiencies. Leaks, growths & rectify
	✓	✓	**Overflow pipe & orifice:** check for blockage, damaged/leaking pipe & rectify

4.4.10 Worked example

The worked example below, including the spreadsheet for the calculation of the stormwater planting sizings, is for the following case (note that this is for a flow attenuation-only case – refer Section 4.3.9 for a comparable worked example for a water quality control situation):

Base data:

(i) Areas (m^2):

Roofs (multiple units):	250
Other impervious	110
Pervious:	90
Lot total:	450

(ii) Soil type: alluvium

(iii) Planter performance standards:

- o flow: attenuate to 'as existing' in a 10% AEP storm event
- o water quality: no requirement

(iv) Applicable time of concentration (Tc): 20 minutes

Hydrological data and calculations:

The methodology uses the Rational Method – refer Section C3.2 or details.

(i) Rainfall depth-duration-frequency data (for Tc = 20 mins & 10% AEP) gives I =75 mm/hr

(ii) Design hydrographs for the following cases (refer spreadsheet below for results):

- o roof
- o rest-of-site
- o target (ie 60% impervious area equivalent)

Sizing storage to meet flow control:

Refer spreadsheet overleaf for the case where the storm duration (D) is equal to the time of concentration (Tc); a comparable example of the case where D > Tc is given for a rain tank in Section 4.5.10.2.

Planter dimensions and sizing of hydraulic components:

(i) Planter dimensions:

From the spreadsheet above, the sizings are as follows:

- planter surface area, W = 7.2m^2
- maximum water depth (above the planter soil surface), M = 0.25m
- planter storage capacity, R = W x M = 1.8m^3
- orifice diameter, E = 70 mm
- orifice height (above the planter soil surface), N = 125 mm

(ii) Sizing of hydraulic components:

- orifice standpipe diameter, F = 1.5 x orifice diameter E = 105 mm
- siphon diameter, G = E = 69 mm (say 70 mm)
- emergency overflow:
  - 2% AEP peak inflow is approximately 7 l/s (ie Q = 0.007 m^3/s)
  - select discharge slot depth, S = 30 mm (flow depth, h_1 = say 25 mm = 0.025 m)
  - computed weir length, L, by weir formula $Q = 1.8 \times L \times h_1^{1.5}$, is 1.0 m

(iii) Planter wall height, K = M+ G+ S = 0.35 m

Note: Eliminating the orifice (ie matching the original CoP (2002) version of the stormwater planter) has the effect of increasing appreciably the required planter area. Note that to simulate this case, the spreadsheet must be modified so that when the water level reaches the siphon, the siphon flow equals the roof inflow less the infiltration.

STORMWATER PLANTER - FLOW ROUTING ANALYSIS

(A) SITE DATA:

Soil Type	Clay		C value
Roof area	250	m2	0.9
Other impervious area	110	m2	0.86
Pervious area	90	m2	0.43
Lot area	450	m2	

(B) PLANTER DETAILS:

Target performance standard: reduce peak flow to the equivalent of that from the site with an impervious area coverage of 60 %

Planter area	7.2	m2
Storage height	0.25	m

Orifice:

height	0.125	m
diameter	0.069	m
discharge coeff	0.75	

Trial orifice diameter calculation:

Peak orifice flow:	*3.76*	*l/s*
Max orifice head:	*0.125*	*m*
Trial diameter:	*0.066*	*m*

Infiltration rate 0.3 m/day

(C) HYDROLOGY - BY RATIONAL METHOD:

(refer comparable calculations in Appendix C - Section C3.5)

Tc	20	min
Storm duration (D)	20	min
Rainfall intensity (10% AEP)	75	mm/hr

Case		C Value	Peak discharge (l/s)
Peak runoff from roof		0.9	4.69
Peak runoff from site impervious area		0.86	1.97
Peak runoff from site pervious area		0.43	0.81
Target peak site outflow:	60 % impervious	0.70	6.56

(D) SIMULATION:

Time step 2 mins = 120 sec

								SITE RUNOFF CALCULATION		
Time (mins)	Planter Inflow l/s	Infiltration Flow l/s	xs Flow to Storage m3	Planter Storage m3	Planter WL m	Av orifice Head (m)	Orifice Flow l/s	Planter Outflow l/s	Rest Site Flow l/s	Total Site Flow l/s
0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.0	0.47	0.03	0.03	0.03	0.00	0.00	0.00	0.03	0.28	0.30
4.0	0.94	0.03	0.08	0.11	0.01	0.00	0.00	0.03	0.56	0.58
6.0	1.41	0.03	0.14	0.24	0.03	0.00	0.00	0.03	0.83	0.86
8.0	1.88	0.03	0.19	0.44	0.06	0.00	0.00	0.03	1.11	1.14
10.0	2.34	0.03	0.25	0.69	0.10	0.00	0.00	0.03	1.39	1.41
12.0	2.81	0.03	0.31	0.99	0.14	0.00	0.00	0.03	1.67	1.69
14.0	3.28	0.03	0.36	1.36	0.19	0.04	2.26	2.29	1.94	4.23
16.0	3.75	0.03	0.42	1.50	0.21	0.07	3.14	3.17	2.22	5.39
18.0	4.22	0.03	0.48	1.60	0.22	0.09	3.49	3.51	2.50	6.01
20.0	4.69	0.03	0.53	1.72	0.24	0.11	3.76	3.78	2.78	6.56
22.0	4.22	0.03	0.53	1.80	0.25	0.12	3.99	4.01	2.50	6.51
24.0	3.75	0.03	0.48	1.79	0.25	0.12	4.08	4.10	2.22	6.32
26.0	3.28	0.03	0.42	1.72	0.24	0.12	3.99	4.02	1.94	5.96
28.0	2.81	0.03	0.36	1.61	0.22	0.11	3.77	3.79	1.67	5.46
30.0	2.34	0.03	0.31	1.46	0.20	0.09	3.43	3.45	1.39	4.84

32.0	1.88	0.03	0.25	1.30	0.18	0.07	2.98	3.01	1.11	4.12
34.0	1.41	0.03	0.19	1.13	0.16	0.04	2.42	2.45	0.83	3.28
36.0	0.94	0.03	0.14	0.98	0.14	0.02	1.71	1.74	0.56	2.29
38.0	0.47	0.03	0.08	0.86	0.12	0.00	0.59	0.62	0.28	0.89
40.0	0.00	0.03	0.03	0.81	0.11	0.00	0.00	0.03	0.00	0.03

NOTE: If/ when planter WL exceeds storage height, site runoff calculation should include planter overflow
(ie overflow = inflow - orifice outflow - infiltration)

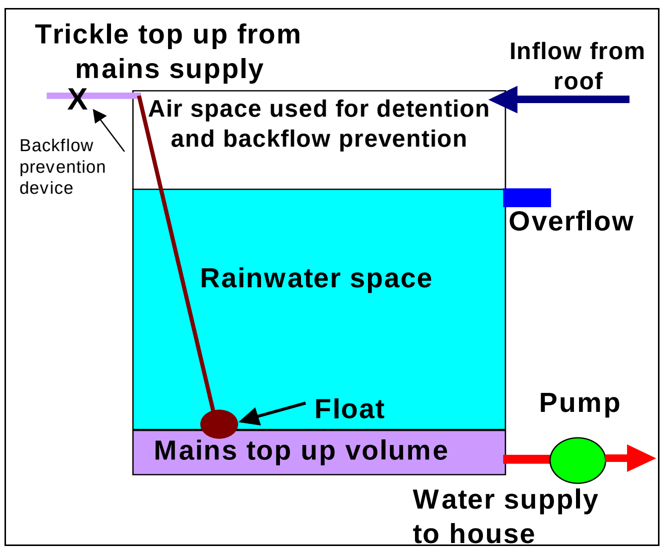
4.5 Rain tank

4.5.1 Description

A rain tank, or dual-use tank, is fed from roof runoff and serves to not only attenuate peak flow but also allow re-use of stored water. As illustrated in Fig 4.5.1, in order to do this, the rain tank has two 'zones', namely:

- temporary storage (or 'air space'):
  - the upper part of the tank, dedicated to retaining runoff in short duration, high intensity storm events
  - has an orifice outlet at the bottom (ie this defines the interface between the temporary and permanent storage zones); this serves to "throttle" the flow
  - has an overflow at the top of the tank, connected to the stormwater system
- permanent storage (or 'rainwater space'):
  - the bottom portion, dedicated to storing water for re-use
  - in areas with mains water supply, it includes a mains connection for "topping-up" the storage to ensure continuity of supply in dry periods

Figure 4.5.1 Rain tank – elevation



Tanks are generally made of concrete, plastic, steel or fibreglass and are typically fabricated off-site. Other types of specifically designed tanks can be used. Rain tanks as described in this guideline take only roof water and are typically placed above ground.

The difference between the rain tank and the detention tank (refer Section 5.2) is that the latter is designed to accomplish only the peak flow attenuation function. The rain tank is nowadays generally preferred, for the following reasons:

- the potential for re-use to be cost-effective, due to the modest extra cost of the larger tank needed to provide the permanent storage
- avoiding the potential maintenance problems of underground detention tanks (refer Section 5.1)
- the re-use benefit of a rain tank, in parallel with avoiding potential public health problems of underground detention tanks, refer to section 4.5.3.1 below, leads to the use of rain tanks being seen as encouraging sound maintenance practices

4.5.2 Capability

Rain tanks are able to:

- provide detention to achieve peak flow attenuation of roof runoff (note that a rain tank alone can often meet the “greenfield” site peak runoff standard, by over-throttling the flow to compensate for the extra runoff from the site impervious area)
- settle-out the roof-derived sediment in the tank
- allow stored water to be re-used for domestic purposes (in turn, this leads to a reduction in the volume of stormwater discharged from the site)

Rain tanks:

- are not able to treat site runoff (apart from removing roof derived sediment)

4.5.3 Applicability

4.5.3.1 General

- are normally installed on the ground or partially buried (ie as needed to ensure gravity feed from the roof gutters)
- can be installed underground, provided that they incorporate adequate structural strength to avoid cracking (note that cracking has the potential to lead not only to leakage, but also the ingress of microbiological contaminants from adjoining soil, with potential risks to public health)
- must be sited at an elevation to allow adequate fall from the orifice at the base of the temporary storage zone to the connection point with the stormwater receiving system, noting that provision may be needed for heading-up of latter (this requirement most often only poses a problem if the rain tank is located below the road and/or is partially/fully buried).
- can be used in rural areas without mains supply to meet all domestic water supply needs
- can be used in areas with mains supply, can be used as a supplemental water source
- allow access for maintenance

4.5.3.2 Re-use component

The issues below need to be considered for the re-use component.

(a) Water quality

In urban areas, airborne contaminants, including hydrocarbons, can intercepted by rainfall, either in the air or on the roof, and washed into the rain tank. Without treatment, the water cannot be considered potable and so should not be plumbed to fixtures where human consumption is likely. A study for Auckland City Council (Ogilvie, 2002) explored the public health risk and recommended that the use of water from urban rain tanks be limited to outdoor taps, toilets and cold water feed to the washing machine and shower.

In rural areas where there is no mains supply, roof tanks have long been the sole source of supply. While the risks are less than in urban areas, tests on rural tank water have found it fails the potable standards (ACE, 2003) set out in the NZ Drinking Water Standards (MoH, DWSNZ, 2000). It is believed that rural dwellers may develop a resistance to illness from E Coli and the like through persistent exposure, but vigilant adherence to maintenance practice (e.g. SDC, 1997; MoH, 2001) is nevertheless warranted. Added safety can be achieved through the implementation of first flush water diverters on tanks (RWHWWS, 2004 - refer Section 4.5.7 for details) and/or water filters on kitchen taps.

(b) Ownership of tanks

In high density urban developments in which the Local Authority requires the installation of on-site devices, there are particular issues with rain tanks in this context. This arises from the fact that on say a multi-unit development, it will be much less costly to implement a single large tank than having one tank per dwelling unit. At issue then is the maintenance obligations and the rights to the re-use water. Although this will be an issue for the controlling local authority, as an example, Auckland City Council has the following policies on the use of rain tanks in multi-dwelling developments (ACC 2002):

- options for ownership, connection and maintenance of tanks (any one of the following to apply):
    - multiple tanks, one connected to each dwelling, with each owner responsible for operation & maintenance
    - one tank fed by multiple roofs, with one particular dwelling having legal responsibility for owning and operating/maintaining the tank; in such a case the owner may choose to plumb water from the tank to other dwellings
    - in the case of a Body Corporate having legal responsibility for owning and operating/maintaining the tank, at least 50% of the dwellings must be plumbed to a tank(s)
- connection of roofs: provision must be made to connect the entire roof area of the development to rainwater tank(s)

4.5.4 Summary of design approach

Note: ***items (1) – (4) are covered in Section 4.5.5 and the remainder in Section 4.5.6***

1. Confirm the suitability of the rain tank to the particular site and application
2. Establish device parameters and the applicable performance standard
3. Establish the water re-use targets
4. Establish the site parameters
5. Assemble the requisite hydrological data applicable to the general area in which the rain tank is to be sited
6. Size temporary storage capacity
7. Size the permanent storage capacity
8. Complete the attendant tank sizing and hydraulic design

Note 1: *There is in practice an interaction between the two storage zones (e.g. at the start of a summer storm, the water level may be drawn down into the permanent storage zone), meaning that steps 6 & 7 should ideally be computed by a "whole of tank" simulation approach, as is possible through modelling[5] (refer note 2). In practice, most practitioners will find it more convenient to use the two-stage approach presented in Section 4.5.6: studies show that this approach produces more conservative (ie slightly larger) temporary storage capacities than the modelling-based approach (ACC, 2002).*

Note 2: *Computer based models can be used in place of or to augment the detailed approach set out in this guideline. For details of the model-based approaches, refer Appendix C – Section C2.4. The procedure set out below uses manual methods, assisted with spreadsheets.*

[5] For further information on this topic, refer Coombes & Kuczera, 2001

4.5.5 Preparatory steps

4.5.5.1 General

1. Confirm the applicability of using a rain tank, noting that it accepts flow from roofs only
2. Confirm the peak flow control performance standards, ie:
   - o design storm frequency (e.g. 10% AEP and/or 50% AEP, with the latter only applicable where there is a stream channel erosion protection imperative – refer Section 3.7)
   - o the target peak site outflow; this is typically as existing, or greenfield – refer Section 3.7 (note that a rain tank alone can often meet the greenfield site runoff standard, by over-throttling the flow to compensate for the extra runoff from the site impervious area)
3. Establish applicable design time of concentration (Tc – refer Appendix C, Section C2.2 for further details), e.g.:
   - o in the receiving stormwater system, at the dwelling connection point (= Tc_1 say)
   - o at key points on down to the outfall (e.g. major watercourse, sea); = Tc_2 , Tc_3, etc
4. Define water re-use targets:
   - o define appropriate uses [refer Section 4.5.3.2(a)]
   - o set target percentage of domestic use to be met (e.g. typically 100% for rural and up to 50% in urban areas with mains supply)
   - o establish the drought frequency condition to be met (applicable in rural areas only where the tank is the sole supply source[6])
5. Establish the water re-use demand, in turn a function of:
   - o the number of persons (it is wise to account for the number usually resident in summer, since this is the critical season)
   - o the per capita demand: this can vary from 100 l/h/d (ie where tanks are the sole source of supply and the users are conservation-minded) to 200 - 250 l/h/d (this figure is representative of an urban situation with unrestricted supply and high water-use facilities, e.g. dishwasher, wastemaster)
   - o where garden/lawn watering and the like is to be met from the tank, this should be catered for by allowing a higher demand in summer
6. Establish key site parameters, e.g.:
   - o site area
   - o impervious area (roof and on-ground)
   - o pervious area (and cover type)
7. For multiple dwelling units case – refer Section 4.5.3.2(b)
8. Identify site/device layout constraints, e.g.:
   - o tank location
   - o tank above ground or underground (note that special structural requirements apply in respect to the latter)
   - o stormwater system connection points (and corresponding elevations)
   - o overland flow paths (from tank outlet)

4.5.5.2 Hydrological data

1. Obtain rainfall depth-duration-frequency data applicable to the general area in which the rain tank is to be sited, for the following cases, as applicable (refer Section 3.12):
   - o 50%, 10% and 1%/2% AEP

[6] It will normally be prudent to size tanks to enable the demand to be met over a dry summer: the severity of the drought to be catered for will depend on factors such as:
- the householders willingness to curb demand in a dry period
- the cost of supplementary supply, if available (eg tanker-delivered)

- o applicable Tc values (from 3 above)

2. Using the data from (1) above, establish design storm runoff peaks and hydrographs, according to the rational formula or other method (refer Section 3.12 and Appendix C for details), for:
   - o target site outflow (only the peak flow is required)
   - o roof runoff
   - o rest-of-site runoff (ie surface impervious and pervious)
3. Obtain monthly rainfall sequence applicable to the general area in which the rain tank is to be sited (e.g. from NZMS 1983); options are:
   - o mean monthly totals where security against droughts is not an issue (for example for the urban case)
   - o mean monthly totals for a representative dry year (ie selected from a review of a long-term record, to meet the required severity
   - o the dry year case can be approximated by applying a factor to the mean monthly totals; this is the "dry period factor" (factor values can be found by analysing long-term local records, but are typically in the range 0.9 - 0.85 for the 2 – 5 year return period drought)

4.5.6 Design steps

(a) Summary:

1. collate the design data/parameters from Section 4.5.5
2. size the temporary storage capacity – refer (b) below
3. size permanent storage capacity – refer (c) below
4. add results from 2 & 3 to establish the total storage (ie tank capacity) and size the allied hydraulic components – refer (d) below

Note: it should be appreciated that the design process\set out below is quite long and involved, due to the need to size both the temporary and permanent storages, in turn involving collation and analysis of the requisite hydrological data. The process can be streamlined appreciably through the use of spreadsheets (see examples below and in Section 4.5.10). Note also that some local authorities have undertaken analyses with local data to prepare design charts or tables, and/or spreadsheets developed by others are available, for example:

- for temporary storage:
  - o ARC TP10 (Chapter 11) has charts enabling the reading-off of the tank storage volume for a range of roof area and paved area combinations (applies to meeting the greenfield flow attenuation target, on clay soils)
  - o North Shore City Council has a spreadsheet available to compute the temporary storage (NSCCWS 2002), where the user input the various site and development parameters (note that the January 2002 version current at the time of writing this Guide is understood to be under review)
- for permanent storage:
  - o ARC TP10 (Chapter 11) includes tables of roof area versus demand and relates these to the percentage of the demand that can be met by a tank of a given size
  - o Ashworth, 2002 includes a spreadsheet on CD to size the permanent storage

(b) Temporary storage:

Typically, a spreadsheet is used to size the temporary storage capacity. This involves performing routing calculations to quantify the way the storage provided in the tank modifies the inflow hydrograph (refer Appendix C – Section C3.4 for details); in turn it applies the following general relationships:

Device outflow =	function of the applied head on the outlet flow control device (e.g. orifice, weir)
Change in storage =	device inflow – device outflow
Site outflow =	device outflow + rest-of-site runoff (ie from pervious plus other impervious area)

Table 4.5.1 illustrates the layout of a typical spreadsheet used to perform the tank routing calculation, together with generalised explanations of the cell arithmetic (the worked example in Section 4.5.10 presents the full spreadsheet). The box below that is below Table 4.5.1 discusses points arising from the analysis.

Table 4.5.1 Illustration of spreadsheet-type routing computation

Time (min)	*Roof runoff*		Tank Storage (C, m)	Tank water level (E, m)	Tank orifice outflow (F, l/s)	Net Tank Storage (G, m³)	SITE RUNOFF CALC	
	Hydrograph (A, l/s) *Note 1*	Volume (B, m³)					Rest of Site 3 H (l/s)	Total Site I (l/s)
Go to 2-3 x Tc in about 0.1 x Tc increments	*Design hydrograph (contributing area)*	*= A(l/s) [averaged] x time*	*= volume G at prior time step + inflow B*	*= volume C / tank area*	*Function of head, E refer note 1*	*= volume C – F x time*	*= design hydro-graph for rest of site*	*= tank outflow F + rest of site runoff H*
0	0	0	0	0	0	0	0	0
2.5	1.05	0.16	0.16	0.05	0.31	0.11	1.12	1.42
5.0	2.1	0.32	0.43	0.14	0.59	0.34	2.23	2.83
7.5	…	…	…	…	…	…	…	…
…	…	…	…	…	…	…	…	…

Notes:

1: It is usual to use the average head over the prior and current time steps; note also that once the tank is full, the outflow is set to match the inflow

2: Hydrograph from the pervious plus other impervious area (e.g. as in Section C3.5 – Case 2B below)

The routing computation spreadsheet is used as follows to size the on-site device, involving applying a trial and error approach:

(1) Set storm duration D = Tc_1 (refer Section 4.5.5.1(3) for details)

(2) Compute the corresponding design hydrographs, for the following refer Section 4.5.5.2(2) for details):
  - o roof runoff
  - o rest-of-site runoff (ie surface impervious and pervious)

(3) Select the trial tank sizing parameters:
  - o plan area of tank
  - o top outlet pipe diameter and height above the permanent/temporary storage interface
  - o outlet orifice diameter (ie located at the permanent/temporary storage interface)

(4) Run the spreadsheet and:
  - o identify the peak site outflow rate (also, it is useful to check if/when device overflow occurs)
  - o compare this to the target peak site outflow (e.g. greenfield)

(5) Select new trial device sizing parameters (e.g. smaller/larger tank, smaller/larger orifice) and re-run the spreadsheet: continue until the required device performance standard is met

(6) Re-run steps (2) – (5) with storm duration D = Tc_2, then again for Tc_3, etc (refer Section 4.5.5.1(3) for details)

(7) Select the largest tank capacity arising from the above runs; noting:
  - o this is the "temporary storage" volume, V (m^3)
  - o the corresponding orifice diameter, E (mm), applies

Allied issues for sizing temporary storage

Dual orifice arrangement to meet stream channel erosion protection requirement:

The normal requirement is to size the tank and orifice to meet the required performance standard (e.g. greenfield, or as existing) in a 20% or 10% AEP storm (ie matching the sizing basis for the piped stormwater receiving system. However, where the tank discharges to a watercourse where channel erosion protection is an issue, it may be necessary to attenuate the 50% AEP flood event, over and above that for the 20% or 10% AEP storm and provide extended detention(refer Section 3.7 for details). Often a single orifice cannot easily meet the dual performance requirement, with the result that the tank will have the following (note that the North Shore City Council 2002 rain tank spreadsheet incorporates this provision):

- a small diameter orifice at the permanent/temporary storage interface to meet the 50% AEP requirement
- a larger diameter orifice, located higher up in the tank, to meet the 20% or 10% AEP requirement

Case where the tank cannot meet the flow attenuation performance target:

There may be cases where the spreadsheet identifies that no tank/orifice combination can meet the meet the required flow attenuation performance target; this will be evident when even very large tanks with small orifices cannot meet the required flow target. This situation occurs in cases where the site impervious area is large in comparison to the roof area, because even fully absorbing the tank inflow and throttling the tank outflow is not enough to compensate for the extra runoff from the site impervious area. In such cases the potential solutions are:

- reduce the site impervious area, or
- in conjunction with the tank, use a separate on site device (e.g. rain garden) to attenuate the site impervious area runoff

(c) Permanent storage

Typically, a spreadsheet will be used to perform the permanent storage calculations, applying the following general relationships:

Change in tank storage = inflow (from roof) – outflow (ie demand)
Inflow from roof = rainfall x roof area x loss/drought factor

Note on data sources and assumptions used in the computation:

- security against droughts: refer Section 4.5.5.1(4)
- rainfall: refer Section 4.5.5.2(3)
- losses: not all of the rainfall measured at a rain gauge will reach the tank; correspondingly, a loss factor of 0.05 – 0.10 is typically applied to the rainfall to account for:
  - losses due to wind currents (e.g. on the lee side of a steep-pitched roof, the rainfall settling on the roof will be lower)
  - evaporation losses (e.g. in summer, in light showers especially, the first millimetre or so will evaporate off a hot roof)
- demand: refer Section 4.5.5.1(5)

Table 4.5.2 illustrates the layout of a typical spreadsheet used to perform the permanent storage calculation, together with generalised explanations of the cell arithmetic, accounting for the above factors (the worked example in Section 4.5.10 presents the full spreadsheet). The computation should start in winter with the tank nominally full and continue over successive months until the minimum storage is found (this is typically in late summer, e.g. Feb, March or April) - the required permanent storage (S, m^3) is then equal to the nominal starting/full storage minus the minimum storage. The box below discusses points arising from the analysis.

Points arising from the permanent storage analysis example in Table 4.5.2
In rural cases where the demand is relatively high in comparison to the roof area, large increments in tank capacity will be required to get from a target supply percentage of about 90% to the full 100%: in this case, an "economic" supply percentage can be calculated if required by running different tank sizes and comparing: - the cost of providing the extra tank capacity, versus the alternative of: - buying water – in turn, there is a frequency question
Similarly, in the urban case, especially where dwellings are 2/3-storeyed, it may be uneconomic to meet more than a modest fraction of the demand (e.g. calculations referenced in Auckland City, 2002 show that, in targeting to meet 50% of the total water demand, this cannot be met where the roof area per person is less than 25 m^2 – in such cases, a storage capacity of 1.5 m^3 per person is recommended)

Table 4.5.2 Illustration of tank permanent storage spreadsheet computation

Month	Mean Monthly Rainfall (A, mm)	Inflow (B, m3) [note 1]	Demand (C, m3) [note 2]	Storage Change (D, m3)	Net Storage (E, m3) [note 3]
Start in mid-winter when tank will be full		*Rainfall A (mm) x loss factor(s) x roof area (m2) / 1000*	*Litres/day x days in the month / 1000*	*D = C - B*	*= volume E at prior time step + D (but not greater than full)*
Aug		-	-	-	100.0 (nominal)
Sep	91	15.6	15	0.6	100
Oct	76	13	15.5	-2.5	97.5
Nov	83	14.2	15	-0.8	96.7
Dec	79	13.5	18.6	-5.1	91.6
Jan	67	11.5	18.6	-7.1	84.5
Feb	78	13.3	16.8	-3.5	81
Mar	84	14.4	18.6	-4.2	**76.8**
Apr	94	16.1	15	1.1	77.9
…	..	…	…	…	…

Notes:

1: Figures in example are with:

- roof area 200 m^2
- dry period factor 0.9
- rainfall loss factor 0.05 (ie runoff = 0.95 x rainfall)

2: Demand basis in this example: 4 persons @ 125 l/h/d = 500 l/d, plus extra 100 l/d in summer (December – March)

3: Continue computation over successive months until minimum storage is found; then required storage = nominal starting/full storage – minimum storage (ie in this case, 100.0 – 76.8 = 23.2 m^3)

(d) Tank sizing:

Note*:: the symbols E, F, etc used below are as defined in Fig 4.5.2*

(1) The required total storage, T (m^3), is the sum of:
- temporary storage, V, is as determined in (b) as above
- permanent storage, S, is as determined in (c) as above
- dead storage allowance = 0.1 x (V + S)

(2) Tank size details:
- volume T (m^3, from 1 above)
  (select next largest available tank size)
- base area, R (m^2), is as determined in (b) as above
- diameter K (m) = $1.128 \times R^{0.5}$
- height G (m) = T / R

(3) Orifice at permanent/temporary storage interface:
- Diameter, E (mm), is as determined in (b) as above
- height above tank base = H (m) = S / R + 0.1 m (dead storage)

(4) Compute the top overflow pipe diameter (F, mm) as follows:

(i) Compute the design discharge Q (l/s) to allow the overflow to discharge the 2% AEP storm without the gutters overflowing:
- identify the 2% AEP rainfall intensity for the 10 minute storm = I_2 mm/hr
- for roof area A (m^2), Q (l/s) = $0.00028 \times A \times I_2$

(ii) Use the orifice discharge formula (refer Appendix C – Section C4.0) to compute the orifice diameter, F, ie:

$$Q = 3470 \times Cd \times d^2 \times h^{0.5}$$

where:

Q = discharge (l/s)

Cd = orifice discharge coefficient (typically 0.6 – 0.7)

d = orifice diameter (m)

h = head on orifice (m)

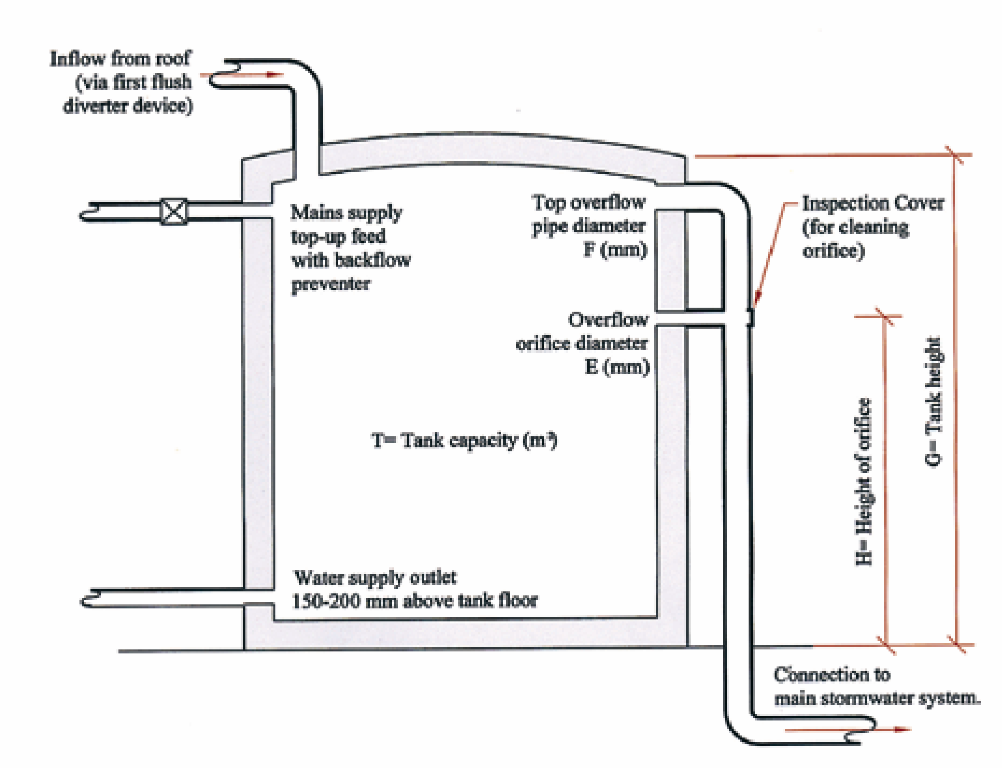
Assuming h = 0.1 m and Cd = 0.6, this can be simplified to:

$F = 39 \times Q^{0.5}$, where Q is in l/s and F in mm

(choose the next largest available pipe size)

As a guide, a 100 mm diameter overflow is sufficient to cater for a roof area of about 200 m^2 in Auckland.

Figure 4.5.2 Rain tank – definition sketch



4.5.7 Design detailing and drawings

The standard design details applicable to rain tanks, to be shown on drawings to be submitted with the consent application(s), are as listed below. In parallel, it will be appropriate to check any regional, city or district council requirements for resource consent, building consent or drainage permit or compliance with other standards.

Rain tank worked example

Note: items shown with an asterisk are only applicable to tanks with a mains water feed.

Item	Requirement
Inlet	To enter through the roof of the tank, via an appropriate first-flush diverter device (e.g. Rain Water Harvesting & Waste Water System P/L's First-flush water diverter, Australian Patent #692835[7], or similar[8]); this device to be sized and installed according to the manufacturer' instructions
*** Mains water feed**	At top of tank, 25 mm minimum above the top outlet and controlled by a float-operated shut-off (minimum level 100 mm above water supply outlet)
*** Backflow preventer**	To be installed to NZS 3500.5 (2000) to avoid cross-contamination
Tank construction:	
- Materials	Concrete, steel, plastic or fibreglass
- Siting/Foundation	Level, on a sand or scoria base (minimum 100 mm depth; where weak sub-soil conditions exists, the foundation to be designed and certified by a geotechnical engineer)
Stormwater outlets:	(refer Fig 4.5.2 for definitions of the parameters referred to below)
- Lower orifice	Diameter E (machine drilled) at height H above base of tank; connect to pipe from top overflow (the orifice is to be accessible for maintenance by an inspection cover)
- Top overflow	Pipe diameter F; connect to main/public stormwater sstem
Water supply outlet:	
- Location	150 – 200 mm above the tank base (ie allow 100 mm dead storage for sediment accumulation)
- Feed to	Plumbing fixtures in dwelling - note that in urban situations, it is recommended that connection is limited to non-potable uses, e.g.: - outdoor watering - toilets - cold water feed to clothes washing machine - cold water feed to shower(s)
Pump	Size to meet the required household duty; plumb so that in the event of pump or power failure, mains water can be used directly
Plumbing	To NZS 3500.5 (2000) and by a certified/registered Plumber. Refer also to Building Industry Authority approved document G12/AS1: Water supply for signage and plumbing identification.

[7] RWHWWS, 2004

[8] The cited product is illustrative of the type of equipment available: note that neither the authors and publishers of this Guide nor NZWERF endorses this or any other proprietary product

4.5.8 Implementation provisions

Following the completion of the design and detailing, the steps to implementation are:

- apply for the appropriate consent (refer Section 3.13 for details of the type of information that will need to be included)
- receive the consent and account for any design changes required under the consent
- construction: requires close attention to ensuring that the following are met:
    - design details
    - specifications, including materials specifications
- commissioning:
    - once constructed, the device will need to be commissioned and tested
    - checks need to be made for flaws such as leaks, blockages, etc
- certification: once commissioned and operating satisfactorily, the device will need to be certified under the provisions of the Building and/or Resource Consent – ARC TP10, Chapter 11 provides examples of the checklists used by certification authorities
- O&M (ongoing): the routine maintenance provisions set out in Section 4.5.9 will need to be undertaken, in accordance with either (as applicable):
    - the provisions of the consent (where nominated), or
    - as per an appropriate O&M model (refer to Appendix D2.0)

4.5.9 Operation and maintenance

The routine maintenance activities that should be undertaken in respect to a rain tank are as tabulated below (note that ARC TP10 – Chapter 11 provides an alternative checklist). It is recommended that the owner be issued with a copy of the checklist, along with a description of the rain tank, covering how it works and explaining the maintenance imperative (refer ACC 2002 – Appendix C, pages C6 and C7 for an example handout).

Frequency				Action
After storm	Quarterly	Annually	2-Yearly	
✓	✓	✓	✓	**Spouting & downpipes:** check for problems such as debris /blockages and leaks and rectify
✓	✓	✓	✓	**First-flush diverter device:** check for blockages; empty debris/sediment
	✓	✓	✓	**Tank water quality:** check for clarity and odour
	✓	✓	✓	**Tank inlet/outlet pipework, orifice, float valve & backflow preventer:** perform visual check for problems e.g. debris/blockages/leaks and rectify
		✓	✓	**Tank structure:** check for leaks and rectify
		✓	✓	**Pump & electrical system:** check and carry out any necessary maintenance
			✓	**Float valve, backflow preventer and first-flush device:** test for correct functioning; repair/replace where faulty or badly worn
			✓	**Tank water quality:** collect water sample (before emptying tank, as below), submit for testing & results to check compliance with DWSNZ, 2000; if exceedances are found, review maintenance practices to identify the cause of the problem(s) and rectify
			✓	**Tank cleaning:** empty the tank and clean out any sediment accumulations and growths
			✓	**Plumbing:** examine plumbing from the tank to the dwelling and rectify any faults

4.5.10 Worked example

4.5.10.1 Case A - Temporary and permanent storage

The worked example below, including spreadsheets for the calculation of the temporary and permanent storage capacities are for the following example case:

Example case A

Base data:

(i) Areas (m^2):

Roof:	250
Other impervious:	100
Pervious:	350
Lot total:	700

(ii) Soil type: clay

(iii) Tank performance standard: attenuate to "greenfield" in 10% AEP storm event

(iv) Applicable time of concentration (Tc): 15 minutes

(v) Storm duration D = Tc (refer Section 4.5.10.2 for an example with D > Tc)

(vi) Water re-use demand: refer input data to the second spreadsheet below

Hydrological data and calculations:

The methodology is the rational method – refer Section C3.2 for details.

(i) Rainfall depth-duration-frequency data (for Tc = 15 mins & 10% AEP) gives I = 100 mm/hr (ie as worked example case 1 in Appendix C – Section C3.5)

(ii) Design hydrographs for the following cases (refer first spreadsheet below for results):

- greenfield: refer worked example case 1 in Appendix C – Section C3.5
- roof: refer worked example case 2A in Appendix C – Section C3.5
- rest-of-site: refer worked example case 2B in Appendix C – Section C3.5

(iii) Mean monthly rainfalls: refer the second spreadsheet below (data is for Albert Park, Auckland)

RAIN TANK - FLOW ROUTING ANALYSIS

(A) SITE DATA:

Soil Type: Clay

AREAS:			C value
Roof area	250	m2	0.9
Other impervious area	100	m2	0.86
Pervious area	350	m2	0.43
Lot area	700	m2	

(B) TANK DETAILS:

Tank area	3.0	m2	(ie	1.9	m dia)	
Tank height	1.2	m				
Orifice dia	0.03	m	D^2=	0.0009		
Orifice discharge coefficient				0.69		

Trial orifice diameter calculation:

Peak orifice flow:	*1.79*	*l/s*
Max orifice head:	*1.2*	*m*
Trial diameter:	*0.026*	*m*

(C) HYDROLOGY - BY RATIONAL METHOD:

(refer comparable calculations in Appendix C - Section C3.5)

Tc	15	min
Storm duration (D)	15	min
Rainfall intensity (10% AEP)	100	mm/hr

	C value	Peak discharge (l/s)
Peak roof discharge:	0.90	6.25
Peak rest-of-site discharge:	0.53	6.57
Permissible site discharge	0.43	8.36

(D) SIMULATION:

Time step 2.5 min

Time	TANK INFLOW		Tank Storage	Tank WL	Adjusted Av WL	Tank Outflow	Net Device Storage	SITE RUNOFF CALC Rest of Site	Total Site
(mins)	l/s	m3	m3	m	m	l/s	m3	l/s	l/s
0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.5	1.04	0.08	0.08	0.03	0.01	0.25	0.04	1.09	1.34
5.0	2.08	0.23	0.28	0.09	0.06	0.53	0.20	2.19	2.72
7.5	3.13	0.39	0.59	0.20	0.15	0.82	0.46	3.28	4.11
10.0	4.17	0.55	1.01	0.34	0.27	1.12	0.84	4.38	5.50
12.5	5.21	0.70	1.54	0.52	0.43	1.42	1.33	5.47	6.89
15.0	6.25	0.86	2.19	0.74	0.63	1.72	1.93	6.57	8.29
17.5	5.21	0.86	2.79	0.94	0.84	1.98	2.50	5.47	7.46
20.0	4.17	0.70	3.20	1.08	1.01	2.17	2.87	4.38	6.55
22.5	3.13	0.55	3.42	1.16	1.12	2.28	3.08	3.28	5.57
25.0	2.08	0.39	3.47	1.17	1.16	2.33	3.12	2.19	4.52
27.5	1.04	0.23	3.35	1.13	1.15	2.32	3.01	1.09	3.41
30.0	0.00	0.08	3.08	1.04	1.09	2.25	2.75	0.00	2.25
32.5	0.00	0.00	2.75	0.93	0.98	2.14	2.42	0.00	2.14
35.0	0.00	0.00	2.42	0.82	0.87	2.02	2.12	0.00	2.02

NOTE: If/when tank WL exceeds tank height, site runoff calculation should include tank overflow (ie overflow = inflow - orifice outflow)

RESULT:

Tank area:	3.0	m2
Tank height:	1.2	m
Orifice diameter:	30	mm
Tank capacity (V)	**3.6**	**m3**

Permanent storage calculation – refer spreadsheet below
(note that for illustrative purposes, a "dry period" factor" of 0.9 is applied: in practice, in urban situations with mains water supply, the factor is normally set at 1.0)

RAIN TANK - REUSE COMPONENT
SIZING OF PERMANENT STORAGE

Nom Tank: 100	m^3		
Roof Area: 250	m^2		
Demand Calculation:	No. persons	5	
	Per capita use	200	l/h/d
	Non-summer	1000	l/d
	Summer xs (Dec - Mar)	100	l/d
	Total summer	1100	l/d
Target % of total demand to be met from tank:		50	%
Rainfall loss factor:	0.05	(ie runoff = 0.95 x rainfall)	
Dry period factor:	0.9		

Month	Average Rainfall (mm)	INFLOW (m^3)	DEMAND (m^3)	Storage Change (m^3)	Net Storage (m^3)	
Jul	118	25.22	15.50	9.72	100.00	(= start full)
Aug	118	25.22	15.50	9.72	100.00	
Sep	91	19.45	15.00	4.45	100.00	
Oct	76	16.25	15.50	0.75	100.00	
Nov	83	17.74	15.00	2.74	100.00	
Dec	79	16.89	17.05	-0.16	99.84	
Jan	67	14.32	17.05	-2.73	97.11	
Feb	78	16.67	15.40	1.27	98.38	
Mar	84	17.96	17.05	0.90	99.29	
Apr	94	20.09	15.00	5.09	100.00	
May	100	21.38	15.50	5.88	100.00	
Jun	124	26.51	15.00	11.51	100.00	
Ann. Total:	1112			Minimum	97.11	m^3
			Required Storage		2.89	m^3

Tank sizing:

(1) total storage, T:

- temporary storage, V (from first spreadsheet above): = 3.6 m^3
- permanent storage, S, (from second spreadsheet above): = 2.9 m^3
- dead storage allowance, D = 0.1 x (V + S) = 0.7 m^3
- total storage, T = V + S + D = 7.2 m^3

(2) Tank size details:

- volume, T (from 1 above) = 7.2 m^3
  (select next largest available tank size)
- base area, R (from first spreadsheet above, to match sizes/diameters available from manufacturers), = 3 m^2
- height, G = T / R + 0.1m (dead storage) = 2.5 m

Orifice at permanent/temporary storage interface:

- diameter, E (from first spreadsheet above) = 30 mm
- height above tank base H = S / R + 0.1 m (dead storage) = 1.1 m

(3) Top overflow pipe diameter (F):

Compute according to the formulae set out in Section 4.5.6 d (4), ie:

Design discharge Q = 0.00028 x A X I_2, where:

A = roof area = 250 m^2

I_2 = 2% AEP rainfall intensity for the 15 minute storm = 140 mm/hr

Whence Q = 9.8 l/s

Top outlet diameter, F = 39 x $Q^{0.5}$ = 122 mm (choose next largest available pipe size)

4.5.10.2 Case B - Temporary storage with longer duration storms

A spreadsheet is set out below for the calculation of the temporary storage for the same case as in Section 4.5.10.1, but for the situation where storm duration (D) exceeds the time of concentration (Tc). Section C2.2 provides an explanation as to where this will apply; in essence this is where the rain tank needs to meet a flow control target in the downstream receiving system rather than at the outlet to the dwelling site (refer also Section 4.5.5.1(3) for application details). Fig 4.5.3 shows a plot of the relevant hydrographs.

Specifically:

(i) Tc = 15 minutes (ie as in Section 4.5.10.1)

(ii) Storm duration (D) = 30 minutes

(iii) Rainfall depth-duration-frequency data (for 30 mins & 10% AEP) gives I = 70 mm/hr

(iv) Design hydrograph derivation: refer Section C3.3 and Fig. C2b for an illustration of the corresponding hydrograph shape

Note that the tank size derived for this case (ie 4.8 m^2 x 1.2 m) is 60% larger than for the Section 5.4.10.1 example (ie 3.0 m^2 x 1.2 m).

RAIN TANK - FLOW ROUTING ANALYSIS

(A) SITE DATA:

Soil Type: Clay

AREAS:			C value
Roof area	250	m2	0.9
Other impervious area	100	m2	0.86
Pervious area	350	m2	0.43
Lot area	700	m2	

(B) TANK DETAILS:

Tank area	4.8	m2	(ie	2.5 m dia)	*Trial orifice diameter calculation:*	
Tank height	1.2	M			*Peak orifice flow:*	*1.25 l/s*
Orifice dia	0.023	M	d^2	0.000529	*Max orifice head:*	*1.2 m*
Orifice discharge coefficient				0.69	*Trial diameter:*	*0.022 m*

(C) HYDROLOGY - BY RATIONAL METHOD:

(refer comparable calculations in Appendix C - Section C3.5)

Tc	15 min		
Storm duration (D)	30 min		
Rainfall intensity (10% AEP)			70 mm/hr
		C value	Peak discharge (l/s)

Peak roof discharge:	0.90	4.38
Peak rest-of-site discharge:	0.53	4.60
Permissible site discharge	0.43	5.85

(D) SIMULATION:

Time step 2.5 Min

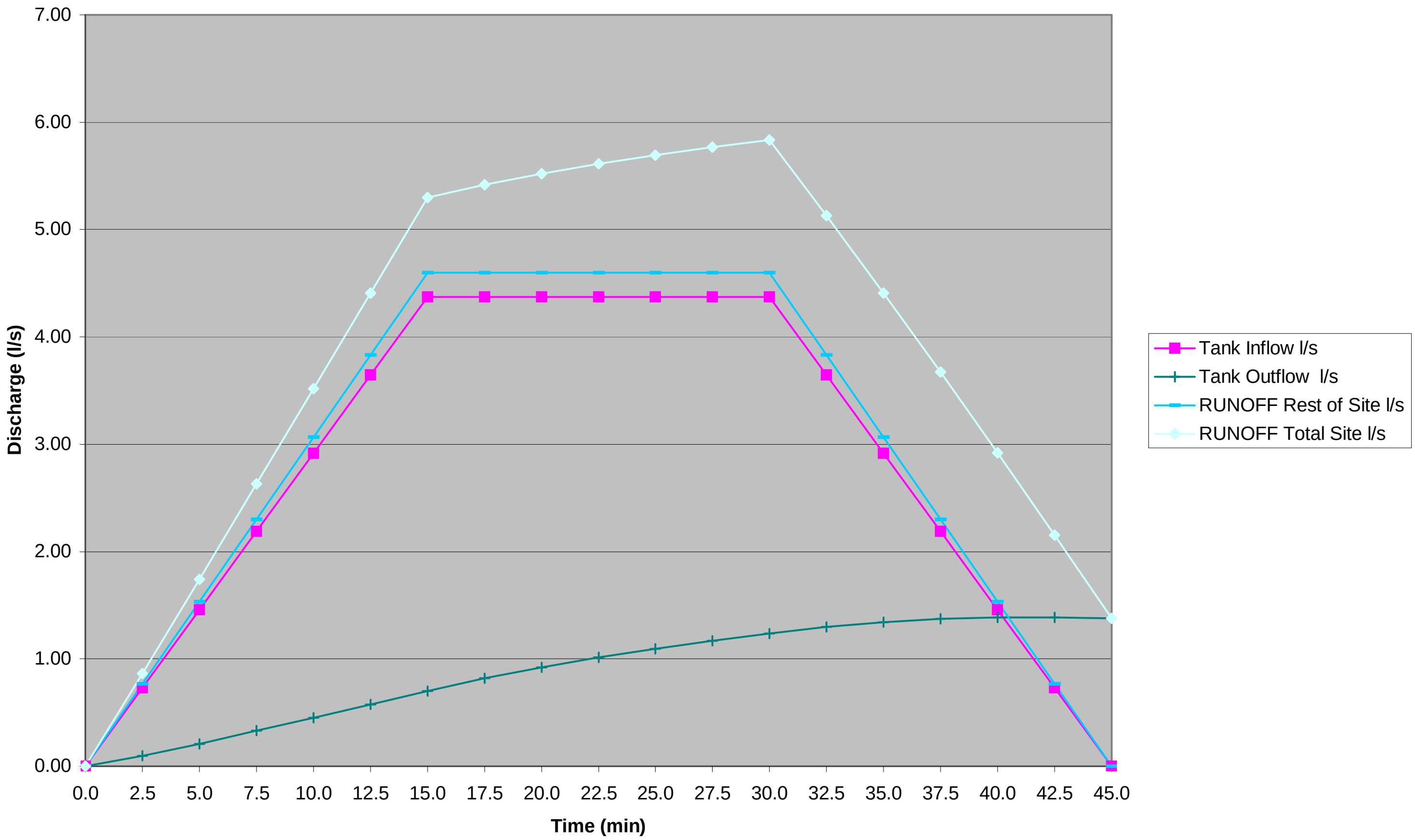
	TANK INFLOW		Tank		Adjusted	Tank	Net Device	SITE RUNOFF CALC	
Time			Storage	Tank WL	Av WL	Outflow	Storage	Rest of Site	Total Site
(mins)	l/s	m3	m3	m	m	l/s	m3	l/s	l/s
0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.5	0.73	0.05	0.05	0.01	0.01	0.10	0.04	0.77	0.86
5.0	1.46	0.16	0.20	0.04	0.03	0.21	0.17	1.53	1.74
7.5	2.19	0.27	0.45	0.09	0.07	0.33	0.40	2.30	2.63
10.0	2.92	0.38	0.78	0.16	0.13	0.45	0.71	3.07	3.52
12.5	3.65	0.49	1.20	0.25	0.21	0.58	1.12	3.83	4.41
15.0	4.38	0.60	1.72	0.36	0.30	0.70	1.61	4.60	5.30
17.5	4.38	0.66	2.27	0.47	0.42	0.82	2.15	4.60	5.42
20.0	4.38	0.66	2.80	0.58	0.53	0.92	2.67	4.60	5.52
22.5	4.38	0.66	3.32	0.69	0.64	1.01	3.17	4.60	5.61
25.0	4.38	0.66	3.83	0.80	0.74	1.10	3.66	4.60	5.69
27.5	4.38	0.66	4.32	0.90	0.85	1.17	4.14	4.60	5.77
30.0	4.38	0.66	4.80	1.00	0.95	1.24	4.61	4.60	5.84
32.5	3.65	0.60	5.21	1.09	1.04	1.30	5.02	3.83	5.13
35.0	2.92	0.49	5.51	1.15	1.12	1.34	5.31	3.07	4.41
37.5	2.19	0.38	5.69	1.19	1.17	1.37	5.49	2.30	3.67
40.0	1.46	0.27	5.76	1.20	1.19	1.39	5.55	1.53	2.92
42.5	0.73	0.16	5.72	1.19	1.20	1.39	5.51	0.77	2.15
45.0	0.00	0.05	5.56	1.16	1.18	1.38	5.36	0.00	1.38

NOTE: If/when tank WL exceeds tank height, site runoff calculation should include tank overflow (ie overflow = inflow - orifice outflow)

RESULT:

Tank area:	4.8	m2
Tank height:	1.2	m
Orifice diameter:	23	mm
Tank capacity (V)	**5.8**	**m3**

Figure 4.5.3 Hydrograph plots for Case B

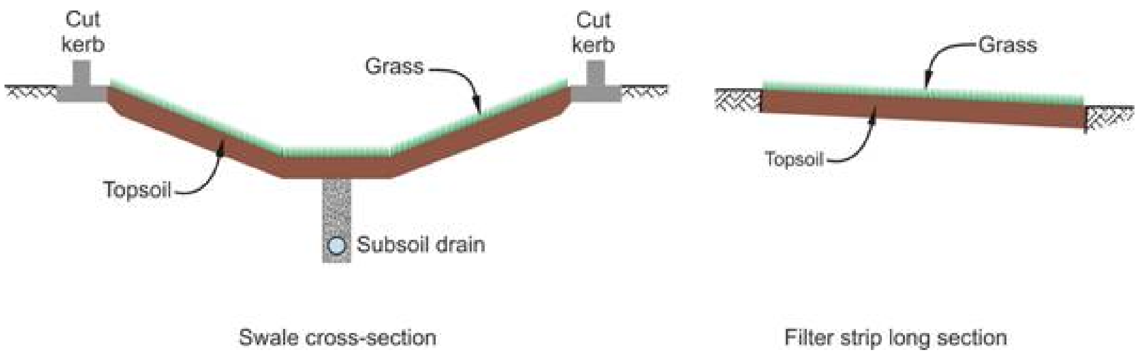


4.6 Swale / filter strip

4.6.1 Description

These devices use vegetation in conjunction with slow and shallow depth of flow to achieve treatment of stormwater. Removal of contaminants is achieved by a combination of filtration, adsorption and biological uptake. Vegetation also decreases flow velocity and allows settlement of particulates. The principal difference between swales and filter strips is that swales accept concentrated flow while filter strips accept distributed or sheet flow

Figure 4.6.1 Swale / filter strip operating principles



4.6.2 Capability

Swales and filter strips are able to:

- treat runoff from impermeable hardstand ground surfaces in commercial, residential and industrial areas
- treat road or parking lot runoff
- provide aesthetic benefit

Swales and filter strips are not able to:

- treat sediment-laden water from construction sites. Install after site works are complete and contributing areas have been fully stabilised in order to prevent excess sediment loading
- provide significant peak flow or volume control

Expected contaminant removal rates for swales / filter strips are (ARC 2003, EPA 1999d):

- suspended solids 81%
- metals (cadmium, copper, zinc, lead) 50 to 90 %
- total phosphorus 9 %
- nitrate 38%
- oxygen demanding substances 67%
- hydrocarbons 62%

4.6.3 Applicability

- can be located in;
    - o median strips in car parks or substitute for kerb and gutter at the side of roads, with kerb cuts to allow entry of runoff
    - o adjacent to site boundaries
- on line or off line location
- for impermeable subsoils, minimum longitudinal slope of 0.5% to avoid pugging of soil
- maximum longitudinal slope: 5% without erosion protection or check dams
- swales require minimum length of 30 m
- maximum drainage flow path to a filter strip is 50 m
- maximum longitudinal slope of contributing slope to a filter strip is 5% unless energy dissipation is provided
- maximum lateral slope of a filter strip is 2%
- require area open to sun, avoid or minimise shading (to encourage vegetation growth)
- device catchment area no more than 4 ha (ARC TP10)
- time of concentration not to exceed 10 minutes
- take care to ensure adequate subsoil drainage is provided in situations where additional infiltration into the subsurface may cause problems, for example adjacent to parking areas or roads, where infiltrating water may weaken the pavement
- use cut kerbs or other measures to prevent vehicles driving on swales

4.6.4 Summary of design approach

Note: This is consistent with ARC TP10

1. Determine the water quality flow rate , refer to section 3.6
2. Adopt trial swale/filter strip cross-section and slope
3. Calculate water quality depth and velocity for water quality flow rate
4. Check that flow depth and velocities are less than allowed maxima and check the hydraulic residence time is at least 9 minutes - this hydraulic residence time is recommended in ARC TP10 and in Minton (2002)

4.6.5 Preparatory steps

1. Confirm quality objective: refer section 3.6
2. Define key site parameters and device needs that determine design details
    - device catchment land use (this is required to be used in design calculations)
    - device catchment impervious area (roof and on-ground areas)
    - device catchment pervious area and cover type (e.g. grass, shrubs, forest)
    - ground slope at location of swale/filter strip
        - o if slope is under 2%, a subsoil drain is required under the base of the swale
        - o if slope is over 5% filter strip is not appropriate
        - o for slopes between 5% and approximately 8%, check dams are required to reduce effective grade to 5% or less

  - o for slopes over 8% swales are unlikely to be appropriate as large check dams will be required
- define maximum flow capacity requirements for the area to be drained and locate overland flow paths for flows in excess of the capacity of the swale/filter strip
- check any regional, city or district council requirements for resource consent, building consent or drainage permit or compliance with other standards.
- provision of adequate access for maintenance

4.6.6 Design steps

4.6.6.1 Sizing for water quality design

Design parameters

- determine the water quality flow rate, refer to section 3.6
- swale cross-section; bottom width between 0.6 and 2m, maximum side slope 1 vertical on 3 horizontal
- longitudinal ground slope and slope. for situations where check dams are required for swales, the effective slope is the slope between the downhill base of one check dam and the crest of next downslope check dam
- swale length (minimum 30m)
- grass height, choose either 50 mm or 150 mm

Design/sizing methodology

Note: This method is generally consistent with design methods per ARC TP 10

1. Adopt trial swale/filter strip cross-section and slope
2. Calculate effective slope
3. Calculate flow depth and velocity for the water quality flow using Mannings equation. Note that this method, including formulae for calculating Mannings n values, is described in ARC TP10
4. Check flow depth is less than
   - 100 mm for swales
   - 25 mm for filter strips
5. Check velocity is less than:
   - 0.8 m/s for swale
   - 0.4 m/s for filter strip
6. If flow depth and velocity criteria are satisfied, proceed to next step, otherwise consider the following options:
   - adopt new trial swale/filter strip cross-section and / or slope. Swale longitudinal slope can be reduced by using check dams.
   - divide the site drainage to flow to multiple swales to reduce the size of the flow per swale/filter
7. Calculate residence time in swale/filter strip. The minimum hydraulic residence time for the water quality flow is at least 9 minutes to achieve the nominated contaminant reductions. If the residence time is less than 9 minutes, revise swale/filter strip cross-section, slope and/or length and recalculate. If the minimum residence time cannot be achieved, use another treatment device or use swale/filter strip in conjunction with another device.
8. Calculate peak flow for the 10 year ARI storm.

9. Calculate 10 year ARI flow velocities using Mannings equation for the 10 year flow. If velocity is greater than 1.5 m/sec, enlarge swale/filter strip size and recalculate. If swale/filter strip size is as large as practical and the 10 year ARI flow velocity is >1.5 m/s, provide erosion protection.
10. Safety check: calculate the mean annual food flow. Calculate flow depth, D and velocity, V using Mannings equation. V x D should not exceed the following:
    - for children: not greater than 0.2 m^2/s
    - for adults not greater than 0.4 m^2/s

4.6.7 Design detailing and drawings

Inlet

- care needed for concentrated inflows to reduce velocity quickly to minimise erosion potential; riprap pads or level spreaders should be used

Cut kerbs

- use cut kerbs or similar to prevent vehicles driving on swales

Swale base

- base width to be no less than 600 mm to facilitate mowing and no greater than 2 m to prevent concentration of flow
- base to be flat , level spreader boards at 15 m centres are useful to prevent concentration of flow, especially for wide bases

Swale depth

overall swale depth to take into account overall drainage requirements for the area served. A common approach is to size the swale and associated depth for the 10 year ARI flow

Filter strip crossfall

crossfall not to exceed 2%

Check dams

o to be used at 15 m centres along swale or filter strip if slope is greater than 5%

Topsoil and vegetation

- minimum topsoil depth of 150 mm
- topsoil to be of good quality and appropriate to support dense grass
- vegetation to be a dense stand of uniform grass or other fine stemmed plants that can tolerate soil saturation and the climatological and pest conditions of the location
- grass length to be maintained at between 50mm and 150mm

Filter fabric

- used to prevent migration of topsoil to underlying subsoil drain

Subsoil drain

- required under base of swale/filter strip if longitudinal grade is less than 2%
- required to protect adjacent pavement subgrades from saturation

ARC TP10 requirements

- check ARC TP10 requirements for detailed requirements for check dams, level spreader boards etc

Check council requirements

- check any regional, city or district council requirements for resource consent, building consent or drainage permit or compliance with other standards

4.6.8 Implementation provisions

Following issuing of the consent, construction will requires close attention to ensuring that the design details and materials specifications in particular topsoil and grass.

Once constructed, the device will need to be commissioned and tested if practical. In the event that the device is commissioned during a dry spell, in some cases it may be appropriate to test the device using a high-capacity hose (e.g. from hydrant or tanker, feeding water to the roof or site impervious area).

Checks need to be made for flaws such as evidence of scour, etc.

Certification: once commissioned and operating satisfactorily, the device will need to be certified under the provisions of the Building and/or Resource Consent – ARC TP10 provides examples of the checklists used by certification authorities.

O&M (ongoing): the routine maintenance provisions set out below will need to be undertaken, in accordance with either (as applicable):

- the provisions of the consent (where nominated), or
- as per an appropriate O&M model (refer to Appendix D2.0)

Operation and maintenance

Item	**Frequency**
Clear debris, litter from entry and contributing areas	As required, at least quarterly
Mow grass to keep height between 50 mm and 150 mm	As required, at least quarterly
Check that there is a thick growth of grass or other approved thin stemmed vegetation. Reinstate vegetation as necessary, remove undesirable vegetation,	As required, at least quarterly
Check that flow is evenly dispersed, remedy concentrated flow or erosion damage by revegetation, earthworks or installation of level spreaders or additional check dams	As required, at least quarterly
Removal of accumulated sediments, restore vegetation as required	As required, at least annually

4.6.9 Worked example

Design of swale

Job name	example
Location	somewhere

design objective	Water quality		100 year ARI	
catchment land use	residential			
impervious area type	seal			
pervious area type	grass, shrub			
catchment imperv area	0.7	ha	0.7	ha
catchment perv area	0.2	ha	0.2	ha
time of concentration	10	min	10	min
rain intensity source	HIRDS 10 min	I/3 2 yr	HIRDS	
rain intensity	18	mm/hr	132	mm/hr
C impervious	0.9		0.9	
C pervious	0.18		0.18	
Catchment CA	0.666		0.666	98
Design Flow	**0.033**	**m^3/s**	**0.244**	**m^3/s**

Swale longitudinal slope S .02
Swale grass height 50 –150 mm
Mannings n = 0.013 $d^{-1.2}$ / (0.75+25s) For d > 60 mm, 150 mm grass ref ARC TP10
Mannings n = 0.009 $d^{-1.2}$ / (0.75+25s) For d > 75 mm, 50 mm grass
Swale shape Trapezoid

Trial calculation using Mannings equation, for water quality flow: select depth to provide calculated flow to match design flow, determine length required for water quality (to provide hydraulic residence time of 9 minutes)

depth, d	bott width	side batter	top width	area	wet. perim p	$r^{2/3}$	slope s	grass length	n	vel. V	flow Q	minimum swale length
m	m	1 on -	m	m^2	m			mm		m/s	m^3/s	m
0.095	2	3	2.57	0.22	2.60	0.19	0.02	150	0.175	0.15	0.033	83
0.084	2	3	2.50	0.19	2.53	0.18	0.02	50	0.141	0.18	0.033	95

Thus flow depth is less than 100 mm; OK. Required swale length is between 83 m and 95 m depending on grass length. Note velocity is well below maximum allowed (0.8 m/s).

Trial calculation for checking swale depth, velocity and safety for 100 year ARI flow: select depth to provide calculated flow to match design flow.

depth, d	bott width	side batter	top width	area	wet. perim p	$r^{2/3}$	slope S	grass length	n	vel V	Q	
m	m	1 on -	m	m^2	m			mm		m/s	m^3/s	
0.19	2	3	3.14	0.49	3.2	0.28	0.02	150	0.076	0.53	0.257	
0.165	2	3	2.99	0.41	3.04	0.26	0.02	50	0.063	0.59	0.244	

Thus required swale depth for 100 year ARI is 0.19 m, velocity is OK as less than max allowed of 1.5 m/s.
Safety: v x d= 0.04 < 0.2 m^2/s OK

4.7 Wetlands

4.7.1 Description

There are two general types of constructed wetlands, surface flow and subsurface flow. Surface flow wetlands mimic natural wetlands and are shallow open ponds with permanent water and submerged and emergent plants. Subsurface flow wetlands include a gravel substrate, which acts as a filter. They are prone to blockage and have high maintenance requirements. The following detailed discussion refers to surface flow wetlands.

Figure 4.7.12 Wetland operating principles

Stormwater flowing through a wetland provides treatment by a variety of mechanisms including settling, filtration, biological degradation, microbial uptake, adsorption, volatilisation and plant uptake. Wetlands can also provide peak flow attenuation and extended detention and landscape and wildlife habitat benefit. Wetlands have a permanent pool ponding volume and an associated permanent pool water level. When stormwater inflows occur, the wetland water level rises above the permanent pool level and the additional storage associated with this rise in water level achieves peak flow attenuation and if the wetland is appropriately designed, provides extended detention.

4.7.2 Capability

Wetlands are able to:

- treat runoff from impermeable hardstand ground surfaces in commercial, residential and some industrial areas, including parking lot runoff. They are well suited for removal of sub 100 micron particulate matter and dissolved chemicals

Expected contaminant removal rates are:

- sediment 60 to 80% (CCC 2003)
- trace metals 40 to 80 % (CCC 2003)
- total phosphorus 40 to 80% (CCC 2003)
- total nitrogen 20 to 60% (CCC 2003)
- BOD 20 to 40% (CCC 2003)
- petroleum hydrocarbons 87% (EPA 1999e)
- bacteria 60 to 100% (CCC 2003)

Wetlands may be able to:

- remove organic contaminants through adsorption, volatilisation, photosynthesis and biotic/abiotic degradation (ARC TP10)
- provide significant peak flow reduction and associated flood protection
- provide extended detention and thus can be used for stream channel protection
- provide aesthetic benefit

4.7.3 Applicability

- require summer baseflow or minimum catchment size to prevent wetland drying out in summer
- minimum catchment size for Auckland area is recommended to be 1 ha. (ARC TP10)
- require impermeable soil base or liner to prevent leakage and potential groundwater contamination
- on line or off line location (refer to glossary for definition)
- require relatively flat ground, maximum ground slope: 5%
- avoid unstable ground
- adequate clearance to existing utilities and to site boundaries
- location of piped outlet to discharge to pipe reticulation or surface dispersal

4.7.4 Summary of design approach

1. Confirm catchment area sufficient, and/or base flows will be sufficient to prevent drying out of wetland in summer.
2. Determine the size required to meet:
    - water quality objectives
    - flood protection peak flow control objectives and extended detention for stream channel protection objectives
3. Check that a device of the required size can be built on the site for all relevant objectives. A device sized to meet the most onerous objective will meet the others
4. If a device of the size required to meet a water quality/peak flow/quantity objective cannot be built on the site but a smaller device will be able to meet a less onerous objective, then adopt the sizing for that less onerous objective and select a separate device to meet the more onerous objective

4.7.5 Preparatory steps

1. Confirm design imperatives
    - quality objective: refer section 3.6
    - peak flow control and stream channel protection: refer section 3.7
2. Define key site parameters and device needs that determine design details
    - device catchment land use (this is required to be used in design calculations)
    - device catchment impervious area (roof and on-ground areas)
    - device catchment pervious area and cover type (e.g. grass, shrubs)
    - for final discharge by infiltration to ground, refer to ground disposal assessment requirements in Section 3.8 and 3.10
    - check any regional, city or district council requirements for resource consent, building consent or drainage permit or compliance with other standards.
    - provision of adequate access for maintenance

4.7.6 Design steps

4.7.6.1 Sizing for water quality design

The ARC method provides a permanent pool equal to the water quality volume with no allowance for porosity of the wetland permanent pool associated with wetland plants. It also allows for water quality benefit in addition to the permanent pool if extended detention is provided. The Christchurch City Council method recommended procedure is to provide a hydraulic residence time of 2 days for at least the first flush and use an assumed vegetation porosity of 0.75.

The recommended approach for this guideline for areas outside the Auckland region and Christchurch City is as follows:

- for typical urban areas, including car parks, low to medium trafficked roads, to provide treatment of sediment, metals and hydrocarbons: provide for at least 1 day hydraulic residence time for the water quality volume using an assumed porosity of the permanent pool of 0.75
- for areas with high contaminant loadings such as busy roads or industrial sites with particular contaminants of concern or for sites where nutrient removal is required: hydraulic residence times of 2 days or more may be required and specialist advice is recommended

Design parameters

- determine water quality volume (WQV) from the appropriate method in section 3.6
- design water level depth: ARC recommendations are:
    - 40% of the wet pool area to be between 0.5 and 1m depth
    - 60% of the wet pool area to be 0 to 0.5 m depth
    - provide banding so there are open areas and vegetated areas and water passes through both sequentially
    - consider safety – use shallow fringe areas
    - care with planting and bed levels to avoid short circuiting
    - need to consider mosquitos if close to residential areas; mosquitos can best be controlled by the establishment of dense vegetation in shallow water and adjacent to the wetland to provide habitat for mosquito predators
- include forebay or separate pond before wetland to capture coarse sediments
- forebay:
    - volume to be 15% of the water quality volume
    - maximum depth of 2 m
    - length to width ratio of between 2:1 and 3:1
    - provide for energy dissipation and even distribution of flow into the wetland
    - minimum length to width ratio for wetland is 2:1 (EPA1999e).

Design/sizing methodology

The required wetland treatment volume, V = WQV x HRT / n

Where:

WQV = water quality volume, m^3

HRT = hydraulic residence time days, refer discussion above

n = wetland permanent pool porosity assume 0.75

The required volume V will be:

- where extended detention is not provided, V = the permanent pool ponding volume
- where extended detention is provided, V = the permanent pool volume plus the temporary storage volume above the permanent pool level provided by extended detention

Determine pond dimensions using the permanent wet pool volume, site topography, available area for the wetland and preliminary construction details such as embankment batters. Obtain specialist geotechnical advice as necessary regarding maximum embankment heights, batters and crest widths

4.7.6.2 Sizing for peak flow control and extended detention

Peak flow control and extended detention are achieved by temporary ponding of water above the wetland permanent water level during a rainfall event. The amount and duration of ponding is dependant on the inflow hydrograph, the characteristics of wetland storage above the permanent water level and the outlet flow characteristics.

Design parameters

- determine catchment parameters, including time of concentration, C values, refer to Appendix C
- determine rainstorm ARI and duration to be considered and associated rainfall depth

Design/sizing methodology

- assess a maximum ponding depth, above the permanent pool water level based on site topography, available area for the wetland and preliminary construction details such as embankment batters and fill or cut soil properties. Obtain specialist geotechnical advice as necessary regarding maximum embankment heights, batters and crest widths
- refer to ARC TP10 section 5 for description of suitable outlet design and Appendix C hydrology for routing methodology
- adopt trial wetland dimensions
- generate hydrograph for existing situation
- generate inflow hydrograph for developed situation
- adopt a trial outlet design, calculate outflow characteristics and route inflow hydrograph (developed) through the wetland
- if objectives are not achieved, decide whether a larger device is practical for the site. If so, increase the surface area and maximum water height to the practical maximum and recalculate the routing calculations
- if the required peak flow control and extended detention objectives can be achieved by the revised design, confirm the device feasibility in relation to the site characteristics, especially topography and available area

Determine device size

- check that the required size can be achieved on the site for all relevant objectives. If so, the device is sized to meet the most onerous objective will meet other objectives
- if a device of the size required to meet a water quality/peak flow/quantity objective cannot be built on the site but a smaller device will be able to meet a less onerous objective, then adopt the sizing for that less onerous objective and select a separate device to meet the more onerous objective

4.7.7 Design detailing and drawings

Inlet forebays

All principal inflow points to be provided with forebays to be designed to trap coarse sediments and be readily accessible for removal of accumulated sediment.

Embankment design

Any embankments must be appropriately designed and constructed to take account of hydrostatic pressure and minimise the risk of slope instability or piping

Permanent pond liners

Lining of the permanent pond to ensure minimal leakage must be achieved by the use of appropriate compacted soil, which may be insitu soils if appropriate or a geotextile liner.

Soil for plant establishment

Place organic soil in the base of the wetland to assist with plant establishment.

Plants

Suitable plant types for the Auckland region are presented in ARC TP10, section 6.9. For other areas of New Zealand, contract appropriately qualified landscape gardeners/architects or regional council staff/publications for advice.

Outlets

- forebay outlet weir to have a length at least 50% of the forebay width
- excess flow by pass to be provided around both he forebay and the wetland
- flow velocities in wetland during the 5 year ARI storm to be less than 0.25 m/s to avoid re-suspension of sediment

Council requirements

Check any regional, city or district council requirements for resource consent, building consent or drainage permit or compliance with other standards.

4.7.8 Implementation provisions

Following the issuing of the consent, the steps in implementing the on-site device are:

- construction: requires close attention to ensuring that the following are met:
    - design details
    - materials specifications in particular soil liner or geotextile
    - protection from sediment entry if catchment is unstabilised during construction
    - specifications

- commissioning:
  - once constructed, the device will need to be commissioned and tested
  - in the event that the device is commissioned during a dry spell, in some cases it may be appropriate to test the device using a high-capacity hose (e.g. from hydrant or tanker, feeding water to the roof or site impervious area) or wait until significant rain occurs
  - checks need to be made for "flaws" such as leaks, blockages, evidence of scour, etc

- certification: once commissioned and operating satisfactorily, the device will need to be certified under the provisions of the Building and/or Resource Consent – ARC TP10 provides examples of the checklists used by certification authorities

- O&M (ongoing): the routine maintenance provisions set out below will need to be undertaken, in accordance with either (as applicable):
  - the provisions of the consent (where nominated), or
  - as per an appropriate O&M model (refer to Appendix D2.0)

4.7.9 Operation and maintenance

Item	**Frequency**
Clear debris, litter from forebay, planted wetland and outlet	As required
Remove noxious weeds and plants	As required but inspect at least quarterly
Check plant species presence, abundance and condition, prune excessive vegetation, replace plants if necessary plants may require watering or replanting during the first three years	As required, but at least 6 monthly
Check that that water is retained in the base of the wetland during dry weather.	6 monthly
Outlet /overflow spillway: check condition, scour, erosion, blockage	6 monthly
Check for mosquito breeding, augment planting as required	6 monthly
Sediment accumulation in forebay: remove if more than 50% of its design volume is occupied with sediment	Annually

4.7.10 Wetland worked example

Job name	Example		
Location	Gisborne		
design objective	Water quality		
catchment land use	residential		
impervious area type	seal		
pervious area type	grass, shrub		
catchment impervious area	8000	m^2	
catchment pervious area	2000	m^2	
catchment time of concentration	10	min	
C impervious	0.83		
C pervious	0.18		
Water quality design storm depth	32.6	mm	1/3 of 2 year 24 hour rainfall from HIRDS
runoff from impervious area = rain - 2 mm	30.6	mm	
pervious area soil drainage	slow		
pervious area depr storage and infiltration	15	mm	
pervious area runoff = rain - depr. stor. & infiltr.	17.60	mm	
total runoff =WQV	280.0	m^3	
assume porosity of wetland water/vegetation, n	0.75		
hydraulic residence time, HRT	1	day-	
required permanent wet pool volume =	WQV x HRT / n		
required permanent wet pool volume =	373.3	m^3	
no extended detention			
trial wetland total surface area	660	m^2	
volume of 0.5 -1.0 m depth (60% area, assume average depth 0.75m)	297	m^3	
volume of 0-0.5 m depth (40% area, assume average depth 0.3m)	79.2	m^3	
trial volume total	376.2	m^3	matches reqd OK

4.8 References

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