# The countryside living TOOLBOX

## Stormwater Management Device Design Details

April 2010

  

PAPAKURA DISTRICT COUNCIL

Waitakere City Council
Te Taiao o Waitakere

Franklin DISTRICT COUNCIL

Rodney DISTRICT COUNCIL

Auckland Regional Council
TE RAUHĪTANGA TAIAO

# THE COUNTRYSIDE LIVING TOOLBOX:

## A GUIDE FOR THE MANAGEMENT OF STORMWATER DISCHARGES IN COUNTRYSIDE LIVING AREAS IN THE AUCKLAND REGION

April 2010

There are 4 publications in this series

**The Countryside Living Toolbox**: Background

**The Countryside Living Toolbox:** Site Design and Prevention of Stormwater Effects

**The Countryside Living Toolbox:** Stormwater Management Device Design Details

**The Countryside Living Toolbox:** Water Supply Public Health Guidelines and Wastewater Management Considerations

*Acknowledgement:*

*This Toolbox is Version 4.0 of several original documents done by and on behalf of the Rodney District Council and the Waitakere City Council over the past eight years. It has borrowed from the earlier versions where changes were not needed and it supercedes those documents.*

Permission was given by Rodney District Council and Waitakere City Council to use information from the earlier documents where use of that information was appropriate.

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## Documents in the Series

The Countryside Living Toolbox is divided into 4 publications.

**Countryside Living Toolbox: Background and Application** – This section defines the applicability of the Toolbox; provides background information on stormwater effects in rural areas; details the regulatory context of this guideline; describes the key stormwater design objectives and approaches; and summarises the different techniques available for use.

**Countryside Living Toolbox: Site Design** – This section provides information on how site design can affect the volume and rate of stormwater which is discharged as a result of development. This section of the Toolbox will assist developers to “avoid” or “prevent” effects.

**Countryside Living Toolbox: Stormwater Management Device Design Details** – This section provides design information for structural stormwater practices. Ponds, wetlands, filter strips, swales, rain gardens, infiltration trenches and rain tanks are discussed. This section of the Toolbox will assist developers to “mitigate” effects.

**Countryside Living Toolbox: Water Supply and Wastewater Management Considerations** – This section of the Toolbox briefly discusses requirements relating to both potable and non-potable water supply. It also provides an overview of the design features and maintenance considerations associated with on-site wastewater treatment and disposal systems.

## Disclaimers

Waitakere District Council
In situations where there are differences to the earlier versions and where they have been relied on or embodied into planning documents such as Structure Plans or Resource Consents then the requirements of the earlier versions shall take precedence over Version 4.0.

Rodney District Council
Infiltration in Rodney District Council
Rodney District Council has significant areas of countryside where soil stability is strongly dependent on and particularly sensitive to changes in moisture content and the hydrological cycle in general.
for that reason infiltration as a means of stormwater management is not seen as a viable management tool.
Water Supply for re-use
This section is not applied in Rodney District Council. RDC has its own provisions for re-use. Where a particular re-use application is required RDC can make available a protocol for calculating storage v consumption requirements to estimate tankage against supply.

Papakura District Council
The guideline provides for a number of methods and tools to mitigate the effects of storm water run-off from countryside living areas but the acceptance of any particular method and tool will depend with the respective TLA (PDC) . The extent and sharing of responsibility by TLA, property owners and developers to ensure continued performance from these methods and tools is not covered in this guideline and will depend on the consenting and approval processes of respective TLA.

# Countryside Living Toolbox:
## STORMWATER MANAGEMENT DEVICE DESIGN DETAILS

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# 1 INTRODUCTION

Parts A and B of this Toolbox discuss the background behind why stormwater management is an issue and prevention of stormwater effects. Part C provides a detailed discussion on how to design those practices that reduce adverse effects.

There are three key elements in designing stormwater management practices:

- Designing to minimise future maintenance,
- Hydrologic design methodology, and
- Detailed practice design.

Maintenance is such a key issue and is most appropriately addressed at the design phase to minimise long-term maintenance problems. As such, it is discussed in the detailed device design section of the toolbox.

# 2 DESIGN TO MINIMISE FUTURE MAINTENANCE

A key element that <u>must</u> be considered during the design phase is operation and maintenance of stormwater management practices. There are several key elements that must be considered during the design phase. Asking and answering some questions or giving serious consideration to operation of the stormwater practice and system can answer them.

- Spend a year at the practice
- Asking maintenance questions such as who, what, when, where and how
- Considering the use of uniform materials or components

## 2.1 Spend a year at the practice

There are three possible entities that may maintain stormwater management practices:

- The District Council,
- A body corporate, or
- Individual property owners.

While the District Council has expertise in asset management, the other two entities probably won't. Practice selection needs to consider the lack of expertise that will exist for maintenance and ensure that maintenance is kept as simple as possible to ensure long-term practice function.

As such, the stormwater designer must imagine conditions at the completed practice throughout an entire year. This should not only include rainy and sunny weather but also consider time of year when evapotranspiration rates are different. Other site conditions may include hot, dry weather or drought when vegetation is stressed or dies. Finally, for safety purposes, the designer should also imagine what the system would be like at night.

As these conditions are visualised, the designer should also imagine how they might affect not only the operation of the practice itself, but also the people who will maintain it or otherwise interact with it. Is there a safety issue related to maintenance of a practice such as an in-ground water tank?

This approach is intended to assist the designer to consider and design for possible conditions at the practice, not just for specific storm events.

## 2.2 Asking maintenance questions

Another key element of design should involve asking specific questions that focus on operation and maintenance characteristics or functions of the practice. The questions should include at least: the following:

- Who will perform maintenance,
- What needs to be maintained,
- When will maintenance need to be performed,
- Where is maintenance needed, and
- How will maintenance be done.

### 2.2.1 Who will perform the maintenance

Does the design of the practice require operation and maintenance specialists or can an average individual be able to do the maintenance.

### 2.2.2 What needs to be maintained

A list of practice components that are part of the design may prompt a revised design with either a shorter list or one that modifies a practice component to facilitate maintenance. An example of this could be a rain garden that has underdrains, dense vegetation and an overflow spillway. Access has to be provided to ensure that a maintenance person can get to the site to conduct contaminant removal or replacement of key elements as needed.

### 2.2.3 When will maintenance need to be performed

Does maintenance have to be done once a day, once a week, monthly or annually? The recurring costs of maintenance can be substantial. In addition, can maintenance only be done during dry weather? If so, what happens during the lengthy time periods of wet, rainy weather? In terms of effort and possible consequences, it is easier for the designer to provide answers to these questions now rather than having a relatively uninformed person developing an approach later.

### 2.2.4 Where will maintenance have to be performed

Recognising that these practices are being done primarily in residential areas, there will always be potential interaction with the public and safety concerns that have to be addressed. Will the maintenance person be able to gain easy access to the practice? Once there, will they have a stable, safe place to stand and work? Can the design provide a means for the maintenance entity to reduce the time on site to conduct maintenance inspections and perform maintenance?

### 2.2.5 How will maintenance be performed

The simple instruction to remove sediment or harvest vegetation can become complicated if there hasn't been any provision made to allow equipment access to the practice or even to the site. Are slopes too steep for mowing equipment to reach or are outfall dispersal trenches located in areas that will be overlooked? Stormwater practices cannot become a liability to the local community.

## 2.3 Considering the use of durable materials

Specify materials that will last for as long as the life expectancy of the intended land use. Reducing construction costs may have a significant adverse impact on long-term maintenance costs.

It is absolutely essential that the designer consider these issues during the design phase so they can be addressed now rather than being left for later resolution. The design phase may be the shortest amount of time given to a project when considering construction time and whole-of-life aspects. It is vital that the design attempts to minimise future maintenance obligations and cost while providing for proper protection of downstream areas.

# 3 HYDROLOGY

## 3.1 Hydrological design method

The hydrologic analysis approach for this toolbox is the Rational Formula. This is not consistent with ARC consenting requirements using TP 108 (ARC, 1999) but this toolbox is for permitted activities under the ALW Plan in addition to ARC consents under the Air, Land and Water Plan. The end result is similar in terms of level of control but the Rational Formula is simpler to use for a broader audience who may be impacted by local requirements.

The Rational Method was developed approximately 150 years ago and is still widely used internationally. There are some limitations to use of the method but it does provide reasonable peak discharge results on small catchments with relatively uniform land use. There is a good discussion on the use of the Rational Method in Appendix C of the NZWERF Guideline (2004). In addition to that the City of Christchurch has a detailed discussion of the Rational Method in their Waterways, Wetlands and Drainage Guide (2003).

It is only suitable for small catchments as the method does not account for catchment storage during flood events, but it is appropriate for small sites (that is it fits within limitations on the use of the toolbox set out in Part A. NZWWA (2004) recommends that it not be used for catchment areas in excess of 50 hectares.

The Rational Formula is the following:

$Q_{wq} = 0.00278CIA$

Q = peak discharge ($m^3/s$)
C = Runoff coefficient (-)
I = Rainfall intensity (mm/hr.)
A = catchment area (ha)

The runoff coefficient = Predevelopment pervious $C_{pre}$ factor + 0.65(%Impervious cover/100)

Where predevelopment C factor related to soil conditions is provided in Table C1.

**Table C1**
**Rational Formula Runoff Coefficients (C Factor)**

| Land use | Soil Group | | |
|---|---|---|---|
| | A | B | C |
| Row crops | .55 | .65 | .70 |
| Pasture | .10 | .20 | .30 |
| Woods - no grazing | .06 | .13 | .16 |
| Impervious Surface | .95 | .95 | .95 |

The soil groups listed are those taken out of TP 108 where:

- Group A soils are volcanic granular loam,
- Group B soils are alluvial soils, and

- Group C soils are mudstone/sandstone.

For much of the Auckland Region, soils will be in the Group C category.

While the design examples will use the Rational Formula for design, the ARC's TP 108 or another hydrologic method acceptable to the approving entity are also acceptable to use for design.

When ARC consents are required, hydrologic design shall be in accordance with ARC requirements, which at this time is for the hydrologic analysis to be done by TP 108.

## 3.2 Calculating water quality volumes

The Rational Formula does not calculate volumes of runoff but rather calculates peak discharges for various storm intensities. Calculate the water quality volume to be treated by using the 1/3 of the 2-year storm as shown in the ARC's TP 108. The City of Christchurch has a simple method of determining the first flush volume in their Waterways, Wetlands and Drainage Guide (2003) where the water quality volume (their first flush volume) is based on the following:

The catchment effective first flush runoff area = $A_{eff}$ = impervious%/100 x total Area (ha)

The first flush volume $V_{ff} = 10 \times A_{eff} \times d_{ff}$ ($m^3$)

Where $d_{ff}$ = first flush water quality depth (water quality storm = 1/3 of 2-year 24-hour storm in TP 108 rainfall table)

Use this method to calculate the water quality volume storage.

## 3.3 Extended detention

As discussed in Parts A and B, critical issues in rural development design, from a stormwater management perspective, are related to increases in stormwater runoff adversely impacting on receiving system physical structure. As such, the extended detention of flows to minimise downstream channel erosion is an important issue. Minimising increases in the total volume of stormwater being discharged will mitigate increases and may through careful design and use of a treatment train approach significantly reduce or eliminate stormwater runoff volume increases to reduce or eliminate the extended detention requirement. An example of this is shown in Table A4 regarding stormwater practices commonly used in a rural environment.

A key difference in calculating stormwater runoff from rural properties versus urban ones is that significant site regrading is minimised in rural areas and there is no change to pervious area predevelopment C factors for most of the site. If impervious surface volume is reduced or mitigated for from a flow perspective, receiving system stability can be maintained.

Another point to mention is time of year. Winter months in the Auckland Region have negligible levels of evaporation so runoff naturally increases over the winter months for a given amount of rainfall. Thus having limited storage volumes for rain tanks does not necessarily compromise their overall stormwater function.

Implementation of the following practices can significantly reduce the need for a formal extended detention on a site design basis.

### 3.3.1 Rain gardens and infiltration trenches

Rain gardens and infiltration trenches (Sections 4.3 and 4.4) having a larger volume of storage beyond the water quality storm can function for extended detention design. To provide for the 34.5 mm rainfall event, the water quality volume should be multiplied by 1.3. This volume would then be used in the design approach equation detailed in the rain garden design and infiltration trench sections.

### 3.3.2 Bush restoration

As can be seen from Table C1, woods have a significantly lower runoff coefficient than other land uses. Using the approach detailed in the bush design section (Section 4.7), revegetation of bush can provide a reduction in total runoff volume and minimise extended detention requirements.

In a similar manner, cutting down existing bush will, of itself, increase stormwater runoff and must be included in any analysis for calculating extended detention volumes. The discharge calculations must be modified in the post-developed condition to account for the change in pervious landuse to the appropriate value and not just account for impervious surfaces.

### 3.3.3 Rain tanks

Rain tanks (Section 4.6) can provide for extended detention in both domestic water consumption and extended detention release.

### 3.3.4 Green roofs

Green roofs (Section 4.8) can also provide extended detention benefits but their depth of media must be at least 70 mm to provide for significant storage. This toolbox recommends 150 mm of media to ensure plant survival.

### 3.3.5 Swales and filter strips

Swales and filter strips (Sections 4.1 and 4.2) do not provide extended detention benefits using a conventional design. The swale design section does discuss the use of underdrains in shallow slope areas and having a modified soil profile with an underdrain can provide extended detention benefits. Filter strips do not generally provide extended detention benefits, as modification of soils on shallow slope areas is not a recommended approach.

### 3.3.6 Wetland swales

In a similar manner as swales and filter strips, wetland swales (Section 4.5) do not provide extended detention. Having a series of check dams can provide for extended detention if designed specifically to provide that function.

### 3.3.7 Ponds and wetlands

Ponds and wetlands can easily be modified to provide for extended detention. There may be situations where an existing pond can be modified to provide for additional storage either by lowering the normal pool level or modifying the embankment or outlet structure. Design of ponds and wetlands are discussed in the ARC's TP 10 and are not addressed further in this countryside guideline.

## 3.4 Peak storm flow control

Control of peak discharges for the 2- and 10-year storms is a requirement of the Permitted Activity Section of Chapter 5 of the ARC's Air, Land and Water Plan. There are two levels of analysis that are necessary to determine peak control levels and requirements.

1. What is the change in peak discharge for the 2- and 10-year storms, and
2. How much storage is required to mitigate downstream effects.

### 3.4.1 Calculating peak discharge

In calculating the peak discharge, the storm duration is normally equal to the time of concentration ($t_c$) of the catchment. For the purposes of this toolbox $t_c$ is calculated as the following:

$$T_c = 0.305(L/180) + 10$$

Where:

$T_c$ = Time of concentration (minutes)

$L$ = Maximum distance from study location to upper catchment boundary limit (m)

Unless a greater duration is indicated by the site analysis, the storm duration to use for peak control purposes is the 1-hour storm. Rainfall intensities can be obtained using territorial authority information or NIWA's High Intensity Rainfall Design System (HIRDS) that is available for purchase from NIWA.

### 3.4.2 Determining volume needs for storage

The estimated volume of storage for a 2- and 10-year storm can be determined by using the following equation.

$$V_{estimated} = 1.5(Q_{post})D$$

Where $V_{estimated}$ = required storage volume ($m^3$)

$Q_{post}$ = Post-development peak discharge rate ($m^3/s$)

D = Duration of storm (sec)

This equation gives the total runoff volume for the storm analyses. For the purposes of this toolbox, the storm duration is 1 hour (3600 seconds). The 1.5 constant was used to provide consistency in volume requirements with TP 108. The general equation is based on a trapezoidal hydrograph with a storm duration greater than the time of concentration. If the storm duration equaled the time of concentration, a triangular hydrograph would have been used but the volume requirements fit a one-hour storm better than a 10-minute $T_c$.

The calculation should be done for 2- and 10-year storms.

#### 3.4.2.1 Control of 2- and 10-year storms within practices

Rural development normally does not involve mass grading of an entire development site. This ensures that the runoff from pervious areas is essentially unchanged from the predevelopment condition. As such, stormwater management practices should only be designed and constructed for those specific areas that need management.

Certain practices can provide control of the differences in the 2- and 10-year storms by providing storage within the practice. Practices that can provide volume control of the difference in peak discharges for the 2- and 10-year storms are the following:

- Swales with underdrains and check dams,
- Rain gardens,
- Infiltration trenches,
- Water tanks,
- Revegetation, and
- Green roofs

The discussion on extended detention control by providing 1.3 times the water quality volume for design also provides for control of the volume increases for the 2- and 10-year storms if designed correctly. The combination of live storage on the practice and void ratios in the practices themselves provides for separation of the 2- and 10-year volume increases from the storm hydrograph.

If this approach is taken, there is no need to design and construct separate peak control devices for developments. A key point is that the practice must manage only those areas that require management such as driveways and impervious surfaces. If runoff from additional land area not needing management drains to the practice, the volumes needed for storage increase greatly and control of the 2- and 10-year storms becomes more difficult. If extraneous drainage cannot be kept out of the stormwater practice, peak control of the 2- and 10-year storms may be necessary and the equations provided above for calculating volumes must be used to determine detention storage.

## 3.5 Flooding analysis

When there exists documented downstream flooding of habitable structures in a catchment, there can be no increased risk as a result of rural land use development. The issue of where in a catchment storage of runoff to prevent increased potential flooding is beneficial and where storage may increase flood potential due to delay in release of water coinciding with peak flows when they arrive from upstream is discussed in Part A. In a similar fashion, criteria are also provided for storage to ensure that potential flood increases are avoided.

The purpose of this discussion is to provide a storm duration for which analysis must be done on the 100-year storm peak discharge. The storm to consider again relates to the $T_c$ of the area being developed. This may be different from design of individual practices where catchment areas are small, as the flooding consideration must consider the development as a whole.

Where there is documented downstream flooding of habitable structures the best approach, in the absence of a catchment management plan, is to ensure that the post-development peak discharge for the 100-year storm not exceed 80% of the pre-development peak discharge for the 100-year storm. This will minimise the potential increase in downstream flooding.

## 3.6 Influence of climate change

The Resource Management Act Amendment Act (March 2004) requires councils to have particular regard to the effects of climate change. Incorporating climate change predictions into stormwater design is important if infrastructure is to maintain the same level of service throughout its lifetime.

In terms of rainfall (Ministry for the Environment, 2008), annual rainfall will slightly decrease in the Region from 3% in the Warkworth area to only 1% in Mangere by 2040. This decrease is expected to increase by 2090 to 5% in the Warkworth area to only 3% in Mangere. In terms of extreme rainfall, heavier and/or more frequent extreme rainfalls are expected. For example, for Auckland, the worst case (most severe) end of the range for 2100 indicates that a rainfall amount currently with a return period of 50 years would have a return period of less than 10 years.

The 2- and 10-year ARI daily storm events are used to confirm a device's ability to convey peak flows under moderately severe conditions. For device components with a design life greater than 25 years the storm event precipitation values (2- and 10-year) should be adjusted to account for climate change. The values obtained from TP 108 should be increased by the percentages listed in Table C-2 unless locally, more detailed data provides more accurate recommendations.

**Table C-2**
**Factors (percentage adjustments) for Use in Deriving Extreme Rainfall Information for Stormwater Design (MfE, 2008)**

| ARI (years) → / Duration ↓ | 2 | 5 | 10 | 20 | 30 | 50 | 100 |
|---|---|---|---|---|---|---|---|
| < 10 minutes | 8.0 | 8.0 | 8.0 | 8.0 | 8.0 | 8.0 | 8.0 |
| 10 minutes | 8.0 | 8.0 | 8.0 | 8.0 | 8.0 | 8.0 | 8.0 |
| 30 minutes | 7.2 | 7.4 | 7.6 | 7.8 | 8.0 | 8.0 | 8.0 |
| 1 hour | 6.7 | 7.1 | 7.4 | 7.7 | 8.0 | 8.0 | 8.0 |
| 2 hours | 6.2 | 6.7 | 7.2 | 7.6 | 8.0 | 8.0 | 8.0 |
| 3 hours | 5.9 | 6.5 | 7.0 | 7.5 | 8.0 | 8.0 | 8.0 |
| 6 hours | 5.3 | 6.1 | 6.8 | 7.4 | 8.0 | 8.0 | 8.0 |
| 12 hours | 4.8 | 5.8 | 6.3 | 7.3 | 8.0 | 8.0 | 8.0 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 24 hours | 4.3 | 5.4 | 6.3 | 7.2 | 8.0 | 8.0 | 8.0 |
| 48 hours | 3.8 | 5.0 | 6.1 | 7.1 | 7.8 | 8.0 | 8.0 |
| 72 hours | 3.5 | 4.8 | 5.9 | 7.0 | 7.7 | 8.0 | 8.0 |

Note: This table recommends percentage adjustments to apply to extreme rainfall per $1^{o}$ C of warming, for a range of average recurrence intervals (ARIs). The percentage changes are mid-range estimates per $1^{o}$ C and should be used only in a screening assessment. The entries in this table for a duration of 24 hours are based on results from a regional climate model driven for the A2 SRES (Special Report on Emissions Scenarios - see MfE, 2008 Appendix 1) emissions scenario. The entries for 10-minute duration are based on the theoretical increase in the amount of water held in the atmosphere for a $1^{o}$C increase in temperature (8%). Entries for other durations are based on logarithmic (in time) interpolation between the 10-minute and 24-hour rates.

# 4 FLOW AND TREATMENT CONTROL

Specific design guidance is provided in this Section for the following practices:

- Swales,
- Filter strips,
- Rain gardens,
- Infiltration trenches,
- Wetland swales,
- Water tanks,
- Bush revegetation,
- Green roofs,
- Access roads and driveways, and
- Dispersal devices

These practices are seen as particularly applicable in countryside living areas.

The issue of stormwater management ponds or wetlands is best discussed in Technical Publication 10 (ARC, 2003) as ponds for stormwater management are not recommended for use in rural areas. They are more appropriate in those areas where a higher intensity of development warrants their use.

The case studies shown for each practice relate to a specific part of rural land use. As an example, the swale case study details treatment for a small access road that will service a rural residential community. Due to the large area of a rural development site that is expected to remain undisturbed and undeveloped, it is considered appropriate to consider the individual elements separately to provide overall site management.

## 4.1 Swale Design

### 4.1.1 Description of practice

A swale is a vegetated earth channel that provides stormwater conveyance and whose vegetation and organic matter provide treatment of the runoff.

**Example of a swale in a rural residential subdivision**

### 4.1.2 Design considerations

The key elements of a swale are low velocities of flow and residence time. Swales impact on stormwater runoff in two ways:

Conveyance of stormwater flows at a low velocity compared with flow in stormwater pipes. As water passes through the vegetation it encounters frictional resistance due to the vegetation, and water quality treatment is provided by passage of the water through the vegetation. Physical, chemical and biological processes occur that reduce contaminant discharge.

Table C3 provides guidance from a design perspective.

**Table C3**
**Swale design elements**

| Design parameter | Criteria |
|---|---|
| Longitudinal slope | < 5% |
| Maximum velocity | 0.8 m/s for water quality storm |
| Maximum water depth above vegetation | The water quality design water depth should not exceed design height for grass. This is a key criterion for ensuring Manning roughness coefficient is provided. |
| Design vegetation height | 100 - 150 mm |
| Manning coefficient | 0.25 for WQ storm, 0.03 for submerged flow (10-yr. Storm) |
| Maximum bottom width | 2 m |
| Minimum hydraulic residence time | 9 minutes |
| Minimum length | 30 m |
| Maximum catchment area served | 2 hectares |
| Maximum lateral slope | 0% |
| Maximum side slope | 4H:1V (shallow as possible for mowing purposes) |
| Where longitudinal slope < 2% | Perforated underdrains shall be provided |

| | |
|---|---|
| Where longitudinal slope > 5% | Check dams shall be provided to ensure effective slope < 5% |
| Where concentrated flows enter the swale (from pipes) | Level spreaders shall be placed at the head of the swale to disperse flows |
| 10-year storm velocities | < 1.5 m/s unless erosion protection is provided |

As can be seen, there are several key differences in Table C3 to the ARC's TP 10 where this standard is more restrictive.

1. The water quality storm should not exceed the design height of grass. TP 10 allows for a maximum flow depth above the grass of 100 mm. This change is felt necessary to improve water quality performance.
2. Using a standard Manning's roughness coefficient of 0.25 rather than the series of equations given in TP 10. This is done for simplicity as either approach is felt to provide credible results.
3. Maximum swale side slopes can be no steeper than 4 horizontal: 1 vertical. This change from a maximum of 3 horizontal: 1 vertical is done based on input from asset managers who feel that mowing is much easier on the flatter slope.
4. Catchment drainage areas should be less than 2 hectares to minimise scour potential.

There are several points that need some discussion and they include:

- Residence time
- Manning's coefficient of roughness
- Lateral inflow

#### 4.1.2.1 Residence time

There have not been many studies that relate water quality performance in swale design. The most recognised work has been done in the U.S. (Metropolitan Seattle, 1992). That study recommended a residence of 9 minutes for flow to pass through the swale and provide approximately an $80^{+}$% removal of total suspended solids. The swale monitored there had a grass height of approximately 150 mm with a grass blade density of approximately 18,000 blades/$m^2$. The grass species was predominantly tall fescue. Most governmental agencies in the U.S. have adopted that criterion. More recently, the recommendation has been recommended upward to 22 minutes due to the uncertainty of performance (Washington State Department of Ecology, 2001). That change in residence time is a significant change from the 9-minute criterion and it is not recommended that the time be increased until further investigation of swale performance is done in New Zealand.

#### 4.1.2.2 Manning's coefficient of roughness

Determining roughness coefficients is more art than science. Many design handbooks provide one value for Manning's coefficient of roughness of 0.2 (Metropolitan Seattle, 1992) or 0.25 (California Stormwater Quality Association, 2003). The ARC funded a swale study (Larcombe, 2003) where dye tests were done on a swale to determine "n" by measuring flow times through the swale. In all of the test trials the values of Manning's coefficient of roughness varied from 0.18 - 0.30. In reality the range is very consistent with the recommendations provided in the literature. The ARC recommended a series of equations for determination of "n" and using those equations provides values lower than Larcombe found in his study.

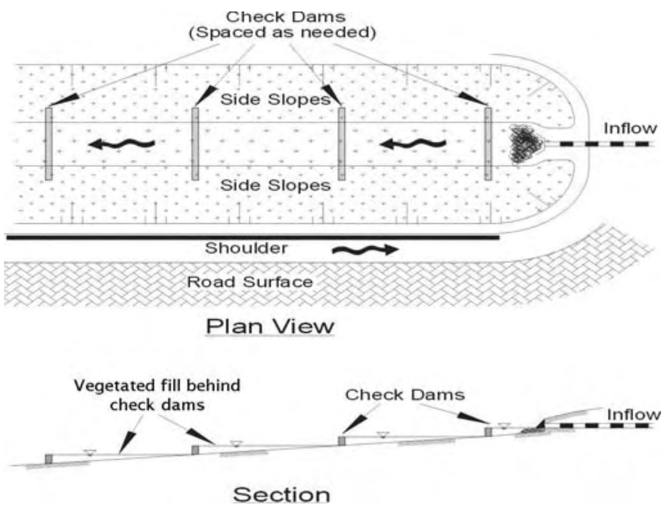
It is recommended that a standardised value for Manning's coefficient of roughness be set at 0.25. It is a mid-point in the Larcombe study and agrees favorably with The California recommendation. It is not felt that using the equations would provide necessarily a better result in design.

For the 10-year storm analysis, it is assumed that the vegetation is submerged so the coefficient of roughness is reduced accordingly. The value selected is 0.03 (Chow, 1959).

#### 4.1.2.3 Lateral inflow

A common concern with swales is lateral inflow from the highway to a point where the flow does not achieve the 9-minute residence time. To the degree that the 9 minutes can be attained it should be. An example of this is Figure C1 that, in addition to check dams, shows a lateral flow diversion that directs the lateral flow to the head of the swale.

**Figure C1 Swale with Check Dams and Diversion of Lateral Inflow**




Where lateral inflow cannot meet the nine-minute residence time for part of the alignment, the normal approach is to accept that the average flow through the swale does take nine minutes. There will be areas in the upper part of the swale that may exceed the required residence time so the average is appropriate in light of the benefits that swales provide.

### 4.1.3 Targeted contaminants

From a water quality standpoint, swales provide good treatment for sediments and metals. Performance for hydrocarbons is moderate as is their effectiveness for nutrients. Swales are more effective for phosphorus removal than for nitrogen due to higher levels of phosphorus being attached to sediments than nitrogen, which tends to be in a soluble form.

### 4.1.4 Advantages

Swales are an excellent practice for small catchment areas and their maintenance obligations are minimal (mowing). Their use in rural environments would eliminate the need for kerbing and have lower stormwater flow velocities than would enclosed pipe systems.

Swales are also a good practice to use as part of a treatment train to mitigate for the adverse effects that result from impervious surfaces.

### 4.1.5 Limitations

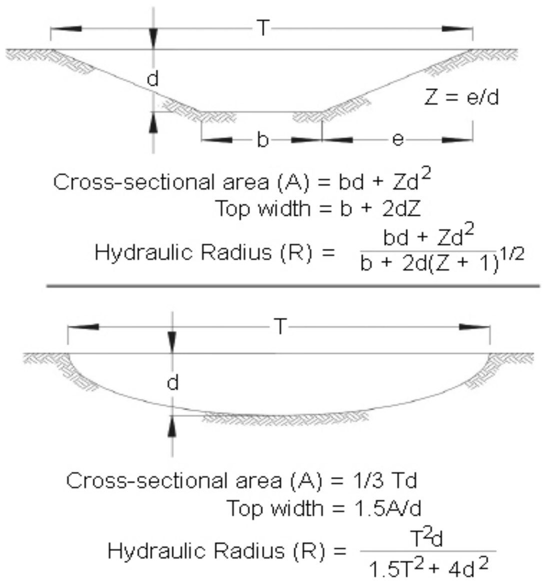
Swales are limited to shallow slope areas to ensure low velocities of flow and meet residence time requirements. They are also only suitable for small catchment areas of less than 2 hectares.

### 4.1.6 Design sizing

The design approach takes the designer through a series of steps that consider swale performance for water quality treatment and consideration of larger flows to ensure that scour or resuspension of deposited sediments does not occur.

1. Estimate runoff flow rate from the water quality storm using 1/3 of the 2-year storm as the water quality storm and calculate the flows. One difference between swale and filter strip design and other stormwater management practices is that they are designed by flow rate where other practices are designed by calculation of the water quality volume.
2. In using the Rational Formula for design, use the entire water quality storm as I. This would be the maximum possible value for discharge calculations and thus be conservative.
3. Establish the longitudinal slope of the swale.
4. Select a vegetation cover. It should be grass and would generally be either perennial rye or fescue.
5. The value for Manning's coefficient of roughness is 0.25
6. Select a swale shape. Two shapes are proposed as they ensure distributed flow throughout the bottom of the swale. Triangular swales are not recommended as they concentrate flow at the bottom of the swale. Channel geometry and equations for calculating cross-sectional areas and hydraulic radius are provided under the individual configurations in Figure C2.
7. An assumption is made on the depth of flow in the swale for the water quality storm. This assumed depth is used for calculating the bottom width of the swale and cross-sectional area.
8. Use Manning's equation for calculating dimensions of the swale by using first approximations for the hydraulic radius and dimensions for selected shape.

**Figure C2 Channel Geometry**



$$Q = AR^{0.67}s^{0.5}/n$$

By making some assumptions about depth and width ratios such as the hydraulic radius for a trapezoid approximating the depth (d), the bottom width of a trapezoid (b) equals the following:

$$b = (Qn/d^{1.67}s^{0.5}) - Zd$$

The slope, depth, discharge and side slope are all known and b can be determined.

Where:

| | | |
|---|---|---|
| Q | = | design discharge flow rate ($m^3$/s) |
| n | = | Manning's n (-) |
| s | = | longitudinal slope (m/m) |
| A | = | cross-sectional area ($m^2$) |
| R | = | hydraulic radius (m) |
| T | = | top width of trapezoid/parabolic shape (m) |
| d | = | depth of flow (m) |
| b | = | bottom width of trapezoid (m) |

For a parabola, the depth and discharge are known so the top width can be solved for.

8. Knowing b (trapezoid) or T (parabola), the cross-sectional area can be determined by the equations in Figure C2.
9. Calculate the swale velocity from the following equation:

$$V = Q/A$$

   If V > 0.8 m/s repeat steps 1 - 9 until the velocity is less than 0.8 m/s.
10. Calculate the swale length (L in metres)

   L = Vt(60 s/minute)

   Where t = residence time in minutes.

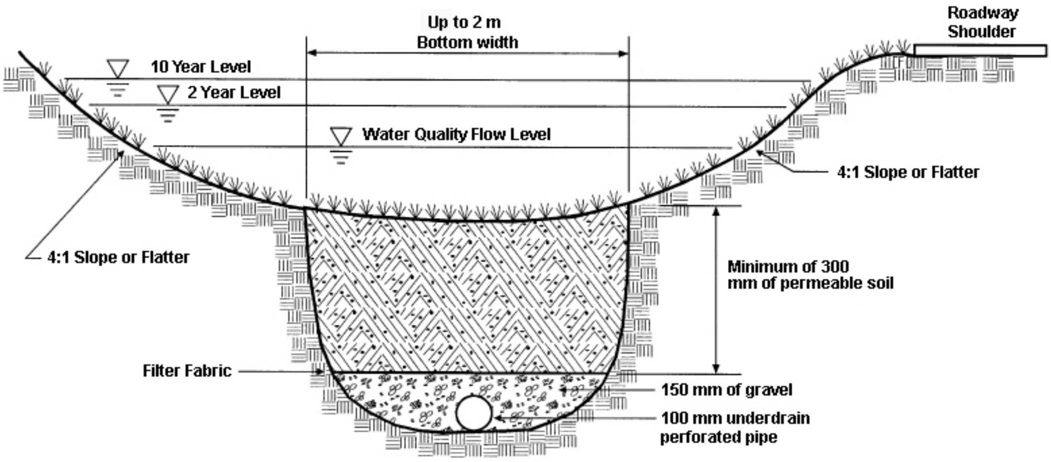
#### 4.1.6.1 Flows in excess of the water quality storm

It is expected that runoff from events larger than the water quality design storm will go through the swale. In that situation, a stability check should be performed to ensure that the 10-year, 24-hour storm does not cause erosion. For the 10-year storm, flow velocities should not exceed 1.5 m/s, although higher velocities may be designed for with appropriate erosion protection. When considering larger storms consideration must be given to increased rainfall values as a result of climate change. Table C5 provides information on the percentage increase for design purposes.

#### 4.1.6.2 Shallow slope situations

Where slopes are less than 2%, an underdrain must be used to prevent soils from becoming saturated during wet times of the year. Figure C3 provides a typical cross-section of what the underdrain system should be designed to ensure that water passes through the invert of the swale, through a loam soil, geotextile fabric and gravel prior to discharge through a 100 mm perforated pipe. The minimum recommended depth of permeable media is 300 mm but greater depth is recommended if extended detention volume is provided.

**Figure C3 Swale schematic showing soils and underdrain**




## 4.1.7 Case study

### 4.1.7.1 Project description

An access road for a rural subdivision is proposed. The lane is 6.4 metres wide and 300 metres long. The project is located in Wainui. The swale will accept runoff from the roadway and approximately 0.2 hectare of overland flow from adjacent pasture (lawn in the developed state) land.

### 4.1.7.2 Hydrology

Using the Rational Formula

$Q_{wq} = 0.00278CIA$

Predevelopment land use is pasture on a C soil so predevelopment C factor is 0.3.

C = 0.3 + .65(%Impervious cover/100) = 0.3 + .65(49%/100) = 0.62
I = Rainfall intensity (mm/hr.) - for Wainui the water quality storm is 30 mm.
A = catchment area in hectares

$Q_{wq} = 0.00278(0.62)(30)(.39) = 0.020\ m^3/s$

For the one hour $Q_{10}$ in Wainui, rainfall is 34.4mm. Effect of global warming on the 10-year storm is 7.4% increase in rainfall. So 34.4 mm for a 10-year storm increased by 7.4% results in 37 mm of rainfall.

$Q_{10} = 0.00278(.62)(37)(0.39) = 0.024\ m^3/s$

#### 4.1.7.3 Swale Design

Slope of swale alignment = 0.015

Several assumptions have to be made regarding the swale, first of which is that the swale will have a trapezoidal design. Side slopes (Z) will then be recommended and an assumption of design storm depth should be made. That value may change depending on the velocity of flow being less than 0.8 m/s.

For this case study, Z = 4 and the depth of flow = 100 mm, which is also the design height of the grass.

Based on the value for Q and s, and the assumptions for n and d, solve for the swale bottom width (b).

$b = (Qn/d^{1.67}s^{0.5}) - Zd$

$b = ((.020)(.25)/(.1^{1.67})(.015^{0.5})) - (4)(.1) = 1.52\ m$

Calculate the top width

$T = b + 2dZ = 1.52 + 2(.1)(4) = 2.32\ m$

Calculate the cross-sectional area

$A = bd + Zd^2 = (1.52)(.1) + 4(.1^2) = 0.192\ m^2$

Calculate the flow velocity

V = Q/A = 0.020/0.192 = 0.1 m/s which is well under than the 0.8 m/s maximum - good.

Calculate the swale length

L = Vt = 0.1(540 sec.) = 54 metres

As the swale will probably have larger flows pass through it, the swale design can be adjusted to account for the larger flows. In this situation the Manning coefficient of roughness will have to be decreased, as flow will be above the grass height so assume n = .03 as the vegetation is completely submerged. Solve for d and ensure that velocities are not erosive. $Q_{10} = 0.024\ m^3/s$.

The following Table C4 relating flow depth to Manning's n to discharge provides information on swale flow under larger flow conditions.

**Table C4 - Flow Depth vs. Manning's n versus Discharge**

| Flow depth (m) | Manning's n | Discharge ($m^3/s$) |
|---|---|---|
| 0.1 | 0.25 | 0.020 |
| 0.1 - 0.2 | 0.03 | 0.19 |
| Total Discharge | | 0.206 |

Even adding only 100 mm to the swale depth provides for conveyance of the 10-year event. In terms of ensuring that the velocity is not greater than 1.5 m/s

Q = AV or Q/A = V = 0.024$m^3$/s/0.442 = 0.05 m/s so the velocities during the 10-year storm are non-erosive.

As the swale is on a slope of 1.5%, an underdrain must be provided to ensure that the swale does not become saturated and adversely affect vegetative growth. Having an underdrain provides the opportunity to provide storage in the swale and provide for control of the 34.5 mm rainfall (extended detention) and for control of the 2- and 10-year peak discharges.

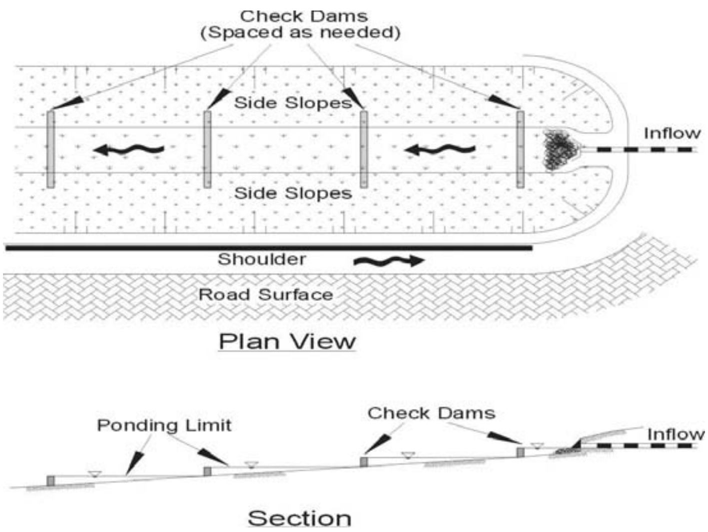
There are two ways that storage can be provided.

- In the media profile that has been placed over the underdrain, and
- Through the use of check dams to provide live storage.

A schematic of the check dam approach is shown in Figure C4.

The approach would be to use the backfilled media according to the rain garden specifications for it and assume a 35% void ratio. Taking the difference in the volumes associated with the pre- and post-development runoff for the 10-year rainfall for the one-hour storm and storing that volumetric difference in the media and as live storage behind check dams would meet the peak control requirements.

**Figure C4 Swale with Check Dams and Diversion of Lateral Inflow**



## 4.2 Filter strip design

### 4.2.1 Description of practice

Filter strips accept stormwater flow as distributed or sheet flow. Filter strip performance, like swales, also relies on residence time that stormwater flows take to travel through the filter strip and the depth of water relative to the height of vegetation. Good contact with vegetation and soil is required to promote the operation of the various mechanisms that capture and transform contaminants, so spreading flow in minimal depth over a wide area is essential.

A key element of filter strips is that they rely on vegetation to slow runoff velocities. If stormwater runoff is allowed to concentrate, it effectively short-circuits the filter strip and reduces water quality benefits. As used in this Standard filter strips are simple designs that must withstand the full range of storm events without eroding.

### 4.2.2 Design considerations

The following Table C5 should be adhered to in designing a filter strip.

**Table C5**
**Filter Strip design elements**

| Design parameter | Criteria |
|---|---|
| Longitudinal slope | 1% - 5% |
| Maximum velocity | 0.4 m/s for water quality storm |
| Maximum water depth above vegetation | The water quality design water depth should not exceed ½ of the design height for grass. This is a key criterion for ensuring Manning roughness coefficient is provided. |
| Design vegetation height | 100 - 150 mm |
| Manning coefficient | 0.35 for WQ storm, 0.03 for submerged flow (10-yr. Storm) |
| Minimum hydraulic residence time | 9 minutes |
| Minimum length | Sufficient to attain residence time |
| Maximum catchment area served | 2 hectares |
| Maximum lateral slope | 1% |

| | |
|---|---|
| Where longitudinal slope < 2% | Filter strips are not recommended for slopes less than 2% unless they are designed for infiltration of runoff |
| Where longitudinal slope > 5% | Level spreaders shall be provided to ensure effective slope < 5% |
| Maximum overland flow distance uphill of the filter strip | 23 m |
| Where concentrated flows enter the swale (from pipes) | Flows entering a filter strip cannot be concentrated. If this is the situation, level spreaders must be used to disperse flows |
| 10-year storm velocities | < 1.5 m/s unless erosion protection is provided |

The Washington State Department of Transportation (WSDOT, 1995) recommends that filter strips treat highway runoff with a maximum of two lanes, and for an average daily traffic of less than 30,000 vehicles per day. The use of filter strips on rural development is very appropriate.

There are two terms used in Table C5; longitudinal slope and lateral slope. Longitudinal slope relates to the slope down the filter strip away from the area being treated. Lateral slope relates to the slope that may parallel the area being treated. The longitudinal slope should not be confused with the longitudinal slope of a roadway.

### 4.2.3 Targeted contaminants

From a water quality standpoint, swales provide good treatment for sediments and metals. Performance for hydrocarbons is moderate as is their effectiveness for nutrients. Swales are more effective for phosphorus removal than for nitrogen due to higher levels of phosphorus being attached to sediments than nitrogen, which tends to be in a soluble form.

### 4.2.4 Advantages

Due to the low density of impervious surfaces in rural areas, filter strips are very suitable for providing water quality treatment. In conjunction with elimination of kerbing or providing kerb cuts, dispersed flow can be maintained and water quality treatment provided. Filter strips are an excellent practice for small catchment areas and their maintenance obligations are minimal (mowing).

### 4.2.5 Limitations

To be effective, filter strips require sheet flow across the entire strip. Once flow concentrates to form a channel, it effectively short-circuits the filter strip. Unfortunately, this usually occurs within a short distance for filter strips in urban areas. It is difficult to maintain sheet flow over a distance of 45 m for pervious areas and 23 m for impervious areas. This may be due in part to the inability to obtain evenly compacted and level soil surfaces using common construction methodology.

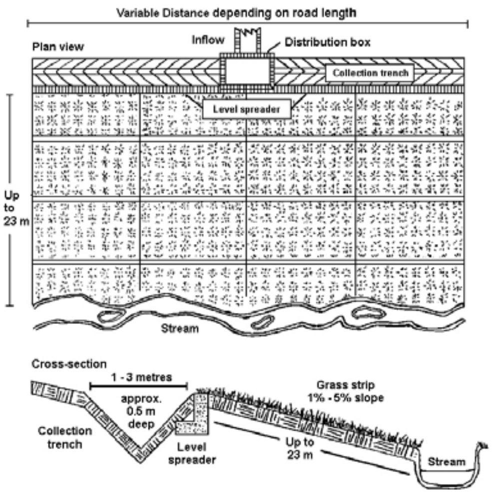
For some applications, a level spreader can be used to help ensure even distribution of stormwater onto the filter strip.

Due to the limited distance that flow can be maintained in sheet flow, they are only suitable for small catchment areas. In addition, they are primarily water quality practices and have little benefit for peak flow control or storage and release of the extended detention volume.

### 4.2.6 Design sizing

A schematic of a filter strip is shown in Figure C5. The schematic shows a collection trench and a level spreader if the flow is from a pipe. In this situation the dispersed flow is maintained across the width of the filter strip.

**Figure C5 Schematic of a Filter Strip**




Design approach

1. The first step is to calculate the discharge (Q) for the area draining to the filter strip. If the filter strip is to take runoff only from an impervious surface, use the Rational Formula where c = 0.95.
2. Once the peak discharge is determined, that discharge can be entered into Manning's equation to determine the width of the filter strip.

$Q = AR^{0.67}s^{0.5}/n$

Where
A = width of filter strip (w) x depth of flow (d - determined by design grass height) ($m^2$)
w = width of filter strip (m)
R = depth of flow (due to very wide flow)(m)
d = depth of flow (m) = R
s = slope
n = roughness coefficient (0.35)

W is known from individual site conditions

So $d = (Qn/ws^{.5})^{.6}$

3. Solve for d based on knowing other design parameters and d must be less than 50 mm in depth

   i. Q = AV where A = wd so velocity of flow can be determined

   ii. Once velocity is determined the length of filter strip can be determined by

L = Vt

Where:
L = length in metres
V = velocity in m/s
t = time in seconds (540 seconds for 9 minute residence time)

### 4.2.7 Case study

A driveway is being constructed with a filter strip providing stormwater treatment. The project is located in Swanson so the water quality storm depth is 29 mm. The slope of land adjacent to the driveway is 3% and the driveway is 100 metres long with a width of 3.4 metres wide.

#### 4.2.7.1 Hydrology

Using the Rational Formula

$Q_{wq}$ = 0.00278CIA

C = 0.3 + .65(%Impervious cover/100) = 0.3 + .65(100%/100) = 0.95
I = Rainfall intensity (mm/hr.) - for Swanson the water quality storm is 29 mm.
A = catchment area in hectares

$Q_{wq}$ = 0.00278(0.95)(29)(0.034) = 0.0026 $m^3/s$

For Swanson, the 10-year 1-hour storm is 34.8 mm; the effect of global warming for the 10-year/1 hour storm is predicted to be 7.4% increase in rainfall. So, design rainfall for the 10-year storm = 37.4 mm.

$Q_{10}$ = 0.00278(.95)(37.4)(0..034) = 0.034 $m^3/s$

#### 4.2.7.2 Filter strip design

1. $Q = AR^{0.67}s^{0.5}/n$

   Where
   Q = water quality discharge ($m^3/s$)
   A = area of filter strip = (w -width in m)(depth of flow - d - in metres)
   R = 0.029 m based on water quality storm and very wide flow path
   s = .03
   n = .35

2. The width is given based on site conditions so solve for y and ensure that it is less than 0.05 m.

   $$d = (Qn/ws^{.5})^{.6}$$

   You will know "w" based on local site conditions. For this example, assume w = 75 metres.

   $d = (.0026(.35)/75(.03)^{.5})^{.6}$

$d = 3.2$ mm which is well under the maximum of 50 mm.

3. Calculate the flow velocity

   $V = Q/wd = .0026/75(.0032) = .01$ m/s which is well under the maximum 0.4 m/s allowed.

4. Calculate the length of the filter strip.

   $L = Vt = .01(540) = 5.4$ metres in length.

As can be seen from this example, the filter strip width can be reduced substantially to adjust to site conditions. The two key elements are a maximum depth of flow during the water quality storm of 50 mm and a residence time of at least 9 minutes (540 seconds) to establish the length of the filter strip.

In terms of a 2- or 10-year storm, the main concern is that velocities of flow not exceed 1.5 m/s. An analysis of the 10-year storm (worst case scenario) is now provided.

$Q_{10} = 0.034\ m^3/s$

Again using Manning's equation:

$Q = AR^{0.67}s^{0.5}/n$ and solve for d through the equation:

$d = (Qn/ws^{.5})^{.6}$

As the depth of flow still does not exceed the grass height the same n factor will be used. If the width of the filter strip were smaller and the depth of flow would exceed the design grass height an appropriate roughness coefficient to be used would be n = 0.15

$d = (.034(.15)/75(.03)^{.5})^{.6}$

$d = 0.009$ m

Using the value to ensure that the velocity of flow during a 10-year storm will not exceed 1.5 m/s

$V = Q/wd = 0.034/75(.009)$

$V = 0.05$ m/s which is well under an erosive velocity.

## 4.3 Rain gardens

### 4.3.1 Description of practice

Rain garden is a common term that is used internationally to describe the storage, passage and eventual discharge of stormwater to a receiving system. Two other terms are commonly used for rain gardens and they are:

- Bioretention
- Biodetention

Bioretention is a description of a process whereby stormwater runoff is treated by passing stormwater through a soil media and then either evapotranspiring the water or infiltrating that water into the ground.

Biodetention is the passage of water through a filter media and then discharging that water downstream to surface waters.

Rain gardens operate by filtering stormwater runoff through a soil media prior to discharge into either the ground or a drainage system. The major pollutant removal pathways within rain gardens are (Somes and Crosby, 2008):

- Event processes
  - ➢ Sedimentation in the extended detention storage, primary sediments and metals
  - ➢ Filtration by the filter media, fine sediments and colloidal particles; and
  - ➢ Nutrient uptake by biofilms
- Inter-event processes
  - ➢ Nutrient adsorption and pollutant decomposition by soil bacteria; and
  - ➢ Adsorption of metals and nutrients by filter particles.

To retain the filter media within the rain garden and aid drainage, one or more layers are used at the bottom of the filter. Figure C6 shows a schematic of a rain garden highlighting key elements

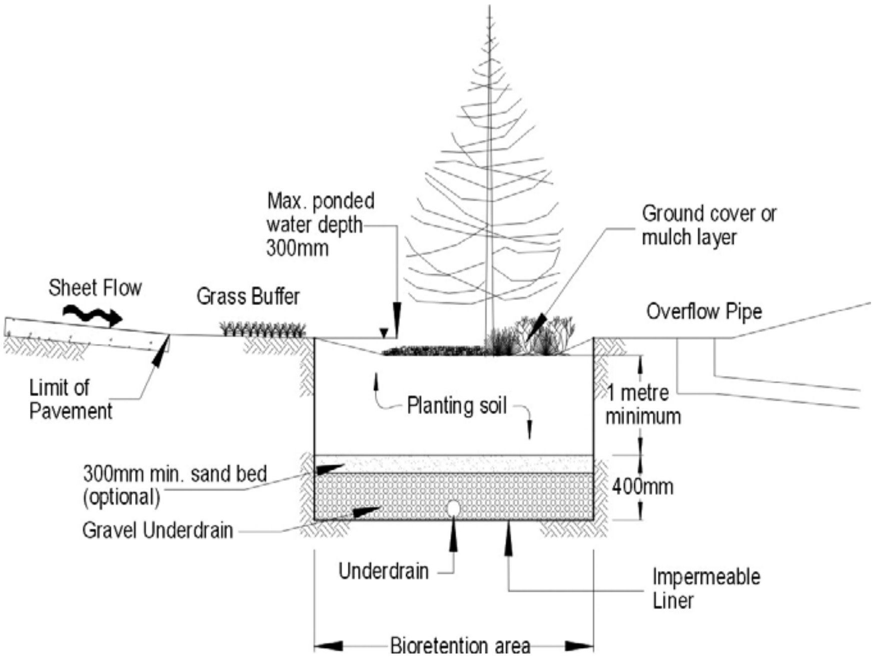
### 4.3.2 Design considerations

The main components of a rain garden include:

- Grass filter strip for minor pre-treatment (where space is available)
- Ponding area in the extended detention zone
- Planting soils
- Ground cover or mulch layer
- Plant material

- Underdrain system

**Figure C6**
**Schematic of Rain Garden showing approximate depth of materials**




Depending on the natural soils in the area that the rain garden has been placed, final discharge of stormwater can be to ground or through a drainage system to surface waters. This will depend on the permeability rates of the underlying soil, depth to groundwater or bedrock and the stability of any slopes that the additional water may be discharged through. In the situation where the eventual disposal of stormwater is to ground, testing of infiltration rates needs to be done consistent with infiltration practices shown in Section 4.4.

It is not recommended that geotextile filter cloth be used between the different media layers in the rain garden, as that will become a point of clogging in the filter. Proper installation of the various layers of media (including drainage layer) will reduce potential migration of contaminants to the drainage system.

Rain gardens are designed as water quality practices and but they can also be used for water quantity control.

## 4.3.3 Targeted contaminants

Rain gardens are effective as treatment to remove sediment, metals and hydrocarbons. They are not as effective at removal of nutrients but efforts are underway in Australia to assess their ability to remove them if an anaerobic zone is created at their base. Final results have not yet verified the success of this modification.

## 4.3.4 Advantages

Of all stormwater practices rain gardens are probably the most attractive if done well. They are effective for small catchment areas and can provide extended detention benefits as permeability rates are fairly low. In addition they are effective at treating a wide range of contaminants due to a high organic content of the media.

### 4.3.5 Limitations

Rain gardens are only suitable for small catchment areas below approximately 2 hectares. For larger areas a constructed wetland would be a better alternative. Another practical limitation is steeper slopes. While a rain garden can be engineered to fit steeper sites there are practical limitations such as retaining wall height or very deep cuts in a given slope.

### 4.3.6 Design sizing

Design approach:

1. Determine the water quality storage volume using 1/3 of the 2-year storm rainfall depth as shown in the ARC's TP 108.
2. Minimum live storage provided above the soil media is 40% of the water quality volume to ensure that the entire water quality storm passes through the rain garden. Failure to provide the storage will result in system bypass and reduced water quality expectations.
3. Calculate the required surface area of the rain garden.

   $A_{rg} = (WQV)(d_{rg})/k(h+d_{rg})t_{rg}$

   Where:
   $A_{rg}$ = surface area of rain garden ($m^2$)
   WQV = water quality treatment volume ($m^3$)
   $d_{rg}$ = planting soil depth (m)
   k = coefficient of permeability (m/day)
   h = average height of water (m) = ½ maximum depth
   $t_{rg}$ = time to pass WQV through soil bed (days)

   The following values should be used.

   $d_{rg}$ = 1.0 metre (limiting media)
   k = 0.5 m/d (slightly higher than TP 10 recommendation)
   h = 0.15 m (maximum water depth 300 mm)
   $t_{rg}$ = 1.0 for residential, 1.5 days for commercial or industrial

4. General comments on rain gardens

   - If less depth of media must be used due to local constrictions (bedrock, groundwater) the area of storage must be increased so the same volume of storage in the media is maintained. The simplest way to ensure the storage volume is maintained is the following ratio:

     $A_{rev.} = A_{rg}/d_{rev.}$

     Where:
     $A_{rev}$ = revised surface area resulting from decreased depth
     $A_{rg}$ = Area of rain garden calculated in step 3 ($m^2$)
     $d_{rev.}$ = actual depth provided

- The coefficient of permeability will initially decline during the establishment phase, as the filter media settles and compacts, but this will level out and then start to increase as the plant community establishes itself and the rooting depth increases.
- Keep drainage areas small and avoid sizing them for too large a catchment area. It is better to have more rain gardens than larger ones.
- Place them in areas where they will not interfere with normal use of the property and where they don't interfere with sight lines, which may present safety issues.
- Where possible, design them as off-line systems so that larger flows do not scour the surface of the rain gardens.

5. Composition of planting soil

The Facility for Advancing Water Biofiltration (FAWB) has been investigating filter media for several years and has developed the following recommendations for the composition of planting soil (FAWB, 2008).

The FAWB bioretention filter media guidelines require three layers of media. The filter media itself (400 - 600 mm deep), a transition layer (100 mm deep) and a drainage layer (50 mm minimum under drainage pipe cover. The FAWB recommendations are shallower than those recommended here as storage is available in the toolbox requirements for storage and release of the 34.5 mm rainfall over a 24-hour period.

The filter media is required to support a range of vegetation types (from groundcovers to trees) that are adapted to freely draining soils with occasional flooding. The material should be:

- Based on natural soils or amended natural soils and can be of siliceous or calcareous origin,
- In general, the media should be loamy sand with an appropriately high permeability under compaction and should be free of rubbish, deleterious material, toxicants, noxious plants and local weeds and should not be hydrophobic.
- The filter media should contain some organic matter for increased water holding capacity but low in nutrient content.

6. Determination of hydraulic conductivity

If required for maintenance reasons, the hydraulic conductivity of potential filter media should be measured using the ASTM F1815-06 method (ASTM International, 2006). This test method uses a compaction method that best represents field conditions and so provides a more realistic assessment of hydraulic conductivity than other test methods.

The hydraulic conductivity, or permeability, has been selected as 0.5 m/day. If the conductivity were measured upon construction completion the permeability may be much higher. The value selected accounts for partial clogging over time, which does occur and the rain garden size is then appropriate in surface area for the partial clogged condition.

7. Particle size distribution

Particle size distribution (PSD) is of secondary importance compared to hydraulic conductivity. A material whose PSD falls within the recommended range does not preclude the need for hydraulic conductivity testing. However, the following Table C6 provides a composition range for appropriate material specification.

**Table C6**
**Composition Range of filter media (FAWB, 2008)**

| Material | Percentage of total composition | Particle size |
|---|---|---|
| Clay and silt | <3% | (<0.05 mm) |
| Very fine sand | 5-30% | (0.05-0.15 mm) |
| Fine sand | 10-30% | (0.15-0.25 mm) |
| Medium to coarse sand | 40-60% | (0.25-1.0 mm) |
| Coarse sand | 7-10% | (1.0-2.0 mm) |
| Fine gravel | <3% | (2.0-3.4 mm) |

Clay and silt are important for water retention and sorption of dissolved contaminants; however they substantially reduce the hydraulic conductivity of the filter media. This size fraction also influences the structural stability of the material (through migration of particles to block small pores and/or slump). It is essential that the total clay and silt mix is less than 3% to reduce the likelihood of structural collapse of such soils.

The filter media should be well graded with all particle size ranges present from the 0.075 mm to the 4.75 mm sieve (as defined by AS1289.3.6.1-1995). There should be no gap in the particle size grading, and a small particle size range should not dominate the composition.

8. Soil media properties

Filter media that do not meet the following specifications should be rejected.

- Organic matter content less than 5% in areas where nutrients are the contaminants of concern. If metals were the primary contaminant then greater organic matter content would be appropriate.
- pH - 5.5-7.5
- Electrical conductivity <1.2dSiemens/m

9. Transition and drainage layers

The transition layer material shall be a clean, well-graded coarse sand material containing little or no fines.

The drainage layer is to be clean, fine gravel, such as a 2-5 mm washed screenings.

10. Plant material

Consider the following when making planting recommendations:

- Native plant species should be specified over exotic or foreign species

- Appropriate vegetation should be selected on its ability to thrive in wet and dry conditions.

The following two tables (Tables C7 and C8) provide some recommendations for rain garden plant species.

| **Table C7<br>Recommendations for Trees and Shrubs (ARC, 2003)** | |
|---|---|
| **Trees and shrubs** | Descriptions |
| ***Brachyglottis repanda*** **rangiora** | Coastal shrub or small tree growing to 4m+. Large attractive pale green leaves with white fuzz on underside. |
| ***Coprosma acerosa*** **sand coprosma** | Grows naturally in sand dunes. Yellow, interlaced stems and fine golden foliage. Forms a tangled shrubby ground cover. Tolerates drought and full exposure. Prefers full sun. |
| ***Coprosma robusta / C. lucida*** **karamu, shining karamu** | Shrubs or small trees growing to 3m+, with glossy green leaves. Masses of orange-red fruit in autumn are attractive to birds. Hardy plants. |
| ***Cordyline australis*** **ti kouka, cabbage tree** | Palm-like in appearance with large heads of linear leaves and panicles of scented flowers. Sun to semi-shade. Prefers damp to moist soil. Grows eventually to 12m+ height. |
| ***Cordyline banksii*** **ti ngahere, forest cabbage tree** | Branching from the base and forming a clump. Long strap-shaped leaves with red-orange coloured veins. Prefers good drainage and semi-shade. |
| ***Corokia buddleioides*** **korokio** | Bushy shrub to 3m, with pale green leaves with silvery underside. Many small bright yellow starry flowers are produced in spring. Prefers an open situation but will tolerate very light shade. |
| ***Entelea arborescens*** **whau** | Fast growing shrub or small tree (to 5m height) with large bright green heart-shaped leaves. Spiny seed capsules follow clusters of white flowers in spring. Handsome foliage plant |
| ***Geniostoma rupestre*** **hangehange** | Common forest shrub with pale green glossy foliage, growing to 2-3m. Tiny flowers give off strong scent in spring. Looks best in sunny position where it retains a bushy habit, and prefers well drained soil. |
| ***Hebe stricta*** **koromiko** | Shrub or small tree growing to 2-5m in height. Natural forms have white to bluish flowers. Many cultivars and hybrids available with other colours, but unsuitable for use near existing natural areas. Full sun. |
| ***Leptospermum scoparium*** **manuka** | Shrub or small tree growing to 4m+ in height. Natural forms have white to pinkish flowers. Many cultivars and hybrids available with other colours, but unsuitable for use near existing natural areas. Hardy and tolerant of difficult conditions. |
| ***Metrosideros robusta*** **rata** | Eventually forms a large tree. Flowers bright red in summer. Will tolerate dryness and exposure. Full sun. |
| ***Pittosporum cornifolium*** **tawhirikaro** | A slender branched shrub grown for its attractive fruiting capsules which are brilliant orange when split open. Sun or semi-shade. |
| ***Pittosporum kirkii*** | A small tree with dark green leaves and large yellow flowers in the summer. Prefers shade |
| ***Pseudopanax crassifolius*** **horoeka** | Very narrow rigid and leathery leaves in its juvenile form. Stunning in amongst bold leaved plants. Sun or semi-shade. |
| ***Pseudopanax lessonii*** **houpara** | Small tree with attractive foliage. Tolerates full exposure and drought. Sun or semi-shade |

| Table C8<br>Grasses, Ground Covers and Other Plants | |
|---|---|
| Grasses, ground covers, and other plants | Description |
| ***Arthropodium cirratum***<br>**Rengarenga, renga lily** | A lily with fleshy pale green – greyish leaves and white flowers. Ground cover in semi shady situation |
| ***Asplenium bulbiferum***<br>**mouku, hen and chicken fern** | A robust fern with small plantlets produced on the fronds. Tolerates dryness and prefers shade |
| ***Asplenium oblongifolium***<br>**huruhuruwhenua, shining spleenwort** | Fern with large shiny fronds. Tolerates dryness. Prefers shade |
| ***Astelia banksii***<br>**kowharawhara, coastal astelia** | Clump forming plant up to a metre high with flax-like leaves. Requires semi-shade. Tolerates full exposure. Frost tender |
| ***Astelia solandri***<br>**kowharawhara, perching astelia** | An epiphytic plant in natural situations. Long drooping bright green leaves. Tolerates dryness. Prefers shade |
| ***Carex flagellifera***<br>**manaia, Glen Murray tussock** | Sedge up to 70cm high with reddish-brown spreading foliage. Prefers damp soil and full sun. Tolerates exposure |
| ***Carex testacea***<br>**sedge** | Coastal sedge up to 40cm high with shiny orange foliage. Prefers full sun and exposure. Tolerates dry soil conditions |
| ***Cortaderia fulvida***<br>**toetoe** | Branching from the base and forming a clump to 4m high. Long strap-shaped leaves with red-orange coloured veins. Prefers good drainage and semi-shade |
| ***Dianella nigra***<br>**turutu** | Lily with reddish leaves, and striking violet-blue fruit. Ground cover; prefers open well-drained situation |
| ***Disphyma australe***<br>**glasswort** | Fleshy leaved ground cover with mauve flowers in the spring. Tolerates drought and full exposure. Frost tender |
| ***Doodia media***<br>**pukupuku, rasp fern** | Hardy fern growing to 25cm. Young fronds coloured bright red when in full sun. Sensitive to frost |
| ***Libertia grandiflora*** & ***L. ixioides***<br>**mikoikoi, native iris** | Clump forming native irises with narrow, upright leaves. Small white flowers in spring. Sun or shade |
| ***Phormium cookianum***<br>**wharariki, mountain flax** | Clump-forming flax with yellow –green drooping leaves, to 2m. Full exposure and sun |
| ***Phormium tenax***<br>**harakeke, flax** | Clump-forming flax with large stiff leaves, to 3 m. Full exposure and sun |

Regarding planting, the following recommendations are made;

1. Species layout should generally be random and natural,
2. A canopy should be stabilised with an understory of shrubs and herbaceous plants,
3. Woody vegetation should not be specified in the vicinity of inflow locations,
4. Stressors (wind, sun, exposure) should be considered when developing the planting plan,
5. Noxious weeds should not be specified,
6. Aesthetics and visual characteristics should be given consideration,
7. Traffic and safety issues must be considered, and
8. Existing and proposed utilities must be identified and considered.

## 4.3.7 Case study

### 4.3.7.1 Project description

A driveway/access interchange (road crowned in the centre so only ½ road area is included) is proposed in Papakura with a rain garden proposed due to aesthetics, peak control and water quality treatment. The total extent of the catchment being served is 820 $m^2$ of which 60% is impervious with the remainder being road or driveway verge.

### 4.3.7.2 Hydrology

1. Calculate the water quality volume to be treated.

   Water quality storm from TP 108 = 23 mm of rainfall

   The catchment effective first flush runoff area = $A_{eff}$ = impervious% x total Area (ha). $A_{eff}$ = (.6)0.082 = 0.049

   The first flush volume $V_{ff} = 10 \times A_{eff} \times d_{ff}$ ($m^3$)
   Where $d_{ff}$ = first flush water quality depth (water quality storm)
   So, $V_{ff}$ = 10 x 0.049 x 23 = 11.3 $m^3$

   To account for extended detention from the impervious surfaces, 30% more storage must be added to the water quality volume to provide additional control.

   Revised WQV = 14.7 $m^3$

### 4.3.7.3 Rain garden design

2. Live volume of storage needed $V_{live}$ = .40(14.7 $m^3$) = 5.9 $m^3$

3. Calculate the required surface area of the rain garden.

   $A_{rg} = (WQV)(d_{rg})/k(h+d_{rg})t_{rg}$

   Where:
   $A_{rg}$ = surface area of rain garden ($m^2$)
   WQV = water quality treatment volume ($m^3$)
   $d_{rg}$ = planting soil depth (m)
   k = coefficient of permeability (m/day)
   h = average height of water (m) = ½ maximum depth
   $t_{rg}$ time to pass WQV through soil bed

   The following values should be used.

   $d_{rg}$ = 1.0 metre
   k = 0.5 m/d
   h = 0.15 m (maximum water depth 300 mm)
   $t_{rg}$ = 1.0 days

$A_{rg}$ = 14.7(1)/0.5(0.15+1)(1.0)

$A_{rg}$ = 25.6 $m^2$

Check to ensure that the surface area calculated and having 0.3 m of live storage provides at least 5.9 $m^3$.
(25.6)(0.3) = 7.68 $m^3$ so the surface area of the rain garden provides for the necessary live storage.

By sizing the rain garden for 1.3 times the water quality volume, extended detention is provided along with storing the difference between the 2- and 10-year storms.

# 4.4 Infiltration trenches

## 4.4.1 Description of practice

Infiltration practices direct urban stormwater away from surface runoff paths and into the underlying soil. This is in contrast to surface detention methods, which are treatment or delay mechanisms that ultimately discharge all stormwater runoff to streams. Infiltration trenches divert runoff into groundwater.

In terms of rural land use, there are two different infiltration practices that could be considered.

- Infiltration trenches, and
- Permeable paving

For the purposes of this toolbox, infiltration trenches will be the only practice that will be discussed in detail. Permeable paving, while being very appropriate for low traffic parking areas such as driveways, are not recommended for roads. Clogging issues and frequent maintenance obligation would limit their use for local roading. There is also the issue of saturation of a road sub-base.

Infiltration trenches are recommended but should be used primarily for groundwater recharge and stormwater volume reduction rather than for water quality treatment. The appropriate use of an infiltration trench would be as part of a treatment train where a swale or filter strip would provide primary contaminant removal with the infiltration trench providing volume control and groundwater recharge.

## 4.4.2 Design considerations

As infiltration trenches are a stormwater management practice that reduces the total volume of stormwater runoff, objectives relate primarily to stream baseflow augmentation and stream erosion protection by reducing the total volume of runoff.

Infiltration practices function primarily by passage of water from the surface into the ground. This passage depends on the following:

- Permeability rates
- Sufficient depth to groundwater or bedrock
- Influent concentrations that would not cause clogging or a threat to local groundwater quality

#### 4.4.2.1 Permeability rates

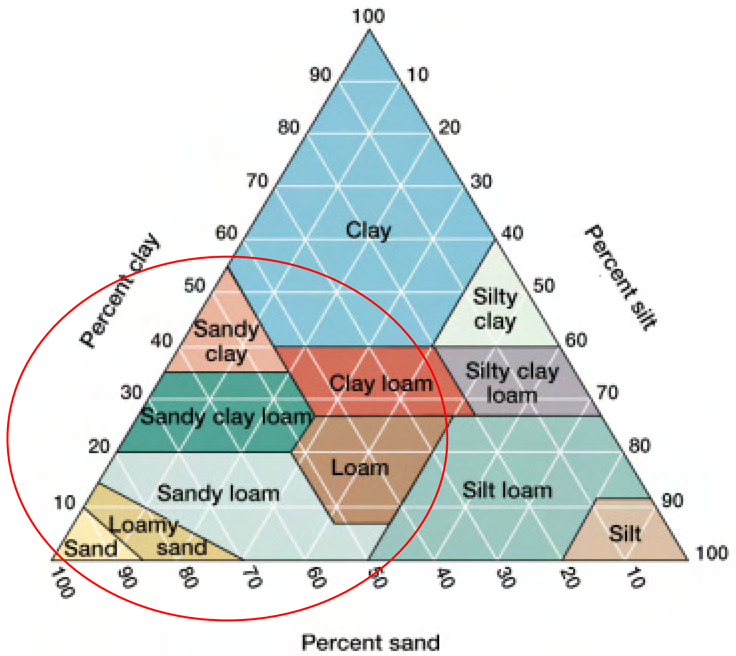
Soil permeability is the most critical consideration for suitability of infiltration trenches. Trenches should be constructed in native soil and Table C9 provides infiltration rates for various soil textures and provides a minimum infiltration rate of 7 mm/hour to determine whether infiltration trenches are a suitable practice for a given highway location.

**Table C9**
**Infiltration Rate for Various Soil Textural Classes**

| Texture Class | Approximate Infiltration Rate in mm/hour |
|---|---|
| Sand | 210 |
| Loamy sand | 61 |
| Sandy loam | 26 |
| Silt loam | 13 |
| Sandy clay loam | 7 |
| Clay loam | 4.5 |
| Silty clay loam | 2.5 |
| Sandy clay | 1.5 |
| Silty clay | 1.3 |
| Clay | 1.0 |
| | 0.5 |

Another way to graphically show infiltration capability is through the use of the Soil Textural Triangle (Davis and Bennett, 1927) as shown in Figure C7. Those soils considered suitable for infiltration trenches are shown in the red circle and are generally in the loam and sand category. Soils suitable for infiltration should have clay content less than 30% and combined silt/clay content less than 40%.

**Figure C7**
**Soil Textural Triangle (Davis, Bennett, 1927)**




In addition, infiltration needs to be carefully considered on sites having fractured rock. There should be at least 3 metres of soil between the invert of the trench and the location of any fractured rock.

At the same time as there is a minimum infiltration rate for infiltration trenches, there also has to be a maximum rate of infiltration to protect groundwater. Infiltration rates in excess of 1 metre/hour may indicate a direct link to a very permeable aquifer. If the rate is in excess of 1 metre/hour, runoff pre-treatment must be provided for water quality treatment to prevent migration of contaminants to groundwater.

#### 4.4.2.2 Sufficient depth to groundwater or bedrock

There are two issues related to depth of groundwater or bedrock.

- Shallow groundwater or bedrock providing no ability to infiltrate stormwater runoff, and
- Having little depth between the ground surface and groundwater or bedrock may allow for groundwater mounding to enter the infiltration practice and limit effective function of the trench.

Due to these concerns there should be at least 3 metres difference between the invert of the infiltration trench and the elevation of the seasonal groundwater table or bedrock. In terms of the seasonal groundwater table, that elevation must be found at the time of year when the water table is at its highest elevation.

#### 4.4.2.3 Influent *concentrations* causing clogging or groundwater quality threat

Due to concerns about clogging, excessive sediment loads should not be allowed to enter the trench. Because of this potential, pre-treatment upstream of the trench is vital to increasing the time period when maintenance has to be accomplished. To further minimise the clogging potential, the design employs an upper gravel or stone layer, approximately 300 mm in thickness with filter fabric placed below at 300 mm below the surface. This upper layer acts as an initial filter and can periodically be removed and replaced as conditions warrant rather than removing the entire rock volume.

There are several other criteria that should be considered before infiltration trenches are used for stormwater management.

- Infiltration trenches must not be constructed in fill material,
- Trenches must not be constructed on slopes in excess of 15%,
- Catchment areas draining to a single trench should not exceed two hectares,
- Trenches should be located at least 300 metres from any municipal or private water supply bore or 30 metres from on-site wastewater systems,
- Infiltration trenches should not be used over limestone where there is potential for a sinkhole to develop,

Figures C8 and C9 provide schematics of infiltration trenches used to receive roof runoff and to receive overland flow runoff.

**Figure C8**
**Infiltration Trench for Roof Runoff**

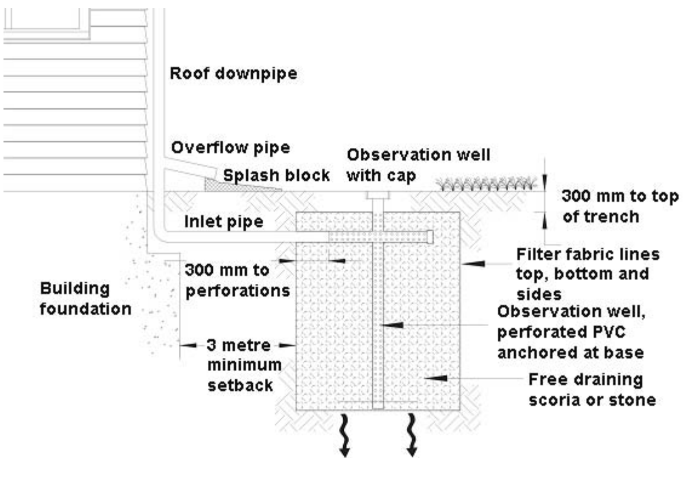
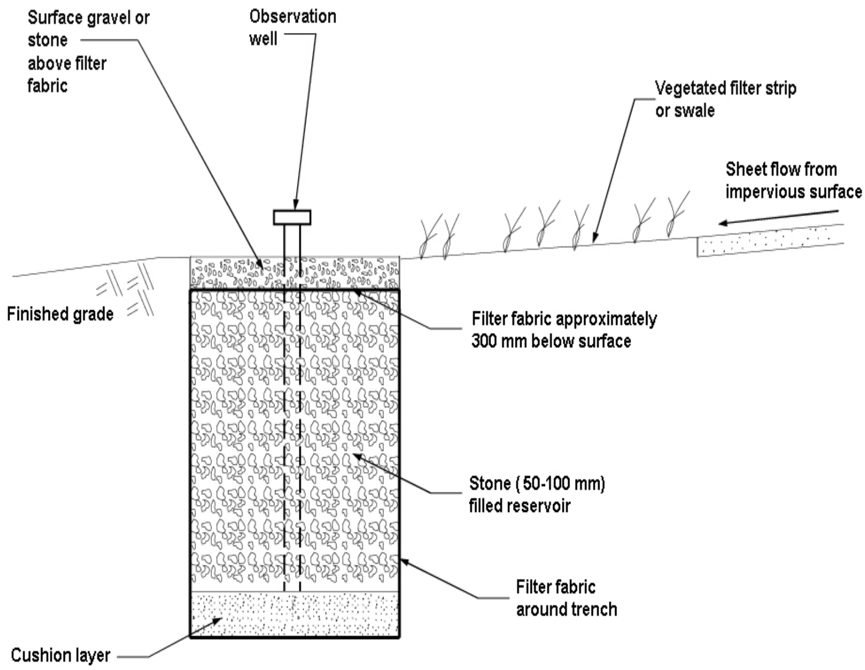


**Figure C9**
**Schematic of an Infiltration Trench for Site Impervious Surfaces**




## 4.4.3 Targeted contaminants

Infiltration trenches are effective at removal of sediment, metals and total petroleum hydrocarbons (TPH). They are moderately effective at removal of phosphorus but are not effective at removal of nitrogen.

## 4.4.4 Advantages

Infiltration trenches control the volume of stormwater being discharged, which is important in preventing adverse impacts to receiving systems (primarily streams) and maintenance of stream baseflow.

## 4.4.5 Limitations

Infiltration trenches are prone to clogging by sediment entry. It is essential that, once constructed, they be protected from sediment being transported from earthwork areas. In addition, there may be groundwater issues if there is a contaminant spill. This is not expected to be a problem with countryside development but the concern is important to recognise if a trench accepts runoff from an industrial site or a highly trafficked road.

### 4.4.6 Design sizing

This approach relies on Darcy's Law (1856), which expresses flow through a porous media. There are two equations that are used: one for surface area of the trench ($A_s$) and the trench volume ($V_t$).

In terms of the design approach:

1. Determine the water quality rainfall by using 1/3 of the 2-year storm in TP 108.
2. Determine the water quality volume from the following equation. The catchment effective first flush runoff area = $A_{eff}$ = impervious% x total Area (ha)
   The first flush volume $V_{ff}$ = 10 x $A_{eff}$ x $d_{ff}$ ($m^3$) = WQV
   Where $d_{ff}$ = first flush water quality depth (water quality storm)
3. Size the trench surface area to allow complete infiltration within 48 hours, including rainfall falling directly on the trench. Use the following equation to determine surface area:

   $A_s = WQV/((f_d)(i)(t)-p)$

   Where:
   $A_s$ = surface area of the trench ($m^2$)
   WQV = water quality volume ($m^3$)
   $f_d$ = infiltration rate (m/hr) - rate reduced by ½ from measured
   i = hydraulic gradient (m/m) - assumed to be 1
   t = time to drain from full condition (hours) - maximum time 48 hours
   p = rainfall depth for water quality storm (m)

   There is a simple test to see how deep an infiltration trench can be to achieve the discharge of the water quality storm. Any deeper than the amount calculated will not achieve the two-day draw down period. The equation is the following:

   $d_{max} = f_d(t/V_r)$

   Where:
   $d_{max}$ = maximum depth of trench
   $f_d$ = infiltration rate (m/hr)
   t = time to drain from full condition (hours)
   $V_r$ = void ratio of reservoir stone (normally 0.35 for stone or 0.5 if scoria is used)

   Once $d_{max}$ has been defined, the actual needed depth can be calculated. If the actual depth exceeds the maximum depth the surface area must be increased to account

4. Find the trench volume to provide storage for 37% of the volume required to infiltrate. This allows for storage of excess runoff during those periods when the runoff exceed the infiltration rate.

   $V_t = 0.37(WQV + pA_s/V_r)$

   Where:

   $V_t$ = trench volume with the aggregate added

5. Calculate the trench depth and compare with the maximum depth

   $V_t/A_s$ = depth of trench (d)

   If $d < d_{max}$ the design is adequate. If $d > d_{max}$ then trench surface area must be increased and depth decreased.

6. If extended detention is a requirement, take the water quality volume and multiply by 1.3 and use this in the equation for calculating the trench surface area.

7. If peak flow control of the 2- and 10-year storms is required, the trench can provide capacity to limit peak discharge increases if the catchment area contributing to the trench is kept as small as possible. If extraneous flows cannot bypass the trench, the difference in volume of the 10-year storm must be calculated and the trench volume increased if the 10-year storm volume exceeds that provided by 1.3 times the water quality volume.

Note

The Franklin District Council has a design manual for soakage retention pits (infiltration trenches) that provides an easy approach to calculating water quality volumes for trenches. Using their approach will provide comparable volumes as would be used for sizing the trench for water quality treatment.

Using the Franklin District Council approach can be used to calculate the water quality volume in steps 1 and 2 of this design sizing but the volume calculated flow will need to be increased to provide for extended detention or peak flow control.

### 4.4.7 Case study

#### 4.4.7.1 Project description

A house is being constructed for in the vicinity of Pukekohe. The house roof area will be 200 $m^2$ and an infiltration trench will be sized to handle the runoff. While water quality is not an issue from a residential roof constructed of a benign material, the trench will provide extended detention control and peak flow control.

The measured infiltration rate for the trench location is 14 mm/hour.

#### 4.4.7.2 Hydrology

1. Calculate the water quality volume

   The 2-year storm for Pukekohe is 70 mm of rainfall so the water quality storm is 23 mm of rainfall.

2. Using that rainfall, the water quality volume is calculated. The catchment effective first flush runoff area = $A_{eff}$ = impervious% x total Area (ha)
   The first flush volume $V_{ff} = 10 \times A_{eff} \times d_{ff}$ ($m^3$)
      Where $d_{ff}$ = first flush water quality depth (water quality storm)
      $A_{eff}$ = (1) x .02 = 0.02

$V_{ff}$ = 10x(0,02)(23) = 4.6 $m^3$

WQV = 4.6 $m^3$

#### 4.4.7.3 Infiltration trench design

3. Calculate the practice surface area

$A_s$ = WQV/(($f_d$)(i)(t)-p)

Where:

| | |
|---|---|
| $A_s$ | = surface area of the trench ($m^2$) |
| WQV | = water quality volume ($m^3$) = 4.6 $m^3$ |
| $f_d$ | = infiltration rate (m/hr) - rate reduced by ½ from measured =14 mm/hour reduced by ½ as a factor of safety, so $f_d$ = 7 mm/hour = 0.007 m/hour |
| i | = hydraulic gradient (m/m) - assumed to be 1 |
| t | = time to drain from full condition (hours) - maximum time 48 hours |
| p | = rainfall depth for water quality storm (m) = .023 m |

$A_s$ = 4.6/(.007)(1)(48) - .023 = 14.7 $m^2$

Calculate the maximum trench depth

$d_{max}$ = $f_d$(t/$V_r$)

Where:

| | |
|---|---|
| $d_{max}$ | = maximum depth of trench |
| $f_d$ | = infiltration rate (m/hr) = 0.007 m/hour |
| t | = time to drain from full condition (hours) = 48 hours |
| $V_r$ | = void ratio of reservoir stone (scoria) = 0.5 |

$d_{max}$ = .007(48/.5) = 0.67 m

4. Find the trench volume

$V_t$ = 0.37(WQV + $pA_s$)/$V_r$ = 0.37(4.6 + 0.023(14.7)/.5 = 3.65 $m^3$

5. Calculate the trench depth and compare with the maximum depth

d = $V_t$/$A_s$ = depth of trench (d) = 3.6/14.7 = 0.25 m

d < $d_{max}$ so the design is adequate

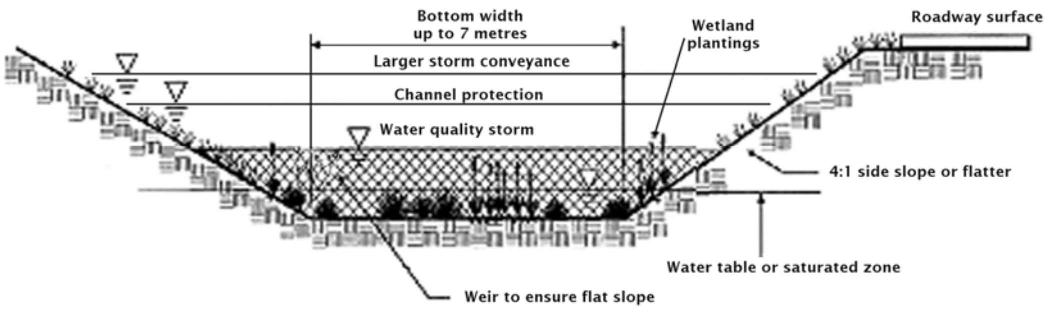
## 4.5 Wetland swales

### 4.5.1 Description of practice

Wetland swales consist of broad open channels in areas where slopes are slight, water tables are high or, on a seasonal basis, there is base flow, and there are saturated soil conditions. If soil is saturated for more than two weeks, normal grasses will not grow.

Wetland swales are similar to normal constructed wetlands in their use of vegetation to treat stormwater runoff. The wetland swale acts similarly to a long and linear shallow wetland treatment practice. Figure C10 shows a typical cross-section for a wetland swale.

**Figure C10**
**Cross-Section of a Wetland Swale (adapted from CWP, 2001)**




### 4.5.2 Design considerations

There are two separate approaches that can be used for sizing wetland swales.

- Storage of the water quality volume generated by the upstream catchment, or
- Ensuring wetland swale residence times exceed 9 minutes.

For the purposes of this toolbox, the recommended approach is ensuring residence times exceed 9 minutes. As the wetland swale will, for the most part, have water in it with standing vegetation, the vegetation will not be as dense as vegetation in a normal vegetated swale. This will result in using a Manning's roughness coefficient of 0.1.

As a result, wetland swales will either be longer or wider than normal vegetated swales. There are several key design elements to a wetland swale.

- As there is no concern about wider channels concentrating flow at one point (as in normal swales), a wetland swale can be up to 7 metres wide.

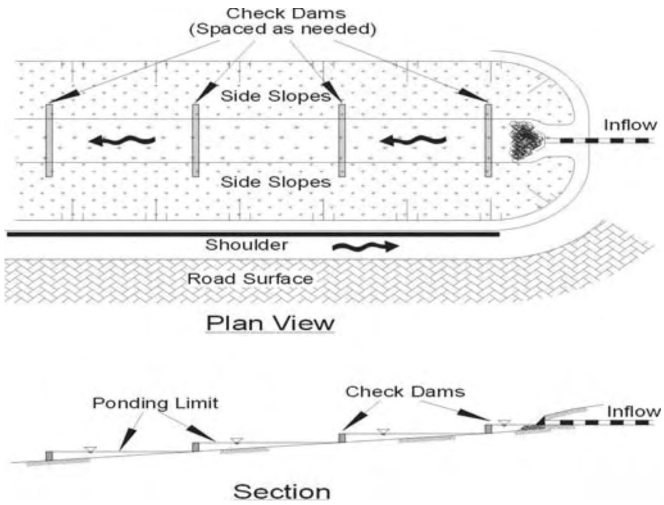
- Due to a reduced roughness coefficient, a length to width ratio of 5 horizontal: 1 vertical must be provided.
- If there is a longitudinal slope, check dams must be used to step the flow, ensure a level bottom on the wetland swale and maintain very shallow side slopes.

A schematic of a wetland swale with check dams is shown in Figure C11. Even though there is a longitudinal slope, the check dams ensure a level invert elevation.

### 4.5.3 Targeted contaminants

Wetland swales are effective at removing suspended sediments and metals. They provide a moderate removal of nutrients and are less effective at removal of oil and grease.

**Figure C11**
**Longitudinal Slope on a Wetland Swale**




### 4.5.4 Advantages

Wetland swales can have the following advantages.

- Having an outlet structure for the wetland swale can provide for peak flow control and extended detention,
- They can accentuate the natural landscape,
- Contaminant removal efficiency can be improved over a normally dry swale, and
- They enhance biological diversity and create beneficial habitat between upland areas and streams.

### 4.5.5 Limitations

Wetland swales are not practical in areas of steep topography and are not practical when driveway crossings are required unless significant opening areas are provided.

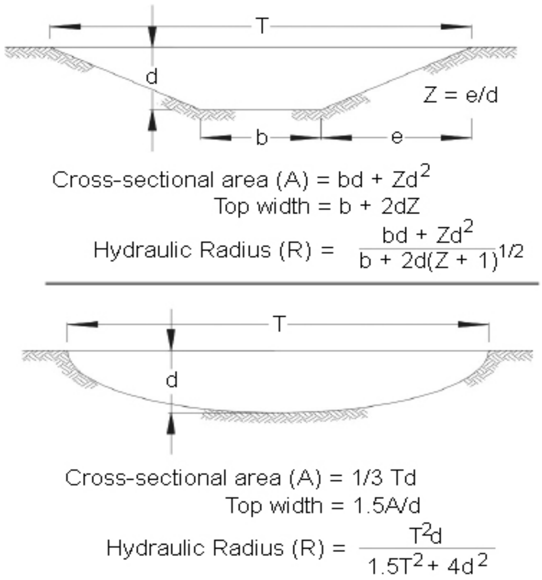
### 4.5.6 Design sizing

The design approach takes the designer through a series of steps that consider swale performance for water quality treatment and consideration of larger flows to ensure that scour or resuspension of deposited sediments does not occur.

1. Estimate runoff flow rate from the water quality storm using 1/3 of the 2-year storm as the water quality storm and calculate the flows. Wetland swales are designed by flow rate as discussed in Section 4.5.2.
2. In using the Rational Formula for design, use the entire water quality storm as i. This would be the maximum possible value for discharge calculations and thus be conservative.

3. Establish the longitudinal slope of the wetland swale. The maximum slope (with or without check dams) should be less than 2%.
4. Select wetland vegetation cover. Types of wetland vegetation to recommend are detailed in the ARC's TP 10, in Section 6.9.1.
5. The value for Manning's coefficient of roughness for wetland swales is 0.10.
6. Select a swale shape. Two shapes are proposed as they ensure distributed flow throughout the bottom of the swale. Of the two shapes, the trapezoidal shape is recommended. Channel geometry and equations for calculating cross-sectional areas and hydraulic radius are provided under the individual configurations in Figure C12.
7. An assumption is made on the normal pool and live storage depth of flow for the water quality storm. This assumed depth is used for calculating the bottom width of the wetland swale and cross-sectional area.
8. It is not required to have a normal pool elevation for a wetland swale but it is important to have a saturated subgrade for wetland plants to thrive. If it can be documented that groundwater is at the surface for the entire year, then a wetland swale is very appropriate.
9. Use Manning's equation for calculating dimensions of the swale by using first approximations for the hydraulic radius and dimensions for selected shape.

**Figure C12 Channel Geometry**




$Q = AR^{0.67}s^{0.5}/n$

By making some assumptions about depth and width ratios such as the hydraulic radius for a trapezoid approximating the depth (d), the bottom width of a trapezoid (b) equals the following:

$b = (Qn/d^{1.67}s^{0.5}) - Zd$

The slope, depth, discharge and side slope are all known and b can be determined.

Where:

Q = design discharge flow rate ($m^3/s$)
n = Manning's n (dimensionless)
s = longitudinal slope (m/m)
A = cross-sectional area ($m^2$)
R = hydraulic radius (m)
T = top width of trapezoid/parabolic shape (m)
d = depth of flow (m)
b = bottom width of trapezoid (m)

For a parabola, the depth and discharge are known so the top width can be solved for.

8. Knowing b (trapezoid) or T (parabola), the cross-sectional area can be determined by the equations in Figure C2.
9. Calculate the swale velocity from the following equation:

$$V = Q/A$$

   If V > 0.8 m/s repeat steps 1 - 9 until the velocity is less than 0.8 m/s.
10. Calculate the swale length (L in metres)

   $L = Vt$ (60 s/minute)

   Where t = residence time in minutes.

#### 4.5.6.1 Flows in excess of the water quality storm

It is expected that runoff from events larger than the water quality design storm will go through the wetland swale. In that situation, a stability check should be performed to ensure that the 10-year, 24-hour storm does not cause erosion. For the 10-year storm, flow velocities should not exceed 1.5 m/s, although higher velocities may be designed for with appropriate erosion protection. When considering larger storms consideration must be given to increased rainfall values as a result of climate change. Table C5 provides information on the percentage increase for design purposes.

If extended detention and/or peak flow control is a requirement for a specific project, the outlet of the wetland swale can be modified so that storage volumes are provided. In that situation, outlet design is based on a typical detention pond design and guidance on detention pond design is given in the ARC's TP 10.

### 4.5.7 Case study

#### 4.5.7.1 Project description

An access road, driveways and pasture for a 3 lot rural subdivision is proposed to drain to a wetland swale. The lane is 6.4 metres wide and 400 metres long, the three driveways are each 3.4 metres wide and 75 metres long and the pasture area is approximately 10,000 $m^2$. The project is located in the upper Flatbush catchment.

#### 4.5.7.2 Hydrology

Using the Rational Formula

$Q_{wq} = 0.00278CIA$

Predevelopment land use is pasture on a C soil so predevelopment C factor is 0.3.

C = 0.3 + .65(%Impervious cover/100) = 0.3 + .65(25%/100) = 0.46

I = Rainfall intensity (mm/hr.) - for Flatbush the water quality storm is 23 mm.
A = catchment area in hectares = 1.33 ha.

$Q_{wq}$ = 0.00278(0.46)(23)(1.33) = 0.039 $m^3/s$

For the one hour $Q_{10}$ in Flatbush, rainfall is 33.6mm. Effect of global warming on the 10-year storm is 7.4% increase in rainfall. So 33.6 mm for a 10-year storm increased by 7.4% results in 36.1 mm of rainfall.

$Q_{10}$ = 0.00278(.46)(36.1)(1.33) = 0.06 $m^3/s$

#### 4.5.7.3 Swale Design

Slope of swale alignment = 0.02

Several assumptions have to be made regarding the swale, first of which is that the wetland swale will have a trapezoidal design. Side slopes (Z) will then be recommended and an assumption of design storm depth should be made. That value may change depending on the velocity of flow being less than 0.8 m/s.

For this case study, Z = 4 and the depth of flow = 100 mm. The static water level (or dead storage) in the wetland swale is estimated to be 100 mm deep as check dams have been designed to maintain a level bottom, but that storage cannot be considered in terms of flow velocities. Since storm flow will overtop the check dams, the slope to use in calculations is the longitudinal slope and not permanent water elevation slope.

Based on the value for Q and s, and the assumptions for n and d, solve for the swale bottom width (b).

$b = (Qn/d^{1.67}s^{0.5}) - Zd$

$b = ((.039)(.1)/(.1^{1.67})(.02^{0.5})) - (4)(.1) = 0.94$ m

Calculate the top width

T = b + 2dZ = 0.94 + 2(.1)(4) = 1.74 m

Calculate the cross-sectional area

$A = bd + Zd^2 = (0.94)(.1) + 4(.1^2) = 0.1\ m^2$

Calculate the flow velocity

V = Q/A = 0.039/0.1 = 0.39 m/s which is well under than the 0.8 m/s maximum - good.

Calculate the wetland swale length

L = Vt = 0.39(540 sec.) = 210 metres long

The wetland swale length can be reduced significantly if it were made wider. A wetland swale can have a bottom width up to 7 metres as standing water will not cause flow to concentrate in one area. As an example, if the swale bottom width were increased to 3 metres, the following calculations will provide an adjusted length.

$b = 3$ metres
$T = 3 + 2(.1)(4) = 3.8$ m
$A = 0.34\ m^2$

$V = Q/A = 0.039/0.34 = 0.11$ m/s
$L = 0.11(540) = 59.4$ m (150 m less length than the previously calculated length)

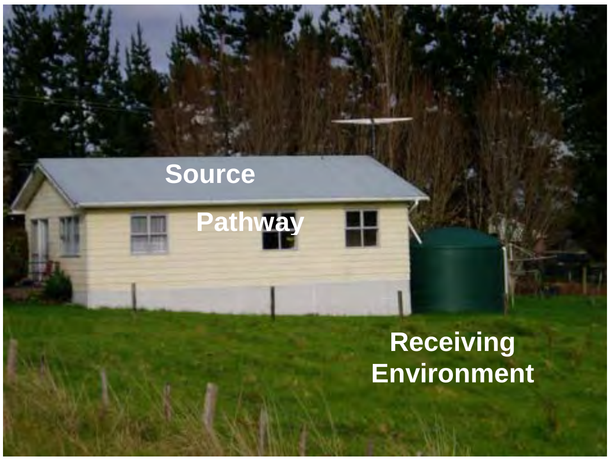
As the swale will probably have larger flows pass through it, the swale design can be adjusted to account for the larger flows. In this situation the Manning coefficient of roughness will not have to be decreased as wetlands vegetation is expected to be considerably higher than the static water level, so assume n = .1. Solve for d and ensure that velocities are not erosive. $Q_{10} = 0.06\ m^3/s$.

$b = (Qn/d^{1.67}s^{0.5}) - Zd$
$0.94 = (0.06(.1)/ d^{1.67}s^{0.5}) - 4d$
by trial and error, the wetland swale must have a depth of 120 mm to convey the 10-year storm
$A = bd + Zd^2 = (0.94)(.12) + 4(.12^2) = 0.17\ m^2$

$Q = AV$ or $Q/A = V = 0.06\ m^3/s/0.17 = 0.35$ m/s so the velocities during the 10-year storm are non-erosive.

## 4.6 Water tanks

### 4.6.1 Description of practice




A water tank is a storage receptacle for stormwater runoff that is generated from roof areas. The stored water can then be used for site needs.

The primary function of water tanks in a rural area is to provide water supply for residential, commercial and industrial use. In addition to the water supply benefits water tanks also reduce the total volume of stormwater runoff by redirecting the runoff to a storage tank for subsequent use for site water needs.

In terms of source, pathway and receiving environment, the source of stormwater is the structure roof, with the pathway being the gutters and down pipes and the water tank is the receiving environment.

It is recognised that in many situations the water tank will be the only source of water for a given site. As such, the tank water will be used for potable purposes. This can involve several health and safety related issues including treating and disinfecting the roof runoff to meet appropriate water quality standards. It is suggested that professional assistance be solicited in these situations. For more information it is suggested that a copy of the Ministry of Health's "Household Water Supply" (2004) document be read.

Much of the design approach in this Section comes from TP 10 (ARC, 2003).

### 4.6.2 Design considerations

There are a number of elements that need to be considered when designing a water tank.

- How much water can be captured from the roof,
- The anticipated water use,
- The percent of water from the roof that can be used,
- Peak flow considerations, and
- Sizing outlets

It is assumed that water tanks, in the context of this toolbox, will be full service tanks and not limited to non-potable uses.

It is not intended in this toolbox that roof areas compensate for impervious surfaces beyond the roof area itself. A major concern in rural land use is rill and gully erosion from very localised areas. As such, compensation of roof storage for a driveway is not recommended.

### 4.6.3 Targeted contaminants

For the most part, rainfall in rural New Zealand areas is not contaminated. The major source of contamination may be from the roof materials themselves or from animal or plant organic matter. Contamination issues can be minimised by using roofing materials that don't generate contaminants or by screening gutters for minimising the entry of organic matter.

### 4.6.4 Advantages

Water tanks have several advantages.

- They reduce the total volume of stormwater runoff by separating the site water use from stormwater runoff,
- They provide for site water use in areas where groundwater supply may be limited,
- Through storage and use, they can provide for detention of excess flows and reduce downstream effects.

Water tanks require minimal maintenance if filtering of roof runoff is provided through screens or first flush diverters.

### 4.6.5 Limitations

The most obvious limitation of water tanks is the potential for them to run dry during drought times. This issue can be minimised through provision of excess storage that ensures adequate capacity during drought times. In addition during extreme drought, water can be purchased to fill the tank.

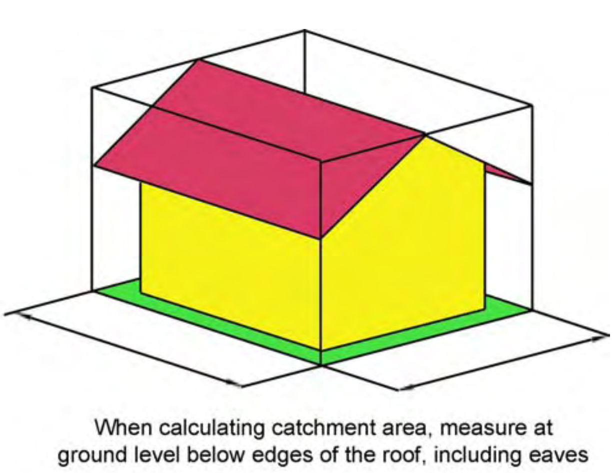
### 4.6.6 Design sizing

As mentioned in Section 4.6.2 there is a logical progression of analysis that needs to be done for water tank sizing.

#### 4.6.6.1 How much water can be captured from the roof

The first aspect of design is to calculate the roof area that will be drained to a water tank. Figure C13 details how that is done. The area that is green and covers the whole plane of the green area is the roof

**Figure C13**
**Calculation of Rood Surface Area**



area that is then used in calculations.

Another component of roof runoff capture is what percentage of stormwater that runs off the roof can be used depending on roof area, tank size and daily usage. Table C10 provides this information.

There are two key assumptions with considering Table C10.

- Tank water is the only water supply for rural development, and
- Water tank sizes are at least 25,000 litres. It is recommended that rural residential land use have two 25,000 litre tanks or a total water supply of 50,000 litres to ensure an adequate supply and, for commercial or industrial land use, to provide detention for times when the tank is full.

Several points can be observed. Daily water use for a residential home on tank water for all water needs will be at least 500 litres/day for three people so 100% of the runoff from the roof can be used if the roof area is 150 $m^2$.

As the roof area gets larger, the runoff from the roof is too great for the water tank to completely handle so there is runoff. A 200 $m^2$ roof with the same water consumption will have approximately 20% of the roof runoff released. A review of Table C10 provides information on various roof sizes and rainfall runoff expectations.

**Table C10**
**Percent of Stormwater Captured**

**150 $m^2$ Roof Area**

| Water use in litres per day | Average Yearly % of Water Captured from Roof | | | | | |
|---|---|---|---|---|---|---|
| | Rain Tank Capacity (Litres) | | | | | |
| | 200 | 1000 | 3000 | 4500 | 9000 | 25000 |
| 125 | 15% | 25% | 25% | 30% | 30% | 30% |
| 225 | 20% | 35% | 45% | 45% | 50% | 50% |
| 325 | 25% | 40% | 55% | 60% | 65% | 72% |
| 500 | 35% | 50% | 65% | 70% | 80% | 100% |
| 600 | 40% | 50% | 70% | 75% | 95% | 100% |
| 1000 | 45% | 55% | 75% | 80% | 100% | 100% |

**200 $m^2$ Roof Area**

| Water use in litres per day | Average Yearly % of Water Captured from Roof | | | | | |
|---|---|---|---|---|---|---|
| | Rain Tank Capacity (Litres) | | | | | |
| | 200 | 1000 | 3000 | 4500 | 9000 | 25000 |
| 125 | 10% | 20% | 20% | 20% | 20% | 20% |
| 225 | 20% | 25% | 35% | 35% | 35% | 35% |
| 325 | 20% | 30% | 40% | 45% | 50% | 55% |
| 500 | 30% | 40% | 55% | 60% | 70% | 80% |
| 600 | 30% | 45% | 60% | 65% | 75% | 85% |
| 1000 | 35% | 45% | 65% | 70% | 80% | 90% |

**250 $m^2$ Roof Area**

| Water use in litres per day | Average Yearly % of Water Captured from Roof | | | | | |
|---|---|---|---|---|---|---|
| | Rain Tank Capacity (Litres) | | | | | |
| | 200 | 1000 | 3000 | 4500 | 9000 | 25000 |
| 125 | 10% | 15% | 20% | 20% | 20% | 20% |
| 225 | 10% | 20% | 30% | 30% | 30% | 30% |
| 325 | 15% | 25% | 35% | 40% | 40% | 45% |
| 500 | 25% | 35% | 45% | 50% | 60% | 65% |
| 600 | 35% | 40% | 50% | 55% | 65% | 80% |
| 1000 | 40% | 40% | 55% | 60% | 70% | 85% |

**300 $m^2$ Roof Area**

| Water use in litres per day | Average Yearly % of Water Captured from Roof | | | | | |
|---|---|---|---|---|---|---|
| | Rain Tank Capacity (Litres) | | | | | |
| | 200 | 1000 | 3000 | 4500 | 9000 | 25000 |
| 125 | 10% | 10% | 15% | 15% | 15% | 15% |
| 225 | 10% | 20% | 25% | 25% | 25% | 25% |
| 325 | 15% | 20% | 30% | 35% | 35% | 35% |
| 500 | 20% | 30% | 40% | 45% | 50% | 55% |
| 600 | 25% | 30% | 45% | 50% | 55% | 65% |
| 1000 | 30% | 35% | 50% | 55% | 55% | 70% |

**500 $m^2$ Roof Area**

| Water use in litres per day | Average Yearly % of Water Captured from Roof | | | | | |
|---|---|---|---|---|---|---|
| | Rain Tank Capacity (Litres) | | | | | |
| | 200 | 1000 | 3000 | 4500 | 9000 | 25000 |
| 125 | 5% | 5% | 5% | 5% | 5% | 5% |
| 225 | 5% | 10% | 10% | 10% | 15% | 15% |
| 325 | 10% | 15% | 15% | 20% | 20% | 20% |
| 500 | 10% | 15% | 15% | 20% | 25% | 30% |
| 600 | 15% | 20% | 25% | 30% | 35% | 40% |
| 1000 | 20% | 25% | 35% | 40% | 50% | 60% |

#### 4.6.6.2 The anticipated water use

Table C11 provides information on anticipated water use for residential properties. The values can be extrapolated for more or less members but an average assumption of three members is reasonable given the potential of people relocating. It is recommended that water use for a residence be 500 litres/day unless there is evidence that the actual number will be more or less and that number is expected to remain standard for at least 10 years. Otherwise use 500 litres/day as the average water use.

**Table C11**
**Estimated Residential Demand Based on 500 l/d for a 3 member household**

| Water use | Average litres/day |
|---|---|
| Bathroom | 125 |
| Toilet | 125 |
| Laundry | 100 |
| Gardening | 100 |
| Kitchen | 50 |
| **Total** | **500** |

The Rodney District Council has minimum water tank sizes for household water supply where the tank is the only water supply to the home. Table C12 provides the tank size needs for variable roof areas and number of bedrooms.

**Table C12**
**Minimum Tank Size for RDC Homes Having Tanks as Sole Water Source**

| Usable Roof Area ($m^2$) | Bedrooms | | | | |
|---|---|---|---|---|---|
| | **1** | **2** | **3** | **4** | **5** |
| **100** | 20 | 50 | | | |
| **120** | 15 | 35 | 75 | | |
| **140** | 10 | 30 | 60 | | |
| **160** | | 20 | 50 | | |
| **180** | | | 45 | 75 | |
| **200** | | | 35 | 65 | |
| **220** | | | 30 | 55 | 90 |
| **240** | | | 30 | 50 | 80 |
| **260** | | | | 45 | 70 |
| **280** | | | | 40 | 65 |
| **300** | | | | 35 | 60 |

Colours indicate units of 25 cubic metres (5,000 gallons):

| | | | |
|---|---|---|---|
| 1 x 25 $m^3$ | 2 x 25 $m^3$ | 3 x 25 $m^3$ | 4 x 25 $m^3$ |

The same assumption cannot be made for rural commercial or industrial land use. In this situation, assumptions need to be made regarding the number of people that will occupy the workplace. Table C13 provides information on occupancy ratios.

**Table C13**
**Building Occupancy Ratios for Different Activities (NSCC, 2008)**

| Activity | Floor to Person Ratio |
|---|---|
| Office | 25 $m^2$ |
| Showroom | 35 $m^2$ |

| | |
|---|---|
| Warehouse | 50 $m^2$ |
| Shops, retail | 35 $m^2$ |
| Restaurant/dining areas | 15 $m^2$ |
| Local shopping centres | 35 $m^2$ |
| Manufacturing | 25 $m^2$ |

The number of individuals occupying the building will be the gross floor area divided by the floor to person ratio.

The amount of water used per day is the number of individuals times 25 litres/day. At a minimum the value should total 125 litres/day.

Industrial sites will have to be considered on an individual basis as the industrial usage may require water use in its operation. The total expected amount of use will then be based on employee and operations usage.

#### 4.6.6.3 The percent of water from the roof that can be used

Knowing the roof area and the anticipated need, the next step is to determine whether the water supplied can meet daily needs. Table C14 provides information on water demand, roof area and the percentage of time that roof runoff can meet daily demand.

In a similar fashion to the percentage of runoff that can be captured, Table C14 shows how important tank size and roof area are to determining whether daily water demand can be met.

What can be seen is that a daily water need of 500 l/d cannot be supplied 100% of the time even with a 25,000 litre tank and a 500 $m^2$ roof area. This is one reason why the toolbox recommends 50,000 litres of tank storage for rural residential properties.

**Table C14**
**Percentage of Demand that can be Met Relating Tank Size, Roof Area and Daily Water Needs.**

**150 $m^2$ Roof Area**

| Water use in litres per day | Average Yearly % of Water Supplied — Rain Tank Capacity (Litres) | | | | | |
|---|---|---|---|---|---|---|
| | 200 | 1000 | 3000 | 4500 | 9000 | 25000 |
| 125 | 50% | 80% | 95% | 100% | 100% | 100% |
| 225 | 40% | 65% | 85% | 90% | 100% | 100% |
| 325 | 35% | 50% | 70% | 80% | 90% | 100% |
| 500 | 25% | 40% | 55% | 60% | 70% | 75% |
| 600 | 25% | 35% | 50% | 50% | 60% | 60% |
| 1000 | 20% | 30% | 35% | 35% | 50% | 55% |

**200 $m^2$ Roof Area**

| Water use in litres per day | Average Yearly % of Water Supplied — Rain Tank Capacity (Litres) | | | | | |
|---|---|---|---|---|---|---|
| | 200 | 1000 | 3000 | 4500 | 9000 | 25000 |
| 125 | 55% | 85% | 95% | 100% | 100% | 100% |
| 225 | 40% | 65% | 85% | 95% | 100% | 100% |
| 325 | 35% | 55% | 75% | 85% | 95% | 100% |
| 500 | 25% | 45% | 60% | 65% | 80% | 90% |
| 600 | 25% | 40% | 50% | 60% | 70% | 80% |
| 1000 | 20% | 30% | 40% | 40% | 50% | 60% |

**250 $m^2$ Roof Area**

| Water use in litres per day | Average Yearly % of Water Supplied — Rain Tank Capacity (Litres) | | | | | |
|---|---|---|---|---|---|---|
| | 200 | 1000 | 3000 | 4500 | 9000 | 25000 |
| 125 | 55% | 85% | 100% | 100% | 100% | 100% |
| 225 | 40% | 65% | 90% | 95% | 100% | 100% |
| 325 | 35% | 60% | 80% | 85% | 95% | 100% |
| 500 | 25% | 45% | 65% | 70% | 85% | 95% |
| 600 | 25% | 45% | 60% | 65% | 75% | 90% |
| 1000 | 20% | 35% | 45% | 50% | 60% | 70% |

**300 $m^2$ Roof Area**

| Water use in litres per day | Average Yearly % of Water Supplied — Rain Tank Capacity (Litres) | | | | | |
|---|---|---|---|---|---|---|
| | 200 | 1000 | 3000 | 4500 | 9000 | 25000 |
| 125 | 55% | 85% | 100% | 100% | 100% | 100% |
| 225 | 40% | 70% | 90% | 95% | 100% | 100% |
| 325 | 35% | 60% | 80% | 90% | 95% | 100% |
| 500 | 25% | 45% | 70% | 75% | 85% | 95% |
| 600 | 25% | 45% | 60% | 70% | 80% | 95% |
| 1000 | 20% | 35% | 55% | 60% | 65% | 70% |

**500 $m^2$ Roof Area**

| Water use in litres per day | Average Yearly % of Water Supplied — Rain Tank Capacity (Litres) | | | | | |
|---|---|---|---|---|---|---|
| | 200 | 1000 | 3000 | 4500 | 9000 | 25000 |
| 125 | 55% | 85% | 100% | 100% | 100% | 100% |
| 225 | 45% | 85% | 90% | 95% | 100% | 100% |
| 325 | 40% | 75% | 85% | 90% | 95% | 100% |
| 500 | 30% | 55% | 75% | 80% | 90% | 95% |
| 600 | 30% | 45% | 65% | 75% | 85% | 95% |
| 1000 | 25% | 35% | 55% | 60% | 75% | 85% |

#### 4.6.6.4 Peak flow consideration

When sizing a water tank, there are two possible storage components.

- The water needs component, and
- For commercial and industrial land use, an attenuation volume that reduces peak rate of discharge.

The attenuation volume occupies the upper storage area of the tank with its outlet orifice placed immediately above the water needs volume.

It is possible that the combined storage would provide more benefit than is estimated. A higher level of attenuation may be achieved in some instances when the tank water level is lower than the orifice level at the start of the storm. These benefits are very difficult to estimate and are not taken into account in design.

There will be a portion of the year when roof runoff will exceed water use and runoff during that time needs to be considered in terms of attenuation.

The volume can be determined by use of Table C13 in conjunction with knowledge of the roof area and intended water use. The amount of water that needs to be addressed through this approach is determined through the following equation.

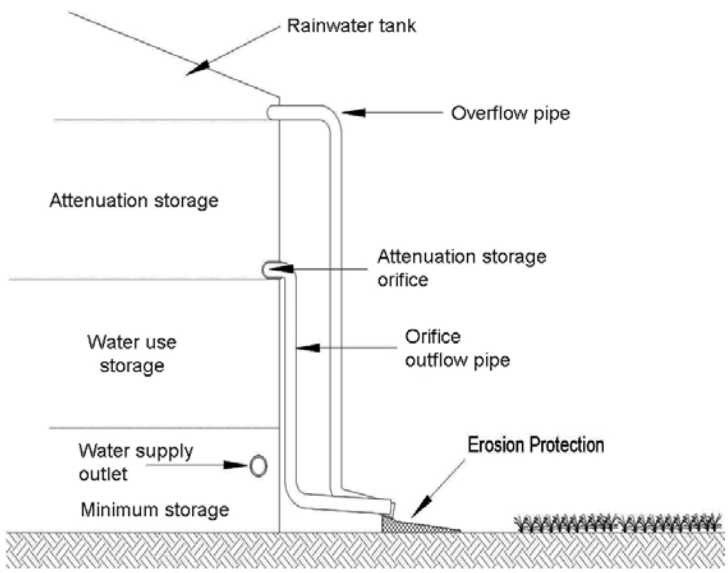
> % of rainfall becoming runoff = 1 – (0.75 x fraction of rainfall used)
>
> 0.75 is the percentage of the time that the water tank is full due to excess rainfall.
>
> > As an example, 500 l/d for a roof of 225 $m^2$ and a 25,000 litre tank would equate to the following:
> >
> > % of rainfall becoming runoff = 1 – (0.75 x 0.95) = 29% of the rainfall becoming runoff.

TP 108 analysis of a one hectare pasture for a 2-year storm indicates that approximately 45% of the rainfall becomes runoff while 58% becomes runoff during a 10-year storm. This would indicate that peak control of stormwater flow rates is not needed in the situation where a residence uses 500 l/d until the % of rainfall that becomes runoff is greater than 45%.

Commercial and industrial sites will have more concern over the percentage of rainfall that becomes runoff than residential development. As detailed above, the percentage of time that rainfall becomes runoff needs to be calculated using daily water use, roof area and tank size. Roof areas above 500 $m^2$ need to be considered individually and a water budget established.

In those situations, attenuation of runoff may be required due to a possible larger expanse of roof area in conjunction with smaller water use. A dual use water tank for attenuation and water use is shown schematically in Figure C14.

**Figure C14**
**Combination Attenuation and Water Use Tank**




As detailed, the water tank has three outlets:

- Water supply outlet for site water use,
- An outlet for the attenuation storage, and
- An overflow pipe for those flows that exceed the tank storage.

The water supply outlet is a standard hose connection to a pump or outlet depending on gravity feed to the water use. The outlet from the attenuation storage provides a controlled release for larger storms to reduce downstream stormwater flow increases and the overflow pipe is for all storms to flow when the tank is full of water.

#### 4.6.6.5 Determining detention volumes and sizing outlets

The volume of storage needed for detention purposes can be addressed with one storage volume, as the volume needed for the 2- and 10-year storms is very similar. The only difference would relate to the size of the outlet orifice. For the purposes of this toolbox it is recommended that storage and release be based on the 2-year storm, as that would retard both flows. Table C15 provides information on storage requirements versus roof areas. If the designed roof area is not listed extrapolate between those that are given to find the appropriate volume.

**Table C15**
**Roof area versus Attenuation Storage for Commercial/Industrial Water Tanks**

| Roof area ($m^2$) | Attenuation storage ($m^3$) |
|---|---|
| 150 | 2.3 |
| 250 | 3.3 |
| 350 | 4.2 |
| 500 | 5.9 |

Once the attenuation storage has been calculated, the attenuation orifice outlet must be sized. Table C16 provides orifice sizes for various roof areas and two different tank diameters.

| Table C16<br>Attenuation Orifice Sizes (mm) for Two Different Tank Diameters | | |
|---|---|---|
| Roof Area ($m^2$) | 2.2 m Diameter Tank | 3.4 m Diameter Tank |
| 150 | 23 | 32 |
| 250 | 27 | 34 |
| 350 | 29 | 37 |
| 500 | 33 | 42 |

The tank elevations can be calculated once the attenuation storage and orifice size have been determined per the following.

1. Select a tank size based on site water needs and needed attenuation storage.
2. Set the water supply outlet at least 200 mm above the tank bottom to allow for debris settlement.
3. Total volumes needed for attenuation and site use are added together. These volumes then must be added to the minimum storage level (volume of tank/height of tank x 200 mm) to ensure that the tank is large enough to accommodate the three storages.
4. Determine the elevations of the various storages. Minimum storage level = 200 mm. Site water use = height of tank/volume of tank x site water use volume = height of water use elevation. This must be added to 200 mm to get elevation in tank of attenuation orifice invert.
5. Calculate invert height of overflow pipe. Overflow invert height = height of tank/volume of tank x attenuation storage volume = height of overflow pipe invert elevation. This must be added to the site water use orifice invert elevation to get the correct overflow elevation.

## 4.6.7 Case studies

Two case studies are provided relating to a rural residential household and for a small commercial building, such as a dairy.

### 4.6.7.1 Case Study 1

A water tank has to be sized for a home. The architects design plans show that the home has a roof area of 260 $m^2$ and it is being designed for a daily water use of 500 litres/day. The water tank is 25,000 litres as the minimum acceptable size.

#### *4.6.7.1.1 Design steps*

1. With the roof area being 260 $m^2$ and a water use of 500 l/d, calculate the percent of stormwater capture from Table C10. As 260 $m^2$ is between the table's two values (250 $m^2$ and 300 $m^2$) the value is interpolated as 63% or that 37% is discharged.
2. If the percentage of roof runoff that can be discharged by the tank is less than 45%, peak flow control does not need to be done.
3. As the water discharged is less than 45% on an annual basis, peak control is not needed for the residence.
4. Final result is a 25,000 litre water tank with an 80 mm overflow pipe at the top of the tank.

### 4.6.7.2 Case study 2

A dairy is constructed in a rural area. The cross sectional area of the roof is 200 $m^2$ and the gross floor area is 165 $m^2$.

#### *4.6.7.2.1 Design steps*

1. Based on the gross floor area calculate the number of individuals who will be working in the dairy. Use a local shopping centre figure of 35 $m^2$/person to calculate the number of individuals. The result is 4.7 individuals so say 5 people working at any one time.
2. Calculate the water used by 5 individuals at 25 l/individual or 125 l/d of water being used.
3. Select a storage amount that will supply needs. At 125 l/d 3000 litres would last 24 days if there is no additional rain during that period. The selection of volume should be based on a reasonable assumption of storage needs during the summer months when several weeks can go by without rainfall.
4. From Table C14, the amount of detention storage needed is 2.8 $m^3$ or 2,800 litres.
5. From Table C15 the peak flow control orifice is 33 mm.
6. Calculate the minimum storage volume needed. If the tank is a 10,000 litre tank its dimensions will be 2.2 m diameter and 3.2 metres tall. To check the minimum storage volume amounts, divide the volume of the tank by its height and multiply by 200 mm. 10,000/3200 x 200 = 625 litres.
7. The tank volume needs to be 3000 litres + 2,800 litres + 625 litres = 6,425 litres so the 10,000 litre tank has more than enough capacity.
8. Determine the elevations of the various storages. Minimum storage level = 200 mm. Site water use = 3200/10,000 = 0.32 mm/l x 3000 l = height of water use elevation or 960 mm. This must be added to 200 mm to get elevation in tank of attenuation orifice invert = 1,160 mm from the bottom of the tank.

9. Calculate invert height of overflow pipe. Overflow invert height = height of tank/volume of tank (0.32) x attenuation storage volume (2800) = height of overflow pipe invert elevation or 896 mm. This must be added to the site water use orifice invert elevation to get the correct overflow elevation = 2,056 mm from the tank invert as a minimum elevation. As the tank is 3.2 metres tall, additional storage can be provided to the site water use to increase total storage for additional safety of supply.

Figure C14 shows a tank cross-section with elevations provided. Figure C15 shows a detail of the attenuation orifice and the exterior overflow pipe.

**Figure C14**
**Water Tank Schematic Showing Case Study Elevations and Volumes**

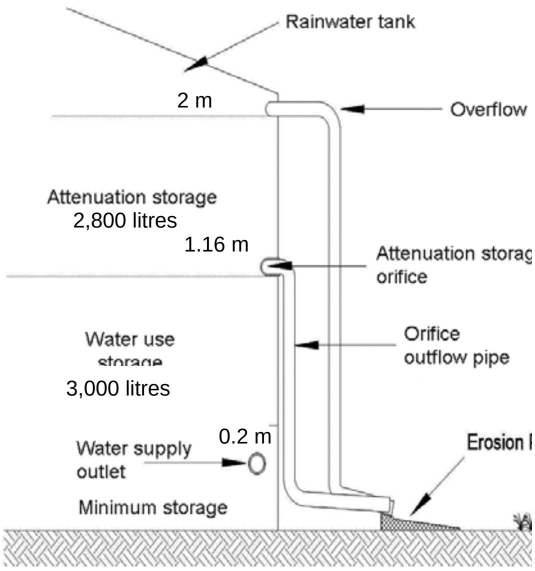
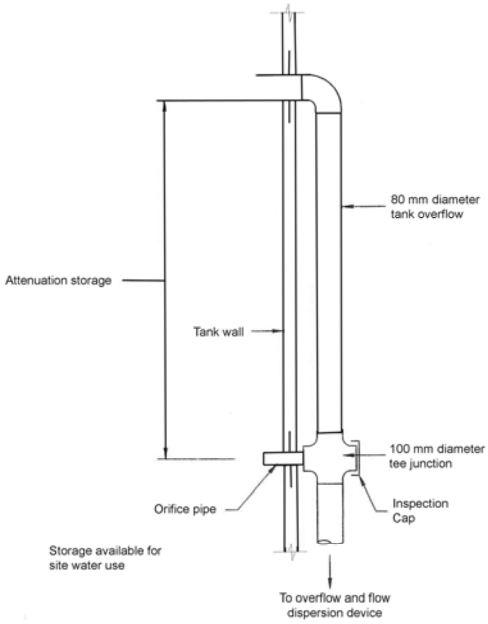



**Figure C15**
**Orifice and Exterior Pipe Details**




Notes

1. For orifice pipe diameter and attenuation storage head use Tables C16 and C17
2. Maximum orifice pipe length is 150 mm. Allow 75 mm clearance from end of pipe to outside of tank wall
3. Fix orifice pipe to 100 mm diameter tee junction using reducer fittings

## 4.7 Bush Revegetation

### 4.7.1 Description of practice

Relating to land use, stormwater runoff is greatest from impervious surfaces. Less runoff is generated from pasturelands. Native bush that is protected from grazing and having litter and brush covering the ground generates the least amount of stormwater runoff.

When land is being converted to rural residential, commercial or industrial land use the total volume and peak rate of stormwater runoff are increased. As pastureland has a greater volume of stormwater runoff than does bush, conversion of existing pastureland into bush can reduce future runoff and mitigate for the effects of increased impervious surface generation.

### 4.7.2 Design considerations

The approach is based on extent of area that is set aside for re-establishment of bush. Key considerations related to re-establishment are the following:

- Existing areas of bush that can be extended,
- Natural site features,
- Slope, and
- Location of waterways.

Providing additional bush to existing bush areas would increase the value of existing bush by increasing bush interior areas. This would reduce fringe vegetation that could become a weed maintenance problem.

When sizing bush restoration for various lot sizes, the level of imperviousness will be very important. As lot size reduces from 1 hectare to 2,000 $m^2$, the proportion of the site that is impervious will increase the required bush area. Under an assumption of a 1 ha lot having 600 $m^2$ of imperviousness; it will take 3500 $m^2$ of bush to compensate for that impervious surface. If a lot is 0.5 ha and the imperviousness of the lot remains at 600 $m^2$ the amount of bush to compensate for the impervious surface is still 3,500 $m^2$ but that will represent approximately 70% of the site area rather than 35%.

If the site area goes below 0.5 ha bush cannot compensate for 600 $m^2$ of imperviousness.

### 4.7.3 Targeted contaminants

While native bush vegetation having a good ground cover can provide contaminant reduction benefits, the main purpose in this toolbox is the reduction of stormwater runoff volumes. Organic matter on the bush floor will remove metals and assist in removal of sediments but residential land use in rural areas does not generate large contaminant loads. Commercial and industrial land use may increase contaminant loads but other practices provided in the toolbox would provide greater levels of treatment.

### 4.7.4 Advantages

Native bush grows over time and maintenance concerns diminish. Where other stormwater management practices need maintenance to ensure long-term performance, bush revegetation improves its hydrological function over time and maintenance needs become minimal.

Native bush also provides benefits for wildlife habitat, shading and cooling during summer. It can act as a windbreak and can be an aesthetic amenity.

### 4.7.5 Limitations

Native bush planting can have fairly high maintenance needs during the first 2-3 years of growth relating to weed control and possible watering needs during drought conditions.

Native bush can also be seen as limiting site usage. If some livestock were a desired activity on the site, they must be excluded from access to the bush areas to ensure that bush growth is not adversely affected.

When planted in widths of less than 20 metres, weeding can remain a problem for years.

### 4.7.6 Design sizing

Bush re-establishment is based on the following Table C17.

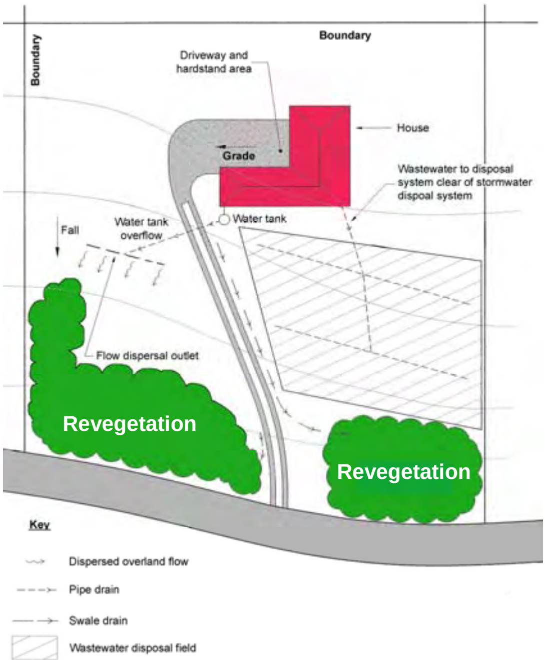
**Table C17**
**Bush Planting Requirements**

| Proposed site impervious area ($m^2$) | Area of bush required ($m^2$) |
|---|---|
| 100 | 1,000 |
| 200 | 1,500 |
| 300 | 2,000 |
| 400 | 2,500 |
| 500 | 3,000 |
| 600 | 3,500 |

The calculations, using a variation of TP 108 that calculates storm and base flow under various landuse scenarios (Beca Carter Hollings & Ferner, 2000), work out to be very even throughout the Region. For every 100 $m^2$ of imperviousness beyond the first 100 $m^2$ of imperviousness there is a 500 $m^2$ requirement for bush establishment.

Recognising the significant areal extent of bush replacement, it may be best to isolate various impervious surfaces and address them separately. That would allow for several practices to provide site management without using too much of a given portion of the site to any one practice.

**Figure C16**
**Bush Revegetation for Runoff Control**




### 4.7.7 Case study

A house on 1 hectare is being constructed and the footprint for the house and driveway is 550 $m^2$ of imperviousness. The site, as shown in Figure C16, has a house, driveway, septic system and needs 3,250 $m^2$ to compensate for impervious surfaces.

Since the roof of the house has a water tank that was designed as in Case Study 1 (Section 4.6.7.1.1) then the 260 $m^2$ can be excluded from the bush revegetation approach. In that case, the impervious surface is now 290 $m^2$ so the bush replacement area is now 1,950 $m^2$, which is a significantly reduced area.

Using practices in conjunction with one another can significantly reduce the size of a practice if it is used to address all of the areas.

## 4.8 Green roofs

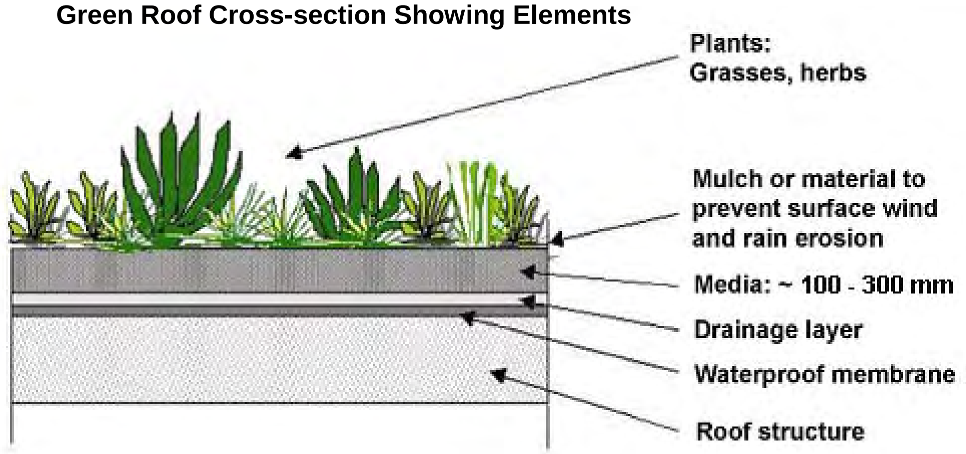
### 4.8.1 Description of practice

Green roofs are roof systems that incorporate a growing media and plants to provide a semi-permeable surface on roofs that would normally consist of impervious surfaces. A green roof more mimics a natural environment to filter precipitation through the media and allowing for the wetted media to evapotranspire between storm events. A green roof may eliminate runoff during small rainfall events and will retard the onset of stormwater runoff and increase the time of concentration from a conventional roof, thus reducing downstream stormwater effects.

### 4.8.2 Design considerations

Typically, as shown in Figure C17, a green roof consists of the following:

**Figure C17**
**Green Roof Cross-section Showing Elements**




- A waterproof membrane to prevent water from leaking into the structure,
- A drainage layer to allow lateral movement of water to the down spout,
- Filter media for passage of stormwater and a growth media for plants,
- Mulch or other material to prevent surface wind and rain erosion, and
- Plants.

Green roofs are engineered systems, which address all of the critical aspects of design, including the following:

- The saturated weight of the system and load bearing capacity of the underlying roof,
- Moisture and root penetration resistance through use of a waterproof membrane,
- Resistance to wind sheer, management of drainage, and
- The suitability of the proposed plant material.

There are generally considered to be two types of green roofs.

- Extensive green roofs, which are shallow systems having less than 100 mm of media, which are not being advocated by this toolbox, and
- Intensive green roofs, which are deeper systems having more than 150 mm of media.

### 4.8.3 Targeted contaminants

From a water quality perspective, green roofs would be effective in retention of fine, wind blown sediments and dissolved metals.

### 4.8.4 Advantages

Overseas data indicates that green roofs can be very effective at reducing the total volume of stormwater runoff. A study in North Carolina (Moran, Hunt and Smith, 2005) indicated that a green roof retained 45% of total annual runoff.

Green roofs can be used on a variety of roof types and on any property size, as their installation will not require the use of additional land. In Auckland's temperate climate, green roofs should not be limited by the ability to establish and maintain vegetative cover.

Another key advantage of green roofs is that they are aesthetically pleasing. They can be very attractive. There are also benefits related to urban cooling during the summer months and insulation benefits for air conditioning and heating.

### 4.8.5 Limitations

The first consideration that might limit the use of green roofs for rural development is that most rural development will need the runoff that comes from their roof for site water use. A significant reduction in annual runoff may not be beneficial for water needs.

There are several other issues that may be considered as limitations.

- Green roofs, as recommended in this toolbox, will necessitate increased structural strength of the roof that would increase costs.
- Maintenance needs, while expected to be minimal, may be costly and difficult depending on height above ground.
- Establishment of plants and their overall survival may require watering during dry periods, at least for the first several years.
- Weed removal may be a requirement depending on individual conditions.

### 4.8.6 Design sizing

There are several key elements of design that need to be addressed.

- Depth of media,
- Composition of media,
- Plant selection,
- Additional support consideration,
- Roof slope,
- Drainage layer and impermeable liner, and
- Stormwater management benefits

#### 4.8.6.1 Depth of media

There are two green roofs in the Auckland Region that are being studied: the University of Auckland Engineering Building green roof, and the Waitakere City Council Headquarters building green roof.

While these are both fairly new installations, some guidance can be given on plant propagation that relates to the depth of media. The University of Auckland site has media between 50 mm - 70 mm in depth. Over the past summer, plants were severely stressed due to the lack of moisture in the shallow subgrade. The Waitakere City green roof fared much better due to its depth being 70 - 150 mm.

Deeper media depths are better than shallower ones.

It is recommended that there be at least 150 mm of media to promote a sustainable plant community.

#### 4.8.6.2 Composition of media

The University of Auckland site investigated a number of different media and has found that the mixture of the following provides the best results and that mixture is recommended for use.

- 30% zeolite,
- 50% pumice, and
- 20% composted bark.

#### 4.8.6.3 Plant selection

New Zealand does not have any native succulents, which is the plant of choice internationally due to their ability to thrive in both wet and dry conditions. There are New Zealand plants that are suitable for green roofs, especially with the recommended depth of media being at least 150 mm.

Recommended plants include the following:

- Disphymae australe (NZ ice plant)
- Pimelea prostrata (NZ daphne)
- Libertia peregrinans (NZ Iris)
- Festuca coxii (native tussock)
- Comprosma Hawera
- Acaena microphylla (NZ bidibid)
- Lepostigma setulosa

Other plants will be acceptable, but a plant specialist should be consulted prior to use due to the shallow media depths and the extremes of wetting and drying that will be encountered.

#### 4.8.6.4 Additional support consideration

The additional load of materials comprising the various components and an assumption of having saturated media conditions needs to be considered when accommodating the roofs structural load. The calculation has to be based on an assumption of a saturated state.

A chartered Professional Engineer must be consulted in the design and construction of a green roof system.

#### 4.8.6.5 Roof slope

Generally, the construction effort and cost of green roofing increases with slope. Minimal slopes slow down water flow and slopes above $5^o$ will have more rapid runoff. Due to native plants not providing the density of vegetation that would bind the media, it is recommended that green roof slopes not exceed $5^o$ unless steps are taken to prevent media slippage and erosion.

#### 4.8.6.6 Drainage layer and impermeable liner

The drainage layer should be a Delta NP drainage layer, or equivalent, with a nonwoven geotextile, which is a two-layer drainage and waterproofing system with the cloth facing the media.

The impermeable liner should be Permathene flexible polypropylene geomembrane (250 um), or equivalent.

Both of these products can be substituted for if the substitution meets the same standards as the two presented.

#### 4.8.6.7 Stormwater management benefits

Green roofs provide an excellent media for water quality treatment of any airborne contaminants and thus meet water quality treatment guidelines.

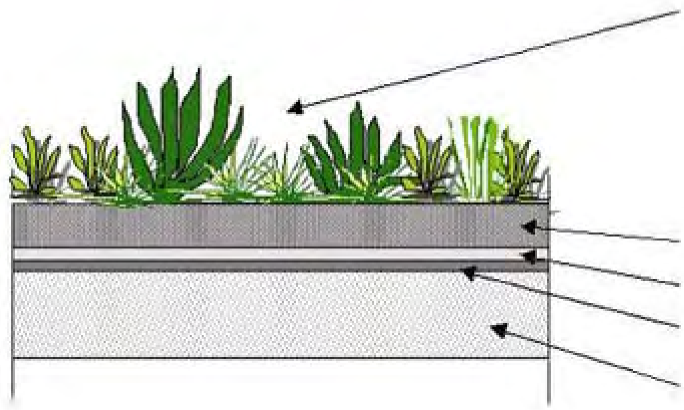
The media recommended includes zeolite, which is a hydrated aluminosilicate mineral having a micro-porous structure. Pumice also has a very high porosity and being highly porous is very lightweight. Design can assume a 50% void ratio for the compost bark, zeolite and pumice.

Stormwater quantity control is not required for green roofs.

### 4.8.7 Case study

This is a typical green roof design is shown in Figure C18.

**Figure C18**
**Case Study Parameters for a Green Roof**



## 4.9 Access roads and driveways

### 4.9.1 Introduction

Development usually results in the need for more roads and an associated increase in road runoff.

In designing and locating new roads, developers will be expected to take account of the general objectives set by the local territorial authorities to minimise stormwater flow increases associated with development and provide ongoing stormwater management.

General guidelines for new roads include:

- Roads should be located, where practicable, away from watercourses and existing areas of established bush,
- Roads should be aligned to minimise cuts and fills,
- Road and pavement widths should be kept to a minimum, and
- Houses should be sited to minimise the length of access road needed and the opportunity for shared access should be explored.

Runoff from roads increases the volumes and rates of stormwater runoff above that of pervious areas and contains contaminants that require treatment.

This section of the toolbox will focus on driveway design although management of new access roads will also require stormwater management approval; either from the ARC through a resource consent or through building consent approval by the territorial authority.

### 4.9.2 Driveways

The driveway design is the responsibility of the property owner.

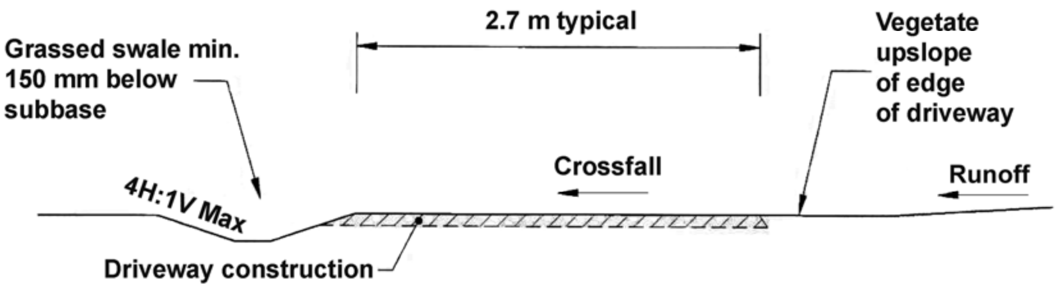
The general principals identified above for access roads also apply to driveways. Options to consider for driveways include:

- The use of dual strip driveways with a grassed central strip, and
- Shared driveways serving several sites.

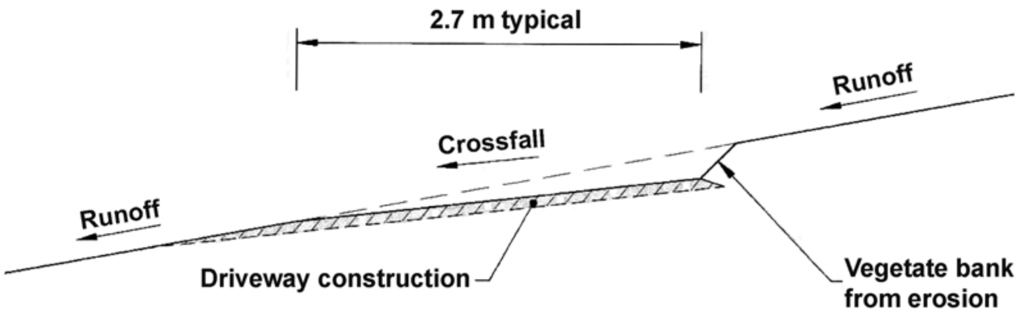
Figure C19 shows typical cross-sections of a driveway. It is important that the driveway design takes into account the ground conditions under the driveway.

Figure C20 shows the detail for a strip driveway.

**Figure C19**
**Driveway Cross-Sections**




Typical section crossing flat ground




Typical section crossing steep ground

**Notes:**

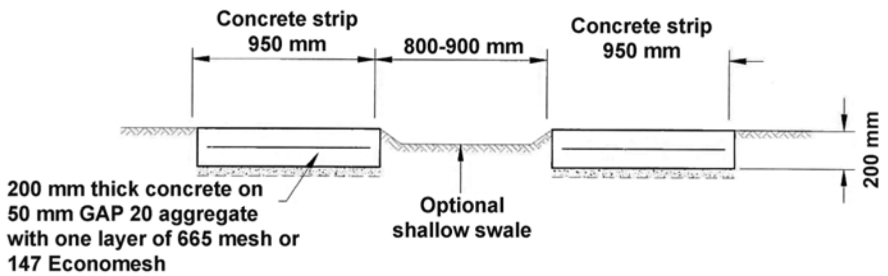
1. Depth of driveway construction based on subgrade CBR of 7. If actual on site is less, either basecourse to be thickended, geotextile to be used or subgrade strengthened.

2. Reduced impervious area driveways are preferred as they produce less runoff. Typical examples are dual strip concrete or precast permeable turf blocks. All precast turf blocks used shall be in accordance with manufacturers specification for bedding material and basecourse that take into account the slope, subgrade and drainage requirements.

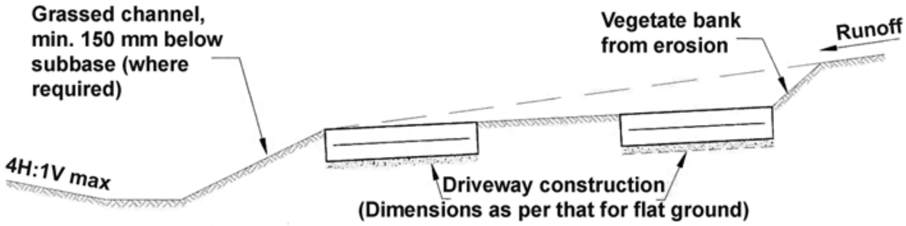
3. Where driveway grades are steeper than 1 in 8, kerb and channel or fully lined channels are required for stormwater control.

4. All channels should discharge into swale and dispersal devices.

**Figure C20**
**Twin Strip Driveway Cross-Sections**




Twin strip driveway section crossing flat ground




Twin strip driveway section crossing steep ground

| Maximum spacing of free joints (m) | | | | | | |
|---|---|---|---|---|---|---|
| Steel | Steel mm$^2$ | Slab thickness | | | | |
| | Area | 100 | 125 | 150 | 175 | 200 |
| 665 | 145 | 26 | 21 | 17 | 15 | 13 |
| 664 | 186 | 33 | 27 | 22 | 19 | 17 |
| 663 | 205 | 37 | 30 | 25 | 21 | 18 |
| 662 | 260 | | 33 | 31 | 27 | 23 |
| 661 | 290 | | | 35 | 30 | 26 |
| 661/0 | 330 | | | | 34 | 30 |
| HD12 at 250 mm | 452 | | | | 41 | 36 |
| HD12 at 225 mm | 503 | | | | | 40 |
| For mesh Fy = 485 MPa: for HD12 Fy = 430 MPa | | | | | | |

**Notes**

1. Concrete strength to be 20 MPa
2. Design is for non-commercial vehicles
3. Free joints shall be provided in the concrete strips at the spacings given in the table above. Free joints must be constructed in accordance with FJ3, FJ4, or FJ5.
4. Tied joints shall be provided at quarter distances between the free joints. Transverse saw cuts to a minimum depth of 50 mm shall be sufficient for the purpose.
5. Stormwater control can be affected by sloping the concrete strips to the centre and providing a shallow swale between them.

### 4.9.3 Drainage across a driveway

There will be times when a driveway must cross a swale or open drain. The main issues of concern with the crossing are the following:

- Water overtopping the driveway,
- Backwater effects flooding upstream areas, or

- Erosion of the outfall.

All of these issues can be addressed through good design and that design will depend on the capacity of the swale.

In most instances, the swale will be designed for a 10-year storm and the swale design already considers the 10-year velocity to ensure that they do not exceed 1.5 m/s.

It is recommended for conveyance of a 10-year storm for a 1-hectare site that a pipe size for driveway entrances is at least 200 mm in diameter. In addition to the size, the pipe should be buried approximately 50 mm below grade to allow for sedimentation of the invert that will reduce flow velocities. Flows beyond a 10-year storm should be allowed to overtop the driveway to prevent pressure flow in the pipe.

Larger catchment areas draining via an open vegetated channel will require an individual design for pipe sizing.

This criterion only applies to countryside living areas where levels of imperviousness are small.

## 4.10 Dispersal devices

### 4.10.1 Introduction

When flows exceed the ability of water tanks to accept water, tanks have an overflow system through an 80 mm outfall pipe. In the same regard, rain garden underdrain pipes must also discharge in a manner that does not cause erosion. Swales and wetland swales also need to discharge in a non-erosive manner to minimise receiving system impacts.

These practices all need a dispersal device to distribute flows to avoid concentration and possible downstream scour.

### 4.10.2 Description

There are three alternative dispersal devices: a trench, an above ground dispersal device and a rigid boundary device. The intended function of these devices is to spread any discharges from the system over a sufficiently large area to avoid concentrations of flow. In this way it attempts to mimic the predevelopment conditions on site. The length of the dispersal devices may require confirmation based on site conditions during the consent or permit approval stage.

The use of an above ground dispersal device must consider the effects of UV rays and the approach must be submitted to the local council for review and approval.

### 4.10.3 Application

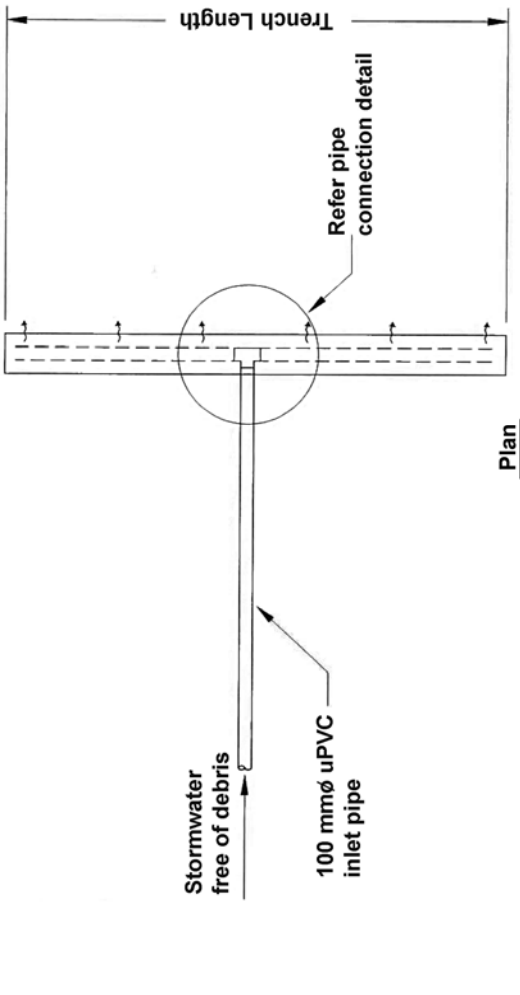
Dispersal devices are the preferred means to disperse concentrated flows from stormwater management practices on rural projects. Figures C21, C22 and C23 provide conceptual details for the design of dispersal devices.

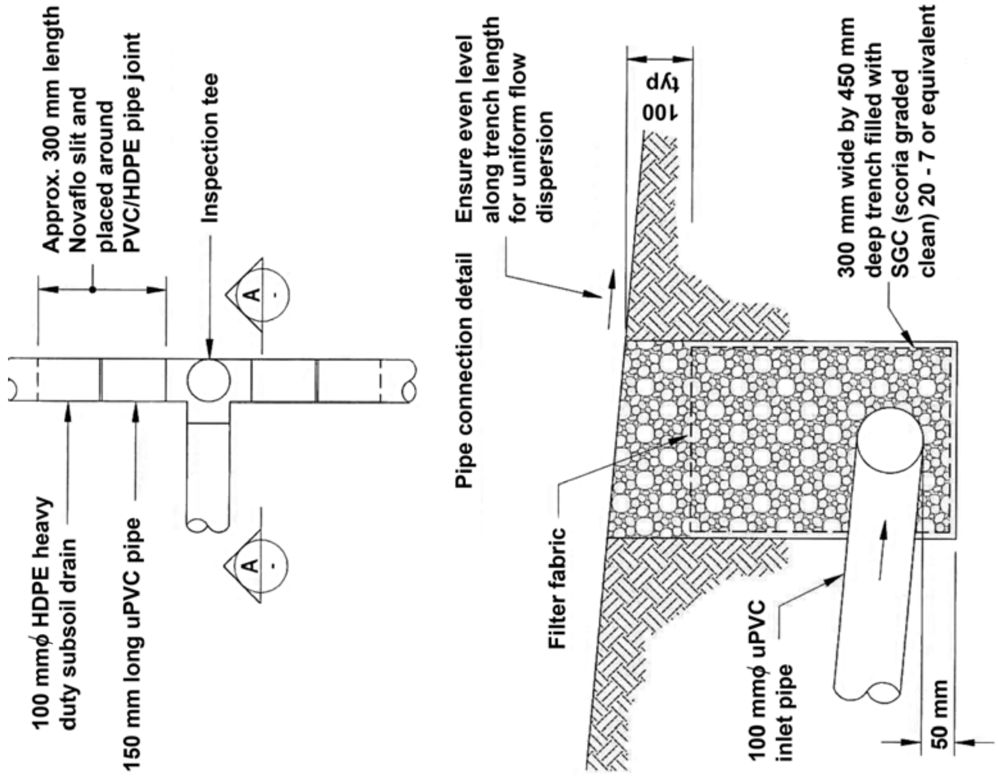
### 4.10.4 Design considerations

There are several key elements related to the placement of dispersal devices.

- They should be sited clear of any wastewater effluent disposal fields.
- They should not be located above or on a slope that has geotechnical stability issues. This is especially true of the dispersal trench that may have water ponding in it on a routine basis causing saturated subsoil conditions.
- The dispersal devices should be sited such that flows will not concentrate for a distance of at least 30 metres past the outlet.
- The dispersal device should be constructed of durable materials and should minimise to the degree possible future maintenance requirements.
- The dispersal device design should give due regard to the potential conflict that may occur with any proposed activity on site. In this regard, fencing or isolation from other activities may be required.

**Figure C21**
**Conceptual Layout of Flow Dispersal Trench**




Keep dispersal trench 30 m clear of property boundary

**Design Parameters**

| Effective catchment area drained ($m^2$) | Trench Length (m) |
|---|---|
| 100 | 8 |
| 200 | 12 |
| 300 | 14 |
| 400 | 16 |
| 500 | 18 |
| 600 | 20 |

(Note: effective catchment area drained = impervious area + 0.72 x pervious area

**Figure C22**
**Above Ground Flow Dispersal**

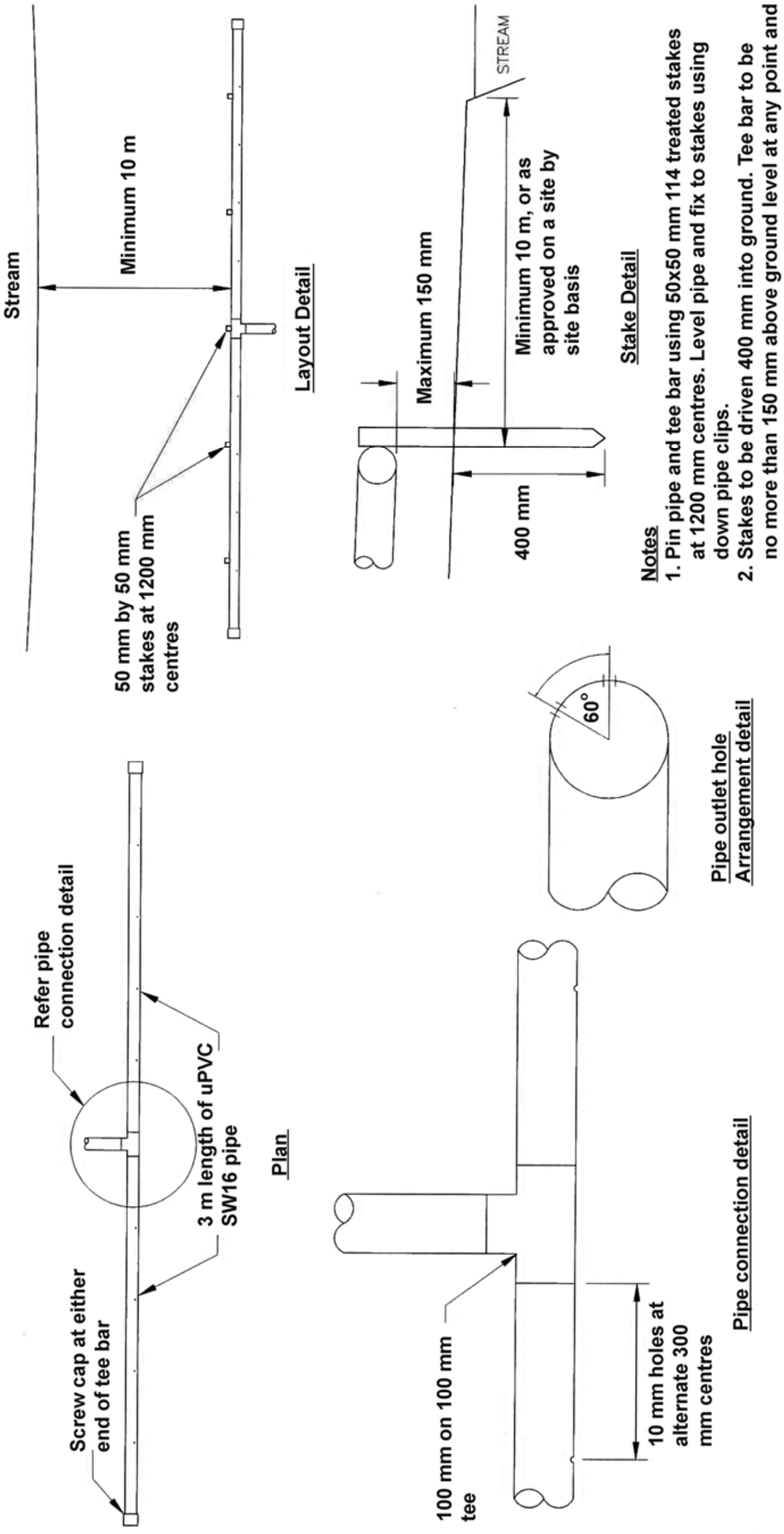
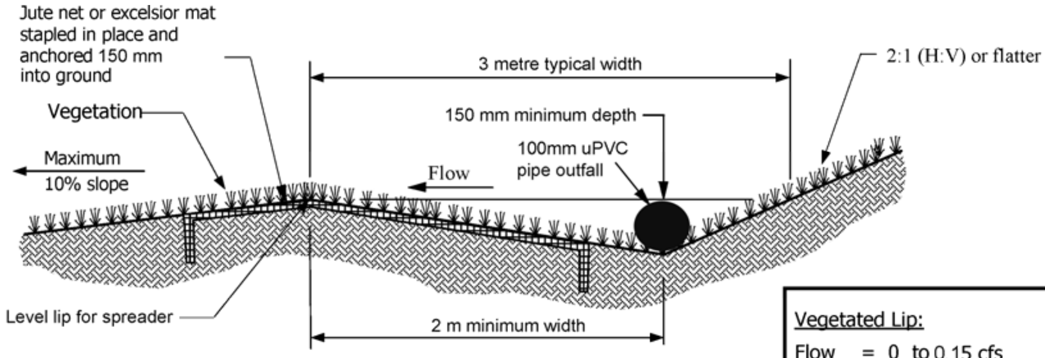


**Figure C23**
**Rigid Boundary Flow Dispersal**



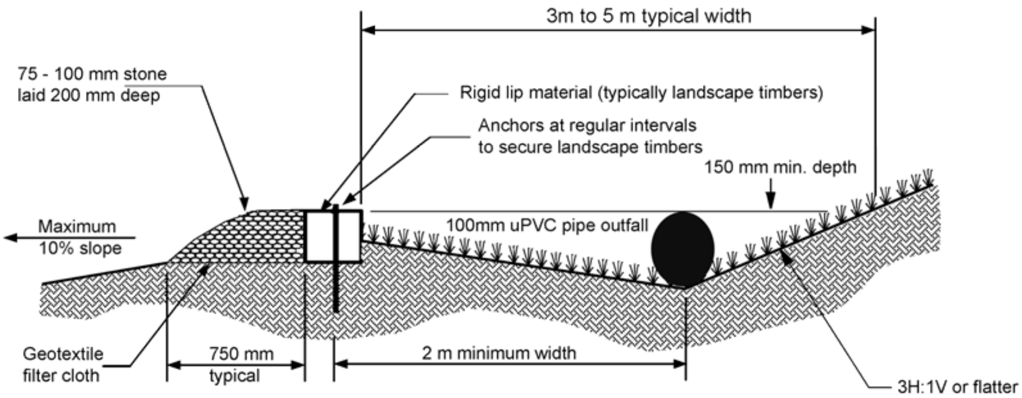

**Level Spreader — Vegetated Lip**

Vegetated Lip:

Flow = 0 to 0.15 cfs
Length = 3 metres
Width = 3 metres

Rigid Lip:

Flow = 0.15 to 0.4 m/s
Length = 3 to 10 metres
Width = 3m to 5 m




**Level Spreader — Rigid Lip**

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