APPENDIX A: STORMWATER MANAGEMENT AREA MAPS

Figure 1 Map of the Timaru Stormwater Management Area

Figure 2 Map of the Temuka Stormwater Management Area

Figure 3 Map of the Pleasant Point Stormwater Management Area

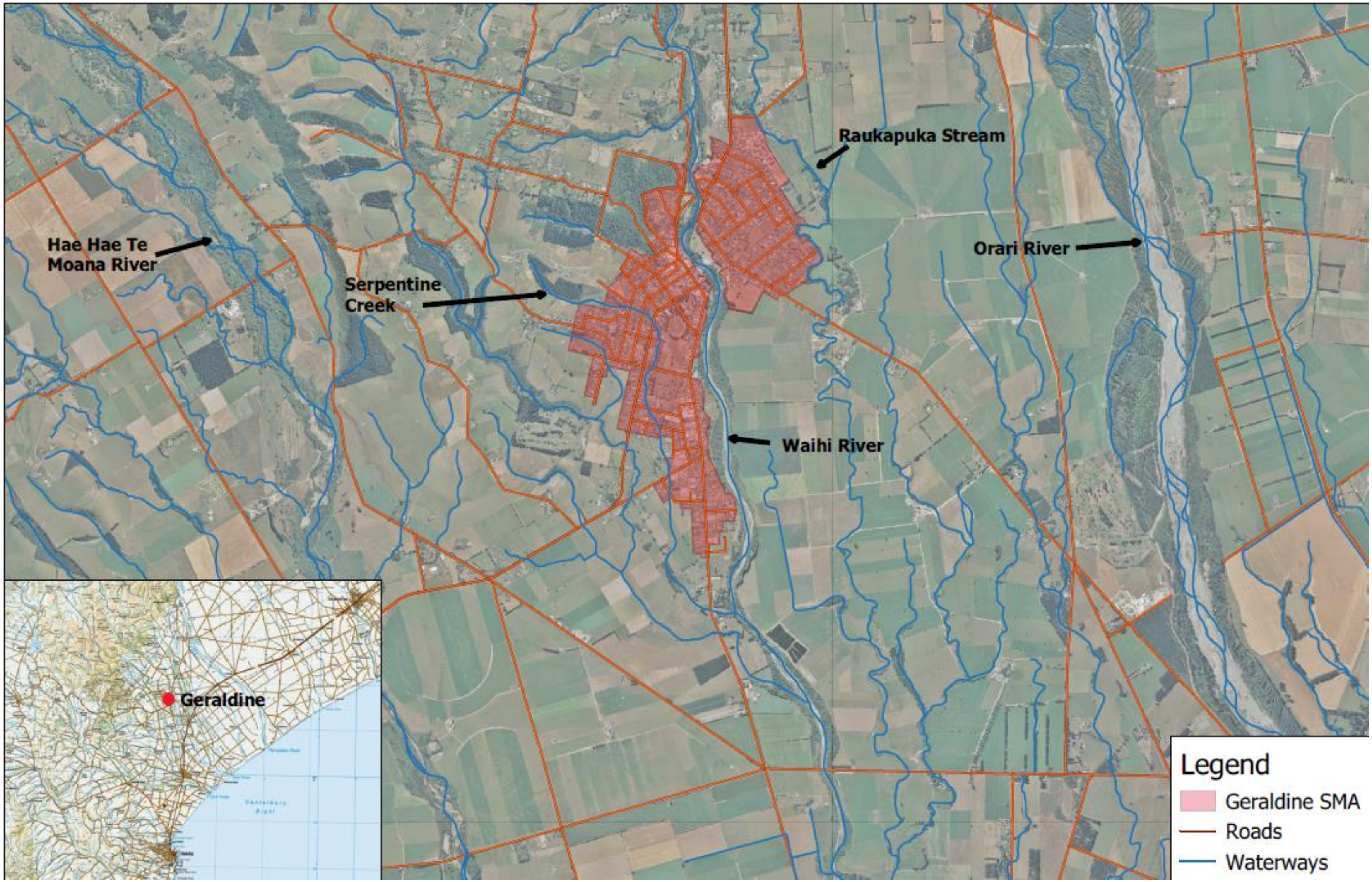

Figure 4 Map of the Geraldine Stormwater Management Area

APPENDIX B: STORMWATER MANAGEMENT DEVICES

Introduction
A number of devices can be used to achieve stormwater quality, neutrality, or a combination of both. Detailed design guidance is provided for common stormwater devices, including:

- Swales – vegetated or grassed
- Bioretention – raingardens, tree pits or planters
- Infiltration – basins, permeable pavements and soakage pits
- Wetlands
- Ponds – wet and dry basins

Design considerations are also provided for proprietary treatment devices.

Acceptable Solutions
A number of Acceptable Solutions have been prepared by TDC and are available on the website. These provide a means of achieving discharge certification, as per Figure 1-1. The use of the Acceptable Solutions are limited to application on 'Minor' developments only – refer to Tables 3-1 and 3-2.

Stormwater Devices
Stormwater management options are tabulated in Table 6-1 below, with commentary on the function, common limitations and comparative advantages of each device. Physical constraints of the site, such as contributing catchment size and soil conditions should also be considered when choosing a treatment or attenuation system.

Many devices can perform both quantity management and treatment functions. The ability to provide this function is identified with a ✓. However this is subject to site conditions and suitable design.

Table 1: Stormwater Device Options						
Device	**Function**	**Comparative Advantages**	**Design Limitations**	**Additional Values**	**Quantity**	**Treatment**
Swale	Trap sediment Some filtration Conveyance	Narrow design Cost-effective, simple construction	Suitable for longitudinal slopes < 4%. On steeper slopes check dams required to reduce velocities Requires minimum 30 m length Shallow design levels	Streetscape	✓	✓
Bioretention	Filtration Removal of dissolved contaminants and fine particles	Enhanced treatment capability More natural functioning system	Under drain required where limited by soil infiltration Depth limited by high groundwater	Groundwater recharge Landscape	✓	✓
Infiltration	Discharge to ground, achieving hydraulic neutrality Filtration	Can be located below ground (small footprint) Can remove need for separate detention system	Unsuitable in poorly drained soils Requires pre-treatment for removal of sediment to prevent clogging Depth limited by high groundwater	Groundwater recharge Landscape	✓	✓
Dry Basin	Stormwater detention Removal of coarse to medium particles	Large detention capacity	Large space requirement Depth limited by high groundwater	Recreation Landscape	✓	✓
Wet Pond	Stormwater detention Removal of coarse to fine particles	More natural functioning system	Large space requirement Free draining soils may require lining of basin	Bird and aquatic habitat Landscape	✓	✓

Table 1: Stormwater Device Options						
Device	**Function**	**Comparative Advantages**	**Design Limitations**	**Additional Values**	**Quantity**	**Treatment**
			Consider safety, access and planting through slopes and planting			
Wetland	Enhanced treatment capability Removal of dissolved contaminants and fine particles	Enhanced treatment capability More natural functioning system	Requires permanent standing water Not suitable on steep slopes Large space requirement Shallow operation levels Requires diversion of flood flows	Bird and aquatic habitat Landscape	✓	✓
Water Tanks	Stormwater detention	Cost-effective, simple installation	No treatment capability	-	✓	-
Permeable Paving	Discharge to ground, achieving hydraulic neutrality	No additional area requirement – reduction of impervious surfaces Can remove need for separate detention system	Requires flat slopes less than 1V:10H Expensive solution Unsuitable in high traffic areas May clog in high sediment generating catchments and are prone to clogging in long term. Unsuitable in poorly drained soils Difficult to maintain	Groundwater recharge	✓	-
Gross Pollutant Trap	Trap coarse sediments and macro pollutants	Prevents blockage of system Cost-effective	Limited treatment capability No detention capability	-	-	✓

Table 1: Stormwater Device Options						
Device	**Function**	**Comparative Advantages**	**Design Limitations**	**Additional Values**	**Quantity**	**Treatment**
Proprietary Systems	Filtration and/or physical separation	Small footprint Good treatment capability 'Plug and Go' systems	Little ecosystem value No detention capacity	-	-	✓

Swales

Swales are broad grassed channels used to treat and convey stormwater runoff. Swales help to filter sediments, nutrients, and other contaminants from stormwater before discharge to receiving environments. Swales treat stormwater runoff by the following:

- Filtration
- Infiltration
- Adsorption, and
- Biological uptake.

Stormwater Management Function	
✓	Water quality ✓ Metals ✓ Sediment ~ TPH
x	Flood protection
x	Stream channel erosion protection

Figure 1: Typical grass-lined swale

In addition to providing water quality treatment, swales can also be used to convey stormwater runoff in place of a conventional piped reticulation system. Swales are generally constructed using in situ topsoils, rather than engineered media. Hence whilst they may provide limited infiltration of runoff, they are not primarily designed for this purpose.

Swales are appropriate to use adjacent to roadways as they can occupy a linear corridor. Although swales may vary in their purpose in different areas, their overall objective is to convey and treat stormwater, slow stormwater flows and provide limited infiltration of stormwater runoff depending on the condition of the subsoil and whether or not the swale is lined.

Water quality treatment is provided by passing stormwater flows through vegetation. Passage through vegetation and providing contact with organic matter allows physical, chemical and biological processes to occur that reduce contaminant delivery downstream.

Design Parameters

Site suitability should be based upon the parameters provided in Table 6-2.

Table 2: Site Suitability Parameters	
Parameter	**Limitation**
Swale catchment area	Swales are most effective for lower flows and volumes. Therefore, are suitable for small / medium sized catchments, generally 3 hectares or less in size.
High sediment loadings	High sediment loadings will clog up the swale invert and promote concentrated flows, which degrade water quality

	function. Swales should be protected from high sediment loads with pre-treatment. Dense planting and level spreaders at inlet can reduce sediment loads.
Swale slope	Swales should have a longitudinal slope less than 5%. In areas with steeper slopes, check dams can be used to reduce the overall swale slope.
Soils	Swales can be implemented in any soil although karst geology may require an impermeable liner on the swale invert to avoid instability issues.
Groundwater	The invert of the swale should not intersect with seasonal high groundwater. The swale base should be at least 1m above the seasonal high groundwater level.
Setback	Swales >1m from a property boundary should have a lined vertical surface if within 5 m of structures. Swales should not be within 3m of a structure.

The following Table 3 should be used for swale design elements.

Table 3: Swale Design Elements

Design parameter	Criteria
Water quality event	Maximum velocity: 0.8 m/s Flow depth: less than vegetation height, maximum 150mm for grassed swales and 300mm for vegetated swales
Primary event	Maximum velocity: 1.5 m/s unless erosion protection is provided. Flow depth: 150 mm below top of swale (unless swale is part of overland flow path)
Longitudinal slope	Swales are not suitable on slopes greater than 8% Slopes of 5-8% require check dams Swales on slopes less than 2% require an underdrain
Check dams	Required when longitudinal slope is >5% to reduce velocities. Maximum height of check dam to equal the depth of flow for the water quality event
Inflow points	Usually a slotted kerb. Care should be taken to ensure sheet flow from the catchment is directed to the swale through inflow points. Where concentrated flows enter the swale (from pipes) level spreaders shall be placed at the head of the swale to disperse flows.
Vegetation	Grass or vegetated. If vegetated plants should be selected that are tolerant of both drought and inundation and that don't shed leaves.
Maximum water depth above vegetation	The water depth for the water quality event should not exceed design height of the grass. This is a key criterion for ensuring Manning roughness coefficient is provided.
Design grass length	100 - 150 mm
Manning coefficient	0.25 for WQ storm 0.03 for a grassed swale (10-yr. Storm)

Table 3: Swale Design Elements	
Design parameter	**Criteria**
	0.25 for a vegetated swale (10-yr. Storm)
Minimum hydraulic residence time (HRT)	9 minutes
Minimum bottom width	0.6 m
Maximum bottom width	2 m
Minimum length	30 m
Maximum catchment area served	3 hectares
Maximum side slope	< 5H:1V for mown grass < 3H:1V for vegetated or unmown grass
Underdrain (not always present)	Required when longitudinal slope of a grassed swale is <2%, optional in other instances. Recommended where local soils have poor infiltration, to prevent stagnation and saturation of swale base. Underdrains are buried under the swale channel to capture filtered stormwater (usually a perforated pipe) and connect directly to the catch pit or stormwater manhole. Access must be provided for backwashing slotted drain.
Outlet	Outlets are usually a catch-pit with a flat grate or a scruffy dome

There are several points that need further discussion. They include:

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

Hydraulic Residence Time

The Hydraulic Residence Time (HRT) is the time that the water takes to travel through the swale. This is a key factor for water quality performance in a vegetated swale. The residence time depends on the following items:

The longitudinal slope of the swale:

- cross-sectional area of the swale
- Velocity of the flow

The velocity of flow is a function of the flow area, slope and frictional resistance of the vegetation and a common equation for calculating velocity is Manning's Equation.

$$v = \frac{R^{\frac{2}{3}}\sqrt{s}}{n} \quad \textbf{Eqn (6-1)}$$

Where:

v = average velocity (m/s)

R = the hydraulic radius of the swale (m)

s = slope of the swale (m/m)

n = Manning's coefficient of roughness

Residence time can then be determined by the following equation:

$$t = \frac{L}{60v} \quad \textbf{Eqn (6-2)}$$

Where:
t = residence time (minutes)
v = average velocity (m/s)
L = average flow length in swale (m)

A minimum average HRT of 9 minutes is recommended to design treatment swales in the Timaru District. Depending on how the swale is configured, there may be areas in the upper part of the swale that exceed the required HRT, and areas in the lower portions of the swale where HRT may not be met. As such, it is required that an average HRT of 9 minutes is achieved.

Manning's Coefficient of Roughness
There are many variables used to determine the Manning's roughness coefficients. A standardised Manning's roughness coefficient of 0.25 shall be applied for water quality flow calculations.

For the 10% or 20% AEP event analyses, it is assumed that the vegetation is submerged so the coefficient of roughness shall be reduced accordingly. A Manning's roughness coefficient of 0.03 for grassed swales, and 0.25 for vegetated swales is applied in these calculations.

Swale Inflow
There are two common ways that flow enters swales: via concentrated flow or dispersed lateral inflow. In addition to lateral flow diversion, Figure 6-2 also illustrates the use of check dams along a swale.

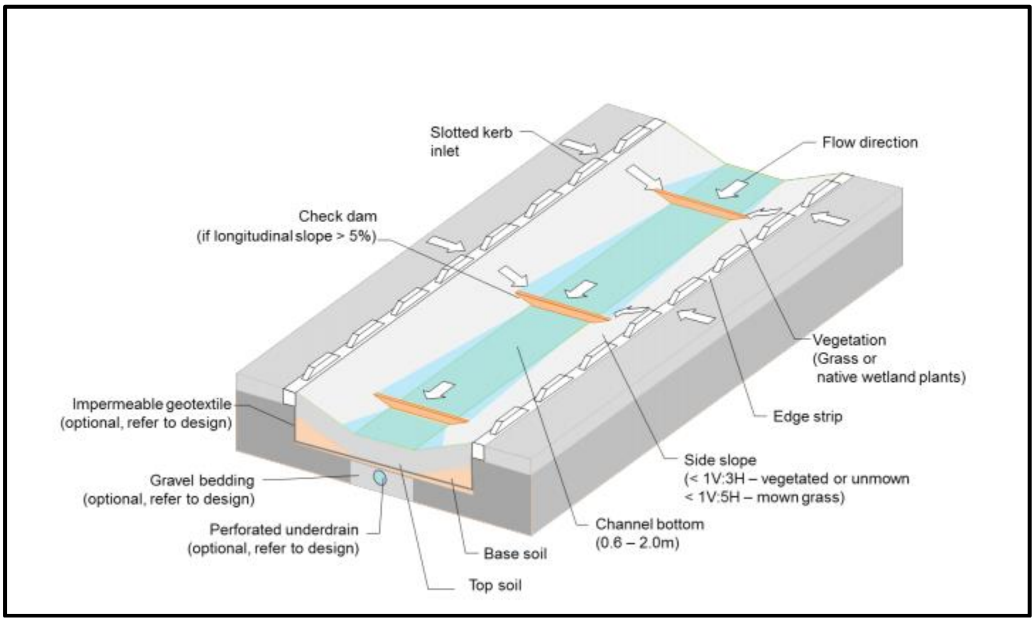

Figure 2: Schematic of a typical swale cross section

Detailed Design Procedure

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. Calculate Water Quality Flow for runoff from impervious areas only using a design intensity of 10 mm/hr.
2. Calculate conveyance flow from all contributing areas for the primary rainfall event (as per Table 4-2), using a 10 minute rainfall design event.
3. Establish the longitudinal slope of the swale.
4. Select a vegetation cover and corresponding a value for Manning's coefficient of roughness
5. Select preliminary swale geometry including shape, side slope, base width and water quality flow depth (based on vegetation length)
6. Calculate cross sectional area (A), and hydraulic radius (R). The following equations can be used to calculate A and R based on a trapezoidal cross section.

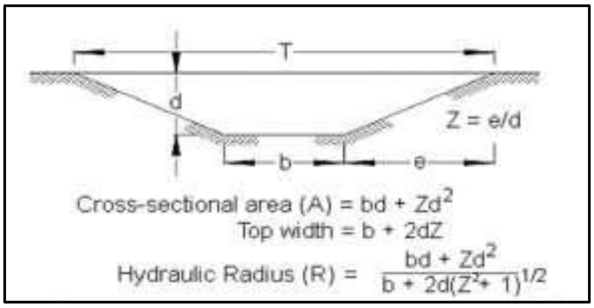

Figure 3: Swale channel geometry

$$A = bd + Zd^2 \quad \textbf{Eqn (6-3)}$$

$$R = \frac{bd + Zd^2}{b + 2d\sqrt{Z^2 + 1}} \quad \textbf{Eqn (6-4)}$$

Where:

A = Cross-sectional area (m^2)

R = Hydraulic radius (m)

T = Top width of trapezoid/parabolic shape (m)ex

d = Depth of flow (m)

b = Bottom width of trapezoid (m)

Z = Swale side slope (Z H : 1 V)

7. Use Manning's equation to check the flow in the swale. If Q calculated is less than water quality flow increase swale dimensions to accommodate flow (repeat steps 4- 7).

$$Q = \frac{AR^{\frac{2}{3}}\sqrt{S}}{n} \textbf{Eqn (6-5)}$$

Where:
Q = Flow rate (m^3/s)
n = Manning's n (dimensionless)
s = Longitudinal slope (m/m)

8. Check the swale velocity from the following equation:

$$v = Q/A \quad \textbf{Eqn (6-6)}$$

If v > 0.8 m/s, repeat steps 4 – 8 until the velocity is less than 0.8 m/s.

9. Check the length of flow in the swale is sufficient to meet minimum HRT requirements as per equation 6-2.

Flows in Excess of the Water Quality Storm

It is expected that runoff from events larger than the water quality design storm will be conveyed via the swale. In that situation, a stability check should be performed to ensure that the 10-year, 10 minute ARI event 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.

Shallow or steeper 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 6-4 provides a typical cross-section of the underdrain system ensuring that water passes through the invert of the swale, through a loam soil, then geotextile fabric and gravel prior to discharge through a 100 mm perforated pipe.

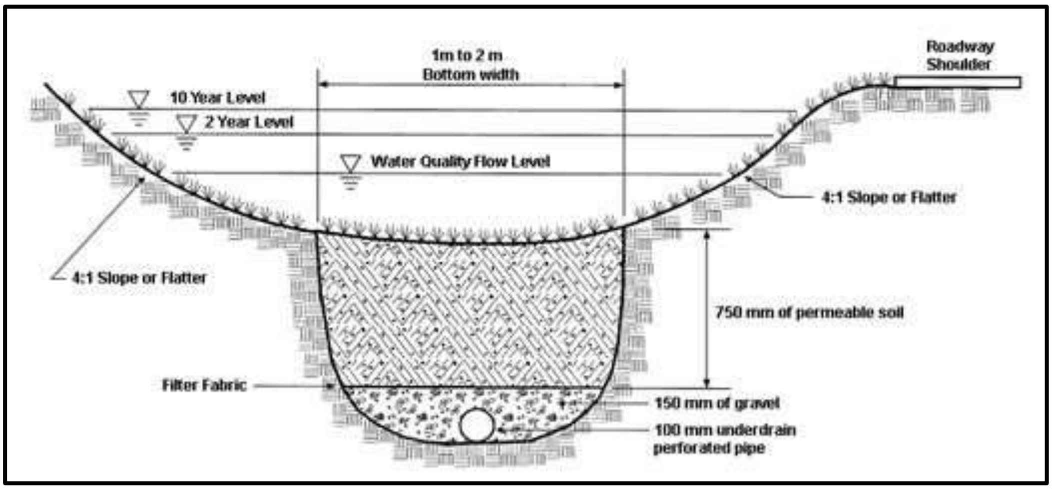

Figure 4: Swale schematic showing soils and underdrain

Where slopes exceed 5%, swales can only be used if check dams are included so that the ultimate post-construction slope between check dams is less than 5%. A key design element is that, as shown in Figure 6-4, the crest of the downstream check dam extends backwater to the toe of the upstream check dam.

To determine the spacing between check dams the following equation is to be used:

$$L_{cd} = \frac{h_{cd}}{s} \textbf{Eqn (6-7)}$$

Where:
L_{cd} = length between check dams (m)
h_{cd} = height of the check dams (m)
s = longitudinal slope (%)

Determining the number of check dams required for the swale is determined by the following equation:

$$N = \frac{L}{L_{cd}} \textbf{Eqn (6-8)}$$

Where:
N = number of check dams
L = total length of the swale (m)
L_{cd} = length between check dams (m)

When using check dams it is important to ensure that scour does not occur at the toe of the check dams. If in doubt, erosion control measures such as stone should be placed at the toes of the check dams to ensure stabilisation.

Larger storms should be calculated assuming the storage behind the check dams is full and the overall slope of the flow would be the slope from the upstream top of the swale to the bottom.

Case Study – Swale

Project Description:
A small rural lifestyle development is proposed with an overall impervious area of 40%. The area of the development is 1.5 hectare with each lot uniformly sloping at 1% towards the road. A swale will be used to treat runoff from impervious surfaces and convey it to an existing water drain downstream of the development. Runoff from the road is dealt with separately. The soil has medium soakage and pre development land use was pasture.

Design Procedure:
As the development is rural residential the swale must be designed to a 1 in 5 year event (as per Table 4-2, Appendix D). Using the High Intensity Rainfall Design charts in Appendix J (Table 2) the design intensity for a 10 minute storm is 48 mm/hr. The design intensity for the water quality event is given as 10 mm/hr.

Using Tables 4-4 and 4-5 the following runoff coefficients are chosen.

Table 1. Chosen runoff coefficients.

	Description	**Runoff coefficient**	**Slope adjustment**	**Intensity adjustment**	**Final value**
Impervious	Roofs, sealed roads and paved surfaces	0.9	-0.05	+0.15	1

Pervious	Medium soakage soil types: pasture and grass cover	0.3	-0.05	+0.15	0.4

composite coefficient is then calculated using equation 4-3.

$$C = \frac{((1.5\times0.4)\times1)+((1.5\times0.6)\times0.4)}{1.5} = 0.64$$

1. Water Quality Event:

Using the rational method for the impervious area of the site only, the flow is calculated using equation 4-1.

$$Q = 2.78 \times 1 \times 10 \times (0.4 \times 1.5) = 16.68\, L/s = 0.0167\, m^3/s$$

2. Conveyance Flow for a 20% AEP storm and a 10 minute rainfall design event for the whole site:

$$Q = 2.78 \times 0.64 \times 48 \times 1.5 = 128.1\, L/s = 0.128\, m^3/s$$

3. Establish a longitudinal slope for the swale:

A slope of 2.5% is chosen, as a result no check dams or an underdrain are required.

4. Vegetation is chosen. Because the swale is vegetated, Manning's coefficient for roughness will be 0.25 for the water quality storm and the 1 in 5 year storm.
5. A trapezoidal swale shape is applied to the swale. A side slope of 1V: 5H is chosen and initial values for b and d respectively, b = 1.05m and d = 0.1m.
6. Combining Manning's equation and first approximations for the hydraulic radius an iterative process is used to determine the dimensions of the swale. Whilst meeting the minimum and maximum requirements outlined in Table 6 - 3. The following process is used:

$$A = bd + Zd^2$$

$$A = 1.05 \times 0.1 + (5\, x\, 0.1^2) = 0.155\, m^2$$

$$R = \frac{bd + Zd^2}{b + 2d(Z^2+1)^{\frac{1}{2}}}$$

$$R = \frac{0.155}{1.05 + 2 \times 0.1(5^2+1)^{\frac{1}{2}}} = 0.0749\, m$$

7. Manning's equation is used to find the flow in the swale and hence the velocity in the swale.

$$Q = \frac{AR^{\frac{2}{3}}\sqrt{s}}{n}$$

$$Q = \frac{0.155 \times 0.0749^{\frac{2}{3}}\sqrt{0.025}}{0.25} = 0.0174\, m^3/s$$

$$v = Q/A$$

$$v = \frac{0.0174}{0.155} = 0.112\, m/s$$

At this stage the values are checked against the required values.
Is the base width between 0.6m and 2m? Yes.
Is the swale velocity less than 0.8 m/s? Yes.
Is the swale flow greater than the design water quality flow? Yes.
If these requirements are not met the values of b, d and s must be adjusted until these are

achieved.

8. The final requirements are a minimum HRT of 9 minutes and minimum swale length of 30m.

Using a HRT of 9 minutes (t = 9)

$$L = v \times t(\frac{60s}{1minute})$$

$$L = 0.111 \times 9\left(\frac{60s}{1minute}\right) = 60.2m$$

9. The conveyance in the swale for a 20% AEP 10 min event is also checked using the same iterative process. The only variable that should be changed when determining the swale flow and velocity is the depth of flow.

Using the same equations as above and adjusting d the following values are determined respectively, d = 0.273m whilst keeping b = 1.05m, $v =$ 0.194m/s and Q=0.128m^3/s.

Is the swale velocity less than 1.5 m/s? Yes.
Is the swale flow greater than the primary event flow? Yes.

10. The primary event must have a flow depth no greater than 150mm from the top of the swale.

Therefore, the final dimensions of the swale are d = 0.423m, b = 1.05m, T = 5.28 and L = 60.2m.

Bioretention

Bioretention is a process whereby stormwater runoff is treated by passing the water through a filtration media. The water then evapotranspires, infiltrates into the ground or provides a slow release to surface waters when infiltration to ground cannot be achieved.

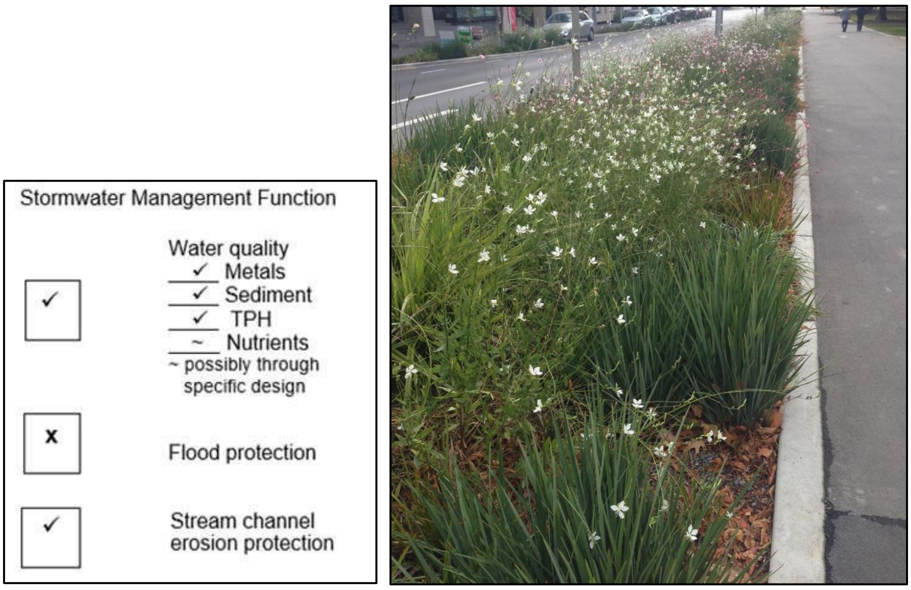

Figure 5: Bioretention

The major pollutant removal pathways within bioretention devices are:

- Sedimentation in the extended detention storage, sediments and metals
- Filtration by the filter media, fine sediments and colloidal particles; and
- Nutrient adsorption and pollutant decomposition by soil bacteria; adsorption of

metals and nutrients by filter particles; and bio-accumulation by plants.

To retain the filter media within the bioretention device and aid drainage, one or more layers are used under the filter as a transition from the fine sands and soils of the filter media to the drainage layer at the bottom of the device. The surfaces of most bioretention devices are planted with a range of vegetation.

Bioretention devices are also known by other names such as raingardens or bio filters and can range in size from tree pits to very large systems of 1,000 m^2 filter area or greater.

Rain gardens

Rain gardens are planted gardens made up of layers of specified soil media which promote the filtration and retention of stormwater. Rain gardens function by promoting the ponding of stormwater in the planted area, which slowly filters through the soil media. The combination of this filtration process, and biological uptake by the plants help absorb and filter contaminants before stormwater soaks into the underlying soil, or conveyance networks.

The key components of a rain garden are shown in Figure 6-6. Including each component in the design is critical to ensure effective operation, and to reduce the long term maintenance requirements.

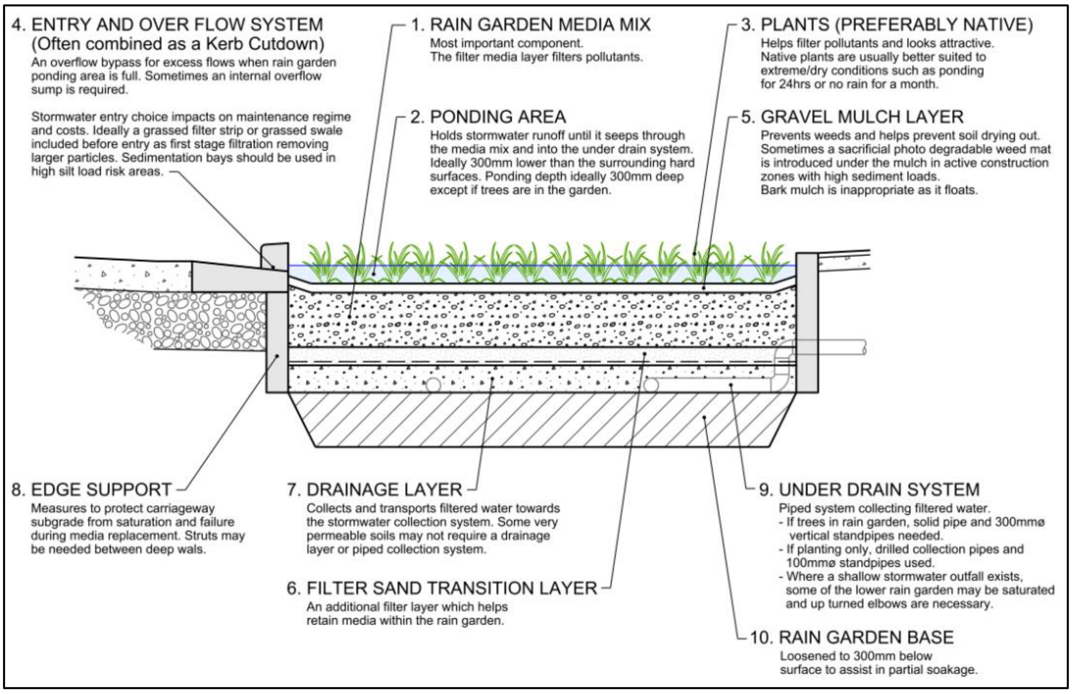

Figure 6: Typical Rain garden (source CCC, 2016

Rain gardens are best suited for retrofit into existing developed areas, or in development areas with minimal available space. For large developments, ponds, swales and wetlands remain TDC's preferred stormwater treatment device due to the lower maintenance costs and higher amenity value. Rain gardens are preferred over proprietary filtration units as they provide additional benefits including improved street amenity.

Rain gardens are excellent solutions for retrofit in existing roadways, where they can be installed in existing on-street parking space, or in the existing carriageway when road narrowing is taking place. It is important to install sediment removal pre-treatment upstream of rain gardens as they primarily fail due to blinding of the surface with the sediment.

For urban intensification, rain gardens can help reduce the effect of increased impermeability, help improve stormwater quality and attenuate flows from frequent minor rainfall events. The added street amenity is another positive factor to consider when eventuating stormwater management options.

A regular maintenance regime is critical to ensure adequate long term performance of a rain garden. The cost of such regime is likely to be higher than what is expected for a basin or wetland. Maintenance and irrigation over the first 2 to 3 years are critical to ensure plant establishment and the long term performance of the systems.

Rain Garden Design Parameters
Site suitability should be based upon the parameters provided in Table 4.

Table 4: Site Suitability Parameters	
Parameter	**Limitation**
Catchment area	The allowable catchment area of a rain garden is dictated by the maximum allowable foot print area of a rain garden being 1200 m^2. As a general rule of thumb, the rain garden filter area should have a minimum foot print of 1.5% of the impervious catchment area. This means that for a 1200 m^2 rain garden, the maximum impervious catchment area should be roughly 4 ha.
Sediment loadings	High sediment loadings will clog up the soil media and ultimately affect the performance of the rain garden. Pre-treatment is required in such scenarios to reduce the impact of sediment loads and improve long term performance. The maintenance regime must also reflect this.
Soils	The infiltration capacity of the underlying soil must be determined to design the rain garden. In areas of poor infiltration rates, a sub soil drain to a suitable outfall must be included in the design.
Groundwater	The base of the rain garden must be at least 800mm higher than the seasonally high ground water table.

The following Table 5 should be used for rain garden design elements.

Table 5: Rain garden Design Elements	
Design parameter	**Criteria**
First flush depth	21 mm (to achieve 80% annual volume capture)
Media depth	600 mm preferred if nitrogen is not the primary pollutant (300 mm minimum). 900 mm if nitrogen is the primary pollutant
Media infiltration rate	Initially 50 – 150 mm/hr Use 30 mm/hr for design purposes
Extended detention ponding volume	Minimum 40% of first flush volume
Extended detention ponding depth (EDD)	Maximum 300mm. Consider reducing this depth in high pedestrian areas.
Depth to seasonally high groundwater	800mm minimum from base of rain garden
Outlet configuration	Downed outlet with soffit of outlet at top of transition layer.

Rain Garden Detailed Design Procedure

The methodology below outlines the rain garden design procedure.

1. Calculate the water quality volume for the draining catchment area
2. Calculate the minimum extended detention ponding volume (40% of first flush volume).
3. Calculate the filter area of the rain garden:

$$A_{rg} = 41.67 \times \frac{V_{ff} d_{rg}}{k(h + d_{rg})t_{rg}} \quad \textbf{Eqn (6-9)}$$

Where:

A_{rg} = Filtration area of rain garden (m^2)
V_{ff} = First flush volume (m^3)
d_{rg} = Filter depth (m) – 0.6m recommended
k = coefficient of permeability (mm/hr) – 30 mm/hr recommended
h = average height of water (m) – Half of extended detention depth recommended
t_{rg} = time to pass first flush volume through soil (day) – 1 day recommended

4. Calculate the minimum live storage area of the rain garden:

$$A_{EDD} = \frac{V_d}{2h} \quad \textbf{Eqn (6-10)}$$

Where:

A_{EDD} = Ponding area of rain garden (m^2)
V_d = Extended detention ponding volume (m^3)

Rain Garden – Case Study

Project Description:

An existing area of land in Timaru town centre is being turned into a carpark. A rain garden is proposed between the carpark and existing road to add to the amenity of the street and help reduce the added area of permeability the carpark brings. The carpark is 3000 m^2 with a slope of 2%.

Design Procedure:
The water quality depth for Timaru is 21mm.
Using Tables 4-4 and 4-5 the following runoff coefficients are chosen.

Table 1. Chosen runoff coefficients

	Description	Runoff coefficient	Slope adjustment	Intensity adjustment	Final value
Impervious	Roofs, sealed roads and paved surfaces	0.9	-0.05	+0.05	0.9

1. Water quality volume;

$$WQV = \frac{0.9 \times 3000m^2 \times 21mm}{1000} = 56.7m^3$$

2. Minimum extended detention volume (live storage needed):

$$V_d = 0.4 \times 56.7 = 22.68m^3$$

3. Filter Area of the Rain Garden

$$A_{rg} = 41.67 \times \frac{V_{ff}(m_3) \times d_{rg}(m)}{k\left(\frac{mm}{hr}\right)(h(m) + d_{rg}(m))t_{rg}(days)}$$

$$A_{rg} = 41.67 \times \frac{56.7 \times 0.6}{30 \times \left((0.3 \times 0.5) + 0.6\right) \times 1} = 63m^2$$

4. Calculate the minimum live storage area of the rain garden

$$A_{EDD} = \frac{22.68}{2 \times 0.15} = 75.6m^2$$

5. Check to see if the calculated filter area meets the required live storage area

$$A_{rg}(63m^2) < A_{EDD}(75.6m^2)$$

Because A_{rg} < A_{EDD} the filter area of the rain garden must be increased to meet the minimum required live storage area,

$$New\ A_{rg} = \frac{22.68}{2 \times 0.15} = 75.6m^2$$

Generally, the live storage requirement will govern the size of the rain garden. However, if live storage can be provided upstream of the raingarden the rain garden can be sized based on treatment area only.

Stormwater Tree pit

A stormwater tree pit is a tree pit which is designed to treat stormwater runoff. The design of such tree pit is the same as a rain garden however is found to provide better retention than a rain garden due to the higher evapotranspiration rate associated with the tree compared to typical rain garden vegetation. Figure 6-7 below shows a typical stormwater tree pit configuration.

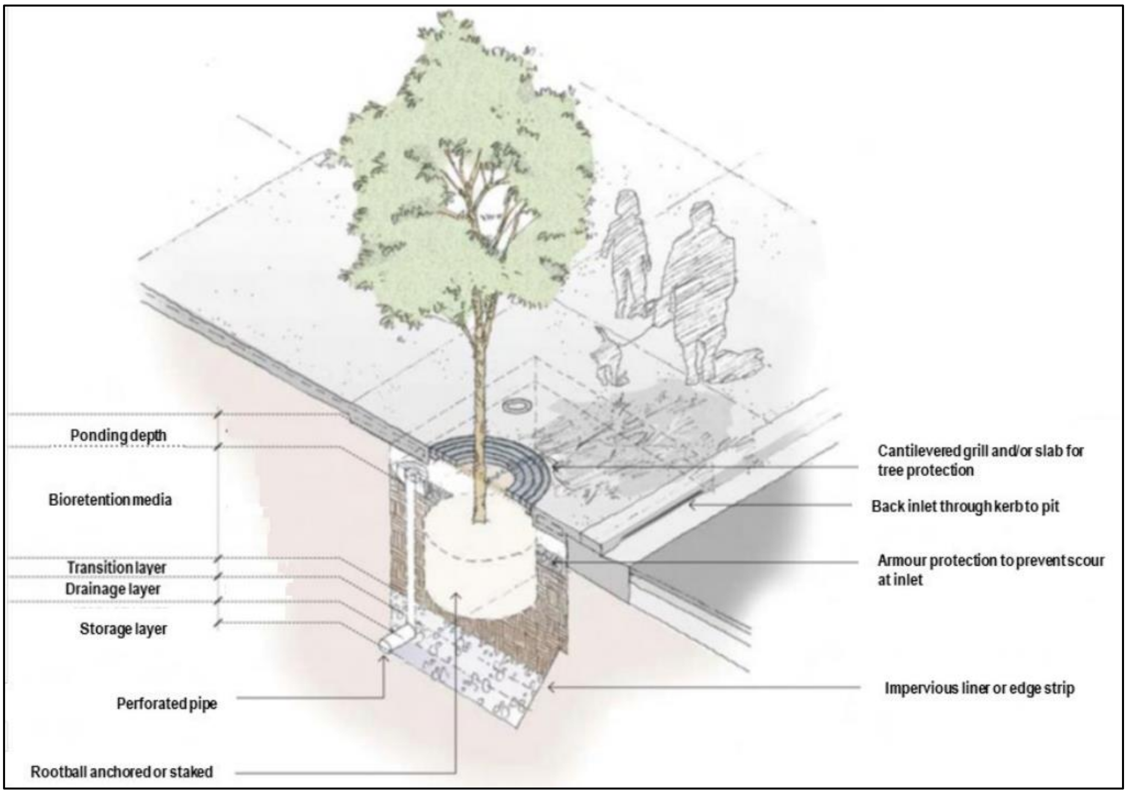

Figure 7: Typical Stormwater Tree pit

The maximum size of the tree and the tree's root ball must be considered in sizing the tree pit to ensure the tree's future needs can be accommodated. The need for future soil replacement and irrigation during establishment must also be considered. Due to these constraints, stormwater tree pits are usually unsuitable for when services are in close proximity to the location.

Infiltration

Infiltration devices are designed and constructed to capture and treat stormwater runoff through:

- Filtration
- Infiltration,
- Adsorption, and
- Biological uptake

Stormwater Management Function

✓	Water quality ✓ Metals ✓ Sediment ✓ TPH
x	Flood protection
✓	Stream channel erosion protection

Figure 8: Typical infiltration device

An infiltration device can be used to direct urban stormwater away from surface runoff paths and into the underlying soil. In contrast to surface detention methods, which are treatment or delay mechanisms that ultimately discharge all runoff to streams, infiltration diverts runoff into groundwater. Of all the traditional stormwater management practices, infiltration is one of the few that can achieve volumetric hydraulic neutrality by reducing the overall volume of stormwater being discharged to surface water receiving environments. This practice most closely matches the hydrological cycle of a greenfield site, allowing groundwater recharge and baseflows in groundwater-linked waterways.

It is critical to provide pre-treatment prior to infiltration devices to remove sediment which can clog the filter media.

There are a wide variety of infiltration devices, including but not limited to:

- Infiltration basins
- Soakage pits
- Permeable pavements

Infiltration Basin

An infiltration basin is essentially a pond that has no surface outlet other than for extreme events. The only way for the water to leave the ponded area is through infiltration to ground. Figure 6-9 shows a schematic of an infiltration basin with an overflow outlet.

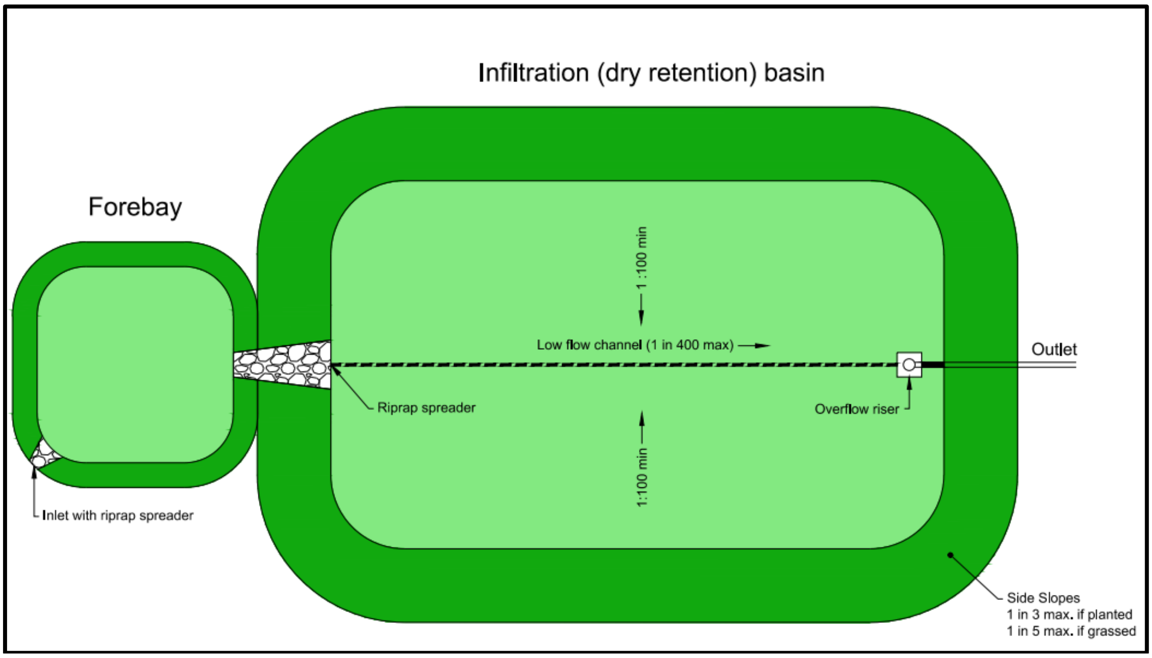

Figure 9: Schematic of an infiltration basin

Infiltration basins are detention facilities which temporarily store stormwater runoff and infiltrate stormwater to ground. Due to the interaction of stormwater with soils and vegetation as it infiltrates, they can achieve a high level of stormwater treatment.

Infiltration basins can also be designed to provide flow attenuation. This is accomplished by providing "dry" storage above the designated infiltration volume. This attenuated volume is then released through an outlet system.

Soakage Pit

Soakage pits provide a stormwater discharge function in a similar fashion to infiltration basins. However, the excavated subgrade is filled with rocks and the void spaces provide for stormwater storage until the runoff infiltrates as shown in Figure 6-10. Due to the lack of interaction between soils and vegetation prior to discharge, soakage pits are not considered to provide stormwater treatment. Pre-treatment may also be required, depending on the source of stormwater directed to them.

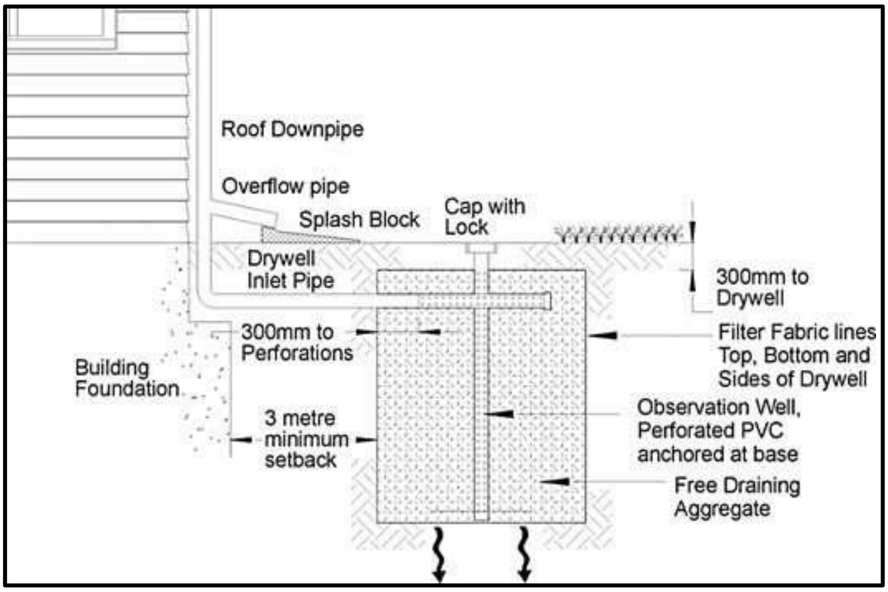

Figure 10: Schematic of a soakage pit

Permeable Pavements
Modular block permeable pavement permits precipitation to drain between paving blocks with a pervious opening as show in Figure 6-11. Paving blocks are appropriate only for areas with very light or no traffic, or for parking areas with minimal turning movements. They are laid on a gravel subgrade and filled with sand or sandy loam but can also be used with grass in the voids which may require irrigation and lawn care during the summer months.

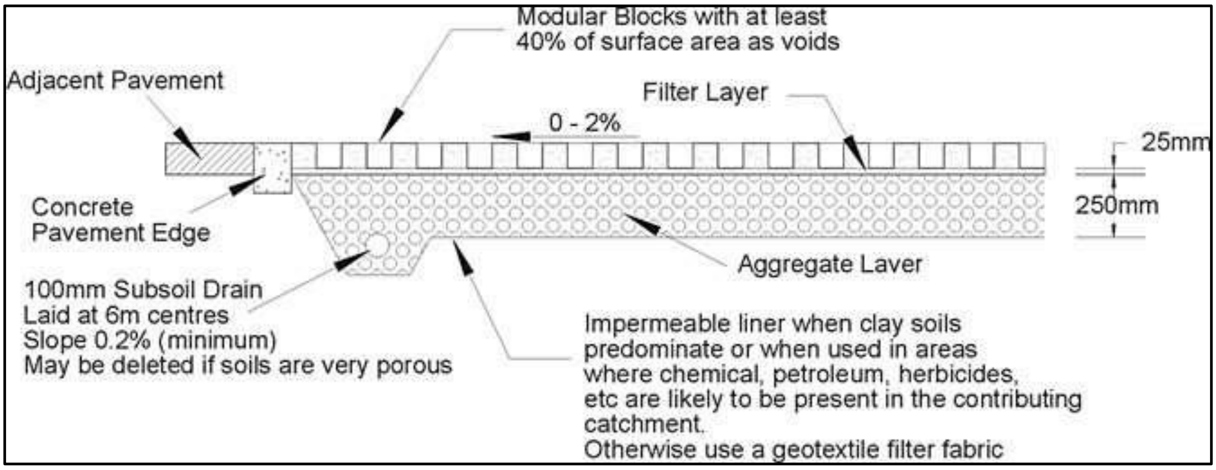

Figure 11: Schematic of permeable pavement

Evolving materials such as permeable concrete, enable stormwater to pass through the hardened material with inter-connected pore spaces. This enables discharge of rain falling directly on the surface. However, the surface can become clogged with fine particles. Reference and specifications should be sourced from suppliers for the design, installation and ongoing maintenance of these products.

Infiltration Device Components
Details for the standard components for infiltration devices are provided in Table 6 below:

Table 6: Infiltration Device Components		
Parameter	**Device**	**Description**
Pre-treatment	Basin	Pre-treatment is provided via a forebay which limits sediment entering the infiltration basin. The only situation where pre-treatment is not required is when runoff has low sediment load, such as roof water from residential areas.
	Soakage pit Permeable pavement	Pre-treatment limits sediment entering the infiltration device and extends the life of the device. The only situation where pre-treatment is not required is when runoff has very low sediment load, such as roof water from residential areas.
Storage (optional)	Basin	Storage is provided within the infiltration basin and is to be sized accordingly.
	Soakage pit Permeable pavement	Storage may be provided within, above or below the aggregate. Storage chambers such as crates, arches and pipes can also be incorporated.
Aggregate	Soakage pit Permeable pavement	Aggregate in the form of gravel, is used to create storage space within the device. Clean drainage aggregate to provide retention and detention storage comprising washed gravel 20 – 40 mm diameter with defined void ratio of at least 0.3.
Geotextile	Soakage pit Permeable pavement	The sides (and top for a soakage pit) of the infiltration device are lined with geotextile to prevent the migration of aggregate and sediments. Geotextile must be secured at edges and base and all joins overlapped to prevent the movement of fine sediment between the device layer and base soils. Geotextile is not to be placed between aggregate layers within the device.
Overflow	Basin Soakage pit Permeable pavement	Infiltration systems should be fitted with an overflow system for the management of rainfall events which exceed the infiltration and storage capacity of the device. This overflow is to be directed towards an approved outfall.
Observation well	Soakage pit Permeable pavement	An observation well should be installed so that inspections can be made. It should consist of a perforated PVC pipe, 100-200 mm in diameter and have footplate and an impermeable lockable cap.

Infiltration Device Design Parameters

Site suitability should be based upon the parameters provided in Table 7.

Table 7: Infiltration Device Site Suitability Parameters	
Parameter	**Limitation**
Catchment	Soakage pits and permeable pavements are suitable for small – medium catchments. Infiltration basins are suitable for medium – large catchments. Devices needs to be located at the lower end of the catchment.
Groundwater	The invert of an infiltration device should be at least 1 m from the seasonally high groundwater level, or any impermeable soil layer.
Infiltration rate	Geotechnical assessment is required to confirm infiltration capacity of subsoils. Soils must have a minimum infiltration rate of 10 mm/hr after including a factor of safety of 3. Therefore, the lowest measured infiltration rate must be equal to or greater than 30 mm/hr.
Underlying soil	Soils need to be evaluated to ensure suitable rates of ground soakage. It must also be ensured that the device is not positioned in an area with instability issues, expansive or saline soils.
Aquifers	The potential impact of infiltration devices on aquifers and downgradient users must be assessed and any risks must be mitigated.
Contaminated Land	Not suitable in an area with contamination nor an area with a high risk of future contamination.
Slope	Implementation on slopes greater than 15% shall only be allowed with design input from a geotechnical expert.
Setback	Infiltration devices must be located at least 3 m from structures, slopes, on-site wastewater systems and roads.
Traffic	Infiltration basins and soakage pits must be located at least 3 m away from trafficked areas. Permeable pavements are suitable for parking or areas with light traffic.

Soil type / infiltration rate

Soil infiltration or permeability is the most critical consideration for the suitability of infiltration devices. Infiltration devices should be constructed in medium textured soils with high permeability. They are unsuitable for use in soils with poor drainage or high ground water. The underlying soils must be tested at the proposed site, refer to Section 3.7.3 and Appendix I for an infiltration testing methodology, minimum infiltration rates, and other site considerations.

Pre-treatment
Pre-treatment is required prior to infiltration devices to reduce the potential for clogging, improve long term performance of the device and minimise operation and maintenance requirements. The exception is infiltration devices accepting only residential roof runoff.

Inlet stability
In addition to pre-treatment, long term function depends on having flows enter the infiltration device through a stable system that does not scour and increase sediment load to the device. If entry is via a reticulated system then velocities entering the device have the potential to cause scour, hence the inlet to the device would need to be stabilized appropriately.

Protection during construction / building phases
Infiltration devices must be protected during site development to ensure construction-phase stormwater runoff does not enter the device. If possible, infiltration devices should not be constructed until the surrounding areas have been stabilised and erosion is no longer a concern. Where this isn't possible, incoming flows must be diverted around infiltration devices until the contributing catchment area is stabilised.

Detailed Design Procedure

In terms of the design approach:

1. Calculate the volume to be managed by the infiltration device V_{runoff}
    a. Water Quality Volume OR
    b. Primary rainfall event runoff volume OR
    c. Volume required to achieve stormwater neutrality

2. Assume an initial depth and surface area of the soak pit.
3. Calculate the volume of soakage for the design storm duration:

$$V_{soak} = A_{sp} S_r D_s \qquad \textbf{Eqn (6-11)}$$

Where:
A_{sp} = Surface area of the infiltration device (m^2)
S_r = Design soakage rate (m/hr)
D_s = Duration of storm (hrs)

4. Calculate the volume of storage required.

$$V_{storage\ req} = V_{runoff} - V_{soak} + pA_{sp-surface} \qquad \textbf{Eqn (6-12)}$$

Where:
$A_{sp-surface}$ = Surface area of infiltration device exposed to rainfall (m^2) (zero for buried soak pits)
p = Design event rainfall depth (m)

5. Calculate the volume of storage available within the infiltration device.

$$V_{storage\ avail} = A_{sp} d_s V_r \qquad \textbf{Eqn (6-13)}$$

Where:
V_r = Void ratio (typically 0.38 for below ground storage in aggregate and 1 for above ground

ponding in infiltration basins)
d_s = depth of infiltration device (m) (this is depth of media below ground for a soakage pit or permeable pavement and ponding height for infiltration basin)

6. Check $V_{storage\ avail} \geq V_{storage\ reg}$
7. Check draw down time is less than 48 hours

$$D = \frac{V_{storage\ req}}{S_r A_{sp}} \quad \textbf{Eqn (6-14)}$$

8. If $V_{storage\ req}$ is significantly less than $V_{storage\ avail}$ or the draw down time is less than 48 hours the infiltration device may be oversized. Steps 2 – 7 can be repeated with varying values for the surface area of the soak pit until the optimum size is accomplished.

Case Study – Soakage pit

Project Description:
Runoff from a 1000m^2 area of road reserve is to be captured and discharged to ground through a soak pit. 70% of the catchment is made up of impervious surfaces (footpaths and roads) and the remaining 30% consists of berms and landscaping. Soakage testing on the proposed site suggests a field soakage rate of 1200 mm/hr. A factor of safety of 3 should be applied for the design soakage rate. The groundwater at the site is 3m below existing ground level. As per Table 4-2 the soak pit needs to be sized to cope with a 1 in 10 year event. The slope of the kerb and channel conveying runoff to the soak pit has a grade of 1:200.

Design Procedure:
Due to the lack of interaction between soils and vegetation prior to discharge, soakage pits are not considered to provide stormwater treatment. Therefore, the water quality volume is not considered. Instead, the runoff volume from for the 1 in 10 year event 1 hr storm is calculated.

Using Table 2 in the High Intensity Rainfall Design Chart, using a duration of 60 minutes and a 1 in 10 year storm event the intensity is found to be 25 mm/hr.

Using Tables 4-4 and 4-5 the following runoff coefficients are chosen.

Table 1. Chosen runoff coefficients

	Description	Area (ha)	Runoff coefficient	Slope adjustment	Intensity adjustment	Final value
Impervious	Roofs, sealed roads and paved surfaces	0.07	0.9	-0.05	+0.15	1
Pervious	Medium Soakage – pasture and grass cover	0.03	0.3	-0.05	+0.15	0.4

The composite coefficient is then calculated using equation 4-3.

$C = \frac{(0.07\times 1)+(0.03\times 0.4)}{0.1} = 0.82$

1. Rational method to calculate runoff volume:

$$Q = \frac{2.78 \times 0.82 \times 25 \times 0.1}{1000} = 0.0057\, m^3/s$$

$V_{runoff} = 0.0057 \times 60min \times 60sec = 20.52m^3$

2. Assume an initial surface area of the soak pit and a depth.

The depth is chosen as d = 2m, to meet the requirement of a minimum of 1m vertical clearance from seasonal groundwater. An initial surface area of 15 m^2 is chosen.

3. Calculate the volume of soakage for the design storm duration:

$$V_{soak} = A_{sp}S_rD_s$$

$$V_{soak} = 15 \times 0.4 \times \frac{60min}{60hrs} = 6\, m^3$$

4. Calculate the volume of storage required.

$V_{storage\ req} = V_{runoff} - V_{soak} + pA_{sp}p$

$$V_{storage\ req} = 20.52 - 6 + 0 \times \frac{25}{1000} = 14.52\, m^3$$

5. Calculate the volume of storage available.

$V_{storage\ avail} = A_{sp}\, d_sV_r$

$V_{storage\ avail} = 15 \times 2 \times 0.38 = 11.4\, m^3$

6. Check $V_{storage\ avail} \geq V_{storage\ reg}$

$V_{storage\ avail}(11.4\, m^3) \leq V_{storage\ req}(14.52\, m^3)$

Criteria not met

7. Increase the surface area to meet $V_{storage\ req}$:

The surface area of the soak pit is increased until the $V_{storage\ avail}$ is equal to or greater than $V_{storage\ req}$. Steps 3 to 6 should be repeated using a new value for the surface area of the soak pit. If it results in zero ponding the design is complete. Otherwise, another iteration should be completed until zero ponding occurs.

In this case an A_{sp} = 18 m^2 is adequate.

8. Check draw down time is less than 48 hours

$D = \frac{V_{storage\ req}}{S_rA_{sp}} = \frac{14.52}{0.4 \times 18} = 1.85\, hrs$

Case Study – Infiltration Basin

Project Description:

Runoff from a 10000m^2 residential subdivision is to be conveyed to an infiltration basin and discharged to ground. 80% of the catchment is made up of impervious surfaces (footpaths and roads) and the remaining 20% consists of berms and landscaping. Soakage testing on the proposed site suggests a field soakage rate of 900 mm/hr. A factor of safety of 3 should be applied for the design soakage rate. The groundwater at the site is 2m below existing ground level. There is no surrounding secondary flow paths or infrastructure to attenuate to so there is no outlet structure. The infiltration must be designed to detain a 1 in 100-year storm event with a 1 hour duration.

Design Procedure:
Using Table 2 in the High Intensity Rainfall Design Chart, using a duration of 60 minutes and a 1 in 100-year storm event the intensity is found to be 50 mm/hr.

Using Tables 4-4 and 4-5 the following runoff coefficients are chosen.

Table 1. Chosen runoff coefficients

	Description	Area (ha)	Runoff coefficient	Slope adjustment	Intensity adjustment	Final value
Impervious	Roofs, sealed roads and paved surfaces	0.8	0.9	-0.05	+0.15	1
Pervious	Medium Soakage – pasture and grass cover	0.2	0.3	-0.05	+0.15	0.4

The composite coefficient is then calculated using equation 4-3.

$C = \frac{(0.8\times 1)+(0.2\times 0.4)}{1} = 0.88$

1. Rational method to calculate runoff volume:

$$Q = \frac{2.78 \times 0.88 \times 50 \, x \, 1}{1000} = 0.12232 \frac{m^3}{s}$$

$V_{runoff} = 0.122 \times 60min \times 60sec = 440.4m^3$

2. Assume an initial surface area of the soak pit and a depth.

The depth is chosen as d = 1m, to meet the requirement of a minimum of 1m clearance from seasonal groundwater. An initial surface area of 355 m^2 is chosen.

3. Calculate the volume of soakage for the design storm duration:

$$V_{soak} = A_{sp} \, S_r D_s$$

$$V_{soak} = 342 \times 0.3 \times \frac{60min}{60hrs} = 102.6 \, m^3$$

4. Calculate the volume of storage required.

$V_{storage\ req} = V_{runoff} - V_{soak} + pA_{sp}$

$$V_{storage\ req} = 20.52 - 6 + \frac{50}{1000} \times 355 = 351.6 \, m^3$$

5. Calculate the volume of storage available.

$V_{storage\ avail} = A_{sp} d_s V_r$

$V_{storage\ avail} = 355 \times 1 \times 1 = 355 \, m^3$

6. Check $V_{storage\ avail} \geq V_{storage\ reg}$

$V_{storage\ avail}(355\ m^3) \leq V_{storage\ req}(351.6\ m^3)$
Criteria is met

7. Check draw down time is less than 48 hours

$$D = \frac{V_{storage\ req}}{S_r A_{sp}} = \frac{351.6}{0.3 \times 355} = 3.3\ hrs$$

Wetland

The creation of wetlands in urban areas to manage stormwater helps to reintroduce natural areas into the urban landform. Wetlands provide many important benefits including the attenuation of flood flows, maintenance of water quality and support aquatic and terrestrial ecological values. From a contaminant removal perspective, wetlands provide a number of different removal processes that are not available in deeper wet ponds.

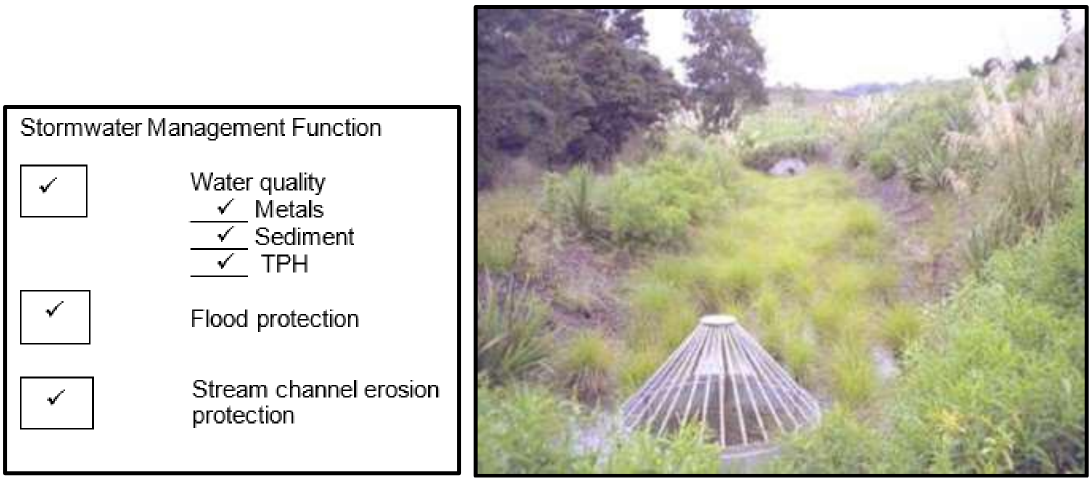

Figure 12: Typical Wetland

The design of wetlands is complex and impacted by site and outcome considerations. The Christchurch City Council simplistic method for wetland sizing using Reed's method (Reed *et al.*, 2005) is applied here.

Design Parameters

Hydrology is the single most important criterion for determining the success of a constructed wetland as it dictates the health of the wetland vegetation. They should therefore only be used in areas that have a base flow from rain, upstream runoff or groundwater to ensure the long-term viability of wetland processes.

Site considerations for the design of wetlands include the following:

Table 8: Site considerations for wetland	
Catchment	Wetlands should be sized based on the entire contributing catchment and must be located in the catchment's lower portion.

Location	Wetlands should be placed away from slopes or locations where there are potential slope stability issues. Wetlands should be designed with appropriate setbacks from dwellings, property lines, retaining walls, structures and traffic areas. Wetlands should not be located on or near contaminated land or fill materials. Wetlands should be positioned offline from any water course.
Groundwater	A geotechnical investigation is needed to inform all wetland designs. Some permeability may be desired for in instances where groundwater recharge is desired.
Soils	Wetland functionality is not impacted by poor drainage.
Base flow	Wetlands need to receive adequate base flow / water inputs to maintain the health of wetland vegetation. Some permeability can be designed for where groundwater recharge / retention is required as long as it does not impact vegetation health.
Pre-treatment	Pre-treatment is recommended to reduce the long term maintenance costs of wetlands. Ongoing maintenance is also required for the removal of litter, and sediment build up.
Maintenance	Adequate space is required for access for operation and maintenance functions to be performed around the wetland – to all pre-treatment areas and the main body of the wetland and the inlet and outlet structures. Regular maintenance is required to remove gross pollutants and to remove sediment build up from wetland forebays.

Wetland design considerations and parameters include the following (Table 9):

Table 9: Design Considerations for Wetlands	
Wetland shape	Should be designed to promote flows that use the full width of the wetland and that avoid short circuiting. Length to width ratio of the flow path should approximate 10L:1W
Inlets	Inlets need to be located within a forebay bund to capture gross sediments in the forebay and to enable flows to be dispersed into the main body, avoiding short circuiting. Debris screens should be used for safety and to remove rubbish and prevent clogging.

Table 9: Design Considerations for Wetlands	
	Erosion protection should be provided at the discharge point for inlets (rock rip rap on a geotextile layer). The invert of the inlet should be no lower than the designed permanent water level of the wetland. A high flow bypass should form part of the inlet structure, diverting non-design flows upstream of the forebay with erosion protection.
Hydraulic residence time (HRT)	Provide a minimum hydraulic residence time of 2 days to achieve water quality treatment.
Outlets	The service outlet incorporates specific outlets at different levels sized to achieve the required design criteria for the site. The outlet riser should incorporate the specific outlets, a top debris screen and a valve/screw cap located close to the wetland base level to allow for dewatering of the wetland for maintenance. The outlet pipe which discharges downstream must be correctly sized. If discharging to a coastal area, stream, lake or wetland, erosion protection must be provided. A removable weir plate should be included in the outlet arrangement (within an accessible manhole) that allows the permanent water level to be adjusted for maintenance. Anti-seepage solutions must be provided along outlet pipes.
Forebay	To hold a minimum of 15% of the water quality volume, depth is 1.0 m. The base of the forebay should be lower than the main body of the wetland. The base should be hardened for easier maintenance. A vertical depth marker should be included to assess sediment build up. Flow velocities from the forebay to be less than 0.25m/s during a 10 year ARI event. Forebay bund is to be accessible for maintenance. A submerged impermeable bund is recommended (crest level

Table 9: Design Considerations for Wetlands	
	100- 150mm below the permanent water level) to delineate the forebay from the main body of the wetland but to provide a constant depth. The forebay bund ends should be keyed into the side slopes.
Slopes	All slopes must be approved by a geotechnical engineer based on site specific constraints. Wetland bank slopes should generally not exceed 4H:1V and be planted
Water depth	Banded bathymetry; intermittent deeper water and shallow planted areas, as discussed below For design purposes assume an operating depth of 0.25 m to determine overall wetted area.
Wetland safety bench	Is to be provided at least 3m wide around the entire wetland (no more than 300mm below the permanent water level), densely planted to form a natural barrier.
Emergency spillway	Should be armoured and ideally located in natural ground. The spillway embankment should be carefully compacted during construction to prevent settlement. Where possible locate near the inlet to the wetland to minimise resuspension of sediments in large storm events. Invert should be 100mm above the maximum water level in the wetland. Freeboard should be at least 300mm above the maximum peak flow of the design storm event.
Maintenance access	An access track is to be provided that is a minimum of 3.5m width and adequate slope to provide ease of access. A sediment drying area is required near the forebay (sized to accommodate 10% of the permanent water volume at 1m depth), located away from the wetland banks, flat with vehicle access.
High flow bypass	It is recommended that a high flow bypass and maintenance bypass is included in the design. High flow bypasses should be designed to: • Withstand high flows without erosion and scour.

Table 9: Design Considerations for Wetlands	
	• Preferably to be above ground, e.g. a vegetated trapezoidal channel. • Take into account downstream conveyance capacity constraints.
Planting	At least 80% of the wetland zone is to be densely planted (excluding the forebay area) at a minimum density of 4 plants / m^2. Assume a wetland vegetation porosity of 0.75. Suitable plant selection is critical for wetland success. Plant species should be tolerant to the required ranges of depth, frequency and duration of inundation. Taller marsh species should be selected within deep marsh zones. Initial planting densities in deep marsh zones should be higher than in shallow marsh zones, so hydraulic resistance is similar between shallow and deep areas. Vegetation that provides a high level of shading (trees, shrubs and reeds/tall sedges) should be planted around, and within, the wetted margin of the wetland.
Flow velocities	Flow velocities in the wetland must not exceed 0.1 m/s for up to the 2 year ARI event and 0.5 m/s for larger storms.
Fish passage	Should be included in the design where appropriate.

Constructed wetlands must be designed in accordance with:

- New Zealand Society of Large Dams (NZSOLD), Dam Safety Guidelines, 2018
- NZSOLD, Guideline on Inspecting Small Dams, 1997
- New Zealand Building Act, 2004.

Bathymetry

Constructed wetlands are shallow vegetated water bodies that do not contain large volumes of water per surface area when compared to wet ponds.

Wetlands are to be designed to have banded bathymetry, as illustrated in Figure 8-27 below. Banded bathymetry, in long section, has variable depths with alternating deep and shallow marsh sections interspersed with occasional open water areas. Design for 60% of the surface area to have a depth range of 0 – 0.5 m, and the remaining 40% a depth range of 0.5 – 1.0 m. It is assumed that water spreads evenly across the full width of the wetland as a uniform flow.

No areas of a wetland other than the sediment forebay should be deeper than 1 metre.

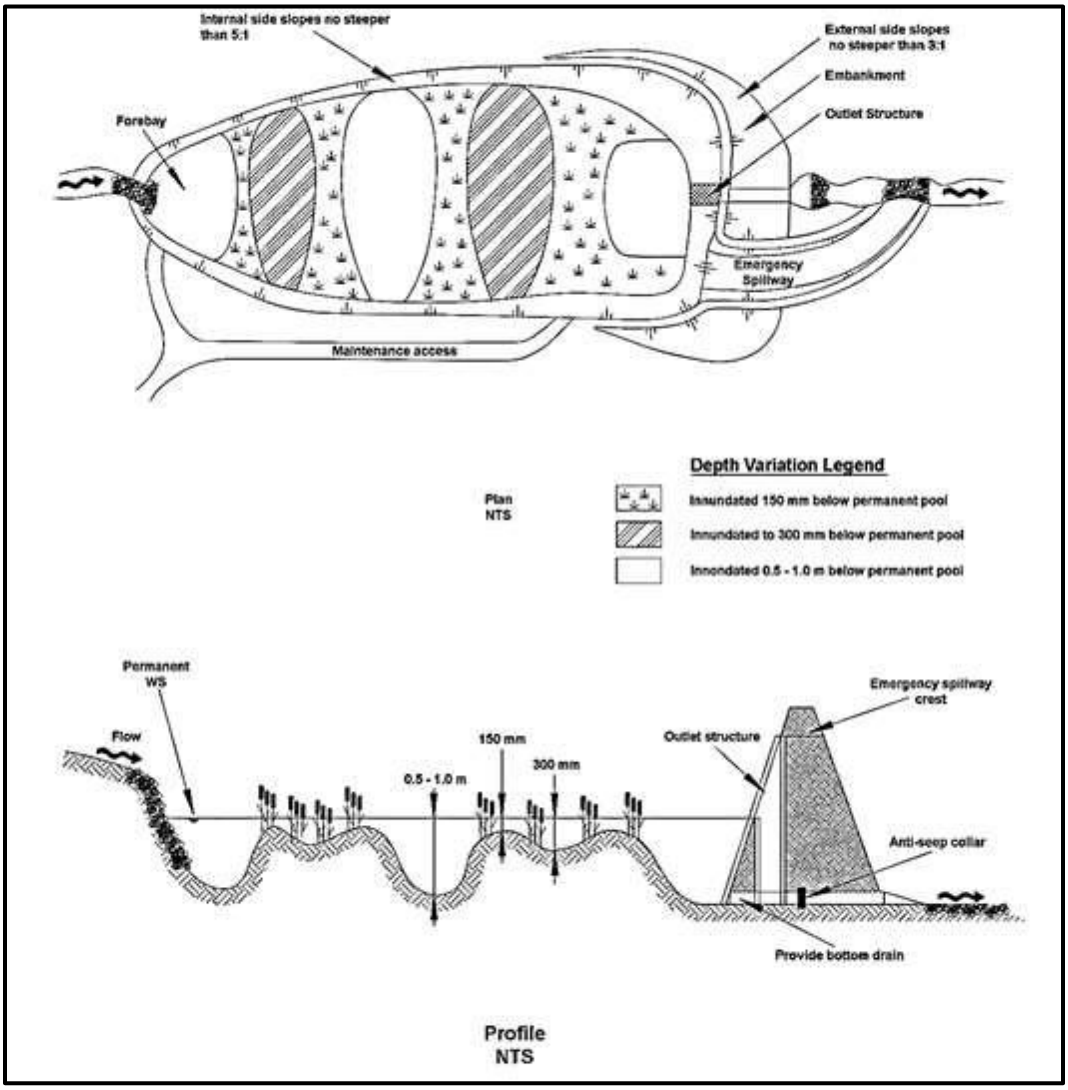

Figure 13: Banded bathymetric wetland schematic

Soils

An important element of wetland function is the need to maintain suitable conditions for wetland plants. As such, a soils analysis of the invert of the wetland shall be undertaken to ensure that the wetland area will retain water.

Where a liner is used in a wetland, a minimum depth of 300 mm of soil (or greater depending on what is suitable for the selected wetland vegetation) is required above the liner to ensure vegetation has enough soil to grow in.

Groundwater Levels

If a wetland is proposed in an area with known high groundwater levels, the groundwater level and water quality must be taken into account when considering the need for a liner/impermeable layer in the base of the wetland.

If the wetland isn't to have an impermeable layer then ideally the full range of groundwater levels needs to be understood in terms of both the impacts of groundwater on storage/detention volumes during winter high groundwater levels, and drawdown and associated effects on vegetation during summer low groundwater levels.

High Flow Bypass
Wetlands can be designed to provide flow attenuation. However, flow velocities must be managed to reduce the risk of resuspension of captured sediments and associated pollutants, prevent scour of biofilms and to protect plants.

Wetlands should be designed with a high flow bypass where possible to protect wetland vegetation from damage during large rainfall events. The bypass should divert high flows upstream of the forebay.

Design Procedure
The design steps are the following:

1. Calculate Water Quality Volume
2. Provide detention prior to entering the wetland, through a combination of wet/dry ponds or size the forebay to detain the full WQV for slow release to the wetland over 4 days.
3. Determine the average flow rate (m^3/day) through the wetland:

$$Q = \frac{WQV}{4\ days} \quad \textbf{Eqn (6-15)}$$

4. Determine the wetland treatment area (m^2) required (A_s):

$$A_s = \frac{Q\ t}{y\ n} \quad \textbf{Eqn (6-16)}$$

Where
t = hydraulic residence time (days)
y = average water depth (m)
n = vegetation porosity (assume 0.75)

5. The shape of the wetland should be designed to promote flows that utilise the full width of the wetland. The length of the wetland flowpath should approximate 10 times its width.
6. Ensure that the percentage of wetland depths meet the above criteria with a banded bathymetric design.
7. Determine the need for extended detention. Wetlands can provide up to 0.5 m detention storage over the WQV.
8. Size outlets using orifice and weir equations provided in Section 6.8 Stormwater Ponds.

Plants for a given project should be considered for suitability by an appropriately skilled practitioner. It is essential that selected plants are very tolerant of wet and dry conditions.

Case Study – Wetland

Project Description:
Runoff from a 2.5ha area commercial site with warehousing and hardstand is to be captured and treated via a wetland before being discharged to the TDC reticulated network. The catchment is 100% impervious surface (including the stormwater management surface area). As per Table 4-3 the stormwater system needs to meet neutrality requirements for

the 1 in 10 year (10%), 24 hour rainfall event.

Design Procedure:
The first flush depth for Timaru is 21mm.

Using Tables 4-4 and 4-5 the following runoff coefficients are chosen.

Table 1. Chosen runoff coefficients

	Description	Area (ha)	Runoff coefficient	Slope adjustment	Intensity adjustment	Final value
Impervious	Roofs, sealed roads and paved surfaces	2.5	0.9	-0.05	+0.15	1

1. Calculate water quality volume:

$$WQV = \frac{0.9 \times 25000 \times 21}{1000} = 472.5m^3$$

2. An extended detention dry pond will act to detain water prior to release into wetland over 4 days

$$Q = \frac{472.5}{4\ days} = 118.13\, m^3/day$$

3. Determine the wetland treatment area. Assume HRT 2 days, average water depth in wetland 0.25 m and vegetation porosity of 0.75.

$$A_s = \frac{Q\ t}{y\ n}$$

$$A_s = \frac{118.13 \times 2}{0.25 \times 0.75} = 1260m^2$$

4. Preliminary wetland sizing is 120m long, approximately 10.5 m wide, 60% shallow @ 0.15 m deep, 40% deep @ 0.40 m deep (average depth 0.25 m)

Wet and Dry Ponds

A stormwater pond is a constructed stormwater management device that collects and detains stormwater runoff from an upstream contributing catchment. A pond can be designed to provide attenuation of flows to help mitigate downstream flood risk and to protect streams from erosion and scour effects. Stormwater ponds can provide some water quality treatment.

Stormwater Management Function

✓*	Water quality ~ Metals ✓ Sediment ~ TPH ~ possibly through specific design
✓	Flood protection
✓	Stream channel erosion protection

* Up-catchment treatment required

Figure 14: Typical Wet Pond

This section provides details about ponds that are either normally dry or normally wet. The following summarises the key differences:

Dry pond - A constructed pond that temporarily detains stormwater runoff to control the peak rate of discharge to mitigate potential flood effects and can provide extended detention to help mitigate downstream erosion and scour effects. These ponds are designed to be dry between storm events.

Wet pond - A constructed pond that has a permanent pool of standing water with live storage provided above this to attenuate peak flows during rainfall events. These ponds can provide some water quality treatment. They can also provide extended detention to help mitigate downstream erosion and scour effects. Increased water temperature associated with the standing water can have a significant adverse effect on receiving environments.

A typical wet pond is shown in Figures 15 and 16.

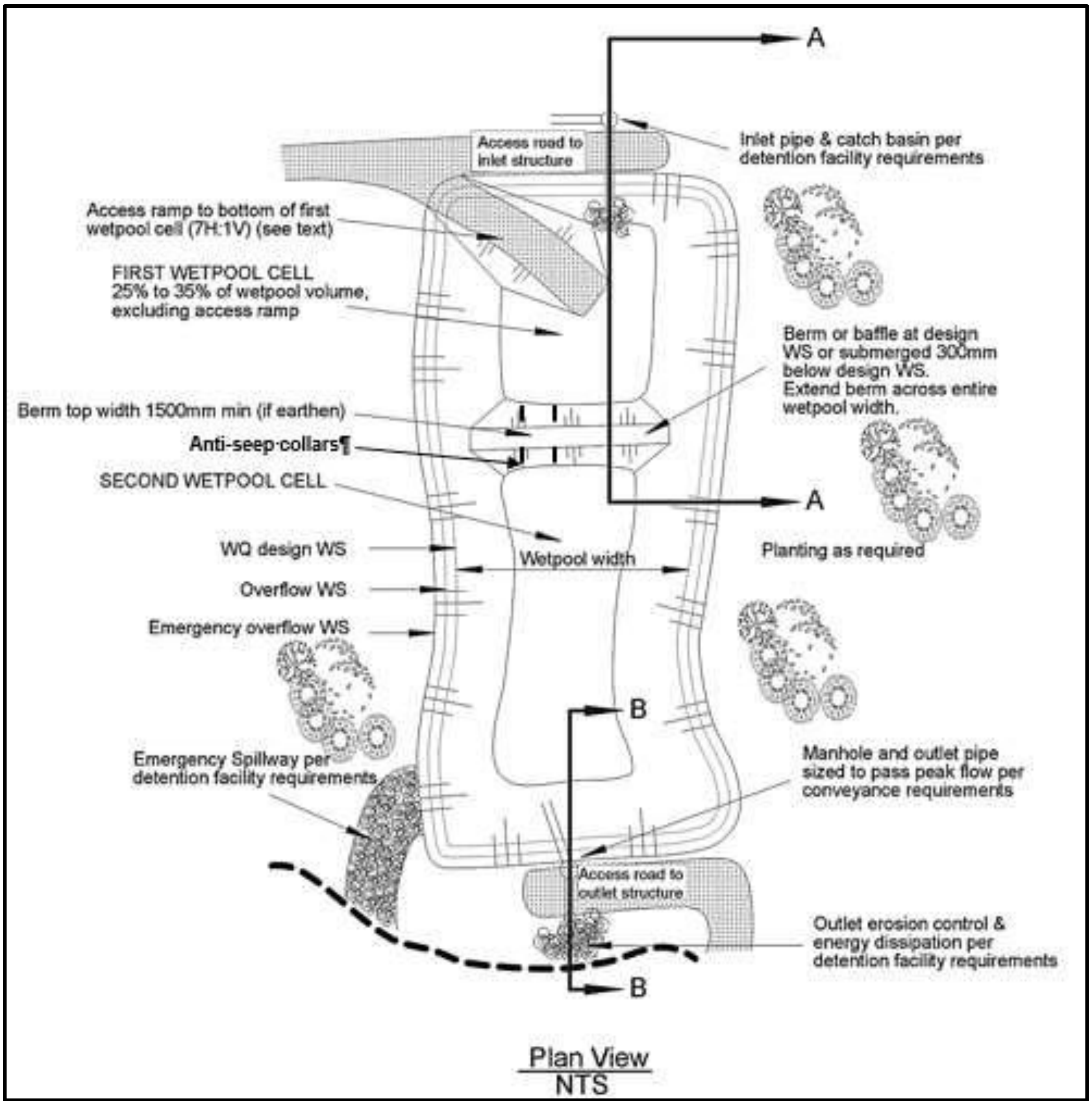

Figure 15: Schematic of a stormwater management pond

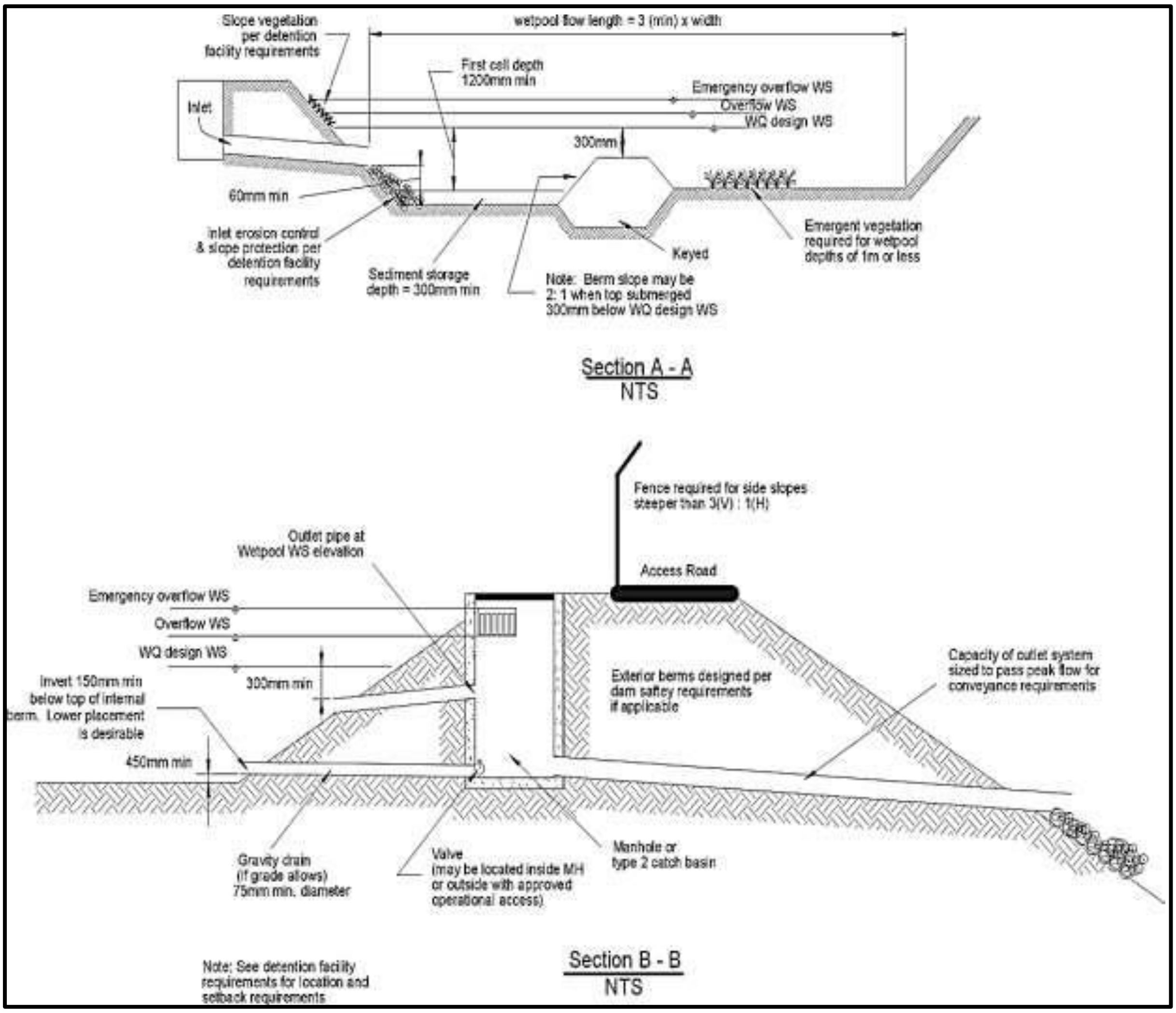

Figure 16: Pond cross-sections

Constraints on the use of ponds

Dry ponds

- Require porous soils or subsurface drainage to ensure that the bottom stays dry between storm events
- Not suitable in areas with high water tables or shallow depth to bedrock
- Not suitable on fill sites or steep slopes unless geotechnically checked
- May not be suitable if receiving water is temperature sensitive

Wet ponds

- Not suitable on fill sites or near steep slopes unless geotechnically checked
- May need supplemental water supply or liner system to maintain permanent pool if not intercepting groundwater
- Minimum contributing drainage area of 6 hectares is needed to maintain the permanent pool
- Not feasible in very dense urban areas or areas with high land costs due to large surface area needs
- May not be suitable if receiving water is temperature sensitive due to warming of pond surface area.

- Safety issues around pool depth need to be addressed

Technical safety criteria for pond design and construction that are beyond the scope of this document include:

- Minimum dam top width
- Embankment side slopes for stability
- Seepage control
- Foundation standards
- Outlet protection
- Access and set aside area for sediment drying

Design Parameters

Site considerations for the design of ponds include the following:

Table 10: Site considerations for Ponds	
Catchment	Ponds should be sized based on the entire contributing catchment and must be located in the catchment's lower portion. Wet ponds should have a minimum contributing drainage area of 6 hectares is needed to maintain the permanent pool
Location	Ponds should be placed away from slopes or locations where there are potential slope stability issues. Ponds should be designed with appropriate setbacks from dwellings, property lines, retaining walls, structures and traffic areas. Ponds should not be located on or near contaminated land or fill materials. Ponds should be positioned offline from any water course.
Groundwater	A geotechnical investigation is needed to inform all pond designs. Dry ponds are not suitable in areas with high water tables or shallow depth to bedrock. Wet ponds may need supplemental water supply or liner system to maintain permanent pool if not intercepting groundwater
Soils	Dry ponds require porous soils or subsurface drainage to ensure that the bottom stays dry between storm events Wet pond functionality is not impacted by poor drainage.
Base flow	Wet ponds need to receive adequate base flow / water inputs to maintain the permanent water level.

Maintenance	Adequate space is required for access for operation and maintenance functions to be performed around a pond. Regular maintenance is required to remove gross pollutants and to remove sediment build up from wet pond forebays.

Pond design considerations and parameters include the following (Table 11):

Table 11: Design Considerations for Ponds	
Pond shape	For wet ponds a minimum length to width ratio of 3L:1W is preferred to facilitate sedimentation
Inlets	Inlets to wet ponds need to be located within a forebay bund to capture gross sediments in the forebay and to enable flows to be dispersed into the main body, avoiding short circuiting. Debris screens should be used for safety and to remove rubbish and prevent clogging. Erosion protection should be provided at the discharge point for inlets (rock rip rap on a geotextile layer). The invert of the inlet should be no lower than the designed permanent water level of the pond. A high flow bypass should form part of the inlet structure, diverting non-design flows upstream of the forebay with erosion protection.
Outlets	The service outlet incorporates specific outlets at different levels sized to achieve the required design criteria for the site. The outlet riser should incorporate the specific outlets, a top debris screen and a valve/screw cap located close to the pond base level to allow for dewatering of wet ponds for maintenance. The outlet pipe which discharges downstream must be correctly sized. If discharging to a coastal area, stream, lake or wetland, erosion protection must be provided. A removable weir plate should be included in the outlet arrangement (within an accessible manhole) that allows the

Table 11: Design Considerations for Ponds	
	permanent water level to be adjusted for maintenance. Anti-seepage solutions must be provided along outlet pipes.
Forebay (Wet Ponds Only)	To hold a minimum of 15% of the water quality volume, minimum depth is 1.0 m. The base of the forebay should be lower than the main body of the pond. The base should be hardened for easier maintenance. A vertical depth marker should be included to assess sediment build up. Flow velocities from the forebay to be less than 0.25m/s during a 10 year ARI event. Forebay bund is to be accessible for maintenance. The crest of the forebay weir should be set to the permanent water level, or 300 mm below the water quality level, whichever is highest The forebay bund ends should be keyed into the side slopes.
Permanent Water Volume (Wet Ponds Only)	Permanent water volume shall provide a minimum permanent water volume of 50% of the water quality volume when extended detention is required, or 100% of the water quality volume when extended detention is not required.
Slopes	All slopes must be approved by a geotechnical engineer based on site specific constraints. Pond bank slopes should generally not exceed 4H:1V if planted or 5H:1V where mowing is required
Water depth	Minimum forebay depth 1m Maximum depth of pond 2m
Safety Features	A reverse slope bench or slope break should be provided 300 mm above the normal standing water pool (where there is a normal pool) for safety purposes. A submerged safety bench is to be provided at least 3m wide around the entire pond (no more than 300mm below the permanent water level)

Table 11: Design Considerations for Ponds	
	Maximum pond side slopes should be 4 H to 1 V. Steeper slopes will make it very difficult for someone who is in the pond to get out of it. TDC does not require fencing of ponds, as it is considered that use of natural features such as reverse benching, dense bank planting, and wetlands buffers (which consist of a dense stand of vegetation) will provide a similar level of protection. The fencing requirement may be reconsidered on a case-by-case basis.
Emergency spillway	Should be armoured and ideally located in natural ground. The spillway embankment should be carefully compacted during construction to prevent settlement. Where possible locate near the inlet to the wet pond to minimise resuspension of sediments in large storm events. Invert should be 100mm above the maximum water level in the pond. Freeboard should be at least 300mm above the maximum peak flow of the design storm event.
Maintenance access	An access track is to be provided that is a minimum of 3.5m width and adequate slope to provide ease of access. A sediment drying area is required near the forebay for wet ponds (sized to accommodate 10% of the permanent water volume at 1m depth), located away from the pond banks, flat with vehicle access.
High flow bypass	It is recommended that a high flow bypass and maintenance bypass is included in the design. Where the pond has an attenuation requirement, then the bypass can be directed to the second cell High flow bypasses should be designed to: • Withstand high flows without erosion and scour. • Preferably to be above ground, e.g. a vegetated trapezoidal channel. • Take into account downstream conveyance capacity constraints.

A schematic of pond safety features is shown in Figure 17.

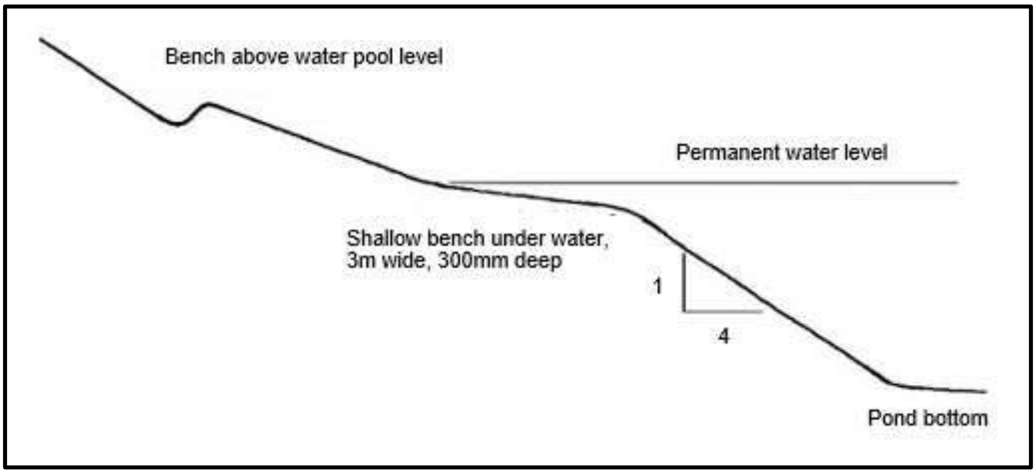

Figure 17: Schematic of safety benches and slopes

Aesthetics must be considered as an essential pond design component. Ponds can be a site amenity if properly designed and landscaped.

Design Procedure

Pond design tasks include the following:

1. Determine the need for stormwater quantity management. Calculate the extended detention requirement which detains the 2 year ARI event and releases it over 48 hours.
2. Determine the need for water quality management. Calculate the water quality volume.

Forebay (Wet Pond Only)

A forebay must be provided for all wet ponds to capture sediment and ensure that flows into the main pond are non-erosive. The sediment forebay is intended to capture only coarse sediments and is the location where most frequent sediment cleanout will be needed.

The forebay bund separating the forebay from the main body of the pond should be formed from impermeable material so that the water level in the forebay can be lowered for maintenance purposes. The crest of the forebay weir should be set to the permanent water level, or 300 mm below the water quality level, whichever is highest.

The forebay should meet the following criteria:

1. The volume of the forebay should be at least 15 % of the calculated water quality volume.
2. Flow velocities from the forebay during the 10-year ARI rainfall event must be less than 0.25 m/s, in order to avoid resuspension of sediment. In some cases this may necessitate increasing the size of the forebay above the minimum criteria provided above.
3. The recommended depth of the forebay is 1 metre or more, to reduce velocities.
4. The forebay should be cleaned out when filled with sediment to 50% of its design volume.

Permanent Water Volume (Wet Pond Only)
The permanent water volume is the volume of water permanently held within a wet pond, between the permanent water level and the base of the pond. It includes the forebay volume and the volume stored in the second wet pool cell. This permanent water pool provides some settling of sediments and amenity value.

The permanent water volume should meet the following criteria:

1. Provide a minimum permanent water volume of 50% of the water quality volume when extended detention is required, or 100% of the water quality volume when extended detention is not required.
2. The volume of permanent water in the forebay is included in the permanent water volume.
3. All ponds should have a shallow bench within the permanent water volume which is 300 mm deep and extends at least three metres from the shoreline, before sloping down to the pond floor.

Hydraulic flow characteristics
The extended detention volume is equal to the 2 year ARI event and includes 50% of the water quality volume. The extended detention volume is located above the permanent water level in a wet pond, or above the base of a dry pond. The extended detention volume should be stored and released over a 48 hour period.

Spillways and Outlet Capacity
There are two primary outlets from a pond: the service outlet and the emergency outlet. They will be discussed in the context of their sizing. Figure 6-18 illustrates the various outlet elements and components.

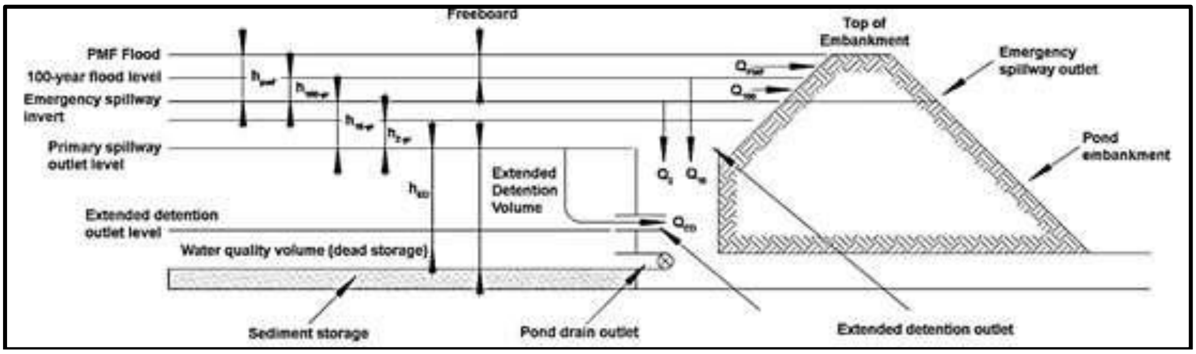

Figure 18: Schematic of pond outlet components

The service outlet should be designed to at least accommodate the flows from the primary drainage system entering the pond. It is important to consider blockage on all outlet devices as smaller orifices will be highly susceptible to blockage unless specifically designed for.

The emergency spillway will convey flows beyond the service spillway's capacity. It should be designed to convey at least the 100-year ARI storm with a freeboard of at least 300 mm.

The emergency spillway should be constructed in natural ground and not placed on fill material unless it is armoured to prevent scour of the embankment. Operating velocities must be calculated for spillways, and particularly at the toe of the spillway, in natural ground to determine the need for additional armouring.

In situations where embankment failure may lead to loss of life or extreme property damage, seek specialist design advice. The New Zealand Society on Large Dams (NZSOLD) can provide guidance on assessing risk and embankment design.

The service outlet may consist of an orifice or pipe outlet, a drop inlet structure, a broad crested weir, a cascade weir or a weir leading to an open channel. As peak control requirements call for both 2 and 10- year ARI storms to be controlled, the discharge is clearly defined in terms of the following equations.

Low Flow Outlet

Required orifice size can be determined by trialling various orifice sizes.

$$Q_i = 0.62A\sqrt{2gh_i}$$

h_i = height of water above centroid of orifice

= the elevation at extended detention - the elevation at normal pool + d/2.

Other devices may be suitable for extended detention design, and all are based on a similar approach to the orifice opening approach. Those designs can include:

- Multiple orifices at the same elevation (n orifices, A area each)

$$Q_i = n0.62A\sqrt{2gh_i}$$

h_i = height of water above centroid of orifice

= the elevation at extended detention - the elevation at normal pool + d/2.

- Vertical slot extending to water surface (width w)

$$Q_i = 1.8wh_i^{\frac{3}{2}}$$

h_i = height of water above invert of slot

- Vertically spaced orifices (situated h_1, h_a, h_b from surface of pond filled to the WQ volume. Each orifice area A)

$$Q = 0.62A\sqrt{2gh_1} + 0.62A\sqrt{2gh_a} + 0.62A\sqrt{2gh_b}$$

- Pipe (area A)

$$h = \left(\frac{1.5{Q_i}^2}{2gA^2}\right) + h_f$$

where h_f is pipe friction loss.

Drop inlet

For moderate flows, the top of the drop shaft acts as a circular sharp weir. For a circular drop inlet, the energy head above the weir lip, (h_{ii}) can be used to calculate the flow according to:

$$Q_{ii} = 3.6pRh_{ii}^{\frac{3}{2}}$$

Where R is the radius of the inlet.

For a box weir:

$$Q_{ii} = 7.0wRh_{ii}^{\frac{3}{2}}$$

Where w is the length of the four sides of the square box, on the inside.

These equations apply only for $h_{ii}/R \leq 0.45$ (or, for a box inlet, $h_{ii}/w \leq 0.45$). For $h_{ii}/R >$ 0.45, the weir becomes partly submerged, and for $h_{ii}/R > 1$ the inlet is fully submerged and the flow resistance is equal to the inlet resistance of a pipe, typically:

$$h_{ii} = k\left(\frac{v^2}{2g}\right)$$

Where v is the velocity at flow Q_{ii} and k is typically 0.5 to 1.0, depending on the details of the inlet

For a circular inlet:

$$v = \left(\frac{Q_{ii}}{pR^2}\right)$$

Starting with the design flow and the chosen pipe radius, the head (h_{ii}) can be found by using the appropriate formula for the h_{ii}/R value. If this head is higher than desired, a large outlet can be used.

Aeration of the flow over the weir should be considered if the flows are so high that inadequate ventilation may cause damage to the drop structure. In general, adequate ventilation will be provided by appropriate sizing of the outlet pipes. It is recommended that the outlet pipe be sized so that when the emergency spillway is operating at maximum flow (Qv), the outlet discharges at 75% full. Standard pipe friction and pipe outlet loss calculations can be performed to determine the required outlet size.

The entry to the outlet should be protected by a screen or grid cage to collect debris.

Broad crested weir

In this case, a weir narrower than the emergency weir is used. The weir could be situated away from the emergency weir, or if sufficient erosion protection is provided, in a lowered section of the emergency spillway.

The flow may pass down a single chute into a small plunge pool or appropriately lined area. Alternatively, a series of small cascades or a stepped spillway may be used. To size the weir, the change in pond elevation (h_{ii}) at the service design flow is found by solution of the following equation:

$$Q_{ii} = 0.57\sqrt{2g}\left(\frac{2}{3}Lh^{3/2} + \frac{8}{15}zh^{5/2}\right)$$

As an approximation, the following formula may be used for a broad-crested weir:

$$Q_{ii} = 1.7Lh_{ii}^{3/2}$$

Where *L* is the weir length.

Weir with channel
This design will be useful for shallower ponds, where the channel can be easily constructed by making a cut in the embankment.

The outflow is controlled by the weir. The following may be used as an approximation for a sharp-crested weir:
$Q_{ii} = 1.8Lh_{ii}^{3/2}$

Where Q_{ii} is the service design flow, h_{ii} is the head over the weir when the emergency spillway starts operation and L is the length of the weir. The outlet channel should be sufficiently large that the water level is below the water level (h_{ii}) at the service design flow (to avoid backwater effects).

Emergency spillway design
The emergency spillway section is normally designed as a trapezoidal channel whose sizing is based on trial and error to the following equation:

$$Q_{ii} = 0.57\sqrt{2g}\left(\frac{2}{3}Lh^{3/2} + \frac{8}{15}zh^{5/2}\right)$$

Where:
Q = discharge through the spillway
L = horizontal bottom width of the spillway
h = depth of flow at design flow
z = horizontal/vertical side slope (recommended to be 3)

Pond and Site Design
Pond shape
The design of pond shape should consider engineering constraints, design parameters to achieve treatment, and the existing topography. For a given catchment the design parameters include water volume, surface area, depth, flow velocity and detention period. A minimum length to width ratio of 3L: 1W is preferred to facilitate sedimentation. The designer should minimise dead zones and short-circuiting to improve the treatment performance of the pond.

An average 5 m buffer width around stormwater ponds is required for access and landscaping. Where possible, ponds should be located alongside reserve areas to maximise green space.

Pond contours
Pond contour profiles are critical to the design of a pond: they determine available storage, the range of plants that can be grown and the movement of water through the pond. The safety features of shallow slopes and reverse slopes will help provide areas suitable for a variety of plants.

Edge form

Edge form influences the appearance of a pond, increases the range of plant and wildlife habitats and has implications for pond maintenance. Edges can include sloping margins where water level fluctuations cause greater areas of wet soils. Such gradually sloping areas can appear a more natural part of the landscape than steep banks, and they provide opportunities for a greater range of plants and habitat. Maximum side slopes of 3H:1V are recommended, with flatter slopes of 5H:1V where mowing is required.

Oil separation

Stormwater will, in most situations, contain oils and greases. Having an extended detention outlet similar to the reverse sloping pipe shown in Figure 8-26 will allow water to be discharged from below the surface and encourage volatilisation of the hydrocarbons on the surface.

Debris screens

Screens are used to trap rubbish and organic debris, which is unsightly, especially if trapped in vegetation. Screens should be used to protect extended detention outlets from clogging. Screens may be installed either at the inlet to the pond or at the outlet from the pond.

Access

Access to the pond for maintenance must be provided for in the design. Access slopes for maintenance vehicles should not exceed 12H:1V for trucks and 5H:1V for excavator access

Proprietary Treatment

Technology and innovation are bringing continuous improvement to the stormwater treatment industry in New Zealand. A number of suppliers have both off-the-shelf products and custom-designed proprietary stormwater treatment systems. These devices typically have a small footprint and have been shown to provide high levels of contaminant removal.

Proprietary devices range from sumps inserts, gross pollutant traps (GPT) and oil-water separators to enhanced systems which use membrane and media filtration. The broadly termed 'enhanced proprietary' systems have significant advantages in performance over other proprietary primary separators and gross pollutant traps. Innovative treatment methods include: hydrodynamic separation, engineered media filtration, membrane filtration and combined media-bio-filtration.

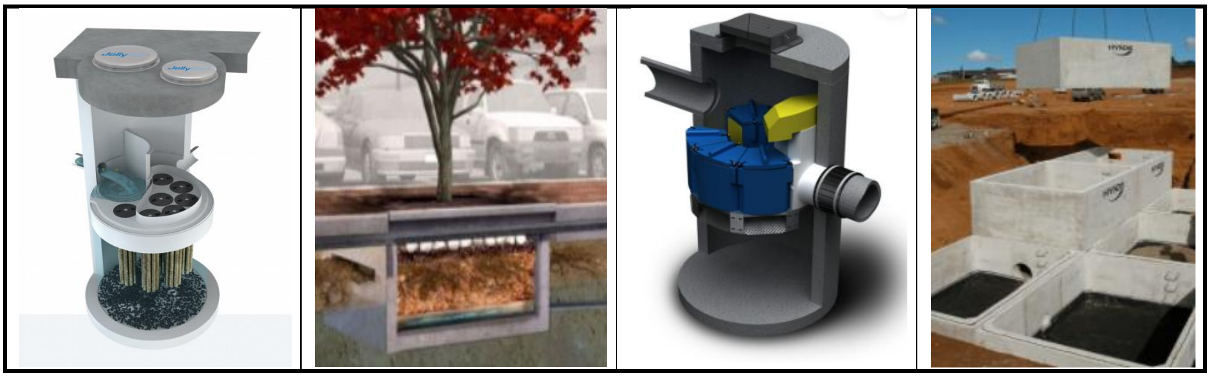

Figure 19: Examples of enhanced proprietary stormwater devices

Design Basis

Design of proprietary devices considers three key parameters; flowrate, contaminant loading and hydraulics. Flow-based treatment devices are designed to treat the water quality flow rate of 10 mm/hr. They require a driving head to operate and typically rely on filtration as their treatment mechanism. Attenuation is incorporated in some treatment devices (such as proprietary raingardens and sand filters). These devices are designed with a ponding depth above the media and a media infiltration rate. Suppliers and manufacturers play a key role in the design process, providing support for the following:

- Selection of the appropriate devices for the contaminant type and loading
- Sizing of devices
- Detailed design to meet hydraulic requirements
- Maintenance requirements and contacts

Calculation sheets, design plans and product specifications are required to be submitted as part of the approval process for the use of any proprietary system. Documentation of water quality analyses should also be supplied to support contaminant removal efficiencies.

Council’s preference is to avoid the use of proprietary devices to achieve stormwater treatment objectives in greenfield residential developments. However, proprietary systems may be an appropriate solution for brownfield and industrial / commercial developments where there may be a constraint on available space for implementing stormwater management systems. TDC will typically not approve the use of a proprietary product that has not already been shown to function successfully in New Zealand.

APPENDIX C: PLANNING, DESIGN AND OPERATION CHECKLIST

Treatment system design and operation should follow the checklist below.

Catchment Hydrology

☐ Determine runoff flows

- ☐ Base flow, summer and winter
- ☐ Frequent events, say 5 times per year
- ☐ Rare flood events
- ☐ First flush volume

Selection of Catchment Management Measures

☐ Determine flood detention objectives

☐ Determine water quality objectives

☐ Estimate the pollutant discharges (dissolved, suspended, and floating)

☐ Identify critical pollutants and receiving water reduction targets

<u>Pre design monitoring:</u>

☐ Design a monitoring/sampling program

- ☐ Select sites for monitoring
- ☐ Assess instrumentation needs

<u>Treatment train selection:</u>

☐ Review types of stormwater treatment systems

☐ Make the treatment train selection based on objectives

☐ Adopt at least first flush offline treatment

<u>Siting the facilities:</u>

☐ Look at physical constraints

☐ Consider landscape and other values

Design Principles

☐ Look at litter trapping measures

☐ Set preliminary basin/pond/wetland depth and shape

☐ Carry out flow routing for:

- ☐ Base flow
- ☐ Frequent events
- ☐ Rare flood events
- ☐ First flush diversion

☐ Look at water treatment of:

- ☐ Base flow
- ☐ Frequent events
- ☐ Rare flood events
- ☐ First flush volume

☐ Determine likely sediment removal effectiveness

- ☐ Assess particle sedimentation efficiency
- ☐ Look at ability to retain settled particles

☐ Consider recreation and aesthetic functions

☐ Refine basin/pond/wetland depths and shape

☐ Resolve inlet and outlet configurations

☐ Select aquatic and riparian plants and design the planting

☐ Look at likely impacts of water quality on ecology

☐ Design structural elements

- ☐ Look at soil substrates, groundwater, and percolation
- ☐ Carry out embankment design
- ☐ Design the shoreline of the structure for wave, water current, aesthetic, and ecological needs
- ☐ Design outlet control structures, allowing for water level control and full drawdown
- ☐ Carry out spillway design
- ☐ Review all components for maintenance requirements including access

☐ Assess and resolve Health and Safety issues

Operation and Maintenance

☐ Produce an Operation and Maintenance manual (Refer to Appendix B.2)

☐ Pond and/or wetland performance assessment

☐ Design a monitoring/sampling program

- ☐ Select sites for monitoring
- ☐ Assess instrumentation needs

APPENDIX D: STORMWATER QUANTITY

Introduction

Adverse environmental impacts can occur when high stormwater flows are discharged into the receiving environment. These flows alter the natural flow patterns and can have negative impacts such as erosion, scour, siltation and flooding. Development typically increases in the area of impervious surfaces such as roads and roof areas. These cause both an increase in volume of stormwater runoff (due to decreased infiltration or interception by vegetation), and a reduction in time it takes to enter the receiving environment, when compared to the pre-development flow patterns.

The aim of stormwater quantity management, or stormwater neutrality, is the reduction of post-development flows into the receiving environment, back to pre-development levels. This can typically be achieved by:

- attenuation – holding the water back and allowing a slow discharge
- on-site discharge – infiltrating the water to ground, or
- alternative methods such as onsite retention or re-use (these are not discussed further in this document)

Design Guidance

To assess the impact of changes to a catchment, analysis of the flows before and after development need to be made. The determination of stormwater inputs such as design rainfall intensity and runoff rates are described in Section 4.4. These inputs can be applied to the Rational Method or a detailed hydraulic analysis. Some guidance on approaches is provided:

- Surface water runoff from non-hill catchments up to 15 hectares, and hill catchments up to 5 hectares, may be calculated using the Rational Method, refer to Section 4.4
- For catchments larger than the areas above, the Rational Method tends to give a conservative result. Therefore for larger areas a dynamic analysis by computer modelling should be used.

It is common practise for stormwater quantity analysis to be undertaken using hydraulic and/or hydrologic modelling software. This document does not provide specific guidance on inputs or methods, however, reference can be made to WWDG[1] Chapter 21.4 Advanced Analysis, NZWERF[2] or use of hydraulic modelling software such as HEC-HMS or Mike Urban.

Certification Requirements

Certification requirements apply to stormwater discharges into the TDC reticulated network and/or assets to be vested and where discharges to ground are within Global Consented Stormwater Management Areas. The process to achieve certification from TDC for discharges

[1] CCC, 2020. Waterways, Wetlands and Drainage Guide: Part B. Christchurch City Council. Updated June 2020.

[2] NZWERF, 2004. On-site Stormwater Management Guideline. New Zealand Water Environment Research Foundation, October 2004.

to the reticulated network is summarised in Section 1.4, with the application form available on the TDC website[3].

With respect to stormwater quantity, the Timaru District Plan sets out the requirements in the Stormwater Management Chapter. All development (including re-development) which creates 30 m² or more of additional impervious surfaces is required to achieve stormwater neutrality.

The events for up to which stormwater neutrality must be achieved is defined by both zone and activity as described in the District Plan. It is expected that an assessment of stormwater neutrality would include rainfall events up to and including the duration specified in Tables 4-1 and 4-2.

Table 4-1: Summary of Minor Development requirements for Stormwater Neutrality				
Zone	**Residential (GRIZ, MDEZ, SZ, MPZ, RLZ)**	**Commercial (all)**	**Industrial (GIZ)**	**Other (NOZ, OSZ, SPRZ)**
Activity	>30 m² and <500 m² <70% impervious			
Stormwater Neutrality	1 in 10 year	1 in 10 year	1 in 50 year	1 in 50 year
Event Duration	1 hour	1 hour	1 hour	1 hour

Table 4-2: Summary of Major Development requirements for Stormwater Neutrality				
Zone	**Residential (GRIZ, MDEZ, SZ, MPZ, RLZ)**	**Commercial (all)**	**Industrial (GIZ)**	**Other (NOZ, OSZ, SPRZ)**
Activity	>500 m²			
Stormwater Neutrality	1 in 10 year	1 in 10 year	1 in 50 year	1 in 50 year
Event Duration	24 hours	24 hours	24 hours	24 hours

Rational Method

The Rational Method is used to estimate runoff from small catchments (non-hill catchments up to 15 ha and hill catchments up to 5 ha). It should be used with caution as the result is highly sensitive to correct selection of the runoff coefficient. For larger catchments, a dynamic analysis by computer modelling is recommended.

The Rational Formula has the form:

$$Q = 2.78\, C\, i\, A \qquad \textbf{Eqn (4-1)}$$

Where

[3] TDC Stormwater Discharge Certification, https://www.timaru.govt.nz/services/environment/storm-water/stormwater-discharge-certification

Q	= runoff in litres per second (L/s)
C	= runoff coefficient (Section 4.4.3)
i	= rainfall intensity (mm/hr) during the design storm duration (D) for the selected return period (Section 4.4.2 and Appendix F)
A	= area catchment above the point being considered (ha) (Section 4.4.1)

See below for procedures for determining A, C, i and D.

Catchment Areas

The catchment area (A) is the catchment upstream of the point under consideration, measured in hectares. The District Plan – Stormwater Chapter refers to impervious surfaces, as per the Definitions. Typical impervious surfaces include:

- Roofs
- Paved areas including driveways and sealed or compacted metal parking areas and patios
- sealed outdoor sports surfaces
- Sealed and compacted-metal roads
- Engineered layers such as compacted clay

TDC Design Rainfall

Design rainfalls specific to the main centres of Timaru District have been developed to account for localised rainfall characteristics. Design rainfall intensities, inclusive of climate change corrections, are provided in Appendix L for the townships of Timaru, Pleasant Point, Geraldine and Temuka.

For sites located away from these towns, source rainfall data from the National Institute of Water and Atmospheric Research (NIWA's) High Intensity Design Rainfall System[4] (HIRDS). The design scenario to be applied is RCP8.5 for the period 2081 – 2100, to account for the potential impact of climate change.

The Design Storm Duration (D) for stormwater devices is specified within the standards of the TDC District Plan – Stormwater Chapter. However, in applications where the duration must be determined, refer to the methodology provided in Appendix J.

Runoff Coefficient

The runoff coefficient is the proportion of rainfall that becomes runoff. Its value depends on the characteristics of the catchment, which mainly depend on the amount of pervious area. For example in sealed urban areas a higher proportion of rainfall will become runoff than would occur on rural catchment areas.

Table 4-4 lists runoff coefficients appropriate to a variety of land uses and soil characteristics. For catchments having a mixture of different types, the runoff coefficient shall be determined by averaging the value for individual parts of the catchment by using the formula:

[4] NIWA, 2017. High Intensity Rainfall Design System V4. https://hirds.niwa.co.nz/. National institute of Water and Atmospheric Research.

$$C = \frac{\sum C_i A_i}{A_c} \quad \textbf{Eqn (4-3)}$$

Where C = the runoff coefficient for the catchment

C_i = the runoff coefficient for a particular land use or surface type

A_i = the area of land to which C_i applies

A_c = the catchment area

Table 4-4: Runoff Coefficients for Soil Types and Land Use

Surface	***C***
Natural Surface Types	
Bare impermeable clay with no interception channels or run-off control	0.70
Bare uncultivated soil of medium soakage	0.60
Heavy clay soil types: – pasture and grass cover – bush and scrub cover – cultivated	 0.40 0.35 0.30
Medium soakage soil types: – pasture and grass cover – bush and scrub cover – cultivated	 0.30 0.25 0.20
High soakage gravel, sandy and volcanic soil types: – pasture and grass cover – bush and scrub cover – cultivated	 0.20 0.15 0.10
Greenspace – Parks, playgrounds and reserves – Gardens, lawns, etc.	Choose appropriate soil and cover type from above
Developed Surface Types	
Roofs, sealed roads and paved surfaces	0.90
Paving stones – with sealed joints – with open joints	 0.80 0.60
Unsealed roads and unsealed yards (e.g. gravel hardstand)	0.50
Note: Table modified from NZBC E1[5].	

The values of run-off coefficient given in Table 4-4 shall be adjusted for slope and rainfall intensity in accordance with Table 4-8, with a maximum C of 1.0 (i.e. if the adjustment to C from Table 4-5 results in a C>1, a value of 1.0 should be used.

[5] MBIE, 2020. Acceptable Solutions and Verification Methods for the New Zealand Building Code Clause E1 Surface Water. Ministry of Business, Innovation and Employment. Amended November 2020.

Table 4-5: Runoff coefficient adjustments	
Catchment Characteristics	**Adjustment to C**
Slope	
0-5%	- 0.05
5-10%	0.00
10-20%	+ 0.05
>20%	+ 0.10
Rainfall Intensity	
< 10 mm/hr	0.00
10-25 mm/hr	+ 0.05
25-50 mm/hr	+ 0.15
> 50 mm/hr	+ 0.20
Note: Table modified from NZBC E1[6].	

Devices for Quantity Management

A summary of devices which can be used for quantity management is provided in Table 4-6. A more detailed discussion of advantages and limitations of devices is provided in Section 6.3, with detailed design methodologies in Sections 6.4 – 6.8.

Table 4-6: Devices for Quantity Management		
Type	**Options**	**Comments**
Tanks	Aboveground or below ground	Effective when limited area is available Provides storage volume with limited rate discharge to the network or surface water
Basins	Wet ponds, dry basins or wetlands	Provides storage volume with limited rate discharge to the network or surface water Provision of additional ecological values
Discharge to Ground	Infiltration, bioretention practises and permeable paving	Achieves stormwater neutrality without requiring a discharge point to the network or surface water

Worked Example – Neutrality

To achieve stormwater neutrality, post-development flows into the receiving environment must be reduced to less than or equal to pre-development flows. A simplified method to determine a site's stormwater neutrality requirement follows, with a worked example below:

1. Using the Rational Method (Section 4.4) or alternative analysis, calculate the flow from the existing site, for the applicable storm event and duration (Table 4-

[6] MBIE, 2020. Acceptable Solutions and Verification Methods for the New Zealand Building Code Clause E1 Surface Water. Ministry of Business, Innovation and Employment. Amended November 2020.

1 above). Convert to a volume of stormwater generated over the duration of the rain event.
2. Repeat Step 1 for the proposed site development
3. The increase in volume/flow between the existing site (pre-development) and the proposed (post-development) represents the volume/flow required to be attenuated or discharged to ground to achieve stormwater neutrality.

- Pre-development Area = 550 m^2, medium soakage soil with pasture average grass cover, 5-10% slope
- Post-development Area = 180 m^2 roof, 230 m^2 driveway, 140 m^2 grass
- Total Impervious Surface added is 410 m^2. This is 75% of the site (410/550 x 100).
- The development site is located in a Mixed Use zone. According to Table 4-3, for sites with >70% impervious area, stormwater neutrality is to be achieved for a 1 in 10 year, 24 hour event.

Table 4-6: Worked Example Parameters

	Catchment Area (ha)	Runoff Coefficient	Rainfall Intensity (mm/hr)	Peak Discharge (L/s)
Pre-development				
Grass	0.055	0.3	4.375	0.20
			Permissible site Discharge	0.20
Post-Development				
Grass	0.014	0.3	4.375	0.05
Roof	0.018	0.9	4.375	0.20
Driveway	0.023	0.9	4.375	0.25
			Post-development Discharge	0.50

- The post-development discharge exceeds the discharge from the site prior to development by 0.3 L/s.
- Acceptable Solution 1 (Rainwater Tanks) cannot be applied as the design requires a 24 hour event.

Options include:

- Roof water tank and Infiltration device – Conversion of the driveway to permeable pavement would allow the runoff to infiltrate to ground and meet stormwater neutrality while roof water is detained in a tank
- Storage – A raingarden or dry basin could provide attenuation at the surface, or tanks located underground

APPENDIX E: STORMWATER QUALITY

Introduction

Water quality refers to the chemical, physical and biological characteristics of water. In the context of stormwater, contaminants may be entrained in water, dissolved or a chemical constituent which can affect the ecological, aesthetic, cultural, recreational or economic value of a receiving environment. Stormwater treatment is the reduction of these contaminants to a level at which they no longer have negative impacts, prior to their discharge to the environment. Table 5-1 summarises common stormwater contaminants and their impacts.

Table 5-1: Common stormwater contaminants

Type	Contaminant	Impacts
Suspended Solids	TSS	Increased sediment bed load. Smothering of fish and invertebrate gills, decreased light availability for aquatic plants and reduced visibility for fish.
Nutrients	Nitrogen, Phosphorus	Nuisance plant growth
Hydrocarbons	TPH, PAH	Oxygen depletion of waters
Metals	Copper, Lead, Zinc	Impact on the physiology of plants, chronic and acute effects on animals
Microbes	E. *coli*	Potential impacts on human health

The most effective method for removing contaminants in stormwater is to use a number of devices to provide multiple water quality benefits. It is important to select a range of treatment devices which provide complimentary contaminant removal. The combination of treatment devices in sequence is referred to as a “Treatment Train”.

Design Guidance

Stormwater treatment design uses two methods for calculation; flow-based and volumetric. The flow-based approach is applied to devices which have a constant flow and little storage volume, such as swales and proprietary devices. A volumetric approach is applied to devices which can store stormwater, such as ponds and raingardens, and this attenuation forms a key part of the treatment mechanism.

Minimum target removal rates for contaminants in stormwater are tabulated in the TDC District Plan Stormwater Chapter (Table 5-2). This may require that devices be combined to achieve the specified removal rates for contaminants of concern.

An alternative approach to treatment calculations is to use contaminant concentrations and percentage removals, to achieve a specified target concentration. This approach is applied by ECan through the Canterbury Land and Water Regional Plan[1] and the Receiving Water Standards identified in Schedule 5 and Water Quality Limits in Schedule 8. A methodology and references are provided in Appendix K. However, this approach does not presently meet the certification requirements for TDC.

Practises such as riparian planting and re-vegetation are encouraged in the district. The ecological goal is to create a sustainable and naturally functioning ecosystem, which requires a minimum of ongoing intervention and maintenance. Planting and habitat creation will also help attract invertebrates, fish, and birds back to urban areas. While the impacts on improved water quality are not quantified, these practises support a holistic approach to stormwater management. Refer to WWDG Section 11 for further detail.

Certification Requirements

Certification requirements only apply to stormwater discharges into the TDC reticulated network or assets to be vested. The process to achieve certification from TDC for discharges to the reticulated network is summarised in Section 1.4, with the application form available on the TDC website[2].

With respect to stormwater treatment, the Timaru District Plan sets out the requirements in the Stormwater Management Chapter. The minimum contaminant removal rates for stormwater treatment are defined by both zone and activity, and are tabulated below (Table 5-2).

Table 5-2: Minimum Target Contaminant Removal Rates

Zone	Residential (GRIZ, MDEZ, SZ, MPZ, RLZ)	Commercial (all)	Industrial (GIZ)	Other (NOZ, OSZ, SPRZ)
Activity	Non-residential Activity (including roads) >30 m²	>50 m²	>30 m²	>30 m²
First Flush	10 mm/hr 21 mm depth	10 mm/hr 21 mm depth	10 mm/hr 21 mm depth	10 mm/hr 21 mm depth
Suspended Solids	> 80 %	> 80 %	> 80 %	> 80 %
Total zinc	> 70 %	> 70 %	> 80 %	> 70 %
Total copper	> 70 %	> 70 %	> 80 %	> 70 %
Total Petroleum Hydrocarbons	> 70 %	> 70 %	> 70 %	> 70 %
Nutrients (Nitrogen, Phosphorus)	> 50 %	> 50 %	> 50 %	> 50 %

Water Quality Volume

[1] ECan, 2019. Canterbury Land and Water Regional Plan. Canterbury Regional Council, February 2019.

[2] https://www.timaru.govt.nz/services/environment/storm-water/stormwater-discharge-certification

The Water Quality Volume (WQV) or First Flush Volume (Vff) refers to the stormwater runoff volume that occurs at the start of a rain event. This runoff typically contains higher concentrations of contaminants which have accumulated on surfaces between rain events. Effective treatment of the first flush volume forms the design basis for many treatment devices.

The Water Quality Volume is calculated according to Eqn 7-1 below.

$$WQV = \frac{\boldsymbol{CAd}}{\mathbf{1000}} \qquad \textbf{Eqn (7-1)}$$

Where

WQV	= water quality volume (m^3)
C	= runoff coefficient (Section 4.4.3)
A	= contributing catchment area (m^2)
d	= first flush rainfall depth

A first flush rainfall depth of 21 mm is applied throughout the Timaru district.

Water Quality Flow

The Water Quality Flow (WQF) refers to the stormwater runoff flow that occurs at the start of a rain event. This runoff typically contains higher concentrations of contaminants which have accumulated on surfaces between rain events. While many stormwater treatment devices utilise the Water Quality Volume detailed in Section 0 above, some flow based devices are sized based on the Water Quality Flow.

The Water Quality Flow is calculated using the rational method according to Eqn 4-4, with runoff coefficients determined based on Table 4-4. For treatment devices that are designed on a flow-based approach, a design intensity of 10 mm/hr is applied.

Removal Rates

The minimum contaminant removal rates for stormwater treatment are defined in Table 5-2. The removal rates achieved for specific contaminants by different treatment devices are summarised (Table 5-3).

Minimum contaminant removal rates are not applicable where there are particularly sensitive or significant receiving environments. In which case, investigation of the receiving environment must be carried out and specific target concentrations achieved for the protection of the receiving environment.

Table 5-3: Removal rates (%) for various treatment devices						
	TSS	Zinc	Copper	TPH	Nitrogen	Phosphorus
Swale	75	50	60	50	20	30
Filter strip	70	50	60	>70	20	20
Sand filter	80	90	90	>70	35	45
Bioretention devices	80	70	75	>70	40	60
Bioretention devices (anaerobic zone)	80	70	75	>70	50	80
Infiltration devices	80	80	70	>70	30	60
Dry ponds	40	10	20	20	10	20
Dry ponds (extended detention)	60	20	30	30	20	30
Wet ponds	75	30	40	30	25	40
Wetlands	80	60	70	70	40	50
Green roofs	Design for quantity management only[1]					
Water tanks	Design for quantity management only[1]					
Permeable paving	Design for quantity management only[1]					
Notes: [1] These are evolving technologies and their ability to treat stormwater will be considered at TDC's discretion on the supply of literature to support contaminant removal rates.						

Removal rates for proprietary devices have not been provided. These devices will be considered at TDC's discretion on the supply of testing data and literature to support their contaminant removal rates.

Treatment by Devices

Multiple devices may be required to achieve the minimum removal rates at the point of discharge, referred to as a Treatment Train. An equation to estimate the percentage total removal of a given contaminant for two or more stormwater management devices in series is the following:

$$\boldsymbol{R = A + B - \frac{(A\ x\ B)}{100}}$$

Where:

R = total removal rate %

A = Removal rate of the first or upstream practice %

B = Removal rate of the second or downstream practice %

Modelling software is available that can be used to develop stormwater treatment trains and give percentage removals and specific concentrations expected for the treatment train given typical contaminant runoff from various catchment types. An example is MUSIC by EWater[3].

[3] https://ewater.org.au/products/music/

Many water quality treatment devices can have limited quantity control capability and may be required to be used in conjunction with another device to achieve the stormwater quality requirements for the site. Other factors including topography, land use characteristics, receiving environment and overall design intent must also be considered to determine the appropriate system to manage water quality.

Devices for Quality Management

A summary of devices which can be used for quality management is provided in Table 5-4. A more detailed discussion of advantages and limitations of devices is provided in Section 6.3, with detailed design methodologies in Sections 6.4 – 6.8.

Table 5-4: Devices for Quality Management

Type	Options	Comments
Swales	Vegetated or planted swales, filter strips	Less effective for removal of nutrients and dissolved contaminants
Basins	Wet ponds or dry basins	Removal of coarse to fine particles
Wetlands	-	Good removal of fine particles and dissolved contaminants
Infiltration	Discharge to ground or underdrainage	Requires pre-treatment for sediment removal to prevent clogging
Bioretention	Anaerobic zones Discharge to ground or underdrainage	Good removal of fine particles and dissolved contaminants
Proprietary	Various manufacturers and operational criteria	Removal efficiencies vary significantly. Requires testing data to validate the removal efficiencies

Worked Example - Quality

Options for devices to achieve the minimum contaminant removal for stormwater quality for a residential subdivision of greater than 6 lots are provided below. This activity creates a road and is therefore a non-residential activity within the residential zone. This requires contaminant removal rates of:

- TSS >80%
- Nutrients (Nitrogen, Phosphorus) >50%
- Metals (Zinc, Copper) >70%
- Hydrocarbons >70%

Example devices and treatment trains to meet the minimum treatment for a greater than 6 lot subdivision may include:

1. A single Bioretention device with an anaerobic zone, such as a rain garden - this can meet quality requirements at removal rates of:
   TSS 80%, Nitrogen 50%, Phosphorus 80%, Zinc 70%, Copper 75%, TPH 70%

2. Multiple Devices in Treatment Train

A swale and a wetland can be sized to meet quality requirements at removal rates of:

R = A + B – ((A x B)/ 100) where A is swale and B is wetland

For TSS,	R = 75 + 80 – [(75x80)/100] = 155 – 60 = 95% removal
For nitrogen,	R = 20 + 40 – [20x40)/100] = 60 – 8 = 52% removal
For phosphorus,	R = 30 + 50 – [(30x50)/100] = 80 – 15 = 65% removal
For copper,	R = 60 + 70 – [(60x70)/100] = 130 – 42 = 88% removal
For zinc,	R = 50 + 60 – [(50x60)/100] = 110 – 30 = 80% removal
For TPH,	R = 50 + 70 – [(50x70)/100] = 120 – 35 = 85% removal

APPENDIX F: OPERATION AND MAINTENANCE

Operation and Maintenance

Routine maintenance and monitoring involves scheduled tasks to ensure the stormwater device/asset is functioning as intended.

Private devices' inspection and maintenance obligations fall to the owners. Inspection reports may be required to be submitted to TDC. Non-compliance may lead to an inspection by TDC and maintenance at the owner's cost. Public assets will be inspected, maintained and monitored by TDC according to the relevant consents and Operation and Maintenance Manual.

Consideration of maintenance requirements i.e. access points, HSE and the ability to replace parts, must be considered during the design of stormwater devices and structures. A list of typical inspections, maintenance requirements and frequencies for a range of stormwater management devices is provided in Appendix H. Responsive maintenance may also be required if conditions in the catchment change or if unusual damage or a high degree of wear and tear occurs to the extent that the device is not operating as intended.

Annual maintenance/inspection records should be kept, including but not limited to:

- Date and details of inspections of the stormwater system; and
- Date and details of any maintenance work, repairs and upgrades to the stormwater system, including removal of material and its disposal.

Preparation of an Operation and Maintenance Manual is required for stormwater infrastructure. Reference the Checklist below when setting up the operation and maintenance requirements for a new asset.

Appendix F.2: OPERATION AND MAINTENANCE CHECKLIST

For Designers and System Managers

☐ Photocopy and use this checklist for all projects.

☐ For all questions, consider impacts on adjacent owners/ community.

Responsibilities

☐ Is it clear who is responsible for the various aspects of maintenance?

☐ What is expected of the adjoining owner? What is expected of the Council?

☐ Have all the necessary consents been obtained?

☐ What is the process for transferring project responsibility between different entities (e.g. between construction, plant establishment, and long-term maintenance phases)?

☐ Who should hold the discharge permit and any other consents? Should they be transferred to the Council, and if so, when?

☐ If the system component is on private land, have the landowner's needs been taken into account?

☐ Have all the written agreements of roles and responsibilities been completed?

Operation and Maintenance Methods and Procedures

☐ Have clear O&M strategies, procedures, or guidelines been issued? Are the documents readily accessible to all personnel involved?

NB: Operation manuals are always required for wetlands, basins, and ponds.

☐ Have the most appropriate maintenance techniques been specified? Are they authorised?

☐ Have alternative techniques been considered? Is the preferred technique the most cost-effective?

☐ Has satisfactory access for maintenance been provided? What type of machinery, if any, will need to gain access?

☐ Do any pools/ponds need to be completely drained for lining repair, sediment removal, etc?

Does pond water level need to be managed or completely drained? How can this be achieved?

☐ Is access width, space, slope, and surface still suitable during storm emergency conditions?

☐ Will the maintenance regime need to change through the life of the project?

☐ Has site security and adjoining owners' security been considered? Is there a need for fencing and locked gates?

Public Safety and Security

☐ Have requirements of the Health and Safety at Work Act 2015 been met (i.e. have all the hazards been identified and avoided, remedied, or mitigated against)?

☐ Does the system meet the Council's health and safety requirements?

☐ Does the proposal conform to the principles of reducing crime through environmental planning and design (refer to Crime Prevention Through Environmental Design – ISO 22341:2021)

☐ Is open-style fencing needed to improve safety and/ or security?

☐ Is the system component safe in terms of the Building Act (2004)?

☐ Is it safe for adjoining public activities? For example, is ready egress available?

☐ Will the facility be available for safe entry and exit for recreation?

Flooding and Other Hazards

☐ Does the design storm capacity meet agreed planning criteria?

☐ Where is the secondary flow path likely to be? Should it be protected by an easement?

☐ Is storm inspection and debris removal needed? If this is so, have all necessary arrangements been made?

☐ Is the level-of-service that has been designed for, adequate in view of the importance and susceptibility of the system component(s) during extreme storms?

☐ Is uncontrolled growth likely to cause a fire hazard during hot, dry summer seasons?

☐ How will the system component perform under each of the Engineering Lifelines hazard scenarios?

For example, seismic activity, River flooding, local flooding, tsunami, wind, snow, and slope hazards?

Design

☐ Have "life cycle" design principles, including project life cycle cost, been considered?

☐ Is the design a relatively low maintenance solution in the short-tern,? In the long-term?

Thus is the solution a sustainable one?

☐ Have potential flow obstructions, erosion, and sedimentation problems during the early development and establishment phases been identified and provided for in the design?

Vegetation

☐ Has the initial planting contract maintenance been defined?

☐ Has the establishment transitional maintenance been defined (i.e. while the planting becomes established; usually year 2-4 after planting)?

☐ Has the ongoing maintenance from year 5 and beyond been defined?

☐ Is there any specific maintenance criteria for the riparian vegetation in the area?

☐ How will the terrestrial vegetation growth be controlled - by sickle, weed eater, mechanical mower, or herbicide? Are the contours and space available compatible with the preferred method? What is the likely frequency of growth control measures?

☐ Will aquatic vegetation need to be controlled by handwork, excavator, harvester, or herbicide? How will algal growth be managed?

☐ Is there any specific maintenance criteria for the aquatic vegetation (including both marginal and submerged plants) in the area?

☐ Is erosion control needed if watering is necessary during plant establishment?

☐ What is the acceptable maximum height and density of vegetation that will not compromise hydraulic requirements?

☐ Could the selected plant species cause other maintenance problems located downstream?

For example, flax and cabbage tree leaves can wrap around the impellers of storm water pumps.

☐ What undesirable terrestrial or aquatic plant species could potentially become established? How will they be controlled?

Debris, Litter, and Sediment

☐ Where will debris, litter, and sediment accumulate? What measures will be needed to interrupt and remove it? How often?

☐ Are debris grills or trash racks needed?

☐ If the system component blocks during a storm, where is the secondary flow path that will divert overflows safely?

☐ Should a pond, debris trap, gross pollution trap, vegetative filter, or other interception device be installed upstream?

☐ Will sediment need to be removed? Is adequate access provided for this?

Nuisances

☐ How will the incidence of pests and nuisances be minimised:

- ☐ problem insects (blackflies, biting midges, and mosquitoes)?
- ☐ stagnant water conditions?
- ☐ smells?
- ☐ algal blooms?
- ☐ rats?

☐ How will complaints be managed?

☐ Will vegetation have adverse effects on adjoining properties?

☐ How many dwellings are close enough to be affected by nuisance

APPENDIX G: CONSTRUCTION STORMWATER MANAGEMENT

Erosion and Sediment

Erosion and sedimentation are natural processes where soil is worn away, transported and deposited by wind or water. However, these processes can be accelerated by manmade factors such as urban development and earthworks. During construction activities bare soil is exposed, which leads to an increased risk of erosion.

Effects of erosion and sedimentation include:

- ecological impacts on downstream waterways due to siltation, smothering of aquatic habitats, abrasion and low light levels
- physical blockages in channels or pipes causing increased flood risk and damage to assets
- effects on Te Mana o te Wai
- impacts on recreational use and visual amenity of waterbodies
- unsuitability of downstream water to be used for irrigation, stock and domestic water supplies

The management of erosion and sediment during construction activities is of high importance. ECan's Erosion and Sediment Control Toolbox[1] (ESCT) identifies the key principles of Erosion and Sediment Control (ESC). These principles are applied through a variety of tools which, when combined in an effective management strategy, form an Erosion and Sediment Control Plan (ESCP).

Erosion and Sediment Control Plans

All construction works where bare soil will be exposed to rainfall require an ESCP in general accordance with ECan's ESCT.

Resource consent from ECan is generally required when undertaking earthworks and discharging water generated during construction. The intention of the Global Discharge Consents are to allow construction discharges to the TDC network for small sites, without requiring a separate ECan consent (Figure 8-1). However, approval must be sought and gained from TDC prior to discharges commencing.

[1] ECan 2021. Erosion and Sediment Control Toolbox for Canterbury. https://esccanterbury.co.nz/. Canterbury Regional Council.

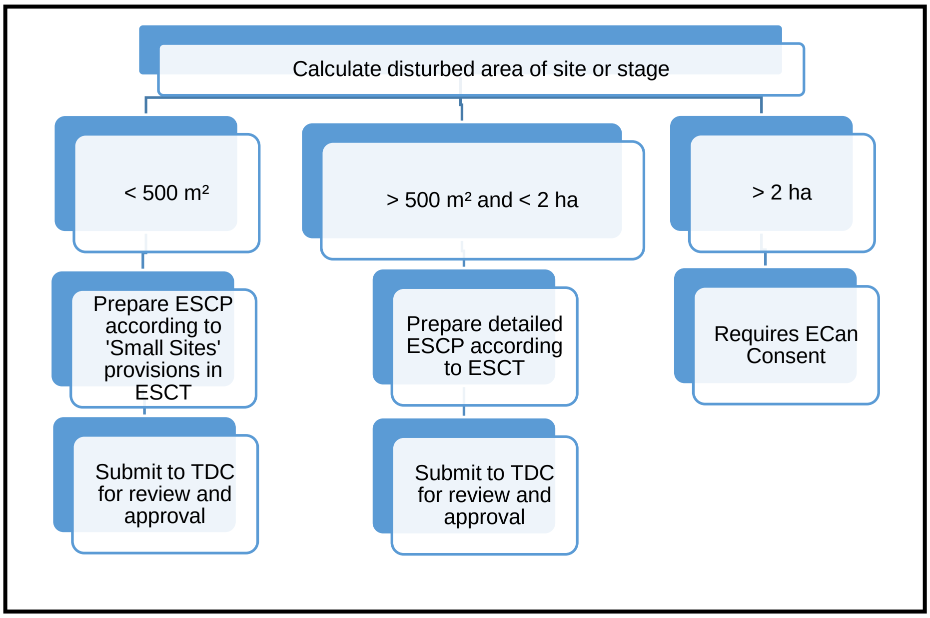

Figure 8-1: Flow chart of ESCP requirements

For minor developments (sites< 500 m²) an ESCP, as outlined in ECan's ECST Checklist 25, shall be submitted to TDC for approval showing where and how the following site management measures, at a minimum, are to be achieved:

- stabilised all-weather access
- control of water at the top of the site
- control of sediment at the foot of the site
- stockpiles and construction materials located within the sediment control zone
- connection of downpipes as soon as possible

For major development (sites greater than 500 m^2 and < 2 ha), the ESCP shall be submitted to TDC for approval including the following detail, at a minimum:

- Location of the works, and cut and fill areas
- Contour information at suitable intervals
- Erosion and sediment controls; including supporting sizing calculations
- Catchment boundaries for the sediment controls
- Details of construction method, timing and duration
- Monitoring and maintenance schedules

Staging of construction is important. Discharge of stormwater runoff during construction from an area of disturbed land exceeding 2 ha will not be authorised by Council's global consent.

Construction should not commence without evidence of an ESCP that has been approved by Council. Council may inspect the development during construction at any time to check for compliance with the approved ESCP and ECan's ESCT.

Care should be taken if sediment-laden runoff is discharged into any infiltration-based stormwater systems during construction. Council maintain the right not to authorise any system under their global consent that they suspect has been compromised through poor construction practices (heavy sediment loads or over compaction of infiltration surfaces) and may require confirmation of performance prior to acceptance.

Table H: Typical Inspection/Maintenance requirements for Stormwater Systems		
Activity	**Maintenance Method**	**Frequency**
General		
Inspect sumps	Visual inspection, ensure grates securely fitted and remove accumulated sediment	6-monthly
Sweeping of kerb and channel	Street sweeper to remove accumulated debris	3-monthly
Inspect structures for erosion, scour or damage.	Remove sediment, debris and litter. Reinstate as required	6-monthly
Check for hydrocarbon accumulation or spills	Remove hydrocarbons that are visible in a layer > 5 mm thick within sumps	Annually
Basins		
Inspect inlet and outlet structures for erosion, scour or damage.	Reinstate levels and vegetation as required.	After significant rainfall events
Clean screens or grates and orifice	Remove sediment, debris and litter.	3-monthly and after significant rainfall events
Repair any structural damage	As required depending on damage	As required
Inspect for sediment deposition or scour within basin and on embankments	Reinstate levels and vegetation as required.	6-monthly
Check for ponding	Ensure basins adequately drain and are dry 48 - 72 hr after rainfall event.	After significant rainfall events
Check vegetation cover	Assess plants for disease or die-off and re-plant if necessary, to maintain healthy coverage of vegetation. Remove weeds as required.	3-monthly
Maintain grass cover	Mow grass to maintain healthy grass cover at a length of between 50mm and 100mm. Remove clippings. Remove weeds as required and re-seed grass in bare areas.	As required
Check for hydrocarbon accumulation or spills	Remove hydrocarbons that are visible over an area >0.5 m²	6-monthly
Wetlands		
Inspect inlet and outlet structures for erosion, scour or damage.	Remove sediment, debris and litter. Reinstate levels and vegetation as required.	After significant rainfall events

Clean any screens or grates and orifice	Remove sediment, debris and litter.	3-monthly and after significant rainfall events
Repair any structural damage	As required depending on damage	As required
Check for sediment deposition	Remove sediment build up which affects the operation of the wetland	6-monthly
Check vegetation cover	Assess plants for disease or die-off and re-plant if necessary, to maintain healthy coverage of vegetation. Remove weeds as required	3-monthly
Check wetland embankment and baffles	Assess for structural integrity and reinstate slumping and vegetation as required.	3-monthly
Check for hydrocarbon accumulation or spills	Remove hydrocarbons that are visible over an area >0.5 m²	6-monthly
Inspect for pest species	Check for signs of pest species and control as required	6-monthly
Swales		
Inspect inlet and outlet structures for erosion, scour or damage.	Remove sediment, debris and litter. Reinstate levels and vegetation as required.	After significant rainfall events
Inspect for sediment deposition, scour and ponding along swale length	Reinstate levels and vegetation as required.	6-monthly
Check vegetation cover	Assess plants for disease or die-off and re-plant if necessary, to maintain healthy coverage of vegetation. Remove weeds as required.	3-monthly
Maintain grass cover	Mow grass to maintain healthy grass cover at a length of between 50mm and 100mm. Remove clippings. Remove weeds as required and re-seed grass in bare areas.	As required
Check for hydrocarbon accumulation or spills	Remove hydrocarbons that are visible over an area >0.5 m²	6-monthly
Bioretention		
Inspect inlet and outlet structures for erosion, scour or damage.	Remove sediment, debris and litter. Reinstate levels and vegetation as required.	After significant rainfall events
Clean grates	Remove sediment, debris and litter.	3-monthly and after significant rainfall events

Repair any structural damage	As required depending on damage	As required
Check for sediment deposition	Remove sediment build up which can block the filter media	6-monthly
Check vegetation cover	Assess plants for disease or die-off and re-plant if necessary, to maintain healthy coverage of vegetation. Remove weeds as required. Do not apply fertilizer.	3-monthly
Check underdrainage is free from blockage	Clear underdrainage with a water jet. CCTV inspection could be utilized if required.	Annually
Check for hydrocarbon accumulation or spills	Remove hydrocarbons that are visible over an area >0.5 m²	6-monthly
Inspect for pest species	Check for signs of pest species and control as required	6-monthly
Infiltration		
Inspect inlet and outlet structures for erosion, scour or damage.	Remove sediment, debris and litter. Reinstate levels and vegetation as required.	After significant rainfall events
Clean any screens or grates and orifice	Remove sediment, debris and litter.	3-monthly and after significant rainfall events
Check for sediment deposition	Remove sediment build up which can block the infiltration media	6-monthly
Check for hydrocarbon accumulation or spills	Remove hydrocarbons that are visible over an area >0.5 m²	6-monthly
Check for ponding	Ensure systems adequately drain and are dry 48 - 72 hr after rainfall event.	After significant rainfall events
Proprietary Systems		
Inspect inlet and outlet structures for erosion, scour or damage.	Remove sediment, debris and litter. Reinstate levels as required.	After significant rainfall events
Clean any screens or grates and orifice	Remove sediment, debris and litter.	3-monthly and after significant rainfall events
Repair any structural damage	As required depending on damage	As required
Check for hydrocarbon or solids accumulation	Remove via vacuum truck or other method as specified by supplier	As specified
Replace media or cartridges	As specified by supplier	As specified

1.1 INFILTRATION TESTING

1.1.1 TEST SELECTION

Accepted testing methodologies and their application are summarised below. Refer to Section 3.7.3 for conversion of test results into a design infiltration rate.

- **Flooded Pit Test** – Designed to mimic a soak-pit's actual operation.
- **Auger Test** – A variation on the flooded pit test.
- **Topsoil Soakage Test** – A shallow depth test carried out at the topsoil level. This test is required to assess infiltration rates through the topsoil that is proposed to be used to line soakage systems.

1.1.2 FLOODED PIT TEST

The flooded pit test is to be in accordance with the BRE Digest 365 Soakaway Design (Building Research Establishment (BRE), 2016) methodology and is designed to mimic the operation of an actual soak-pit in a design event situation. This testing methodology can account for the effects of groundwater mounding when correctly carried out.

Designers should note that the results of tests may be affected by seasonal factors. In the winter and spring the soil moisture and groundwater level will be higher than in the summer (except where up-gradient irrigation has a significant effect). Testing under a worst case basis should be undertaken.

The test procedure is as follows:

1. Excavate a test pit to the anticipated design depth, or a minimum of 1.5 m. Ideally plan dimensions will also be similar to that expected / proposed. Alternatively, the minimum pit dimensions may be 0.3-1.0 m wide, by 1-3 m long, which is easily achieved with an excavator. The pit should have vertical sides trimmed square.

   Note: **Do not enter the pit,** and take care working near the sides of the pit, particularly where they may be unstable. Where there is a risk of people or livestock falling into the pit do not leave it unattended or provide means to prevent entry.

2. Measure the pit dimensions carefully (depth, length / width at top and length / width at base). For safety reasons, do not enter the pit, estimate the base dimensions if necessary.
3. If necessary for stability, the pit should be filled with granular material. When granular fill is used, a full-height, perforated, vertical observation tube should be positioned in the soakage trial pit so that water levels can be monitored with a dip tape.
4. Place a fixed object at the top of the pit to provide a consistent point for measuring the depth to the water surface.

5. Fill the pit with clean uncontaminated water to the maximum design water depth. In the absence of any design information, use a maximum water depth to 300 mm below ground level. Rapid inflow is needed so that the pit can be filled in a short time (this is likely to require a minimum 100 mm hose for filling the pit).

   Note that the test will require a large water tanker (ideally 8-10 m^3) and the means to refill locally in a reasonable time. Council can advise the location of the nearest filling point where tankers can be filled and refilled for the purpose of undertaking this test.

6. If the water drains too fast to fill the pit with a full flowing 100 mm hose, no further testing is required. In this case, the maximum design value of 1,000 mm/hr can be used.

7. Record the depth to the water surface and time from filling, at intervals sufficiently close to clearly define water level versus time, until empty or the water level has at least dropped below 25% of the maximum design water depth.

8. Calculate the soil infiltration rate from the time taken for the water level to fall from 75% to 25% effective storage depth in the soakage trial pit:

   $$f = 1000\frac{V_{\mathrm{p}75-25}}{a_{\mathrm{s}50}\ t_{\mathrm{p}75-25}} \qquad \text{Eqn (3-1)}$$

   where f = soil infiltration rate (mm/hr)

   $V_{75\text{-}25}$ = effective storage volume between 75% and 25% maximum design depth (m^3)

   a_{s50} = internal surface area of the soakage trial pit up to 50% effective storage depth and including the base area (m^2)

   $t_{p75\text{-}25}$ = the time for the water level to fall from 75% to 25 % maximum design depth (hrs).

9. If it is impossible to carry out a full-depth soakage test, the soil infiltration rate calculation should be based on the time for the fall of the water level from 75% to 25% of the actual maximum water depth achieved in the test. The effective area of loss from the soakage trial pit is then calculated as the internal surface area of the pit to 50% maximum depth achieved, plus the base area of the soakage trial pit.

10. Repeat the test a minimum of three times on the same or consecutive days.

1.1.3 AUGER TEST

The flooded test pit method is suitable for use with auger holes with modifications to items 4 and 8 as follows:

4. Fill the pit with clean uncontaminated water to the maximum design water depth and maintain full for at least four hours. In the absence of any design information, use a maximum water depth to 300 mm below ground level.

8. Continue to repeat the test until the calculated infiltration rates for two consecutive tests are within 5%.

1.1.4 TOPSOIL SOAKAGE TEST

This methodology is designed to ascertain topsoil infiltration rates where that topsoil is proposed to be used to line an infiltration swale or basin. The test methodology is designed to be simple and easy to carry out.

The measured topsoil infiltration rate can also be used to verify Horton's Infiltration Values for use in hydraulic models.

The test procedure is as follows:

1. Using a sharp spade cut out a piece of turf approximately 250 x 250 mm and 50 – 100 mm deep. Make sure to score the sides after the hole is dug because a spade can smear and compact the sides, which can affect the infiltration rate.
2. Place a fixed object at the top of the pit to provide a consistent point for measuring the depth to the water surface (or ruler fixed in the hole).
3. Carefully measure the dimensions of the excavation.
4. Fill the excavation to the top with water and time the time taken to fully drain.
5. Where there is minimal separation from ground water (i.e. <1 m) this test should be done in conjunction with the Flooded Pit Test to ascertain the capacity of the underlying soils.
6. Calculating the Infiltration Rate

   $$f = \frac{d}{t}\,\frac{A_{base}}{A_{50}} \qquad \text{Eqn (3-2)}$$

   Where: f = soil infiltration rate (mm/hr)

   d = total excavation depth (mm)

   A_{50} = internal surface area of the trial pit up to 50% the total depth and including the base area (m^2)

   A_{base} = base area (m^2)

   t = the time for the pit to empty (hrs).

7. Repeat the process until the excavation consistently drains within a set period of time – this represents the ultimate infiltration rate.

1.1.5 ALTERNATIVE INFILTRATION TESTING METHODS

The following alternative infiltration test methods will also be accepted:

- Soil Permeability Measurement - Constant Head Test (in accordance with Appendix G of AS/NZS 1547:2012) (Standards New Zealand, 2012)
- ASTM D3385-09, Standard Method for Infiltration Rate of Soils using Double Ring Infiltrometer (ASTM International, 2009).

Appendix E - Time of Concentration Methodology

Design Storm Duration

The maximum peak flow is assumed to occur under an average rainfall of duration just equal to the time necessary for all of the catchment to begin contributing, the so-called Time of Concentration or Tc, which is the time taken for runoff from the furthest point of the catchment to reach the design point. For longer durations the rainfall intensity will be less and for shorter durations not all the catchment will be contributing. In both cases the resulting calculated flow will normally be smaller.

The Time of Concentration to the point of interest is taken as an initial time of entry (Te) (overland flow) plus travel time (Tt) in open channels, road channels and pipes. The minimum Tc shall not be less than 10 minutes in residential or commercial areas, and not less than 25 minutes in parks and rural areas. For hillside flow, ensure that the travel time is evaluated from the very top of the catchment.

Time of Entry (Te)

Time of Entry is the time taken for runoff to travel overland from properties and roofs to the point of entry at the road channels. Overland flow can occur on either grassed or sealed surfaces. In urban areas, overland flowpaths will typically be less than 50 m, due to interception by fences or structures. Standard times of entry are applied to urban areas:

Industrial/Business/Commercial areas	Te = 5 minutes
Residential areas	Te = 10 minutes
For hillside, parks and rural catchments,	Te = the time for overland flow, as estimated by Figure A-1 or the following equation

$$\boldsymbol{Te\ (Overland) = \frac{100nL^{0.33}}{s^{0.2}}} \qquad \textbf{Eqn (A-1)}$$

Where *Te(overland)* = flow time (mins)

n = roughness coefficient for overland flow (Table A-1)

L = length of overland flow path (m)

S = slope of catchment (%)

If the actual slope of sections of the catchment varies significantly from the average, determine the average slope using the Equal Areas Method[1].

[1] MBIE, 2020. Acceptable Solutions and Verification Methods for the New Zealand Building Code Clause E1 Surface Water. Ministry of Business, Innovation and Employment. Amended November 2020.

Table A-1: Horton's n roughness values for overland flow	
Surface Type	**Horton's n roughness coefficient**
Asphalt/concrete	0.010 – 0.012
Bare sand	0.010 – 0.060
Bare clay/loam	0.012 – 0.033
Gravelled surface	0.012 – 0.030
Short grass	0.100 – 0.200
Lawns	0.200 – 0.300
Pasture	0.300 – 0.400
Dense Shrubbery	0.400

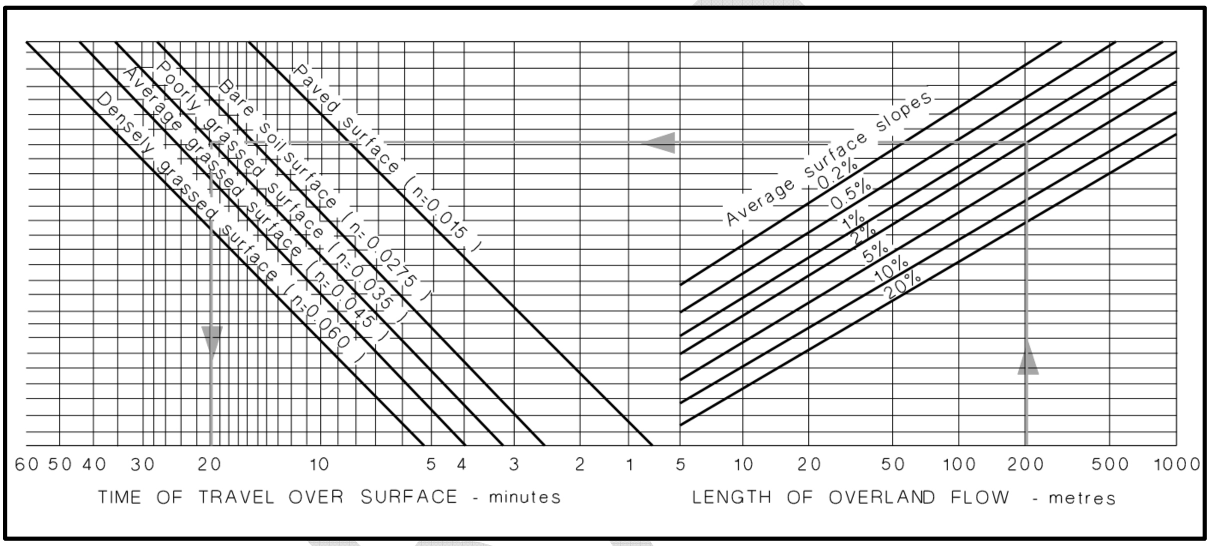

Figure A-1: Nomograph for estimating overland sheet flow times (CCC, 2020)

Time of Network Flow (Tt)

The time of network flow, or travel time (Tt), is comprised of the time of road channel flow, pipe network flow, and open channel flow.

The *Time of Road Channel Flow*, is the time taken for water to flow from the point of entry at the road channel, to the point of discharge to a sump, drain, or other outlet. Figure A-2 gives side channel velocities and flow times with flow depth to top of kerb for flat channels based on a Manning 'n' of 0.016.

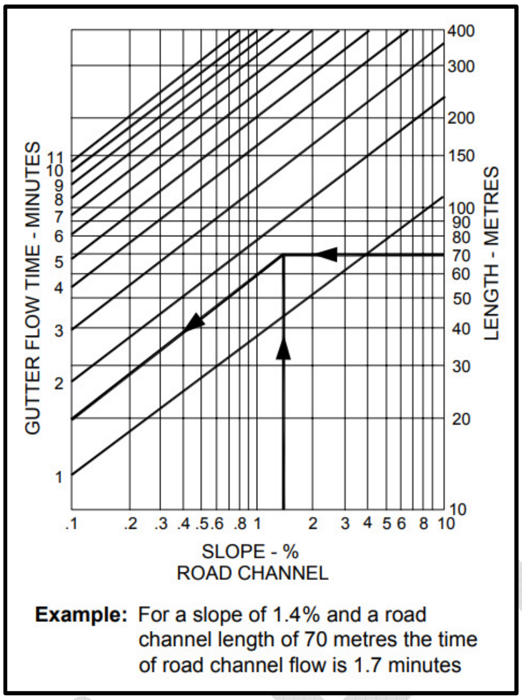

Figure A-2: Side channel flow time from channel length and slope (CCC, 2020)

The *Time of Pipe Flow* can be derived from flow velocity obtained from the Pipe Flow Nomograph[2]. To follow this procedure, longitudinal sections are required of the piped systems, giving internal pipe diameters, lengths, and gradients. For preliminary calculations, if there is little detail of the final pipe systems, then the typical velocities in Table A-2[3] may be used.

Table A-2: Typical pipe flow velocities for various gradients (CCC, 2020)

Gradient	Grade	Typical velocities (m/s) for various diameters (mm)					
		225	300	375	450	600	750
Flat	1 in 500	0.5	0.6	0.7	0.8	1.0	1.1
	1 in 200	0.8	1.0	1.1	1.3	1.5	1.8
Moderate	1 in 100	1.1	1.4	1.6	1.8	2.2	2.5
	1 in 50	1.6	1.9	2.2	2.5	3.1	3.6
Steep	1 in 20	2.5	3.1	3.5	4.0	4.9	5.6
	1 in 10	3.6	4.3	5.0	5.7	6.9	8.0

The *Time of Open Channel Flow* is calculated by means of the Manning equation (refer to WWDG Chapter 22: Hydraulics). If there is insufficient data available to calculate the time of open channel flow, the approximate natural stream velocities given in Table A-3[4] can be used.

[2] MBIE, 2020. Acceptable Solutions and Verification Methods for the New Zealand Building Code Clause E1 Surface Water. Ministry of Business, Innovation and Employment. Amended November 2020.
[3] CCC, 2020. Waterways, Wetlands and Drainage Guide: Part B. Christchurch City Council. Updated June 2020.
[4] CCC, 2020. Waterways, Wetlands and Drainage Guide: Part B. Christchurch City Council. Updated June 2020.

Table A-3: Approximate natural stream velocities (CCC, 2020)		
Catchment Description	**Grade**	**Velocity**
Flat	Flat to 1 in 100	0.3 – 1.0 m/s
Moderate	1 in 100 to 1 in 20	0.6 – 2.0 m/s
Hillside	1 in 20 or steeper	1.5 – 3.0 m/s

Where open channel or stream flow forms a significant part of the catchment (particularly for rural channelised catchments), computer modelling must be considered. However, there are a number of methods that can be used to estimate the travel time for open channel flow e.g. Bransbury Williams, Mannings, Ramser-Kirpich and US Soil Conservation Service procedures. The estimates from these methods will vary because different interpretations of the time of concentration are involved and not all of the formulae are suited to the same conditions.

Table G1: Alternative Water Quality Target Contaminant Concentration Method		
	Methodology	**Reference/Comment**
1.	Identify contaminants of concern to receiving environment, and standards required to be met	Refer to LWRP[1] Schedules 5 and 8 for specific Water Quality Classes. Water Quality Classes can be determined from ECan Canterbury Maps[2].
2.	Identify contaminants of concern for the proposed activity/land use	Typically urban stormwater contaminants of concern include: • Total Suspended Solids (TSS) • Metals (Zinc, Copper, Lead) • Hydrocarbons/Oil and Grease Additionally, consider microbiological contaminants (E. *coli*) and nutrients (nitrogen and phosphorus), or as defined by specific land use.
3.	Identify concentrations of untreated contaminants discharged	Preferably, on-site sampling of stormwater would be undertaken to identify concentrations. Where this data is unavailable, consult resources including: • NIWA Urban Runoff Quality Information System (URQIS)[3] • WWDG[4] Table 6-2 • Brough et al, 2012[5] or other published data
4.	Determine Treatment Train of devices using % removal rates	Typical % removal rates of contaminants by devices can be found from resources including: • Table 5-3 in Section 5.5 • WRF International Stormwater BMP Database[6] • WWDG[7] Table 6-6
5.	Apply Mixing Zone calculation to identify concentrations of contaminants in discharge, beyond the mixing zone	Refer to LWRP[8] Schedule 5 for mixing zone calculation for waterways and lakes.

[1] ECan, 2019. Canterbury Land and Water Regional Plan. Canterbury Regional Council, February 2019.
[2] ECan, 2021. Canterbury Maps. https://canterburymaps.govt.nz/ Canterbury Regional Council, 2021.
[3] NIWA, 2020. Urban Runoff Quality Information System (URQIS). https://niwa.co.nz/information-services/urban-runoff-quality-information-system-urqis. National Institute of Water and Atmospheric Research.
[4] CCC, 2020. Waterways, Wetlands and Drainage Guide: Part B. Christchurch City Council. Updated June 2020.
[5] Brough A, Brunton R, England M, Eastman, R. 2012. Stormwater Quality – An analysis of runoff from modern subdivisions and the implications for stormwater treatment. Water New Zealand Stormwater Conference 2012.
[6] WRF, 2020. https://www.bmpdatabase.org/ International Stormwater BMP Database.
[7] CCC, 2020. Waterways, Wetlands and Drainage Guide: Part B. Christchurch City Council. Updated June 2020.
[8] ECan, 2019. Canterbury Land and Water Regional Plan. Canterbury Regional Council, February 2019.

High Intensity Rainfall Design for Timaru District

TIMARU DISTRICT COUNCIL
Te Kaunihera ā-Rohe o Te Tihi o Maru

The following figures are to be used when calculatiing the required design for stormwater attenuation system within Timaru Distrrict.
The design rainfalls has allow for the effect of climate change to 2090.
The effect of climate change is based on the estimates in the *"Climate Change Effect and Impacts Assessment - A guidance manual for Local Government in New Zealand"* by the Ministry for the Environment (MfE, 2008).

Table 1 - TIMARU: Rainfall Depth Duration Frequency Estimates (mm)										
ARI	**10-min**	**20-min**	**30-min**	**1-hr**	**2-hr**	**6-hr**	**12-hr**	**24-hr**	**48-hr**	**72-hr**
Dist	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*
2.33	5	7	8	11	16	29	38	56	74	80
5	8	12	14	18	24	40	56	82	105	112
10	12	17	20	25	31	51	71	105	130	139
20	15	24	27	32	39	63	88	127	154	163
50	21	34	36	42	51	79	110	157	184	194
100	26	41	43	50	59	90	125	176	204	215

Table 2 - TIMARU: Rainfall Depth Intensity (mm/hr)										
ARI	**10-min**	**20-min**	**30-min**	**1-hr**	**2-hr**	**6-hr**	**12-hr**	**24-hr**	**48-hr**	**72-hr**
Dist	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*
2.33	30	21	16	11	8	5	4	3	2	2
5	48	36	28	18	12	7	5	3	3	2
10	72	51	40	25	16	9	6	4	3	2
20	90	72	54	32	20	11	7	5	4	3
50	126	102	72	42	26	13	9	7	4	3
100	156	123	86	50	30	15	10	7	5	3

Table 3 - TEMUKA: Rainfall Depth Duration Frequency Estimates (mm)										
ARI	**10-min**	**20-min**	**30-min**	**1-hr**	**2-hr**	**6-hr**	**12-hr**	**24-hr**	**48-hr**	**72-hr**
Dist	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*
2.33	5	6	7	10	16	29	39	53	66	71
5	6	8	10	16	22	39	54	72	91	99
10	8	10	14	21	29	49	66	90	114	123
20	9	13	17	25	35	59	78	106	137	147
50	10	16	21	32	43	73	95	129	165	177
100	13	16	24	37	49	82	107	145	184	198

Table 4 - TEMUKA: Rainfall Depth Intensity (mm/hr)										
ARI	**10-min**	**20-min**	**30-min**	**1-hr**	**2-hr**	**6-hr**	**12-hr**	**24-hr**	**48-hr**	**72-hr**
Dist	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*
2.33	30	18	14	10	8	5	4	3	2	1
5	36	24	20	16	11	7	5	3	2	2
10	48	30	28	21	15	8	6	4	3	2
20	54	39	34	25	18	10	7	4	3	3
50	60	48	42	32	22	12	8	5	4	3
100	78	48	48	37	25	14	9	6	4	3

Table 5 - GERALDINE: Rainfall Depth Duration Frequency Estimates (mm)										
ARI	**10-min**	**20-min**	**30-min**	**1-hr**	**2-hr**	**6-hr**	**12-hr**	**24-hr**	**48-hr**	**72-hr**
Dist	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*
2.33	6	7	9	14	20	36	50	68	84	90
5	7	10	14	21	27	50	68	92	117	126
10	10	14	17	26	37	62	84	115	147	158
20	12	16	22	32	44	75	100	136	176	188
50	14	21	27	42	55	94	122	165	211	227
100	16	23	31	48	63	106	137	186	237	254

Table 6 - GERALDINE: Rainfall Depth Intensity (mm/hr)										
ARI	**10-min**	**20-min**	**30-min**	**1-hr**	**2-hr**	**6-hr**	**12-hr**	**24-hr**	**48-hr**	**72-hr**
Dist	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*
2.33	36	21	18	14	10	6	5	3	2	2
5	42	30	28	21	14	8	6	4	3	2
10	60	42	34	26	19	10	7	5	4	3
20	72	48	44	32	22	13	8	6	4	3
50	84	63	54	42	28	16	10	7	5	4
100	96	69	62	48	32	18	11	8	5	4

Table 7 - PLEASANT POINT: Rainfall Depth Duration Frequency Estimates (mm)										
ARI	**10-min**	**20-min**	**30-min**	**1-hr**	**2-hr**	**6-hr**	**12-hr**	**24-hr**	**48-hr**	**72-hr**
Dist	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*
2.33	5	6	7	10	16	29	39	53	66	71
5	6	8	10	16	22	39	54	752	91	99
10	8	10	14	21	29	49	66	90	114	123
20	9	13	17	25	35	59	78	106	137	147
50	10	16	21	32	43	73	95	129	165	177
100	13	19	24	37	49	82	107	145	184	198

Table 8 - PLEASANT POINT: Rainfall Depth Intensity (mm/hr)										
ARI	**10-min**	**20-min**	**30-min**	**1-hr**	**2-hr**	**6-hr**	**12-hr**	**24-hr**	**48-hr**	**72-hr**
Dist	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*	*PE3*
2.33	30	18	14	10	8	5	4	3	2	1
5	36	24	20	16	11	7	5	31	2	2
10	48	30	28	21	15	8	6	4	3	2
20	54	39	34	25	18	10	7	4	3	3
50	60	48	42	32	22	12	8	5	4	3
100	78	57	48	37	25	14	9	6	4	3